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THE DESIGN OF TRANSMISSION LINE SUPPORT FOUNDATIONS

- AN OVERVIEW -

Working Group

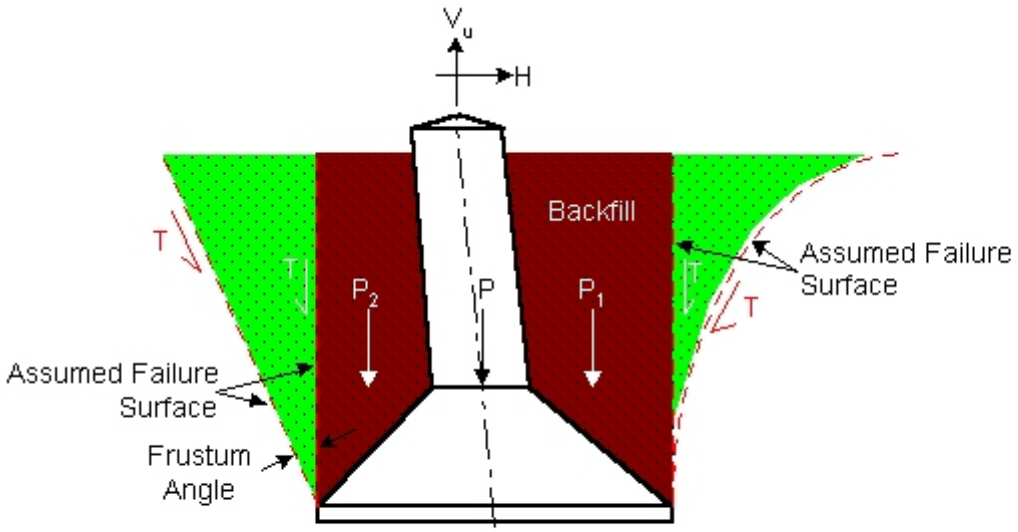
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Working Group 22.07 - Foundations

The Design of Transmission Line Support Foundations - An Overview

February 2002

Synopsis

This report was prepared by a task force drawn from Working Group 07 'Foundations' of Cigre Study Committee 22 and provides an overview of the design of overhead transmission line support foundations.



Transmission line foundations provide the interlinking component between the support and in-situ soil and/or rock. The interrelationship between the support type and the applied foundation loadings are considered, together with the methods of determining the engineering properties of the in-situ soil and/or rock.

For the purpose of this report, two principal categories of foundations have been considered:, Separate and Compact. Within each category the major types applicable to that category are reviewed, e.g. monoblock, spread, drilled shaft, pile, anchor, direct embedment of poles etc. For each type of foundation considered, the geotechnical design principles, the minimum geotechnical data required and the influence of construction methods on the design are examined.

In order to change from deterministic to reliability-based design practice, the corresponding need to determine the e^{th} percent exclusion limit strength (resistance) of the foundation has been identified, together with the associated need to establish probabilistic strength reduction factors based on full scale foundation load tests.

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1 INTRODUCTION

1.1 General

Transmission line foundations are the interlinking component between the support and the in-situ soil and/or rock. However, unlike the other major components of a transmission line, they are constructed wholly or partly in-situ in a natural medium whose characteristic properties may vary between support locations and possibly between adjacent foundations. Correspondingly, transmission line foundation design is an art based on judgement derived from experience and testing.

The foundations for overhead transmission line supports differ from those for buildings, bridges and other similar foundation types from two points of view : the modes of loading they are subjected to and the performance criteria they must satisfy.

Generally, foundations for buildings, etc. are subjected to large dead loads (mass) which result mainly in vertical compressive loads. The allowable movements of the foundations which support these types of structures are limited by the flexibility of the supported structures. Conversely, the forces acting on overhead transmission line foundations are typically an overturning moment. In the case of separate foundations, individual foundation loads become a combination of uplift, compression and horizontal shear loads. These foundation loads arise primarily from dead load and a combination of wind and/or ice action on both the conductors and the support. Correspondingly, these loads have variable and probabilistic characteristics. The allowable displacements of the foundations must be compatible with the support types (lattice tower, monopole and H-frame supports) and with the overhead line function (electrical clearances). For poles located in a populated area, foundation displacement must result in pole displacements which are compatible with visual impression of safety.

This report is an overview of the most common types of overhead transmission line support foundations used in practice. Although, the number of design approaches presented is extensive, this overview is not an exhaustive report.

Many issues have to be considered in the design of overhead transmission line support foundations :

- Support type;
- Load type and duration;
- Geotechnical characteristics of soil and/or rock;
- The reliability of the analytical design model;
- The degree of movement the support can withstand;
- Level of security required and whether the foundation should be stronger than the support, or have the same strength;
- Available materials;
- Access for construction equipment;
- Economics.

The recommended methodology (procedure) for the design of transmission line foundations for both deterministic and reliability-based design (RBD) approaches is shown in Figure 1.1.

This methodology can be described in the following steps:

a) Establish the support type and corresponding level of security required;
b) Establish the variations in geotechnical/ geological conditions along the transmission

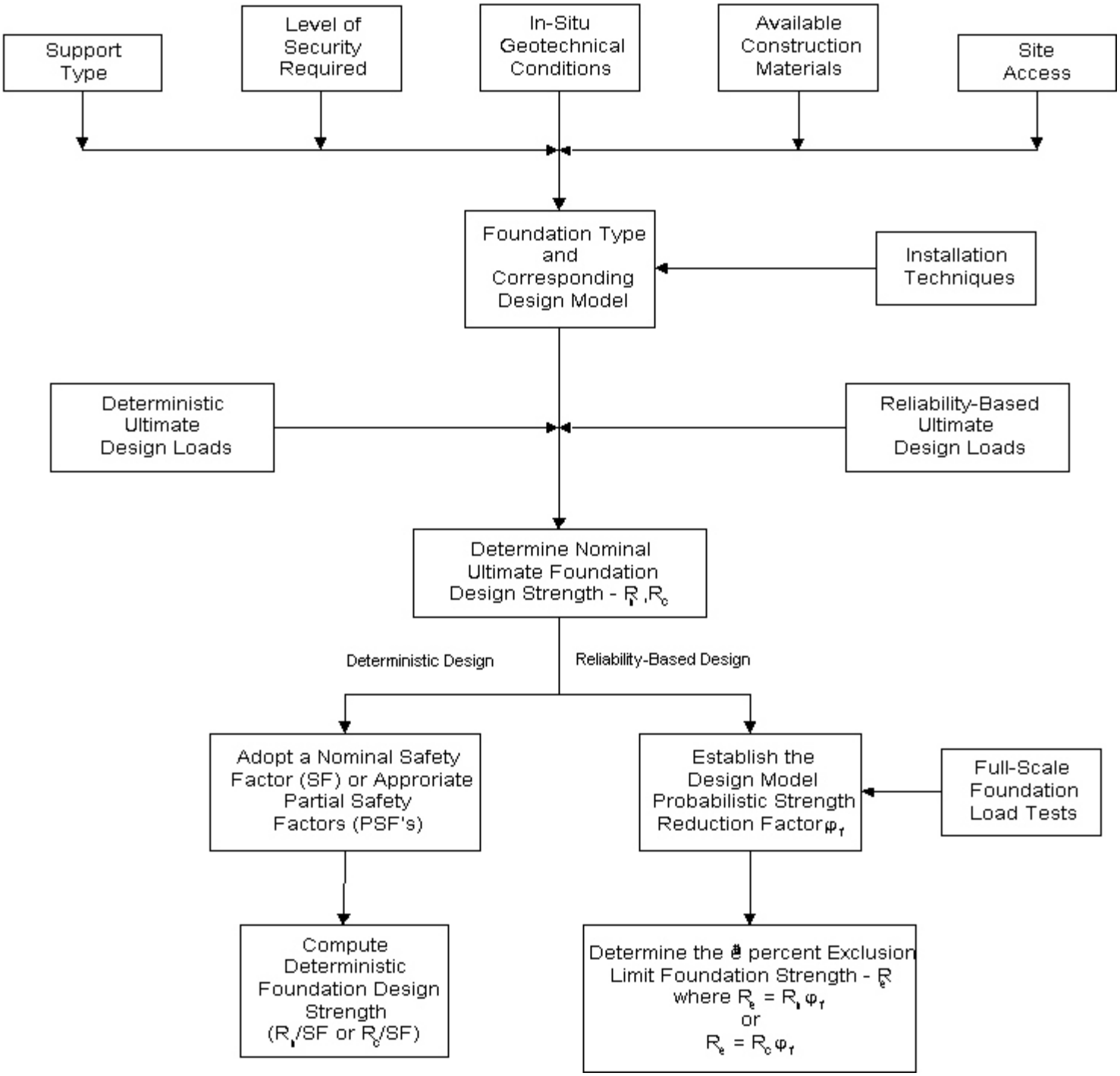


Figure 1.1 - Diagrammatic Representation of Foundation Design Procedure

line route including environmental impacts and the appropriate geotechnical design parameters required for the proposed foundation design model;

c) Consider possible sources of construction materials and any restrictions on site accesses for materials and/or construction equipment;

d) Select appropriate type of foundation and corresponding geotechnical design model, taking into consideration the proposed installation techniques;

e) Obtain and/or calculate appropriate ultimate deterministic or reliability-based foundation design loadings;

f) Determine the nominal ultimate foundation design strength (capacity);
g) For RBD establish the design probabilistic strength reduction factor based on the results of full-scale load tests and hence determine the e^{th} percent exclusion limit foundation strength;
h) For deterministic design approach adopt a nominal factor and compute the deterministic foundation design strength (and if appropriate check, wherever possible, the results from full-scale foundation load tests).

In addition new techniques can influence the design approach to be adopted. These considerations explain the number and the diversity of the available design methods.

1.2 Aims and Objectives

The aim of this report is to provide an overview of the various methods for the design of a number of foundation types. Correspondingly, to achieve this overall aim an extensive literature review has been undertaken to establish the range of potential foundation geotechnical design models. However, it is not the intention of this report to present detailed geotechnical design equations which are described in the text-books or in specialised literature.

For the purpose of this report two principal categories of foundations have been considered Separate and Compact. Anchor foundations have for convenience been included under separate foundations. Within each principal category the major foundation types applicable to that category have been reviewed e.g. monoblock, drilled shaft, direct embedment, pad and chimney, steel grillage, passive and active anchors, helical screw anchor etc.

Correspondingly the primary objective of this report is to outline for each major foundation type their characteristics, preferred range of use, general design methods and any specific limitations to be considered in their design or use.

Section 2 of this report considers the interrelationship between support type, foundation reaction and the potential types of foundation which could be used. This interrelationship is shown diagrammatically in Figure 1.2. Separate and Anchor foundations are considered in Section 3, while Compact foundations are reviewed in Section 4.

The limit state and reliability-based geotechnical design of the foundation including the calibration of geotechnical design models against the results of full-scale foundation load tests is considered in Section 5. A summary of this overview report is contained in Section 6, while Annex A contains a comprehensive reference list.

1.3 Definitions

The following definitions are used throughout this **report:**

Anchors and Anchor foundations: "Anchors can be used to provide tensile resistance for guys of any type of guyed support and to provide additional uplift resistance to spread footing type separate foundations. Common types of anchors are ground anchorages, helical screw anchors, precast concrete guy anchor blocks".

Compact foundations: "A compact (single) foundation is predominately subjected to an overturning moment in association with relatively small shears, vertical and torsional forces, which are usually resisted by lateral soil pressures. Generally this type of foundation is used for single

poles, narrow base lattice towers and for H-frame supports with predominate moment loadings , although raft foundations for wide base lattice towers are included in this category. Common types of compact foundations are monoblock, concrete pads, drilled shafts (augers) and rafts".

Separate foundations: "A separate foundation is predominately subjected to vertical compression or uplift forces in association with relatively small shears and torsional forces. Uplift forces are usually resisted by the dead weight of the foundation, earth surcharges and/or shear forces in the soil. Compression forces are resisted by vertical bearing and/or shear forces in the soil. Common types of separate foundations are spread footings, e.g. concrete pyramid / pad and chimney foundations, drilled shafts (augers) and single piles or pile groups".

Working load: An un-factored load derived from a climatic event with an undefined return period.

Nominal Ultimate Foundation Design Strength [R_n, R_c]**:** "The foundation strength derived from an un-calibrated theoretical design model, i.e. obtained when the geometric and geotechnical parameters are input into the theoretical design equation. This is usually taken to be R_n, however, the term characteristic strength [R_c] used in IEC 60826 is the nominal ultimate foundation design strength".

Nominal Safety Factor: "A factor based on code requirements and or experience".

Probabilistic Strength Reduction Factor [ϕ_n]**:** "Is the probabilistic factor which adjusts the predicted nominal (characteristic) ultimate strength, R_n to the e^{th} percent exclusion limit strength R_e". However, this factor does not take into consideration any desired strength coordination between the support and its foundation nor the number of foundations (components) subjected to the maximum load, i.e. factors Φ_S and Φ_N in IEC 60826.

Deterministic Design Strength: "The deterministic design strength of a foundation is the nominal ultimate strength [R_n, R_c] divided by a nominal factor of safety".

The e^{th} percent Exclusion Limit Strength [R_e]**:** "The foundation strength at the e^{th} percent exclusion limit".

All other definitions are in accordance to IEC 60050(466)-50 [IEC 1990] and IEC 61773 [IEC 1996], unless otherwise stated.

1.4 Safety and Environmental Issues

This report provides solely an overview of present practice for the design of foundations. No attempt has been made to cover aspects of design engineering related specifically to safety or environmental issues. Such matters shall be covered by design engineers in accordance with the required 'Health and Safety' and 'Environmental Assessment' practices.

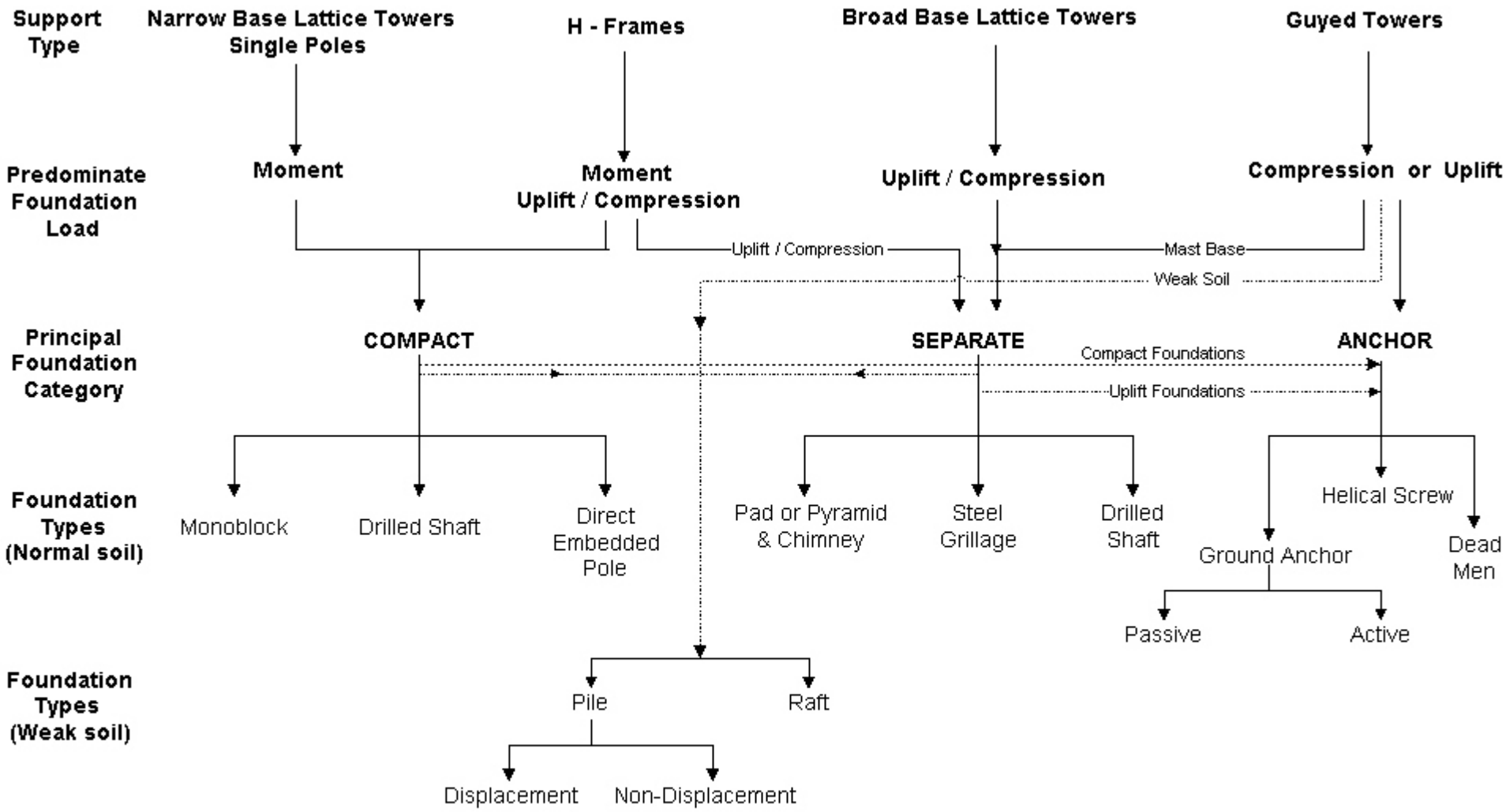


Figure 1.2 Diagrammatic Representation of Interrelationship between Support Type and Principal Foundation Category

Note: Weak Soil defined as soil with an SPT value of less than 10 blows per 300 mm, or an undrained shear strength of less than 35 kN/ sq.m

2 SUPPORT TYPES and FOUNDATION LOADS

2.1 Applied Loadings

2.1.1 Historical Perspective

From a historical perspective there has been a gradual change in the design procedures used to determine the loading applied to transmission line supports and hence to their foundations, initial design procedures were based on deterministic methods; however, in recent years probabilistic or semi-probabilistic limit state methods have been developed and are currently used in practice.

In the deterministic concept a 'working' or everyday loading event multiplied by an overload factor must be resisted by the ultimate strength of the support and the foundation divided by a safety factor or alternatively multiplied by a strength reduction factor. Alternatively, the 'working' load is multiplied by an 'overall global factor' of safety which must be resisted by the ultimate strength of the support or the foundation. In this instance the overall global factor of safety is a combination of the overload factor and the strength reduction factor. The two principal loading events usually considered under this approach are 'normal' everyday climatic events and abnormal or exceptional events, e.g. 'broken wire'. Different overload or global safety factors are applied to the loading event and different strength reduction factors are used depending on the degree of security required, which may in turn vary between different design methods and foundation types.

There are no universally accepted deterministic design codes for the determination of transmission line loadings. The majority of codes - standards are based on national requirements and/or regulations. Two of the codes which have received limited acceptance outside their country of origin are ANSI NESC C2 [ANSI 1998] and DIN VDE 0210 [DIN 1987].

For probabilistic or semi-probabilistic limit state methods a 'Limit state' is defined as having occurred if the transmission line or any part of it fails to satisfy any of the performance criteria specified. The principal limit state condition is climatic loading, whereby the defined climatic loading corresponding to a specific return period multiplied by a (partial) load factor must be resisted by the characteristic strength of the support or foundation multiplied by a (partial) strength reduction factor.

One of the major difficulties in the application of probabilistic design methods, is ensuring that there is sufficient full scale foundation test data available to accurately calibrate each design model with any degree of confidence. The use of a normal distribution curve inherently assumes an infinite number of samples. An approach that has been adopted to overcome the lack of sufficient data is the use of semi probabilistic techniques. In this, the results of the theoretical probabilistic design model are compared with the design of existing foundations with satisfactory service performance and the theoretical design model adjusted to give similar performance.

At present there are two reports on the application of semi-probabilistic methods to the determination of transmission line loadings IEC 60826 [IEC 1991] and ASCE Manual No. 74 [ASCE 1991].

2.1.2 System Design

Both IEC 60826 [IEC 1991] and ASCE Manual No.74 [ASCE 1991] considers the application of system design concepts, whereby the transmission line is considered as a complete integral system. A system design approach recognises the fact that a transmission line is composed of a series of interrelated components, e.g. conductors, insulators, supports, foundations etc.,

where the failure of any major component usually leads to the loss of power. The advantage of this concept is the ability to design for a defined uniform level of reliability or, alternatively, to design for a preferred sequence of failure by differentiating between the strength of various line components.

2.1.3 IEC 60826

IEC 60826 [IEC 1991] considers three principal limit state loading conditions: climatic, security and construction and maintenance. Of these only the climatic event has a probabilistic basis, the other two are deterministic concepts.

The basic design equation for the relationship between climatic loading and design strength may be written as:

$$\gamma_u Q_T < \phi_R R_n \quad \text{. . . . . Eq. 2.1}$$

where γ_u = factor depending on the span (use factor)
Q_T = load corresponding to a return period T
ϕ_R = global strength factor which considers the coordination of the between components, the number of components subjected to the load and the quality level of the component ($\phi_R = \phi_S \phi_N \phi_Q \phi_c$)
R_n = the nominal strength of the component
ϕ_S = factor related to strength coordination between different components
ϕ_N = factor related to number of components subjected to the design load
ϕ_Q = factor related to the quality level
ϕ_c = factor related to the relationship between the actual exclusion limit of R_n and the assumed value of e = 10%

Note: The term characteristic strength [R_c] used in IEC 60826 is the nominal ultimate strength.

The desired level of reliability can be achieved by selecting one of the three specified return periods, i.e. 50, 100 and 500 years and modifying the load event accordingly.

Criteria for the damage and failure (ultimate strength) limit states for foundations, the relationship between characteristic strength and nominal strength of foundations, strength coordination between components and the methods of calculating the characteristic strength of the foundations (based on normal distribution) are all given in IEC 60826.

A diagrammatic representation of the relationship between the probability density functions for the component load effect (f_Q) and the component resistance (f_R) is shown on Figure 2.1.

2.1.4 ASCE Manual No. 74

ASCE Manual No. 74 [ASCE 1991] is similar to IEC 60826 [IEC 1991] with respect to the principal limit state loading conditions considered. However, the approach adopted by the ASCE assumes that the reliability of the overall transmission system is equal to the reliability of the weakest component, whereas the IEC considers that the reliability of the line is a function of both the component reliability and the number of supports effected by the climatic event.

The basic design equation for the relationship between load and strength is given by:

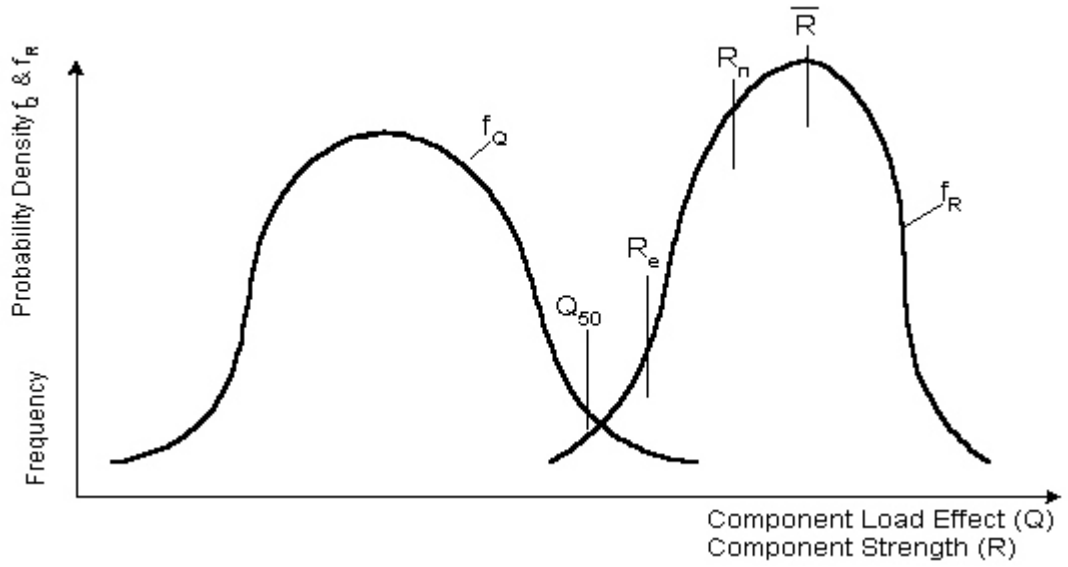
$$\phi R_e > [DL + \gamma Q_{50}] \quad \text{. . . . . Eq. 2.2}$$

where ϕ = strength (or resistance) reduction factor which can be selected to adjusted the reliability of the component
R_e = the e^{th} percent exclusion limit strength of the component
DL = the dead load effects
γ = load factor applied to the climatic load effect Q_{50} under consideration
Q_{50} = loads resulting from a 50-year return period climatic load event

The load factor (γ) can be adjusted on a relative basis from the 50-year base load event to take account of other recommended return periods, i.e. 100, 200 and 400 years, thereby accounting for the importance and possibly the length of the transmission line. The strength factor (ϕ) takes into account both the non-uniformity of exclusion limits and differences in coefficients of variation in the strength of components, it can be used optionally to adjust the relative reliability of each component.

The ASCE has simplified their approach with regards to the strength of the component for the different limit states, in that they consider the damage and failure (ultimate) limit state to be identical and as such the same nominal strength (R_n) can be used, whereas the IEC has different strength requirements for these limit states.

DiGioia [2000] gives an overview of the ASCE reliability-based design procedure with particular emphasis on support foundation design and the calibration of the geotechnical design model.



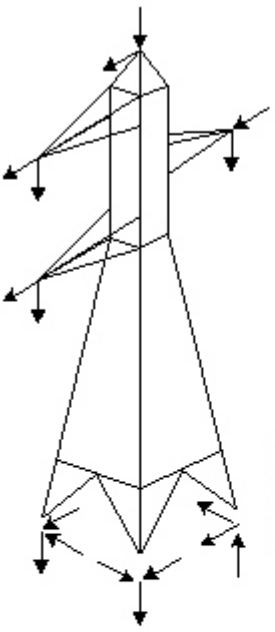
Q_{50} = 50 year Return period climatic load event
R_n = Nominal or characteristic strength
R_e = e% exclusion limit strength
$\overline{R}$ = Average strength

Figure 2.1 - Probability Density Functions for Component Load Effects and Strength

2.2 Support Types - Foundation Loads

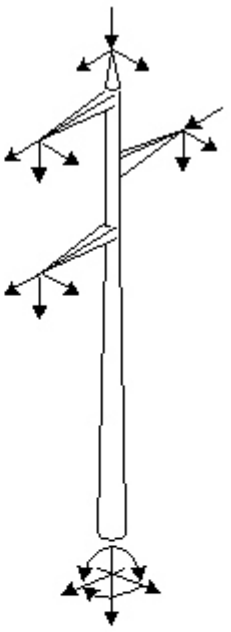
2.2.1 Single Poles and Narrow Base Lattice Towers

The foundation loads for single poles and narrow base lattice towers with compact foundations consist of overturning moments in association with relatively small horizontal, vertical and torsional forces.

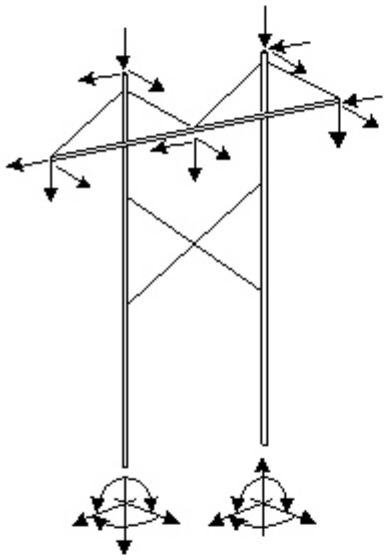


Note:
Rear tension leg applied foundation loading omitted for clarity

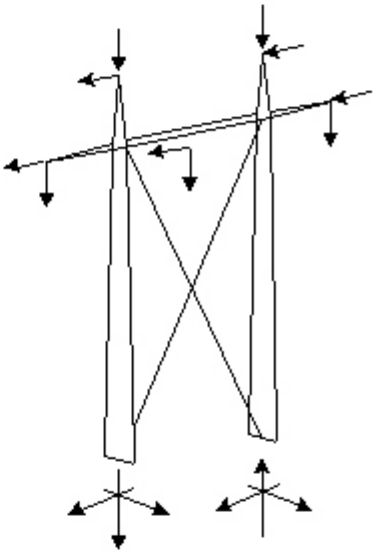
Lattice Tower - Separate Foundations



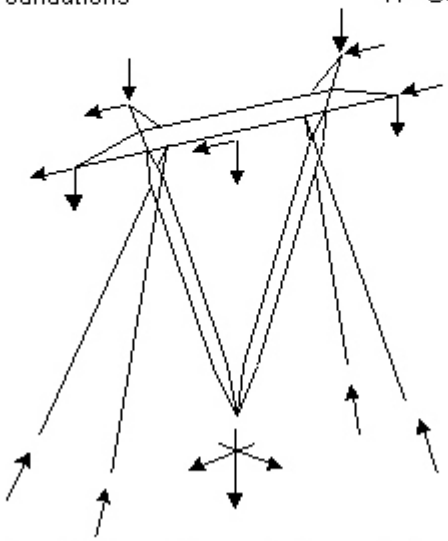
Single Pole - Compact Foundations



H - Support Separate Foundations



H - Support Separate Foundations



Externally Guyed Support - Separate Foundations

Figure 2.2 - Support Type - Applied Foundation Loading

2.2.2 H - Framed Supports

H - Framed supports are basically structurally indeterminate. The foundation loads can be determined either by making assumptions that result in a structurally determinate structure or by using computerised stiffness matrix methods. The foundation loads for H-frame supports consist of overturning moments in association with relatively small horizontal, vertical and torsional forces. If the connection between the supports and foundations are designed as pins or universal joints, theoretically the moments acting upon the foundations will be zero.

2.2.3 Broad Base Lattice Towers

Lattice tower foundation loads consist principally of vertical uplift (tension) or compression forces and associated horizontal shears. For intermediate and angle towers with small angles of deviation, the vertical loads may either be in tension or compression. For angle towers with large angles of deviation and terminal towers one side will normally be in uplift and the other in compression. Under all loading combinations the distribution of horizontal forces between the individual footings will vary depending on the bracing arrangement of the tower.

2.2.4 Externally Guyed Supports

For all types of externally guyed supports, the guy anchors will be in uplift, while the mast foundations will be in compression with relatively small horizontal forces.

Typical support type - foundation load free body diagrams for the above support types are shown in Figure 2.2.

2.3 Geotechnical Data

2.3.1 Key Geotechnical Parameters

The key geotechnical parameters required for foundation design are summarised below:

- Ground water level
- Density (unit weight) of in-situ soil
- Density (unit weight) of backfill
- Strength of in-situ soil
- Strength of backfill
- Deformation modulus of in-situ soil and backfill
- Susceptibility of the soil to seismic deformation

The density and strength of the backfill will only be required for excavated foundations, e.g. pad and chimney and for directly embedded poles.

Besides the geotechnical parameters needed to evaluate foundation capacity, as presented above, the deformation parameters of the geological and backfill materials may also be needed if displacement criteria are being considered in the analysis and design.

2.3.2 Development of Engineering Properties

If existing geotechnical data is available for an existing foundation, the engineering properties of the soils can be used to evaluate foundation capacity and refurbishment requirements. If not, the engineering properties of the soils present at a foundation location can be estimated based on correlations with soil types, correlations with in-situ tests, and from laboratory test results.

a) Correlations with Soil Types

Correlations are available relating the engineering properties of soils to the soil type. Certain types of soils will have a certain range of values for a given engineering property. An estimate of the value of a given engineering property can be made knowing the soil type and the density

and/or consistency of the soil. If the density and/or consistency of the soil are not known, a conservative estimate of the engineering properties should be made.

b) Correlations with In-situ Tests

The engineering properties of the soils can be estimated based on the results of in-situ tests. The results of Standard Penetration Tests (SPT) provide soil samples for classification and determination of soil type. The SPT resistance (N) or blow count can be correlated with the density, strength, and deformation properties of soil. These correlations are generally more reliable for granular (non cohesive) soils than for cohesive soils. The Cone Penetration Test (CPT) provides data which can be correlated to the soil type, strength, density, and deformation properties. Correlations between the tip resistance and side friction and the soil type, density and strength are available. The CPT correlations are considered more reliable for cohesive soils than the SPT correlations for cohesive soils. This test may be difficult to conduct in coarse granular soils. The CPT does not provide a sample of the soil for classification or confirmation of the soil type. Pressuremeter (PMT) and Dilatometer (DMT) tests can be used to measure deformation properties of soil and rock materials.

Details of correlation between in-situ tests and engineering properties of the soil are given in CIRIA Report No.143 [CIRIA1995] for SPTs, by Meigh [1987] for CPTs and Mair and Wood [1987] for PMTs.

c) Laboratory Tests

Laboratory tests can provide direct measurements of the density, strength and deformation properties of the in-situ soils and backfills. Direct shear or Triaxial shear strength tests on soil samples obtained in the field can be conducted to determine the shear strength and deformation properties of the soil at specific sites. Measurement of specimen density will provide information on the unit weight of the existing soil layers. Details of laboratory tests on soil samples are given in national standards or codes of practice, e.g. ASTM D2487 [ASTM 1991], BS 1377 [BSI 1990].

2.4 Foundation Structural Design

The structural design of the foundation is not covered in this overview and reference should made to the appropriate national standard or code of practice, e.g. ACI 318 [ACI 1989], BS 8110 [BSI 1985], DIN 1045 [DIN 1988] etc. However, it should be checked whether the standard or code of practice is in ultimate limit state format or allowable (working) load format.

The design of the interconnection between the support and the foundations will depend on the proposed method of connection, i.e. stubs and cleats / shear connectors, anchor (holding down) bolts or direct embedment in the case of single pole supports. A review of International practice with regards to the design of stubs and cleats for lattice towers with separate foundations is contained in Cigré Electra paper No.131 [Cigre 1990], together with recommendations of 'Good Practice' especially regarding the distribution of load between the stub and cleats. Recommendations for the design of anchor bolts, stubs and cleats /shear connectors for lattice steel towers are given the ASCE Manual No.52 [ASCE 1988], while ASCE Manual No.72 [ASCE 1990] contains recommendations for steel transmission pole structures.

2.5 Foundation Geotechnical Design

An overview of the different geotechnical design models for separate and compact foundations is given in the following sections of the report. For details of the calibration of the theoretical geotechnical design model against the results of full scale foundation load test procedure, reference should be made to Section 5 of this Report.

3 SEPARATE FOUNDATIONS

3.1 General

Separate foundations may be defined as those specifically designed to withstand the loads transmitted by each leg of a support. Generally separate foundations are used for lattice towers or H-frame structures when the face width exceeds 3 m, provided the geotechnical conditions are suitable, or where adequate provision has been made to limit unwanted deformation in lattice towers due to differential settlement between adjacent foundations caused by subsurface mining activities. The connection between the leg of the support and the foundation is normally provided by stubs encased in the foundations or by the use of anchor bolts.

The following types of separate foundations are considered in this section of the report:

- Spread, e.g. concrete pad and chimney, pyramid and chimney and steel grillages;
- Drilled shafts (augered) with and without under-reams (belled);
- Piled foundations either single or multiple piles;
- Anchor foundations,
- H-frame support foundations.

Although anchor foundations have been identified as one of the principal categories of foundations, for convenience they have included within this section of the report.

The selection of the individual type of foundation will depend on design practice, geotechnical conditions, constructional and access constraints, financial and time budgets. For a comparison between the different types of separate foundations reference should be made to Table 3.1.

3.2 Applied Loadings

Separate foundations are principally loaded by vertical compression or uplift forces with small horizontal shear forces in the transverse and longitudinal direction. However, the actual loading will vary depending on the relative inclination of the vertical axis of the foundation with respect to that of the embedded stub or anchor bolts and for spread footings, on the relative orientation in plan of the base of the foundation to the axis of the support, i.e. whether it is set parallel to the face or parallel to the diagonal of the support.

Additional loading may be imposed on the foundations due to external sources, e.g. soil surcharges from uphill slopes, down drag on piles, frost heave etc. and should, where appropriate, be considered in the overall design of the foundation.

3.3 Spread Footing Foundations

3.3.1 General

Under the general classification of spread footings the following types of foundations have been reviewed:

- Concrete pad and chimney including stepped block foundations;
- Concrete pyramid and chimney including normal plain concrete pyramids, shallow reinforced pyramids and pyramids with extended pads;
- Steel grillage foundations.

Table 3.1 - Spread Foundation Types and Applications

Foundation Type	Applicable Soil	Advantages	Disadvantages
Spread - Pad & Chimney	All non cohesive & cohesive soils except very weak	Useable over a wide range of soil conditions, can be undercut or cast-in-situ (if permitted) which gives better uplift resistance.	Normally more expensive in concrete materials than pyramid foundations, Difficulty in obtaining good finish to the upper surface of the pad and reduced durability. If not undercut or cast directly against sides of excavation relies on backfill for uplift resistance.
Spread - Pyramid & Chimney	As above	Useable over a wide range of soil conditions, if used with an extended pad or shallow pyramid. Use of formwork improves durability of concrete.	Cost of formwork. Cannot be undercut. Relies on backfill for uplift resistance. A nominal pad (50 mm) must be provided below the pyramid to ensure no concrete segregation at edge of pyramid.
Spread - Steel Grillage	Dry non cohesive and cohesive soil.	Prefabricated, and light to transport to site in difficult terrain.	Requires suitable backfill material. Not readily adaptable in changing soil conditions. Range may be extended by use of imported backfill and encasing of grillage by concrete in wet conditions.
Drilled Shaft	Any type of soil or rock	Useable for all types of soil or rock, can be used for all types of support.	Initial cost of equipment and mobilization. Cost of testing. Drilled shaft foundations difficult to install in soils with frequent boulders.
Anchor	Any type of soil or rock	Both active and passive anchors can be used.	
Piled	Weak soil	Variety of different types available, adaptable to ground conditions present.	

a) Concrete Pad & Chimney and Stepped Block foundations

Concrete pad and chimney foundations (Figure 3.1a) in their simplest form comprise a cast-in-situ unreinforced pad with a reinforced concrete chimney. The pad may be undercut, depending on the both geotechnical conditions and safety considerations. The thickness of the pad and hence its rigidity is normally sufficient, not to require the application of the concept of the modulus of subgrade reaction.

The structural design of the foundation and hence the necessity for reinforcing the pad will depend on: the applied foundation loading, the geotechnical design model used, the applicable structural design code and the geotechnical parameters. For large pad foundations it is common practice to utilize a secondary upper pad to reduce the bending moment on the lower pad. Both pads in this instance should be effectively tied together.

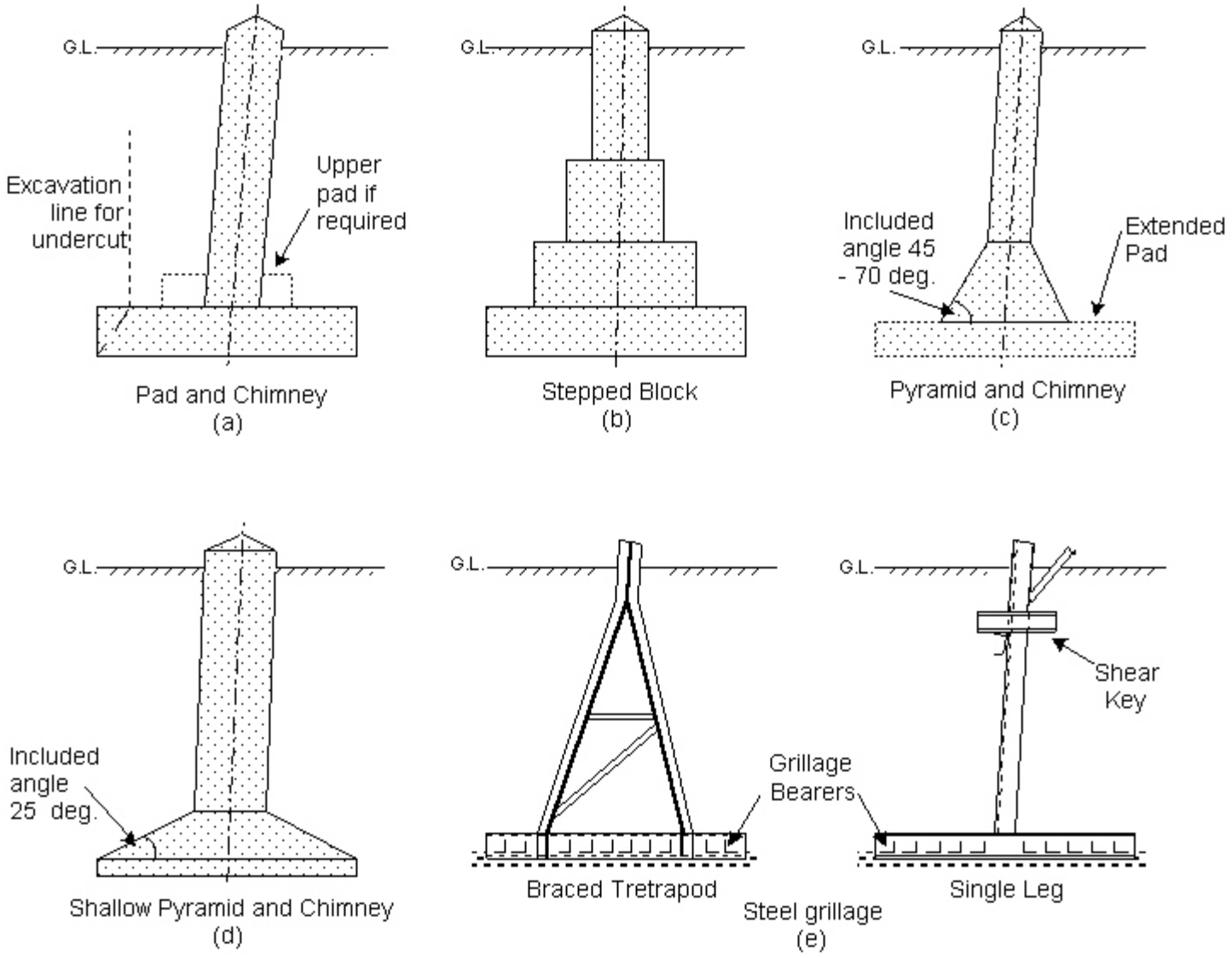
A common variation of the pad and chimney foundation is the stepped block foundation, (Figure 3.1b) whereby consecutively smaller blocks are cast on top of each other. The blocks may be either square or circular in cross-section. The factors previously outlined for the pad and chimney foundation, together with the constructional techniques used, will dictate the necessity or otherwise for reinforcing the blocks.

b) Concrete Pyramid & Chimney, Shallow Pyramid and Pyramid with extended pad.

Normal concrete pyramid and chimney foundations (Figure 3.1c) are cast-in-situ using prefabricated formwork and consequentially the foundation cannot be undercut. Provided the included angle between the base and the sides of the foundation is between 45 and 70 degrees, the pyramid may be designed using plain concrete.

When the base area of the pyramid becomes large and hence the volume of concrete excessive, there are two alternative solutions, i.e. the use of shallow pyramids or the provision of an extended pad beneath the normal pyramid. In the former solution (Figure 3.1d) the included angle of the pyramid will reduce to approximately 25 degrees and consequentially it is necessary to reinforce the concrete. For the latter solution (Figure 3.1c) a normal reinforced pad is cast beneath the pyramid, with the pad and pyramid effectively tied together.

Figure 3.1 - Spread Footings



c) Steel Grillage Foundations

Steel grillage foundations (Figure 3.1e) basically consist of steel angle section grillage members which are effectively connected to two steel angle or channel section bearers oriented normal to the grillage members. Depending on the fabrication process used, the grillage members are either bolted to, or slotted in the bearers. In the latter case it is common practice to 'spot' weld the grillage members to the bearers prior to installation.

The connection of the grillage to the support is by means of a single or multiple leg members, with a braced tetrapod being the most effective. If a single leg member is used, it is common practice to provide a steel or concrete shear key just below ground level to resist the horizontal shear force. Although the use of grillage foundations is normally restricted to dry non cohesive or cohesive soils, the range of the foundation can be increased by encasing the grillage

members in concrete especially in wet conditions, thereby effectively transforming the grillage into a pad foundation.

3.3.2 Foundation Geotechnical Design

The overview of geotechnical design methods for spread foundations has for been grouped into procedures related to the two principal applied loadings, i.e. compression and uplift. The design of the foundation must take account of the direction and orientation of the applied loading and must be designed to prevent excessive displacement or shear failure of the soil.

a) Compression Resistance

The applied compression load is resisted by the in-situ ground in bearing and a typical free body diagram is shown in Figure 3.2.

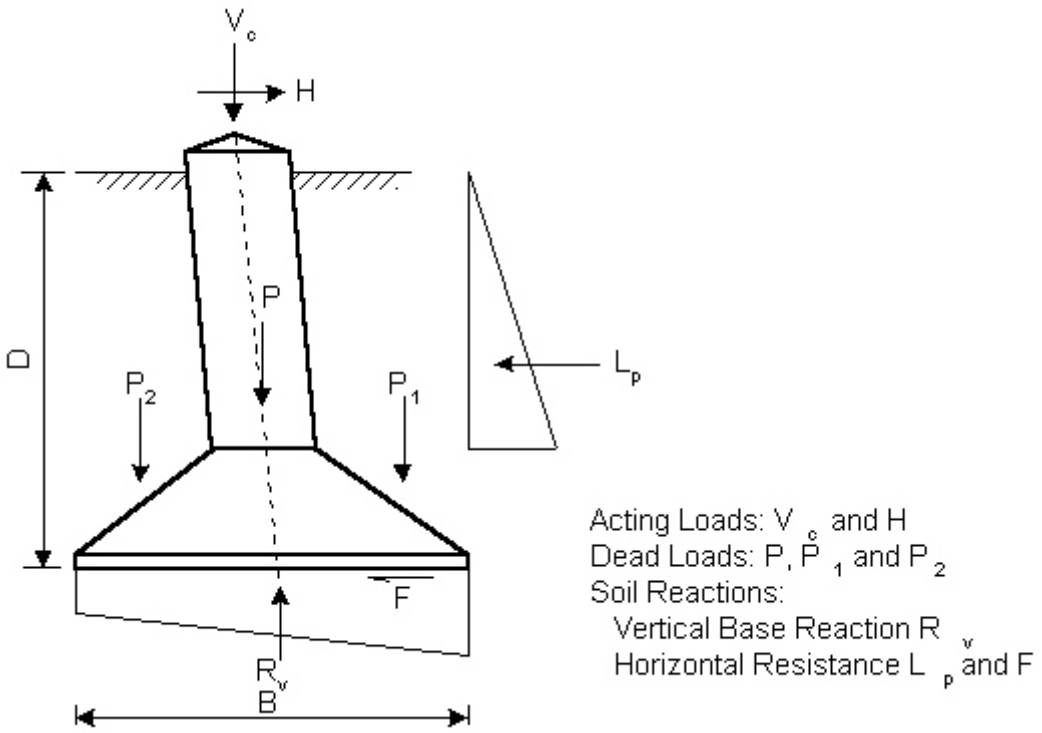


Figure 3.2 - Free Body Diagram - Spread Foundations (Compression)

Depending on the geotechnical design model used, the horizontal shear force will be resisted wholly or partly by the lateral resistance of the soil (L_p) and by the friction / adhesion at the base of the foundation (F). However, the resultant moment due to the applied load (H) will give rise to minor eccentricities in the bearing pressure. The concept shown in Figure 3.2 also applies to pad and chimney, stepped block and steel grillage foundations.

For steel grillage foundations the net area of the base, i.e. the area of the bearers in contact with the soil is normally used for the calculation of the bearing pressure. However, DIN VDE 0210 [DIN 1985] permits the use of the total area of the base provided the spacing between individual grillage members is less than 1/3rd of the width of the individual members.

The ultimate bearing pressure (shear failure) can be calculated using the bearing capacity equations derived by Terzaghi [1943], Meyerhof [1951, 1963], Hansen [1970] or Vesić [1973]. Bowles [1996] makes the following observations regarding the application of the different bearing capacity equations, where D is the depth of foundation and B is the base width:

Method	Application
Terzaghi	Very cohesive soils when D/B≤ 1 or for a quick estimate of q_{ult} to compare with other methods. Do not use for footings with moments and/or horizontal forces or tilted bases and/or sloping ground.
Hansen, Meyerhof, Vesić	Any situation that applies
Hansen, Vesić	When base is tilted, when footing is on a slope or when D/ B > 1.

The weight of the soil above the foundation (forces P_1 and P_2 in Figure 3.2) should only be included in calculations for the applied loading if gross bearing pressures and not net bearing pressures are calculated. For non cohesive (granular) soils the effective stress parameters should be used, whereas for cohesive soils the undrained case usually controls. For submerged foundations the submerged effective unit weight should be used. The "Trial - Use Guide for Transmission Structure Foundation Design" [ASCE/IEEE 1985] gives recommendations with respect to the reduction in the bearing capacity relative to the ground water level.

Calculation procedures for the determination of bearing capacities directly from in-situ test results are given by Bowles [1996] for the Standard Penetration Test based on the work of Terzaghi and Peck and Meyerhof, and for the Cone Penetration Test based on that by Schmertmann.

Presumptive allowable bearing pressures are contained in most national design standards or codes of practice, e.g. BS 8004 [BSI 1986] and DIN VDE 0210 [DIN 1985]. However, caution should be exercised when using these presumptive allowable bearing pressures since generally the assumed safety factor is not stated.

Settlement of a spread foundation can be divided between immediate, consolidation and secondary conditions. Immediate settlements are those that occur as soon as the load is applied to the soil mass and may exhibit significant values for non-saturated clays, silts, sands and gravels. Consolidation settlement is related only to the sustained load component in cohesive soils and may normally be ignored for suspension supports, but can be significant for angle support foundations. Secondary settlement occurs after consolidation settlement is complete and may contribute significantly to the total settlement in highly organic soils due to soil creep. Methods of calculating the settlement of spread footings are given in 'Trial - Use Guide' [IEEE 1985].

b) Uplift Resistance

Various design methods for determining the uplift resistance of spread foundations have been developed using empirical, semi-empirical and theoretical techniques. In a few cases, the design models were developed in conjunction with load tests on laboratory model and/or full scale foundations. The parameters considered have included the weight of the foundation, the weight of the soil contained within the assumed failure surface which vary from vertical extending from the base of the foundation to circular, the shear strength mobilized along the failure or slip surface and whether the foundation is undercut, cast against undisturbed soil or against formwork. However, in order to improve the reliability of a particular method of determining the uplift capacity or the displacement of spread foundations, the geotechnical design model should be calibrated against full-scale uplift load test data.

A typical free-body diagram for a spread foundation in uplift applicable to concrete pad, pyramid or block foundations and steel grillage foundations is shown in Figure 3.3.

A review of various methods of determining the uplift resistance is given in Table 3.2, together with the resisting forces and failure surfaces considered. Provided that the true leg slope is less than 1H : 5V it is normally satisfactory only to consider the vertical component of the leg load in uplift. The effect of the horizontal shear component of the applied loading (H) is usually

ignored in the calculation of the uplift capacity and none of the methods listed in Table 3.2 take account of the horizontal shear component.

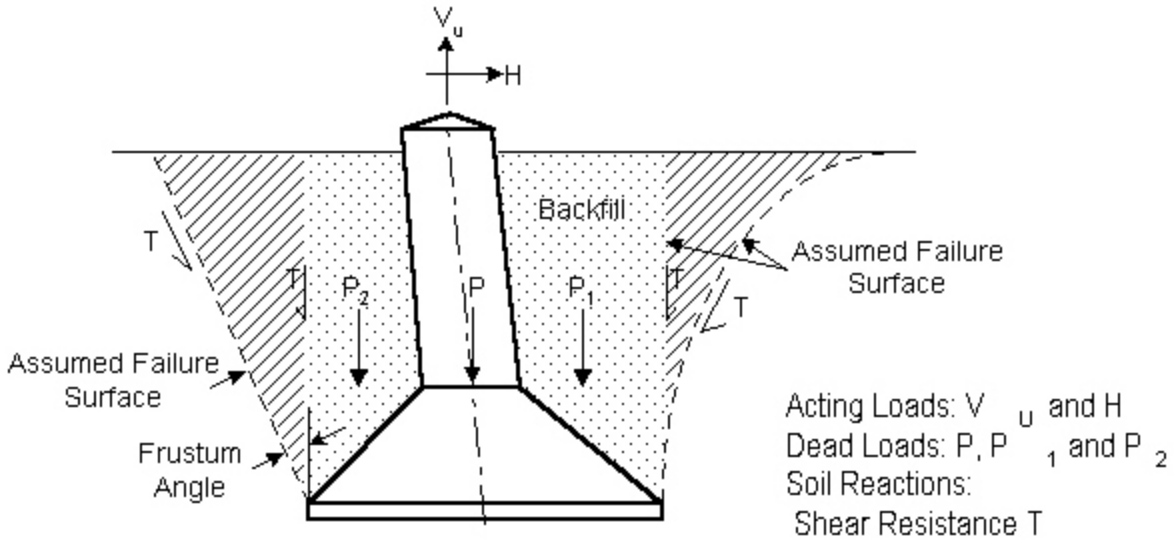


Figure 3.3 Free- Body Diagram - Spread Foundations (Uplift)

Note: The free body diagram is composite and illustrates the application of various failure surfaces and design models.

Table 3.2 - Methods for Determining Uplift Resistance for Spread Footings

Author or Method	Resisting Forces			Assumed Failure Surface	Ultimate or Working Resistance	Comments
	P	P_1 & P_2	T			
Biarez & Barraud (1968)	✔	✔	✔	Along inclined plane from base of foundation	Ultimate	Dependant upon soil type and depth of foundation
Cauzillo (1973)	✔	✔	✔	Logarithmic spiral	Ultimate	Dependant upon soil type and shape of foundation base
Flucker & Teng (1965)	✔	✔	N/A	Along edge of frustum	Ultimate	Frustum angle dependant upon soil properties.
Killer (1953)	✔	✔	✔	Along vertical plane from base of foundation to G.L.	Ultimate	Shear resistance dependant upon soil type
Meyerhof & Adams (1968)	✔	✔	✔	Along vertical plane from base of foundation	Ultimate	Dependant upon soil type and depth of foundation
Mors (1964)	✔	✔	✔	A simplified logarithmic spiral	Ultimate	Frustum based method
Vanner (1967)	✔	✔	✔	Complex frustum	Ultimate	Capacity dependant upon Base to Depth ratio.
VDE 0210 (1985)	✔	✔	N/A	Not quoted	Working	Frustum based method

Based on an extensive series of laboratory model tests in conjunction with a limited number of full-scale load tests, Biarez and Barraud [1968] proposed a series of formula for calculating the uplift resistance of pad and chimney and piled foundations cast directly against undisturbed non

cohesive and cohesive soil. Further calibration was also undertaken against full-scale load test data. The uplift resistance is related to the shear strength along an inclined surface rising from the base of the foundation, at a specified angle depending on the soil type. For foundations set below the critical depth localised shear failure is assumed to occur.

Another theory based on laboratory model tests was proposed by Cauzillo [1973], which relates the failure mechanism to the foundation shape. The failure is assumed to be along the path of a logarithmic spiral, again with a critical depth at which the plastic zone extends just to the ground surface from the junction between the pad and chimney. Calibration was also undertaken against full-scale load test data.

The classical frustum uplift capacity design method assumes a failure surface generated by an inverted frustum radiating from the base of the foundation. Various modifications have been proposed to take account of foundations cast directly against undisturbed soil or undercut. Flucker & Teng [1965] quotes different values for the frustum angle dependent upon the soil type, ground water level and whether the foundation is cast against undisturbed soil or cast in formwork.

Killer [1953] assumes that the failure takes place along a vertical shear plane extending from the base of the foundation to the ground surface. Different values are quoted for the shear resistance factor depending on the soil type.

Separate design models for shallow and deep spread foundations were developed by Meyerhof and Adams [1968], based on laboratory model tests and full-scale tests conducted in both sand and clay. For shallow foundations the failure surface is assumed to reach the ground level, while for deep foundations the compressibility and deformation of the soil mass above the foundation prevents the failure surface reaching the ground surface. Reasonable agreement was obtained between the theoretical value and full-scale load tests in sand. However, for clay it is necessary to distinguish between the short term (undrained) uplift capacity and the long term (drained) capacity.

An adaptation of the frustum theory was made by Mors [1964], who considers a failure surface equivalent to a logarithmic spiral, although simplified assumptions are made for calculation purposes. A review of other methods for calculating uplift capacity is also included in his paper.

One of the few methods developed solely on the results of full-scale load tests on pyramid foundations was proposed by Vanner [1967].The failure surface considered depending on the depth of the foundation, shallow foundations producing a complex frustum, while deep foundations failing due to local soil fracture. However, the tests were restricted to relatively small pyramid foundations with a base width of 0.85 m and depths varying from 1.37 m to 2.74 m in fine silty sand.

A further adaption of the frustum method is given in VDE 0210 [DIN 1985] for pad and chimney (stepped blocks) and steel grillage foundations. Different values are ascribed to the frustum angle dependent upon the soil type and whether the foundation is undercut, cast against undisturbed soil or against formwork.

Many of the theories have been only checked against a relatively small number of full-scale foundation test results, often all of similar size. Scale effects play an important part and so calibration or re-calibration is necessary. This applies to Killer which was based on small blocks, similarly Biarez and Barraud and Cauzillo only used a range of full-scale test foundation data varying from 11 kV up to 132 kV.

For steel grillage foundations it is normal practice to use the gross area of the foundation for the calculation of the uplift resistance, provided that the distance between the grillage members is

not greater than the width of the members. This is based on the assumption that the soil will arch between the bearers.

Seasonal variations in the water level and the affect on geotechnical parameters should be taken into consideration when calculating the uplift resistance, especially if the geotechnical investigation is undertaken at the end of the 'dry' season. Details of the both the variation in uplift and bearing capacities due to seasonal changes in ground conditions are given by Vanner [1982].

3.3.3 Minimum Geotechnical Data

Depending on the design method used, some or all of the following geotechnical parameters will be required:

- In-situ soil type and density, Backfill soil type and density
- Water table depth and potential variations in depth;
- In-situ soil and backfill shear strength parameters, i.e. effective cohesion and angle of internal friction and undrained shear strength;
- Compressibility indexes for the in-situ soil to estimate the amount and rate of consolidation settlement especially for poor soils.
- The susceptibility of the soil to seismic deformation in areas of high seismic loadings.

3.3.4 Influence of Construction Methods on Design

Construction techniques only have a small influence on the behaviour of spread footings in compression. However, they do have a major influence for spread footings in uplift, depending on the in-situ density of the backfill and whether the foundations are undercut, cast against undisturbed soil or cast in formwork.

Undercut foundations are considered to have improved uplift resistance and reduced movement under load compared with foundations cast in formwork, where foundations cast directly against undisturbed soil exhibit behaviour lying between these two extremes. Vanner [1982] quotes the results of full scale load tests undertaken by EdF to investigate the behaviour of foundations undercut into undisturbed soil. For stepped block foundations, EdF normal practice had been to undercut the lower pad by 100 mm; they then investigated undercutting the penultimate pad and providing an undercut of 400 mm. The uplift resistance increased by between 25 and 50%, with the movement decreasing by a similar amount. Based solely on a theoretical application of VDE 0120, Vanner quote's relative ratios for the uplift resistance of 1.4 : 1.2 : 1.0 for undercut : cast-in-situ : cast in formwork, respectively for a foundation 2.55 m square, 3.3 deep in stiff clay.

The density of the backfill has a major influence on the performance of foundations cast in formwork. The interaction between in-situ soil density, backfill density and foundation depth to width ratio and their influence on the uplift resistance of spread footings is described by Kulhawy et al [1985]. Based on a series of laboratory model tests which attempted to reproduce the effects of foundation installation methods, Kulhawy et al proposed the qualitative trends in uplift capacity shown in Table 3.3.

Table 3.3 - Qualitative Trends in Uplift Resistance (Kulhawyet al)

Increase in Parameter	Effect on Capacity	Conditions for Which change in Capacity is most Pronounced
Backfill Density	Increase	Deep (D/B = 3), Dense native Soil, Square
Native Soil Density	Moderate increase	Deep (D/B = 3), Dense Backfill, Square
Depth (D / B)	Substantial increase	Dense Native Soil and Backfill, Square
Length (L / B)	Little, if any, increase	- - - - - - - - - - - -

3.3.5. AdFreeze

The adfreeze phenomenon occurs in northern countries where a combination of extremely low temperatures and ground conditions give rise to frost heave problems sufficient to cause the collapse of a tower.

a) Permafrost

Permafrost may occur in the form of scattered “islands” ranging in size from a square metre to hectares or larger and in depth from less than 3 metres up to one hundred metres or more. There is no fixed pattern to the occurrence of permafrost and it is not unusual to find only part of the ground within a tower site affected by permafrost. The frozen soils might be silty clays containing ice inclusions as well as ice lenses.

The permafrost affected silty clay soils may undergo pronounced changes as the ground passes from the frozen to the thawed state. In the frozen state, the soils have high bearing capacities, upon thawing, however the cohesive forces between the soil particles (mainly the cementation forces of ice) change abruptly. Ice lenses and inclusions are transformed from relatively hard solids into a fluid which is easily displaced even under the action of the weight of the soil itself, resulting in a sudden change of the soil structure and a drastic reduction in strength.

The thawing ground will settle in a non-uniform manner in addition to the change in mechanical properties. The settlement is basically due to the deformation resulting from the soil consolidating under its own weight. Greater settlements may be anticipated under footings of structures whose design permits the thawing soil to squeeze out from beneath the footing. The differential ground settlements within a tower site due to consolidation of the thawing soil may vary between 150 mm and 600 to 900 mm under particularly adverse conditions as quoted by Lecomte and Meyere [1980] .

b) Frost Forces

The freezing of pore water in soils and the formation of ice lenses results in a swelling of the ground and any foundation members which either bear upon such ground or adhere to it through adfreezing forces, may be subjected to high stresses. Direct heave forces acting on the undersides of foundations can generally be minimized or overcome by setting the foundations at a depth below the normal frost penetration. This however, does not eliminate the heaving forces transmitted through adfreeze bond to the foundation members which extend through the active layer to the ground surface. The rate of frost penetration also influences the magnitude of the adfreeze forces.

The heave force is also related to the amount of movement that the structure can tolerate. If the structure is permitted to move in the direction of the ground heave, the forces are relieved, on the other hand if the structure members are restrained, the adfreeze forces may cause stress reversals at the connections and cause direct and bending stresses in the foundation members themselves. These stresses can be quite significant and may have serious consequences if they are not allowed for in the design.

Details of possible methods of alleviating both permafrost and frost forces are contained in Cigre Brochure No.141 [1999].

3.4 Drilled Shaft Foundations

3.4.1 General

A drilled shaft or augered foundation is essentially a cylindrical excavation formed by a power auger and subsequently filled with reinforced concrete. The shaft may be straight or the base may be enlarged by under-reaming or belling. Below 800 mm diameter the foundation would normally be defined as a bored pile.

For broad base lattice towers drilled shafts may be installed vertically or inclined along the hip slope of the leg as shown in Figure 3.4. The shaft shear load is greatly reduced for drilled shafts inclined along the tower leg hip slope. For H-frame supports the shaft would be installed vertically.

Under-reaming of the base can be undertaken in non-caving soils to increase the bearing and uplift capacity of the drilled shaft. The diameter of the under-ream may be up to three times the shaft diameter [ACI 1993]. Provided the under-ream slope is not less than 45 degrees to the horizontal, the shear strength of the unreinforced base concrete should be sufficient to resisting the shaft "punching" through the bell.

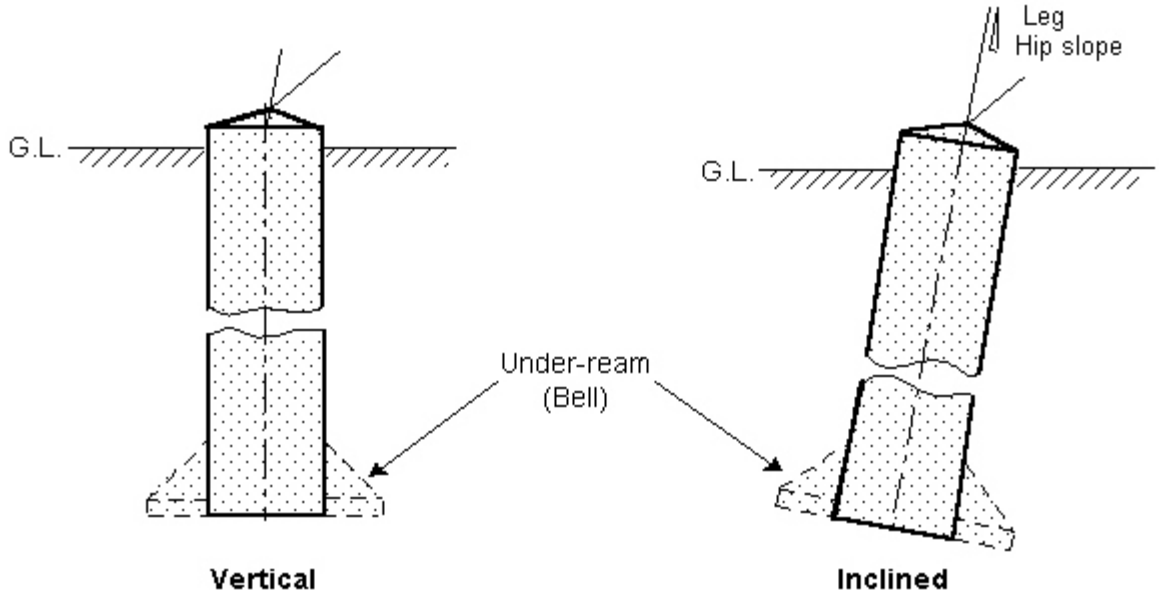


Figure 3.4 - Drilled Shaft Foundations

3.4.2 Foundation Geotechnical Design

The geotechnical design of drilled shaft foundations has been divided into the three principal load components: compression, uplift and horizontal shear, although obviously the shear load acts concurrently with other two design loads. The method of load super-position where each design loads are considered separately was justified by Downs and Chieurrzi [1966] for a ratio of lateral to uplift load of 1:10, based on an extensive series of full-scale foundation load tests. The ACI Report on drilled Piers [ACI 1993] also permits this approach to be adopted.

For details of the geotechnical design of separate drilled shaft foundations subject to an applied moment, i.e. for H-frame supports reference should be made to Section 4 of this report.

a) Compression Resistance

The ultimate compression resistance of a drilled shaft is composed of two components: the base resistance (end bearing) and the skin resistance (skin friction) developed by the shaft. A typical free body diagram for a drilled shaft under compression loading is shown in Figure 3.5.

Since the two resisting components are not fully mobilized at the same time, the skin friction reaching its ultimate value prior to that of the base, it is necessary to consider:

- the ultimate skin friction in conjunction with the end bearing at the transition point from ultimate to limit skin friction or,
- residual skin friction and the ultimate end bearing.

this is particularly true for cohesive soils. Further details of the load distribution of drilled shafts

are given by Reese and O'Neill [1969].

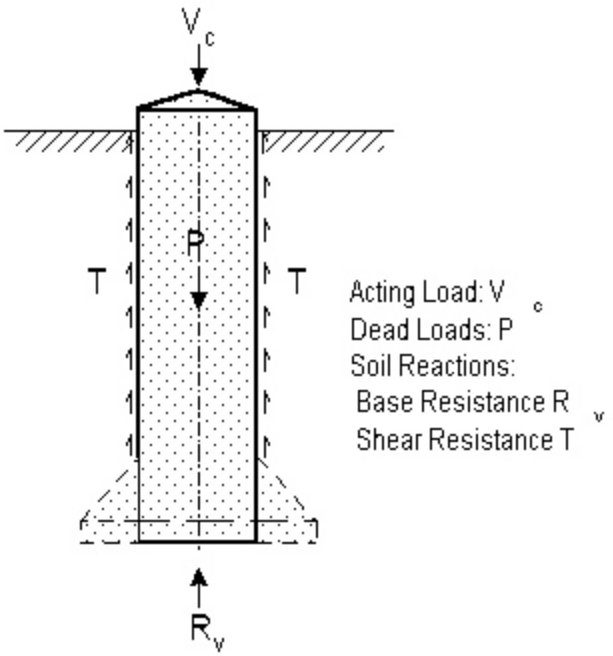


Figure 3.5 - Free Body Diagram - Drilled Shaft Foundation (Compression)

The end bearing resistance can be determined using the bearing capacity equations developed by Terzaghi [1943], Meyerhof [1951, 1963] or Hansen [1970].

Shaft resistance can be determined using either the 'Alpha' method [Tomlinson 1971], or the 'Beta' method [Burland 1973] for determination of the skin friction on the perimeter of the shaft, i.e. the "cylindrical shear" model. In the 'Alpha' method for cohesive soils the ultimate skin friction is related by an empirical correlation to the undrained shear strength of the soil, whereas for non cohesive soils it is a function of both the vertical effective stress and the angle of friction between the shaft and the soil. The 'Beta' method does not differentiate between soil types and the ultimate skin friction is a function of both the effective overburden pressure and the angle of friction between the shaft and the soil.

The effective length of the shaft for determining the skin friction is normally taken as less than the geometric length. According to Reese et al. [1976] the effective shaft length should exclude the top 1.5 m and for belled shafts the bell perimeter or, for straight shafts the bottom 1.5 m.

Since the ultimate shaft and base resistance are not developed simultaneously, it may for a particular soil/rock condition be logical to use either the base or the shaft resistance rather than a combination of the both to determine the overall resistance of the foundation. However, Reese et al. [1976] proposed a method based on the interaction between the two resistances to develop the overall foundation resistance.

For details of skin friction and base resistance of drilled shafts in rock, reference should be made to Horvath [1978] and Benmokrane [1994].

b) Uplift Resistance

There are no generally agreed methods for determining the ultimate uplift resistance of drilled shaft foundations, due to the difficulty in predicting the geometry of the failure surface. This point is further complicated depending on whether the shaft is straight or under-reamed. A typical free body diagram for a drilled shaft foundation in uplift is shown in Figure 3.6.

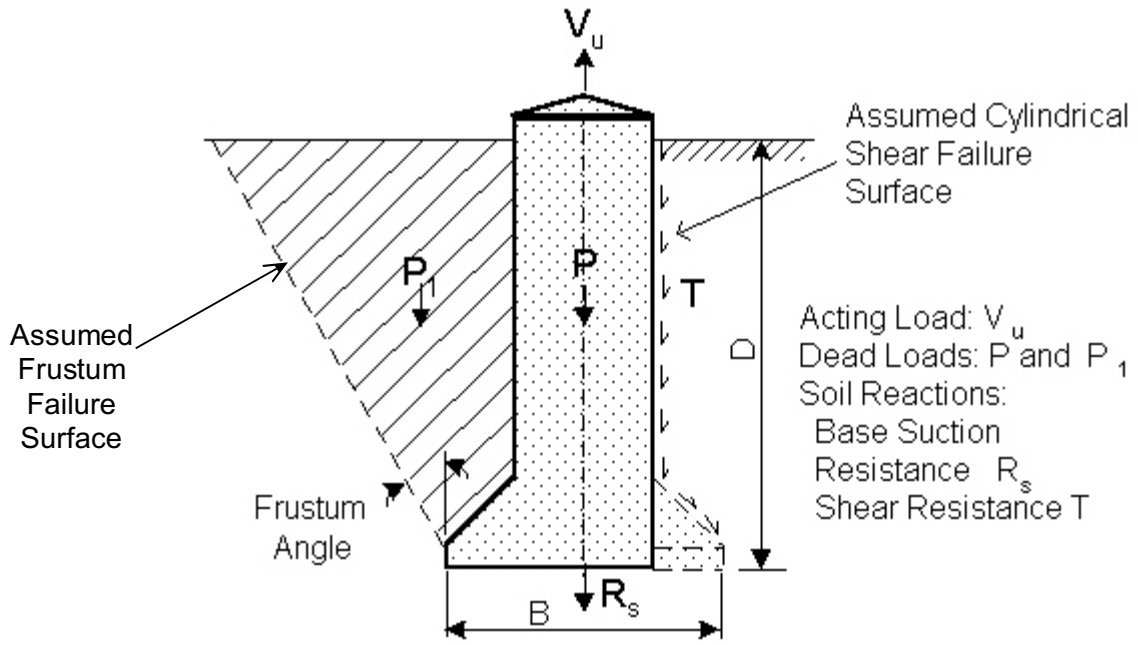


Figure 3.6 - Free Body Diagram - Drilled Shaft Foundation (Uplift)

Note: The free body diagram is composite and illustrates both the Frustum and Cylindrical Shear models. The suction resistance is only applicable for cohesive soils.

A review of various methods of determining the uplift resistance is given in Table 3.4 for straight and under-reamed drilled shaft foundations.

Table 3.4 - Methods for Determining Uplift Resistance of Drilled Shaft Foundations

Author or Method	Shaft Type	Soil Type	Resisting Forces			Assumed Failure Surface
			P	P_1	T	
Adams & Radhakrishna (1975)	Straight	Non Cohesive	✔	N/A	✔	Cylindrical
	Belled		✔	✔	✔	Frustum
	Straight	Cohesive	✔	N/A	✔	Cylindrical
	Belled		✔	N/A	✔	Cylindrical
CUFAD (1989)	Straight	Any	✔	N/A	✔	Cylindrical
	Belled		✔	N/A	✔	Cylindrical
Downs & Chieurrzi (1966)	Straight	Any	✔	N/A	✔	Cylindrical
	Belled	Non Cohesive	✔	✔	N/A	Frustum
Williams (1994)	Straight	Cohesive	✔	N/A	✔	Cylindrical
VDE 0210 (1985)	Belled	Any	✔	✔	N/A	Frustum

Adams and Radhakrishna [1975] design methods are effectively an extensions of the work previously undertaken by Meyerhorf and Adams [1968]. Based on both laboratory and full- scale uplift load tests the following approximate methods were developed for the determination of the uplift capacity of drilled shaft foundations:

- For straight shafts in non cohesive soil an expression based on the horizontal earth

pressure theory was developed, the uplift coefficient K_u was related to the D/B ratio (depth/ diameter). However, for deep belled shafts this method overestimated the uplift capacity, so an alternative solution based on the method developed by Meyerhof and Adams [1968] for spread footings was considered.

- A cylindrical shear model using the Alpha method for the determination of the skin friction was developed for straight shafts in cohesive soils, whereas for belled shafts a bearing capacity theory similar to that proposed by Meyerhof [1963] was used.

CUFAD [Kulhawy,1989] considers the uplift resistance to include the weight of the foundation, tip suction (R_s) and the side shear resistance. For deep drilled shafts (depth to diameter greater than 6) that side resistance is based on the cylindrical shear model. While for shallow shafts in addition to the cylindrical shear the potential for cone breakout is also considered.

Downs and Chieurrzi [1966] proposed two separate design models for the determination of the uplift capacity of drilled shafts based on an extensive series of full-scale uplift load tests. For under-reamed shafts in a non cohesive soil a model based on the weight of the soil contained within a frustum radiating from the base of the under-ream was proposed. The frustum angle was equal to the internal angle of friction of the soil. The cylindrical shear model was proposed for straight shafts in either non cohesive or cohesive soils.

The investigation undertaken by Williams et al. [1994] into the uplift capacity of straight drilled shaft foundations was a direct result of the failure of five 275 kV towers under high wind loading. Both analytical studies using cylindrical shear models (Alpha and Beta) and full-scale foundation load tests were undertaken to estimate the load-transfer along the shaft under uplift loading. The results from investigation indicated that the Beta method gave the best correlation with the test results.

A further adaption of the frustum method is given in VDE 0210 [DIN 1985] for the uplift resistance of under-reamed drilled shaft foundations. Different values are ascribed to the frustum angle dependent upon the soil type and the foundation depth to under-ream diameter ratio.

The long term capacity of drilled shafts under sustained loading was investigated by Adams and Radhakrishna [1975]. The results of their investigation were that in cohesive soils for straight shafts the long term capacity would be equal to 50% percent of the short term capacity, whereas for belled shafts this would vary between 40% and 100% depending on the depth to diameter ratio of the shaft. There was no apparent reduction for shafts in non cohesive soil. The Central Electricity Generating Board (CEGB) [1967] considered a reduction of 30% in the shaft resistance for straight shafts in cohesive soil for tension tower foundations. In addition the CEGB limited the suction resistance of the base to a maximum of 10% of the total uplift resistance of the shaft.

(e) Lateral Resistance

The lateral resistance of drilled shafts can be determined using any of the methods applicable to piled foundations, e.g. Broms, Hansen, Singh etc. For details of these methods reference should be made to Section 3.5.

3.4.3 Minimum Geotechnical Data

Depending on the design method used, some or all of the following geotechnical parameters will be required:

- In-situ Soil type and density and water table depth;
- Water table depth and potential variations in depth;
- In-situ Shear strength parameters, i.e. effective cohesion and angle of internal friction;

- Compressibility indexes for the estimation of the amount and rate of settlement.

3.4.4 Influence of Construction Methods on Design

The ground water level is perhaps the most important geotechnical factor influencing drilled shaft construction. Holes drilled or bored in high groundwater conditions may not stay open because of high seepage gradients generated by the high water table. This condition is especially critical when combined with the relief of confining pressure due to the boring process. These problems are most severe in loose, non cohesive soils, and fissured, jointed clays.

Ground water may also interfere with preparation of the bottom of the hole, cause difficulty in concreting, and may lead to damage of fresh concrete. If the water pressure exceeds the fluid pressure of the concrete, necking of the drilled shaft may occur. Groundwater flow can also leach cement out of the drilled shaft. The problems associated with ground water can sometimes be alleviated through the use of drilling muds, casing or by stabilizing the soil through de-watering.

Boring causes a release of confining pressure in the soil and thus may cause a reduction in the shear strength of the soil with a corresponding reduction in skin friction, particularly in fissured clays and clay shales. Soft and very soft clays and silts may squeeze into the drilled shaft, due to stress relief, before the concrete can be poured.

Soil resistance can vary due to seasonal moisture changes brought on by rain, drought, snow, floods, and frost action. The worst soil-climatic conditions that might reasonably be judged to influence the project should be used in the design. Expansive soils expand and contract with changing moisture content and can induce an upward load on the drilled shaft that can significantly change concrete stresses and side resistance. This may produce a net tension load on the drilled shaft even though the drilled shaft is supporting compressive loads. The geotechnical and structural design of the drilled shafts should accommodate this possibility. During the dry season, expansive soils dessicate near the surface and pull away from the sides of the drilled shaft thus eliminating the load transfer due to skin friction in this zone. In expansive soils it is important to determine the depth at which the moisture content is constant with time in order to calculate the effective skin friction.

Similar problems can occur due to the consolidation of uncompacted soils including fill material inducing negative skin friction loads and hence causing additional compressive loading on the drilled shaft. This can also result from de-watering or vertical surcharges.

Insufficient attention to the removal of disturbed material from the base of the shaft may result in unacceptable medium to long term settlement, especially for heavy angle or terminal supports.
Belled shafts require a soil that is sufficiently cohesive to stand without collapsing until the shaft is completed.

A review of the problems associated with construction of cast-in-place concrete piles (drilled shafts) is given in both the CIRIA Report PG2 [1977a] and the “Trial - Use Guide for Transmission Structure Foundation Design” [ASCE/IEEE 1985].

The influence and use of bentonite drilling mud in bored pile construction is reviewed in CIRIA Report PG3 [1977b]. The conclusions of the report where: in cohesive soils the behaviour of piles seems to be unaffected by the presence of bentonite, for non cohesive soils the shaft friction is not significantly different from normal expectations, in weak rock the level of friction currently permitted were similarly realised in practice. However, to ensure that satisfactory results are achieved, careful attention must be taken during the installation to ensure a minimum delay between boring and concreting and of the handling of the tremie pipe during concreting. These conclusions were further reinforced by the comparison of the test results from six full-

scale pile load tests in cohesive soil, three of the piles being drilled using bentonite [CIRIA,1978]. The results of the tests indicated there was no appreciable difference between in the ultimate capacities of the piles.

3.5 Pile Foundations

3.5.1 General

Pile foundations can either comprise a single pile or a group of piles connected at or just below ground level by a reinforced concrete cap, i.e. a piled foundation. This section of the report reviews the geotechnical design of both individual piles and pile groups.

Until recently piles were either classified as “driven” or “bored”, however, a preferable classification is that suggested by Weltman and Little [1977], who proposed the designation of “displacement” where the soil is moved radially as the pile enters the ground or “non-displacement” when little disturbance is caused to the ground as the pile is installed. The non-displacement piles are generally bored. Displacement piles can be driven using totally preformed sections from steel, pre-cast concrete or timber. Alternatively, where hollow steel or precast concrete sections are used these are normally subsequently filled with concrete, or for steel H-sections post grouted. Non-displacement piles are cast-in-situ using either concrete or grout, the pile section being formed by boring, drilling or driving a retrievable open-ended steel tube.

Specifically excluded from this section is a review of micro-piles, i.e. non-displacement piles less than 300 mm diameter which have been included in Section 3.6.

3.5.2 Foundation Geotechnical Design

The following factors will have a direct influence on the design capacity of an individual pile:

- Whether the foundation consists of an individual pile or a group of piles;
- Whether the pile(s) are vertical or inclined, i.e. raked;
- Relative pile spacing;
- The orientation of the pile group relative to the plan (horizontal) axis of the support;
- For a pile group the depth to the underside of the pile cap relative to the application point of the horizontal shear component of the loading from the support;
- Whether the pile caps are connected together by tie-beams;
- The geotechnical subsurface design parameters, including negative skin friction.
- The susceptibility of the soil to seismic deformation in areas of high seismic loadings.

Typical arrangements of piled footings are shown in Figure 3.6.

In this sub-section a similar procedure has been adopted as that used for drilled shafts, whereby the applied loading components, i.e. compression, uplift and shear, have been considered separately for individual piles. Subsequently the group effect due to the proximity of adjacent piles has also been considered.

In addition to this review where applicable reference should be made to the appropriate national piling standard or code of practice, e.g. ACI 543 [ACI,1974], AS 2159 [SAA,1978], DIN 4014 [DIN 1990] etc.

a) Compression Resistance

The ultimate compression resistance of a pile is composed of two components, the base resistance (end bearing) and the skin resistance (skin friction) developed on the surface area of the pile.

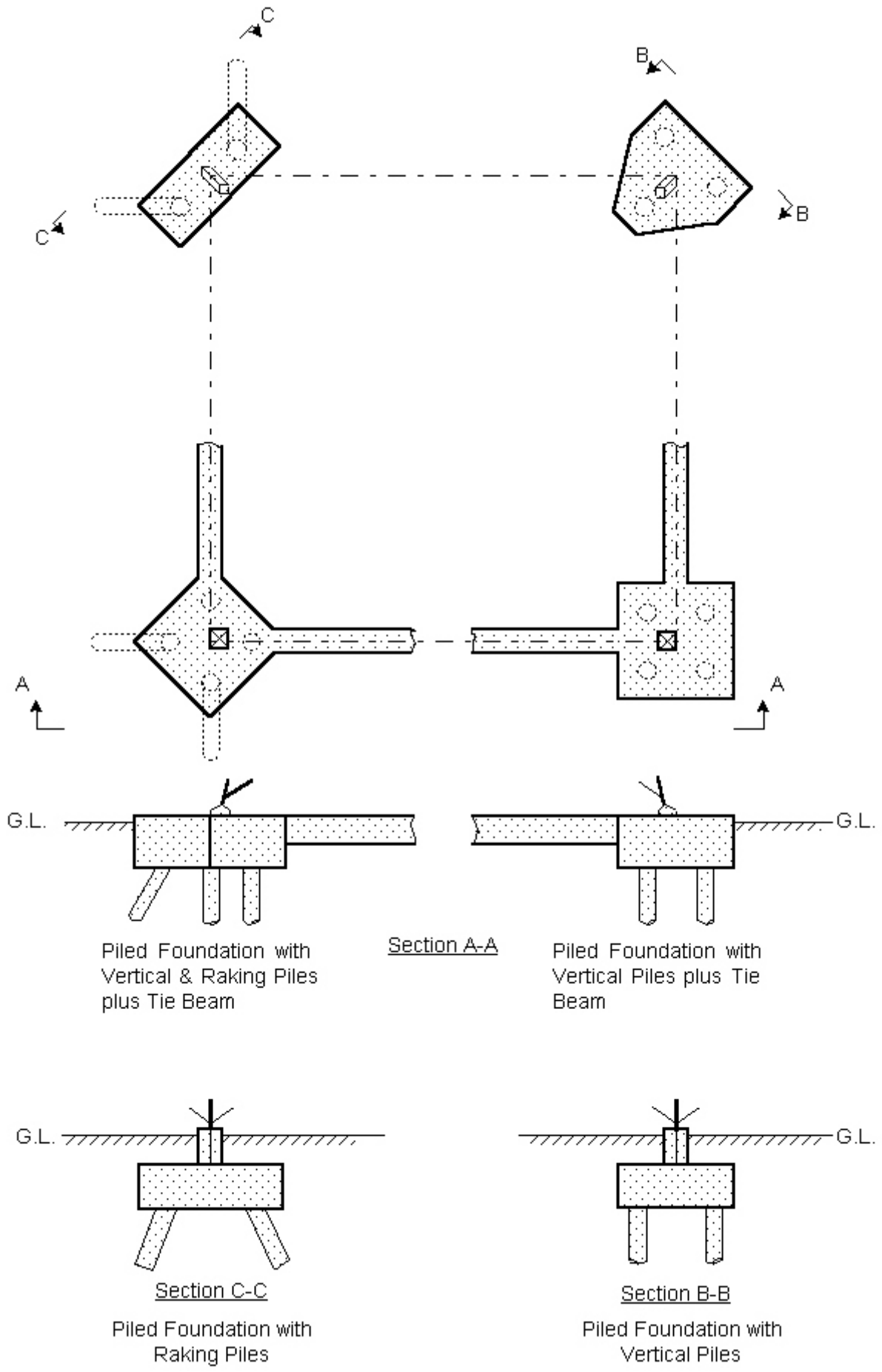


Figure 3.6 - Typical Piled Foundation Arrangements

The end bearing resistance can be determined using the bearing equations or bearing capacity factors developed by Vesić [1975], Berezantsev [1961], Janbu [1976] and Skempton [1951], in addition to those procedures outlined in Section 3.4.2.

In addition to the methods outlined in Section 3.4.2 the procedure proposed by Broms [1966] can also be considered for the determination of the pile skin friction.

Both the ultimate end bearing resistance and the ultimate skin friction can be estimated directly from the results of in-situ strength tests undertaken during the geotechnical investigation.

- Meyerhof [1976] proposed a relationship between the statistical average of the SPT 'N' values in a zone of 8B (pile diameter) above to 3B below the pile base and the ultimate base resistance.
- A similar approach was adopted by Fleming and Thorburn [1983] from the results of the CPT, where a weighted average of the cone resistance from 8B above to 2B below the base of the pile was considered.
- Relationships for estimating the ultimate skin friction have been developed by Meyerhof [1976], Shioi and Fukui [1982], and Thorburn and MacVicar [1971] with the SPT 'N' value and by Meyerhof [1976] and Thorburn and MacVicar [1971] based on CPT cone penetration resistance.
- Hobbs and Healy [1979] have related both the end bearing resistance and the skin friction to the STP 'N' value for driven displacement and non-displacement bored piles in chalk.

Dynamic pile resistance for displacement piles can also be estimated by the use of pile driving formulae. Where the dynamic resistance of the pile is related to the measured permanent displacement (or 'set') of the pile at each hammer blow. A review of the different pile driving formulae was undertaken by Whitaker [1975], who concluded that in some situations, pile capacities predicted by the different pile driving formulae may differ by a factor of 3. Wherever possible pile driving formulae should be correlated against the results of full-scale load tests for the specific pile, pile driving equipment and geotechnical conditions present.

b) Uplift Resistance

The ultimate uplift resistance of a pile can be determined using similar procedures to those outlined in Section 3.4.2 for drilled shafts. Further information on the design of steel piles subject to uplift and lateral forces is contained in the paper by Teng et al. [1969].

c) Lateral Resistance

Traditionally, piles have been raked to provide sufficient horizontal resistance to withstand lateral loads, such that the lateral force is resisted by the horizontal component of the axial pile capacity. Graphical methods being used to find the individual pile loads in a group and the resulting force polygon could only close if there were raked piles in the group. However, it is very conservative to ignore the resistance of a pile to withstand lateral loading, i.e. loading applied normal to the pile axis.

The use of raked piles in areas of major seismic loadings should be carefully assessed, since these can cause major punching loads on foundations during seismic events.

Gillson and Cliffe [1968] outlined the design procedure adopted by the C.E.G.B. for piled foundations, with particular reference to the use of stabilised, i.e. raked piles intersecting at the horizontal shear load application point, semi-stabilised piles where the line of action does not intersect at the shear application point and vertical piles. To simplify the analysis of the pile group the following design method was used: all piles are assumed to act equally, elastic deformation and pile/soil deflections are not significant in calculating pile loads, pile forces can be calculated using triangle of forces and the effect of axial and shear forces can be calculated

separately and added algebraically. Furthermore, under a working load condition of high wind and no ice, a balance is required between the applied uplift loading and the weight of the piled foundation. Where pile tests were undertaken the following acceptance criteria was adopted: for working loads the displacement must not exceed 6 mm and at 90% of the guaranteed pile ultimate uplift capacity the displacement must not exceed 25 mm.

The ultimate lateral resistance of a pile depends on the length of the pile and the stiffness of the pile relative to the stiffness of the soil in which the pile is embedded. As shown in Figure 3.7, short piles will displace as a rigid body, where as the lateral capacity of long piles will be limited to the ultimate moment capacity of the pile. Where piles are embedded in a pile cap, there are similar modes of failure, short piles will translate as a rigid body with the pile cap, while progressively longer piles will first form a plastic hinge at the level of the pile cap and then a second hinge further down the pile.

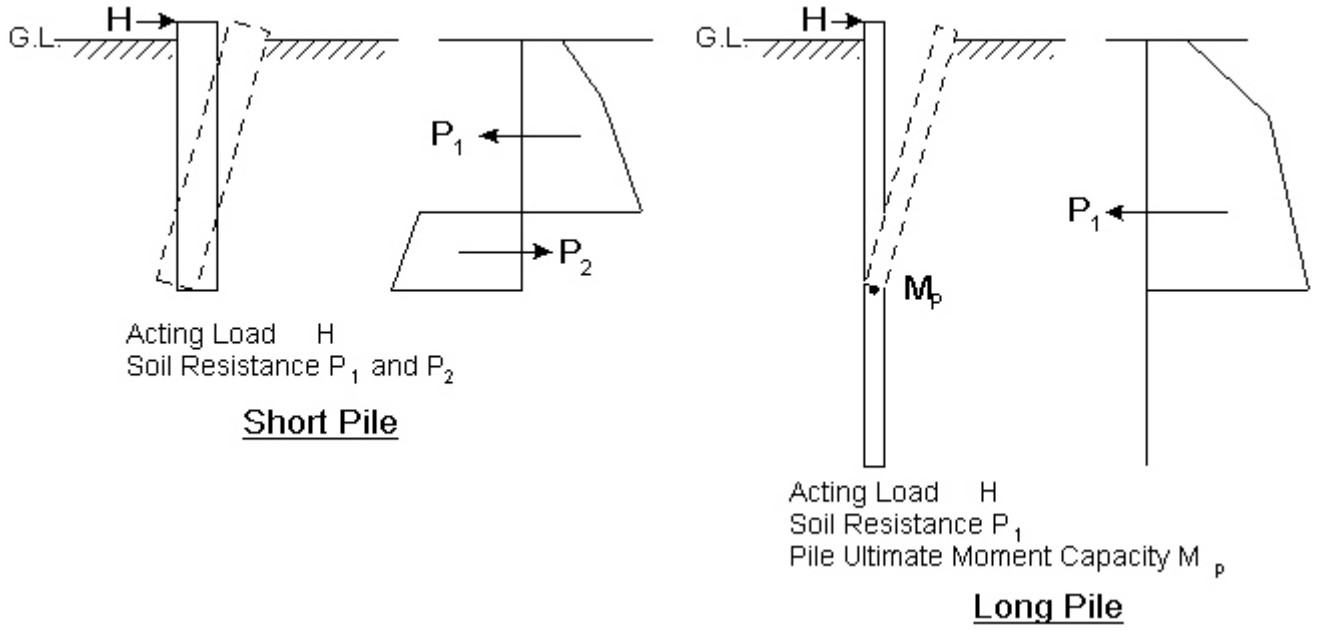


Figure 3.7 - Free Body Diagram - Pile Foundations (Lateral Load)

The ultimate lateral resistance for both long and short piles can be determined statically. For homogenous soils, either cohesive or non cohesive the method proposed by Broms [1964a & 1964b] can be used. Broms's simplification of discounting the lateral resistance of the top 1.5D of soil (D = pile diameter) may be conservative when applied to drilled shaft foundations in cohesive soils. For heterogeneous soils the method proposed by Hansen [1961] would be preferable.

An alternative approach based on the beam-on-elastic-subgrade theory using simplified assumptions regarding soil stress-strain behaviour was proposed by Singh et al. [1971] who developed interrelationships between the lateral resistance, displacement and maximum moment capacity of piles in cohesive and non cohesive soils as a function of the pile dimensions, type of loading and fixity of the head, as a series of design charts.

The beam-on-elastic subgrade problem can also be solved very effectively by using the Finite Element Method. Schmidt has derived algorithms for both single piles [1985a] and pile groups [1985b] in which any load assumptions, boundary conditions and variation of the subgrade modulus along the pile are catered for. The application of the Finite Element Method for solving the non-linear problem (beam-on-plastic subgrade) is also possible without any major difficulty.

The interaction between the soil and a rigid shaft can be idealized using the subgrade modulus.

Under the application of a lateral load, a rigid shaft rotates producing a ground line displacement, which can be uniquely related to the shaft displacement at that depth via the coefficient of subgrade reaction. Analytical solutions have been developed, giving the deflected shape of the pile and the shear force and bending moment distribution down the pile for the following situations:

- Matlock and Reese [1960]; applicable when the coefficient of subgrade reaction is assumed constant (cohesive soils) down the length of the pile;
- Reese and Matlock [1956]; applicable when the coefficient of subgrade reaction varies linearly with depth (non cohesive soils);
- Welch and Reese [1972],assumes a nonlinear coefficient of subgrade modulus model utilizing the *p-y* curves to describe the relationship between the lateral pressure *p* and the lateral displacement *y*. The *p-y* curve can be derived by measuring or calculating values of soil pressure and deflection from the results of instrumented field tests, assuming a correlation with the stress-strain properties measured in a laboratory, or assuming a characteristic shape for the pressure-deflection curve.

CIRIA Report 103 [1984] reviews the currently available methods for the analysis of laterally loaded piles and pile groups. The report highlights the limitations imposed by the available methods and provides guidance on the practical problems of assigning realistic values to the related soil parameters, with particular emphasis on the value of the soil stiffness.

d) Group Effect

For piles under compression loading the ASCE (Committee on Deep Foundations) [1984] suggests that for friction piles in non cohesive soils at the usual pile spacing of s = 2 to 3 pile diameters the group efficiency ≥ 1 (i.e. group resistance divided by sum of individual pile resistances). The reason given is that in non cohesive soils the pile displacement plus driving vibrations increase the soil density in the vicinity of the pile, which is further increased as other piles are driven nearby.

For friction piles in cohesive soils the block perimeter shear plus point bearing of the group in plan should be used as the group resistance, but in no case should the group resistance be considered greater than the single pile resistance times the number of piles in the group. The block bearing resistance should only be included if the cap is in contact with the ground.

There are at present no effective methods for determining the group action of piles subjected to either uplift or lateral loadings, partly because of the difficulty of mathematical modelling and partly due to the lack of full-scale load test data.

3.5.3 Minimum Geotechnical Data

Depending on the design method used, some or all of the following geotechnical parameters will be required:

- In-situ Soil type and density;
- Water table depth and potential variations in depth;
- In-situ Shear strength parameters, i.e. effective cohesion and angle of internal friction and undrained shear strength;
- Compressibility indexes for the estimation of the amount and rate of settlement.

3.5.4 Influence of Construction Methods on Design

The influences outlined in Section 3.4.4 for drilled shaft foundations are applicable to non-displacement piles. For displacement piles the following points should be considered.

During transportation and handling care should be taken to prevent deformation or cracking of the piles. Similarly initial alignment of the piles is important in reducing the subsequent

possibility of creating undesirable bending stresses in the pile. Driving heads to distribute hammer blows and cap blocks to prevent damage to the pile and hammer are necessary for impact driving. Overdriving of a pile may cause structural damage.

Pile driving may induce heave in saturated, fine-grained, slow draining soils, where the displaced soil increases the pore water pressure, so that the void ratio cannot rapidly change. As the pore pressure dissipates, the amount of heave may be reduced. Piles already driven in these materials may be uplifted, the problem being especially aggravated if the piles are closely spaced.

In granular soils a rearrangement of the soil structure from the driving vibrations may result in subsidence of adjacent areas. Previously driven piles may be pre-loaded to some extent by this phenomenon.

Similar changes in soil resistance due to variations in seasonal moisture content described in Section 3.4.4 for drilled shaft foundations are applicable to displacement piles.

A review of the problems associated with installation of displacement piles is given in CIRIA Report PG8 [1980].

3.6 Anchor Foundations

3.6.1 General

Anchors can be used to provide tension resistance for guys of any type of guyed support and to provide additional uplift resistance to spread footing type foundations in which case various types of anchors can be used.

a) Ground Anchors

Ground anchors consist of a steel tendon (either reinforcing steel, wire or steel cable) placed into a hole drilled into rock or soil which is subsequently filled with a cement or resin based grout usually under pressure (Figure 3.8a).

Micro-piles are small diameter cast-in-situ non displacement piles, with a diameter less than 300 mm.

Ground anchors can be grouped together in array and connected by a cap at or below ground level to form a spread footing anchor foundation (Figure 3.8b).

b) Block Anchors

Block anchors comprise a pad and chimney spread type footing whereby the concrete is cast directly against the face of the excavation possibly with an undercut at the base (Figure 3.8c).

c) Helical Screw Anchors

A helical screw anchor comprises a steel shaft which is screwed into the ground (Figure 3.8d). Helical screw anchors can be connected together at or above ground level by a steel grillage or concrete cap to form a helical screw anchor foundation (Figure 3.8e).

d) Deadman/Spread Anchors

Typically these anchors consist of a timber baulk, precast concrete block/pad or deformed steel plate installed in the ground by excavating a trench or augering a hole, placing the anchor against undisturbed soil and backfilling the excavation (Figure 3.8f). The anchor rod may be installed by cutting a narrow trench or drilling a small diameter hole.

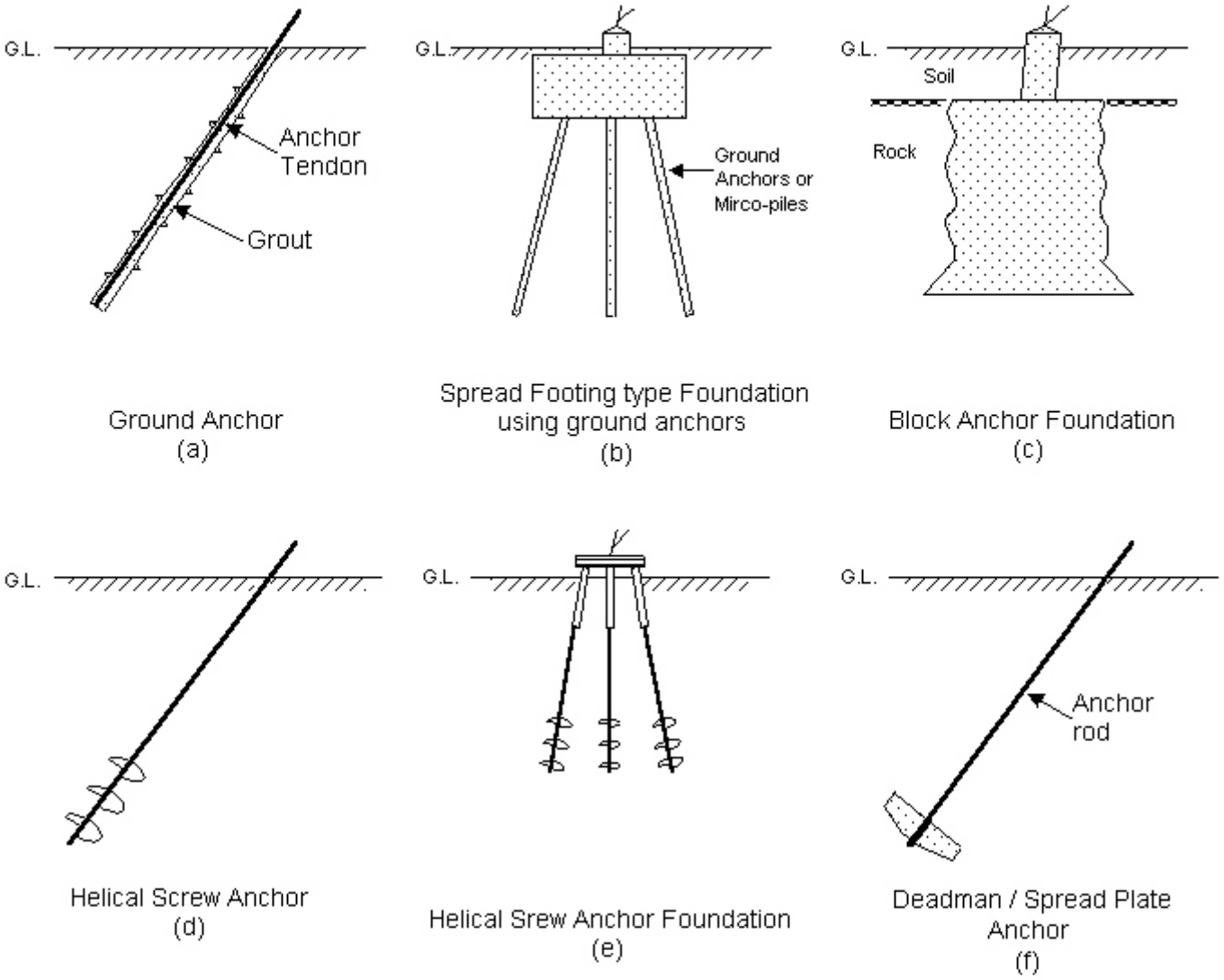


Figure 3.8 - Anchor Foundations

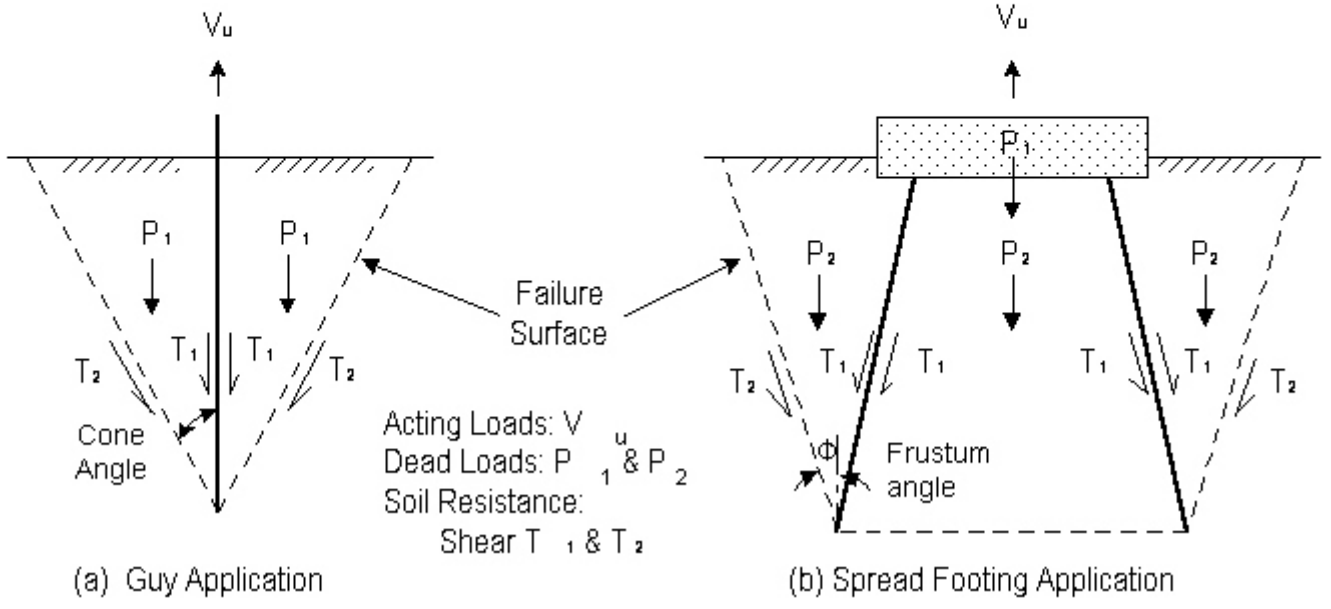


Figure 3.9 - Free Body Diagram - Ground Anchors (Uplift)

3.6.2 Foundation Geotechnical Design

Guy anchors and anchored foundations are principally designed to resist the uplift forces from guys or from a support leg respectively. They may be used singularly or combined in a group (array) connected by a cap.

Anchors are designed to resist tension loads; however, certain types of anchors also have compressive resistance, i.e. micro-piles, block anchors and helical screw anchors. Micro-piles and helical screw anchors would normally be arranged in a group under compressive loading.

Due to marked differences in the geotechnical design of each anchor type, the design of each type of anchor has been considered separately.

a) Ground Anchors

Ground anchors transfer the applied load from the tendon into the surrounding rock or soil by interfacial friction. The interfacial friction in soil may be considerable and can be increased by high pressure grouting. Ground anchors are normally designed to resist only axial tensile forces.

A free body diagram for ground anchors used as a guy foundation is shown in Figure 3.9a. Figure 3.9b shows a ground anchor utilized in a spread footing application.

For ground anchors in rock, the ultimate uplift resistance is determined by the strength of the following materials and critical interfaces:

- Rock mass;
- Grout - rock bond;
- Grout - tendon bond;
- Tensile strength of tendon or connection;
- Free and fixed tendon length.

Similar materials and critical interface strengths apply to ground anchors in soil except that the soil mass is usually not a critical parameter. The intensity of the grout pressure and hence the depth of penetration into the soil will have a marked influence on the effective diameter of the anchor for the determination of the uplift capacity.

Ground anchors may be active where the tendon is prestressed prior to the application of the guy load, or passive where no prestressing is applied.

Ismael et al. [1979] based on the full-scale load tests of passive ground anchors in rock, considered the failure mechanism for both single anchors and group anchors in relation to the ultimate resistance. For single anchors the uplift resistance was based on the weight of the rock cone radiating from the bottom of the anchor plus the shear resistance on the conical surface (Figure 3.9a), while for group anchors a frustum was considered projecting from the perimeter bars (Figure 3.9b). The frustum angle (ϕ) and minimum embedment being dependant upon the rock type and/or quality. Further research correlated the ultimate rock - grout bond to the unconfined compressive strength of the rock or grout, while that for the reinforcing rod tendon - grout bond was related to a function of the square root of the unconfined compressive strength of the grout.

A similar failure mechanism was assumed by Vanner et al. [1986] for passive anchors drilled in hard soil. The results of full-scale load tests indicated that there was no deterioration in the anchor resistance when subjected to 100 load cycles at a level equivalent to 50% of the ultimate resistance. Further tests confirmed this result when the anchor was subjected to 300 cycles equivalent to 78% of the yield stress of the tendon.

Littlejohn and Bruce [1977] published an extensive state of the art review of the design,

construction, stressing and testing of both active and passive ground anchors in rock. Subsequently this formed the basis of BS 8081 [1989] which contains extensive details on all aspects of ground anchor design, installation, testing and corrosion protection.

BS 8081 considers four basic types of anchorages ranging from gravity grouted straight shaft boreholes commonly employed in rock to high pressure multiple stage grouted systems used in fine non cohesive soils. Three testing regimes are proposed varying from proving tests to check the suitability of the design criteria, through suitability tests based on the actual production anchorage, to acceptance tests undertaken on all anchorages.

Spread anchored foundations are a combined foundation whereby the compressive load is transferred by the cap and the uplift load is resisted by the anchors. Depending on the inclination of the anchors, the lateral resistance will be provided by the passive resistance of the cap plus the horizontal component of the ground anchor resistance.

Micro-piles transfer the applied load from the steel reinforcement to the surrounding rock/soil by interfacial friction with minimal end bearing, and are capable of resisting both axial loading (tension and compression) plus lateral loads. Grouting of the micro-pile may vary from a single stage operation under gravity to multiple stage post-grouting under pressure. The intensity of the grout pressure and hence the depth of penetration into the soil will have a marked influence on the effective diameter of the micro-pile for the determination of the load carrying capacity. The uplift resistance may be determined using similar procedures as those for ground anchors, whilst for compressive resistance the Alpha method (reference Section 3.4.2) can be used. A review of the different types of micro-piles is contained in the ASCE Geotechnical Special Publication No.50 [1995].

b) Block Type Anchors

Block type anchors are usually installed in weak or fractured rock and hard soil (SPT 'N'>30), when it is uneconomic to use ground anchors. A free body diagram for a block foundation under uplift is shown in Figure 3.10.

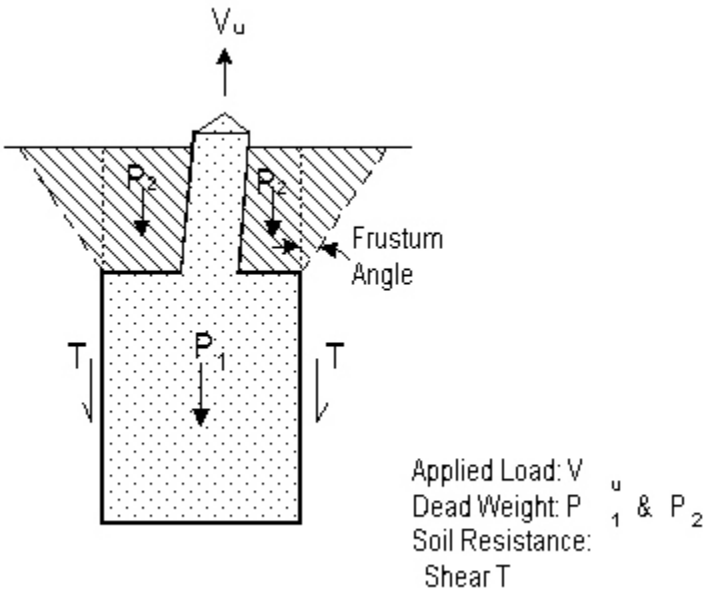


Figure 3.10 - Free Body Diagram - Block Anchor (Uplift)

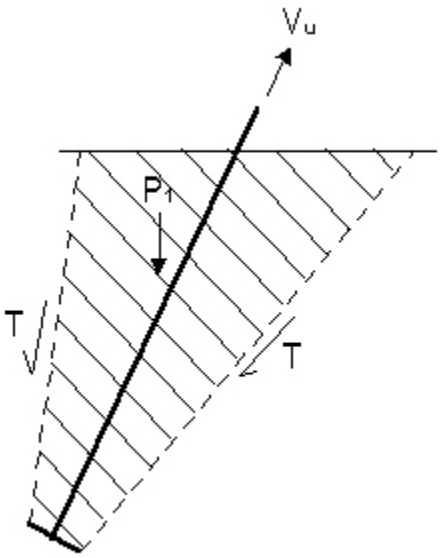


Figure 3.11 - Free Body Diagram - Deadman / Spread Plate Anchor (Uplift)

Compression resistance can be considered in a similar manner to that for spread footings

(reference Section 3.3.2), while uplift resistance is assumed to be provided by the shear resistance developed at the concrete-rock interface plus the weight of the foundation and the soil (if any) above the foundation. Normal practice is to assume a frustum type failure of the soil above the foundation. The Indian Central Board of Irrigation and Power (CBIP) [1996] quotes an ultimate rock - concrete bond stress of 145 kN/m² for fissured rock and 390 kN/m² for hard rock.

c) Helical Screw Anchors

Adams et al. [1976] considered that helical screw anchors could be treated as long slender belled footings with a high depth to width ratio. Correspondingly, a bearing capacity type equation could be used to determine the uplift resistance. The uplift coefficient (Terzaghi bearing capacity factor) was related to the relative density of the soil for non cohesive soils and to the shear strength for cohesive soils.

The results of a series of full scale load tests demonstrated that the compression and uplift resistance of the helical screw anchor were equal, when the depth to helix diameter (D/B) ratio is in excess of 6. The tests further showed that the shaft adhesion contributed a considerable proportion of the total foundation resistance. Tests run over extended time periods indicated that in cohesive soils the long term resistance should be taken as 70% of the short term resistance.

Expressions for multiple helices have been developed by Mitsch and Clemence [1985] for non cohesive soils and for both non cohesive and cohesive soils by Rodgers et al. [1979]. Rodgers' expression is similar to that proposed by Adams except for the inclusion of the resistance of the soil column above the top helices.

Although there are manufacturers' recommendations relating the installation torque to the anchor's resistance for different soil types, anchor depths and helices diameter, it is recommended that the computed capacities should be correlated against full scale load tests.

d) Deadman / Spread Anchors

The uplift resistance of deadman / spread anchors is based on the weight and strength of the soil above the anchor, plus the weight of the anchor. Similar methods to those reviewed in Section 3.3 may be used to determine the uplift resistance. A free body diagram for a block foundation under uplift is shown in Figure 3.11.

Martin [1974], based on a series of model tests and subsequently correlated by full-scale load tests, proposed three different failure mechanisms dependant upon the depth to width (D/B) ratio of the anchor plate. For shallow ($D/B \leq 3$) and medium depth ($3 < D/B < 6$) the anchor failed by movement of the soil above the anchor, whilst at greater depths ($D/B > 6$) localised failure of the soil occurred. The ultimate uplift resistance was related to a bearing capacity type equation, taking into account the dimensions of the plate, depth and inclination of the plate and the soil properties, such that the uplift resistance increases with depth and inclination, but also inversely proportional to the length to width ratio.

For the 500 kV Colstrip project in the USA, Zobel et al. [1976] undertook the full-scale load testing of different types of guy anchors, i.e. helical screw, dead man, augered bell and explosive anchors (whereby the bell is formed by the denotation of an explosive charge). Additional full-scale load tests were also undertaken to evaluate the performance of separate steel grillage and drilled shaft foundations for self supporting lattice towers. The criteria adopted for the evaluation of the ultimate loads with respect to the foundation displacement was 50 mm and 25 mm for self supporting suspension and angle towers respectively and 100 mm for guy anchors. The conclusions of the tests were:

- That helical screw and dead man type anchors were not acceptable due to inconsistent

test results and the overall cost of installation;
- Installation control is essential to provide the uplift resistance of explosive and augered bell anchors;
- Belled anchors provided the most economical solution;
- Drilled shaft foundations were preferable to steel grillage foundations for self supporting towers. However, if for environmental reasons grillage foundations are required a crushed rock backfill should be used.

3.6.3 Minimum Geotechnical Data

Depending on the design method used, some or all of the following geotechnical parameters will be required:

- Soil and/or rock density and water table depth;
- Soil Shear strength parameters, i.e. effective cohesion and angle of internal friction;
- Unconfined compressive strength of the rock;
- Rock Quality Designation.

3.6.4 Influence of Construction Methods on Design

The critical constructional features for ground anchors related to the design are drilling, hole stability and continuity of operation. Drilling necessarily disturbs the ground and the method should be chosen relative to the ground conditions to cause either the minimum disturbance, or the disturbance most beneficial to the anchorage capacity. Hole stability is critical and special care is required to ensure that the drilling or flushing method does not give rise to excessive loss of grout. Continuity of operations such that tendon installation and grouting are undertaken on the same day as drilling, since any delay can have serious consequences due to ground deterioration.

Helical screw anchors require constant rotational speed to ensure satisfactory down pressure and a constant anchor inclination. If spinning occurs, the disturbance to the soil will cause a reduction in the uplift capacity. Excessive downthrust can cause a torsional-buckling failure of the shaft.

The uplift resistance of deadman / spread anchors partly depends on the quality of the backfill and ensuring that the anchor bears against undisturbed soil.

3.7 H - Framed Support Foundations

3.7.1 General

Depending on the configuration of the internal X-bracing of the H-Framed support, the foundations will be subjected to either vertical compression or uplift forces with small horizontal shear forces, or overturning moments with relatively small horizontal, vertical and torsional forces. For the former any type of spread foundations previously considered would be suitable. Separate foundations for the latter condition, i.e. subjected to a moment loading are considered in this Section of the report.

3.7.2 Spread Footings

Spread footings subjected to biaxial overturning moments with small horizontal, vertical compression forces were considered by Teng and Manual [1977]. They proposed a design model for determining the maximum bearing pressure, when the load eccentricity is outside the middle one-third of the base (such that part of the foundation loses contact with the soil), based on the theory of subgrade reaction. A bearing pressure diagram for this condition of loading is shown in Figure 3.12. Stability of the foundation against overturning and sliding are also considered by the authors. In addition, procedures are given for determining both the ultimate bearing pressure and settlement of eccentrically loaded foundations.

3.7.3 Drilled Shaft Foundations
For details of drilled shaft foundations subjected to overturning moments reference should be made to Section 4.4 of this report.

3.7.4 Piled Foundations
Unless a single pile is considered, the use of a piled foundation will permit the resolution of the applied overturning moment into individual pile axial forces either compression or uplift. In which case the design methods reviewed in Section 3.5 are applicable, for single piles reference should be made to Section 4.5 of this report.

3.7.5 Anchor Foundations
Individual anchors are not in themselves capable of resisting an applied moment, therefore they must be arranged in an array interconnected by a concrete cap or steel grillage. Under these circumstances ground anchors (reference section 3.6.2 (b)), or helical screw anchor foundations (reference section 3.6.2(d)) could be used, with the applied moment resolved into individual anchor axial forces.

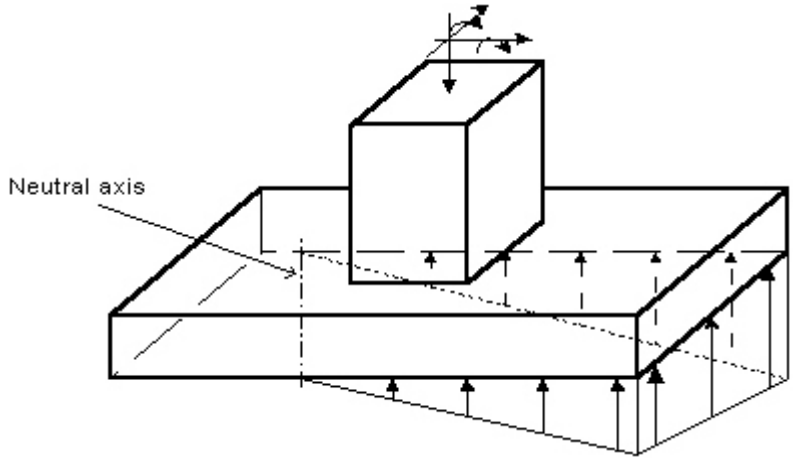


Figure 3.12 - Bearing pressure diagram for a Spread Footing subjected to Biaxial Bending

3.8 Influence of Sustained or Varying Loading on Foundations

3.8.1 Sustained Loading
The effect of long term sustained loading of foundations in cohesive soils was investigated by Meyerhof and Adams [1968], using laboratory spread foundations in clay. The results of the tests indicated that in stiff clay the long term capacity of the foundation is a small percentage of the short term capacity, whereas in soft clay the long term capacity is a much higher percentage of the short term capacity.

The conclusions of their investigation were that the drained or long-term uplift capacity in clay for spread foundations can be appreciably less than the undrained or short-term capacity at shallow depths. The reduction in time is due to the dissipation of negative pore water pressures which allow softening of the soil. The long-term capacity could be estimated using the design model for non cohesive soils. The reduction in capacity being most prevalent in stiff (over consolidated) clays at shallow depths and for each clay there is a certain depth at which the long term capacity will become greater than the short term capacity. However, of practical importance is the fact that only the long term sustained component of the applied loading need be considered.

Reference has previously been made in Section 3.4.2 to investigation by Adams and

Radhakrishna [1975], into the long term capacity of drilled shafts under sustained loading. The results of their investigation were that in cohesive soils for straight shafts the long term capacity would be equal to 50% percent of the short term capacity, whereas for belled shafts this would vary between 40% and 100% depending on the depth to diameter ratio of the shaft. There was no apparent reduction for shafts in non cohesive soil. .

3.8.2 Varying Loading

Cauzillo and Rendina [1980] investigated the effects of varying loading on laboratory model uplift foundations. Pad and chimney foundations were tested in both non cohesive and cohesive soils and pile foundations in cohesive soil. Two different types of varying loads were considered

- Transient loads due to dynamic effects on the line conductor breakage, ice drop, etc;
- Fluctuating load due to wind action on the line.

They discovered that the transient longitudinal loads were filtered by the tower and correspondingly the foundations were only affected by the residual load. This means that, because of the relative frequency responses of the tower and the foundation, the latter can resist a load applied suddenly and for a short duration better than the same load applied for a long time. Consequentially, the foundation only need be designed to resist the residual static loading (after conductor breakage) and not the dynamic shock load.

The fluctuating wind load applied as a succession of load peaks for one hour were transferred to the foundations through the tower, effectively unaltered and as such tended to produce a progressive deterioration of foundations in cohesive soil, but not in non cohesive soil.

Cochard [1979] investigated only fluctuating loads on reduced scale laboratory models, but his investigations included compression - compression, tension-tension and compression-tension cycles. On average the cycles lasted two minutes. The cycles considered were: compression > tension > no load representing light winds on suspension towers; compression > no load > tension representing medium winds on suspension towers and no load > compression > tension representing strong winds on tension towers, and it was the latter which proved destructive to straight drilled shaft foundations in particular. While the two former regimes tended to increase the ultimate resistance of the drilled shaft foundations, under the compression - tension loading, drilled shafts could pull out under cyclic loads, the geotechnical resistance being only some 40% of the ultimate static uplift resistance of the shaft in non cohesive soils. In cohesive soils, accelerated uplift of a shaft or of a flat plate occurs only when the tensile portion of the cyclic load exceeds about 80% of the static uplift resistance. Flat plates behaved as if statically loaded in a non cohesive soils. It was pointed out that a pad and chimney foundation could be considered intermediate between the shaft and the plate, and it would thus be less prone to degradation by a fluctuating load than a straight shaft.

The results of the investigation indicated that Cochard obtained considerable weakening of a pile in non cohesive soil under cyclic loading. However, it should be borne in mind that the tests were only undertaken on laboratory scale models.

Cauzillo and Rendina did not find foundations in sand were effectively weakened.

However, the joint conclusions of both sets of authors was that, until more was known of foundation behaviour, the maxima of known fluctuating loads applied to foundations should be restricted to 75% of static uplift for pad and chimney foundations, and to 60% of static maximum uplift capacity for straight drilled shafts. Care should however, be taken in the application of the authors' conclusions since the fluctuating loads considered may not be representative of real conditions experienced in the field.

For further information on the dynamic load effects on pile foundations reference should be

made to the Special Report [Cigre 1986] by Meyere for the 1986 Session of Cigre.

3.9 Calibration of the Design Model

For details of the calibration of the theoretical design model against the results of full scale load test results reference should be made to Section 5 of this report.

4 COMPACT FOUNDATIONS

4.1 General

Compact foundations may be defined as those specifically designed to resist the applied overturning moment from the support. Generally this type of foundation is used for single poles, for lattice towers with narrow base widths (less than 3 m) and for H-framed supports with a predominate moment loading. In addition, they may be used to replace separate footings for a wide base lattice towers when there is a specific need to limit the differential settlement between adjacent footings, i.e. raft foundations. The connection between the support and the foundation is normally provided by anchor bolts, by a section of the pole directly encased in the foundation, or by stubs encased in the foundation.

The following types of compact foundations are considered in this section of the report:

- Monoblock;
- Drilled shaft;
- Direct Embedded Pole;
- Raft;
- Piled.

The selection of the individual type of foundation will depend on design practice, geotechnical conditions, construction and access constraints, and financial and time budgets.

4.2 Applied Loading

Compact foundations are principally loaded by overturning moments with small vertical forces and horizontal shears in the transverse and longitudinal directions.

Additional loading may be imposed on the foundations due to external sources, e.g. soil surcharges from uphill slopes, down drag on piles, frost heave etc. and should, where appropriate, be considered in the overall design of the foundation.

4.3 Monoblock

4.3.1 General

Concrete monoblock foundations in their simplest form comprises a cast-in-situ reinforced concrete block. A typical one for a single pole is shown in Figure 4.1a. A monoblock foundation for a narrow base width tower is shown in Figure 4.1b. Alternatively they can be cast in-situ using prefabricated formwork or pre-cast, Figure 4.1c.

4.3.2 Foundation Geotechnical Design

The overview of geotechnical design methods for monoblock foundations has for convenience assumed that the principal resistance to the applied loading is provided principally by the lateral resistance of the soil. Where the principal resistance is provided by the base bearing resistance of the soil, the foundation has been classified as a ‘raft’ and reference should be made to Section 4.6.3 of this report.

The design of the foundation should take account of the orientation of the applied loading and should be designed to prevent excessive rotation or shear failure of the soil.

The applied loading is resisted primarily by the lateral resistance of the soil, but also by the ground in bearing and the soil lateral shear resistance on the side and base of the block. A

typical free body diagram is shown in Figure 4.2.

There has been limited interest in recent years in developing methods of determining either the ultimate resistance, or the resistance at a limiting angular rotation of the block for monoblock foundations.

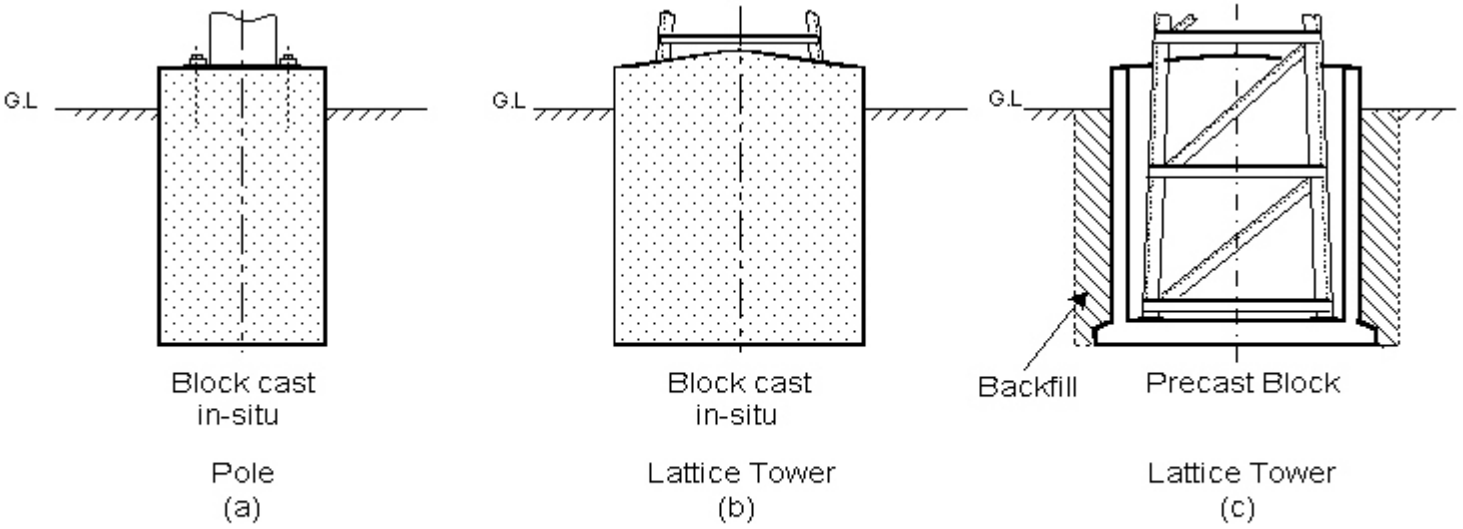


Figure 4.1 - Monoblock Foundations

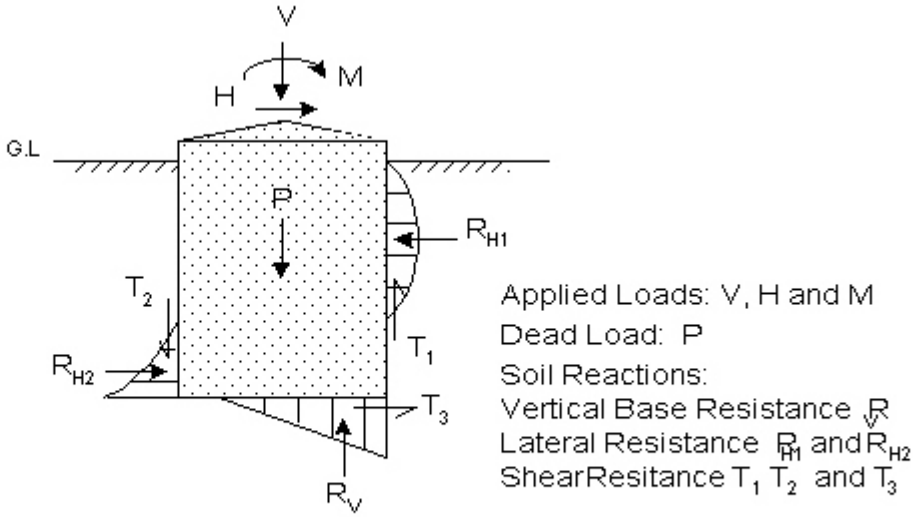


Figure 4.2 - Free Body Diagram - Monoblock Foundations

The Sulzberger [1945] design method is based on a limiting angular rotation of the block of 1%. The design model assumes that the horizontal and vertical resistances (R_H and R_V) are related to the subgrade modulus of the soil, while the effects of the shear resistances (T_1, T_2 and T_3) are ignored. The method is an iterative procedure and depends on the assumed point of rotation of the foundation.

Berio [1954], re-examined the work of Sulzberger and proposed two methods of determining the resistance of the block. By ignoring the effects of both the applied shear force (H) and the horizontal base resistance (T_3), Berio was able to develop a simplified expression which related the applied working load moment (M) (approximately equal to 50% of the ultimate resisting moment) directly to the dimensions of the block and the applied vertical load (V). Various

expressions were developed for different soil types. An exact solution taking into consideration all the applied forces and soil reactions was developed to determine the ultimate resistance of the foundation for non cohesive soils. To assist in the calculations Berio prepared a set of parametric graphs for the determination of the ultimate moment capacity for different values of the internal angle of friction of the soil. Both proposals were correlated against a limited number of full-scale tests previously undertaken by Sulzberger and other workers in this field.

4.3.3 Minimum Geotechnical Data

Depending on the design method used, some or all of the following geotechnical parameters will be required:

- In-situ Soil type and density, Backfill type and density;
- Water table depth and potential variations in depth;
- In-situ soil and backfill shear strength parameters, i.e. effective cohesion and angle of internal friction;

4.3.4 Influence of Construction Methods on Design

Construction techniques will have a major influence on the geotechnical design of precast monoblock foundations and to a lesser extent on those cast-in-situ. The degree of disturbance caused to the surrounding soil and the effectiveness of any backfill compaction will affect the lateral resistance provided by the soil.

4.4 Drilled Shafts

4.4.1 General

Drilled shafts used as compact foundations are similar to those described in Section 3.4 for separate foundations, except they are always installed vertically and are predominately loaded by high overturning moments.

4.4.2 Foundation Geotechnical Design

The geotechnical design of the foundation should take account of the orientation of the applied loading and should be designed to prevent excessive deflection and rotation and shear failure of the soil.

The applied loading is resisted primarily by the lateral resistance of the soil, in conjunction with the vertical side shear resistance, a base axial and shear resistance, and a typical free body diagram is shown in Figure 4.3.

Initially the determination of the ultimate geotechnical capacity of drilled shaft foundations subjected to high overturning moments, was based on the work undertaken by Broms for short rigid piles and Hansen and Reese for long flexible piles with high lateral shears but small overturning moments. For both piles and drilled shafts the principal resistance to the applied load is provided by the lateral resistance of the soil. However, for drilled shaft foundations additional resistance to the applied load is also provided by the vertical side shear, base shear and base axial resistance.

Cigre SC22 WG07 [1993] prepared an Electra paper comparing various methods for determining the ultimate geotechnical capacity of drilled shaft foundations subjected to high overturning moments. The three basic design models considered were:

- MFAD (Moment Foundation Analysis and Design) developed in the USA for EPRI by GAI Consultants Inc. MFAD is a four- spring nonlinear subgade modulus model,

- EdF's model is similar to MFAD in concept, except that it incorporates the results from

the pressure meter test for the determination of both the ultimate capacity and displacements;

- Dembicki and Odrobinski's model which is based on a limit equilibrium solution.

In addition to these three design models, a comparison with the three general purpose pile models previously referred to, i.e. Broms, Hasen and Reese was also made.

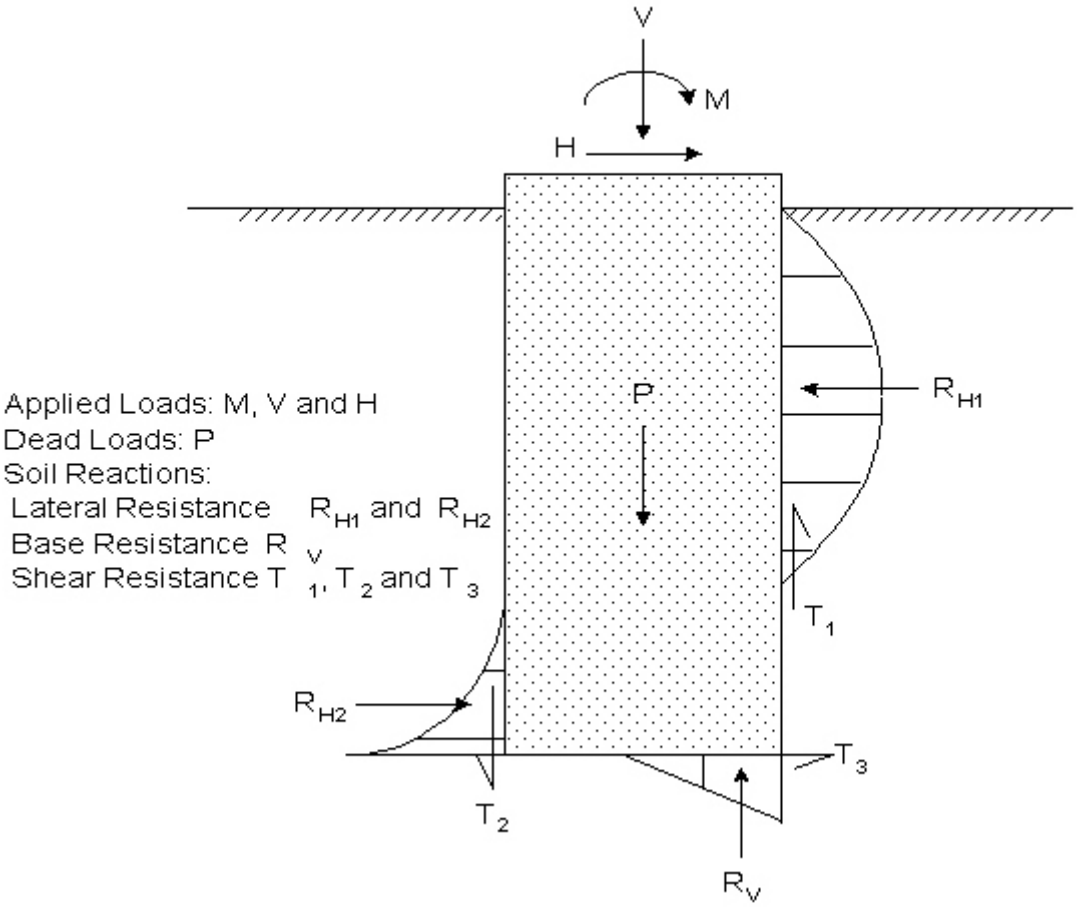


Figure 4.3 - Free Body Diagram - Drilled Shaft Foundations

Both MFAD and the EdF design model take into consideration all the resisting forces shown in the free body diagram. Whereas the Dembicki and Odrobinski, Broms, Hasen and Reese's model ignore the effects of base shear (T_3) and base axial resistance (R_v).

All of the design models were compared against the results of 14 well documented full-scale load drilled shaft tests. The results indicated that all the general purpose models under predicted the ultimate moment capacity when compared with the actual 2° rotation measured moment capacity. MFAD slightly over predicted the capacity, whereas both the EdF model and Dembicki and Odrobinski's model under predicted the capacity.

Details of six full-scale load tests undertaken on drilled shaft foundations partially or totally socketed into rock are described by DiGioia and Rojas-Gonzalez [1994].

Refinement of MFAD design model for rock socketed drilled shaft foundations based on a series of full-scale load tests is described by DiGioia et al. [1998a]. In order to establish the in-situ strength and modulus of deformation of the rock, a literature review of rock mass strength and deformation properties was conducted to obtain the most suitable and consistent

approach. The procedure selected was based on published correlation between the Rock Mass Rating (RMR_{76}) value and the in-situ rock strength and deformation properties of the rock.

4.4.3 Minimum Geotechnical Data

Depending on the design method used, some or all of the following geotechnical parameters will be required:

- In-situ Soil type and density;
- Water table depth and potential variations in depth;
- In-situ soil strength parameters, i.e. effective cohesion and angle of internal friction;
- In-situ soil modulus of deformation and limit pressure from pressure meter test.

4.4.4 Influence of Construction Methods on Design

Similar construction influences to those described for separate drilled shaft foundations (see Section 3.4.4) will also apply to drilled shaft compact foundations.

4.5 Direct Embedment

4.5.1 General

Originally used for the direct embedment of relatively lightly loaded wood poles, this type of foundation is also now used for steel and concrete poles subjected to high overturning moments.

4.5.2 Foundation Geotechnical Design

The design of directly embedded pole foundations is similar to that for drilled shaft foundations, except the effect of the backfill soil annulus surrounding the pole should be taken into account.

A paper by Bragg, DiGioia and Longo [1987], proposed an adaptation of the four-spring nonlinear subgrade modulus model MFAD previously developed for drilled shaft foundations. The major difference is the presence of the backfill annulus surrounding the perimeter of the directly embedded pole. The four-spring design model is modified by the introduction of two further springs modelling the load-deflection characteristics of the backfill material. The design model was evaluated against the results from 10 full-scale load tests on directly embedded single steel poles. Results indicated that the design model under predicted the ultimate geotechnical capacity of the foundation by 20% on average.

Stein [1988], proposed a design model for directly embedded wood pole foundations in cohesive soils, whereby the depth of embedded was related to the lateral coefficient of subgrade reaction of the soil when the pole rotation is less than 1 degree. The design model assumes that the skin friction and adhesion of the soil are negligible around the pole due to backfilling of the excavation, the compaction of the backfill round the pole does not contribute to the skin friction or the adhesion, the pole surface is smooth, consequentially the contributions from the vertical side shear (T_1 and T_2), base shear (T_3) and base axial resistance (R_v) are ignored. Stein also assumed that the ultimate lateral pressure diagram varies directly with depth. A further simplification was included in the model by directly relating the ultimate lateral coefficient of subgrade reaction to the ultimate cohesion of the in-situ soil. No correlation was undertaken against full-scale load tests.

As part of the full-scale load test series previously described for drilled shaft foundations partially or totally socketed into rock [DiGioia and Rojas-Gonzalez 1994], eight full-scale tests on directly embedded steel poles were also undertaken at the same time. The backfill material used at the different test sites varied from a native gravel mix - crushed stone - grouted gravel and concrete. The combined results of both tests series lead to the preparation of a provisional

design guideline, which defines the minimum foundation embedment depending on the thickness of the soil overburden and the ratio of rocket socket depth to the foundation diameter. Haldar et al. [1997] undertook an extensive experimental and analytical investigation into directly embedded steel pole foundations in non cohesive soil. The scope of the experimental investigation included fifty laboratory model tests, four centrifuge model pole tests and eight full-scale load tests. The effect of both different backfill materials, i.e. sand, crushed stone, native soil and flowable material and the addition of a base plate to the pole were considered. The conclusions of the research were that the design models of Hansen [1961] and Petrasovits and Award [1972],which models only a single material surrounding the pole predicted the ultimate capacity of the embedded pole foundations reasonably well, assuming the material surrounding the pole had the properties of the backfill material and not the native soil. Both foundation moment-rotation behaviour and the ultimate capacity were also predicted reasonably well by the design model of Bushan et al.[1981]. A modification of this method taking into consideration the backfill material, together with a design model developed by the authors based on the earth pressure theory for long retaining walls are presented in an implementation guide.

4.5.3 Minimum Geotechnical Data

Depending on the design method used, some or all of the following geotechnical parameters will be required:

- In-situ Soil type and density, Backfill soil type and density
- Water table depth and potential variations in depth;
- In-situ soil and backfill shear strength parameters, i.e. effective cohesion and angle of internal friction;
- In-situ soil Modulus of Deformation.

4.5.4 Influence of Construction Methods on Design

The type of backfill material, the degree of compaction of the backfill and hence its strength and deformation properties relative to the surrounding native soil and width of the backfill annulus will all have a major influence on the performance of a direct embedment foundation. In addition, the influences described for separate drilled shaft foundations (see Section 3.4.4) will also apply to direct embedment foundations.

Details of the research undertaken on the relationship between the strength and deformation properties of granular backfill materials and their influence on direct embedded pole foundation behaviour is given by DiGioia et al. [1998b]. The conclusions of their research was that compacted well-graded granular backfills had the greatest stiffness and strength. In addition the degree of compaction had a greater influence on stiffness than strength.

4.6 Raft

4.6.1 General

Under the general classification of raft foundations, the following types of foundations have been considered:

- Concrete raft foundations;
- Steel grillage raft foundations.

a) Concrete Raft Foundations

Concrete raft foundations in their simplest form comprise a cast-in-situ reinforced concrete pad at or below ground level as shown in Figure 4.4a. Normally, the thickness of the pad and hence its rigidity is sufficient such that the soil pressure can be assumed to be linear. Where the flexural rigidity of the raft is taken into account, the design could be based on the concept of

a beam on an elastic foundation.

b) Steel Grillage Raft Foundations

Steel grillage raft foundations as shown in Figure 4.4b, are normally only used for narrow base lattice steel towers, and basically consist of steel angle section grillage members which are connected to two steel angle or channel section bearers orientated normal to the grillage members. Depending on the fabrication process used, the grillage members are either bolted to, or slotted in the bearers. In the latter case it is common practice to 'spot' weld the grillage members to the bearers prior to installation.

The connection of the grillage to the support is by means of an extension of the tower body.

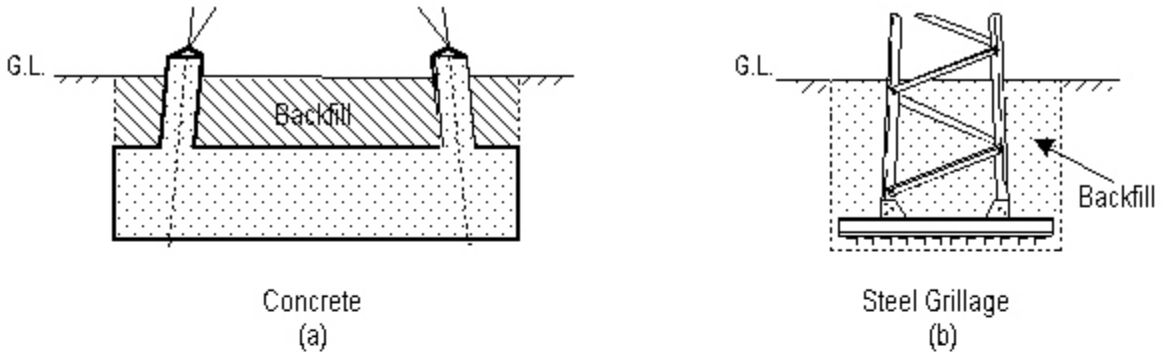


Figure 4. 4 - Raft Foundations

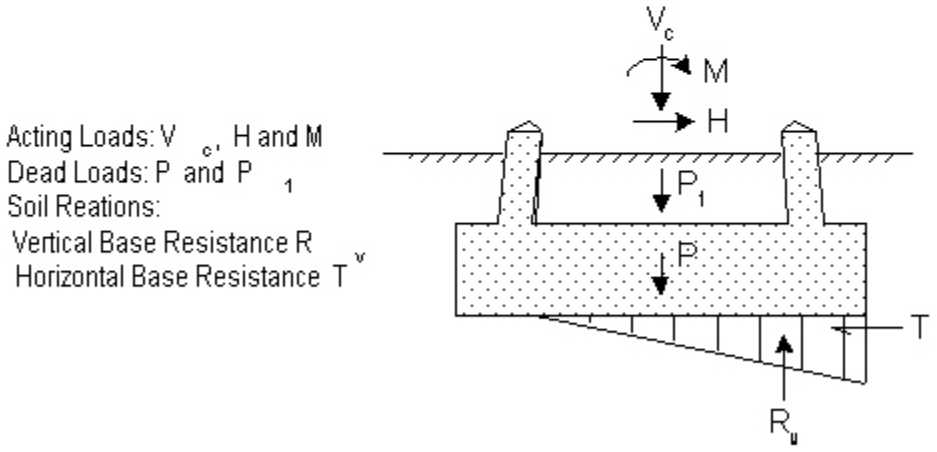


Figure 4.5 - Free Body Diagram - Raft Foundations

4.6.2 Foundation Geotechnical Design

The predominate resistance to the applied loading is provided by the base bearing resistance of the soil. Lateral resistance is generally neglected. The stability against overturning is ensured by the self weight of the raft foundation and the vertical surcharge due to any soil above the foundation. A free body diagram for a raft foundation is shown in Figure 4.5.

Where raft foundations are subjected to biaxial bending and the load eccentricity is outside the middle one-third of the base, part of the foundation loses contact with the soil. Under these circumstances the location of the neutral axis of the foundation cannot be found directly. Either an iterative process is undertaken or use is made of published design charts ASCE/IEEE [1985] and U.S. Dept of Interior, Bureau of Reclamation [1965]. Similarly the maximum design bearing pressure can be determined using the same charts.

The ultimate bearing pressure can be calculated using the bearing capacity equations derived by Meyerhof [1951], Hansen [1970] or Vesić [1973].

Zolezzi [1986] made a comparative study of the design of raft foundations for wide base lattice towers using the methods of Sulzeberger, Burklin and Dembicki. The results of the comparison indicated that there was no appreciable difference in the final foundation size (volume of concrete) for each of the alternative design methods. A further comparison was also made with a raft foundation designed using the flexible plate (mat foundation) method. The numerical solution of the flexible plate was obtained using the grillage beam analogy. The ensuing foundation giving the lowest volume of concrete, approximately 47% compared to the more traditional methods of design.

4.6.3 Minimum Geotechnical Data

Depending on the design method used, some or all of the following geotechnical parameters will be required:

- In-situ Soil type and density, Backfill soil type and density;
- Water table depth and potential variations in depth;
- In-situ soil and backfill shear strength parameters, i.e. effective cohesion and angle of internal friction;

4.6.4 Influence of Construction Methods on Design

The only construction influence that affect the design of raft foundations is ensuring that there is the minimum disturbance to the soil at the setting depth of the raft.

4.7 Piles

The use of a single pile as a compact foundation is unusual unless the applied loading is relatively light. The normal arrangement is for a group of piles connected at or just below ground level by a reinforced concrete cap (see Figure 4.6), similar to the arrangement described in Section 3.5.

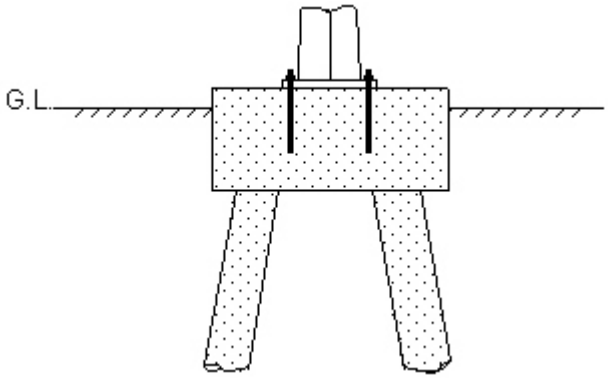


Figure 4.6 - Piled foundation

Using this type of foundation, the applied overturning moment can be resolved into axial compression and uplift loading on the pile. A free body diagram of a piled foundation is shown in Figure 4.7. Correspondingly, reference should be made to Sections 3.5.2, 3.5.3 and 3.5.4 for details of the Geotechnical Design, Minimum Geotechnical Data and Influence of Construction Methods on Design respectively.

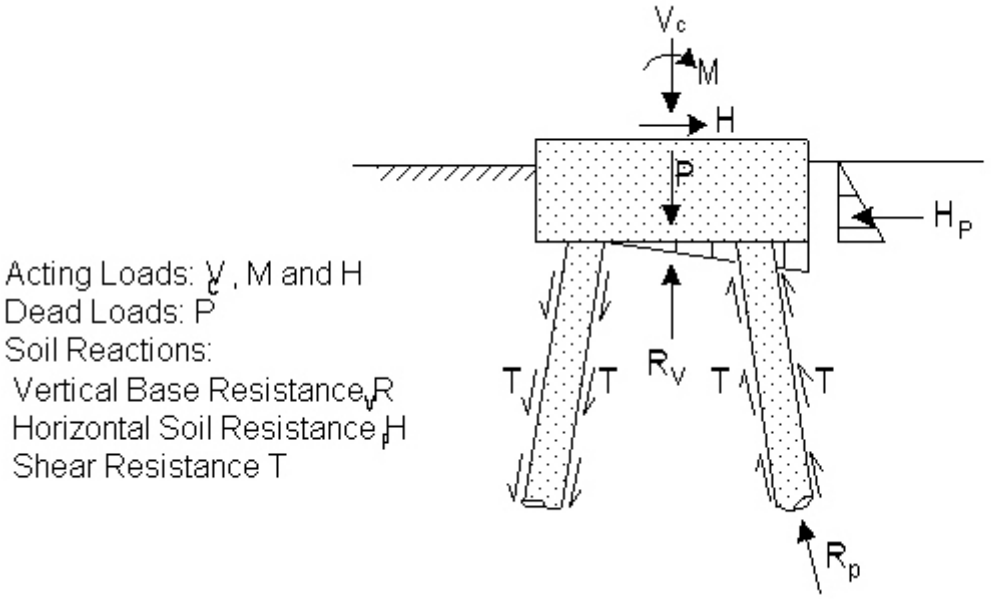


Figure 4.7 - Free Body Diagram - Piled Foundation

4.8 Calibration of the Design Model

For details of the calibration of the theoretical design model against the results of full scale load test results reference should be made to Section 5 of this report.

5 GEOTECHNICAL DESIGN

5.1 General

An overview of various design methods for determining the nominal ultimate foundation design strength (R_n, R_c) for different types of foundations has been given in the previous sections of this report. As shown in Figure 1.1, if a Deterministic Design Approach is to be used, the designer must select a single nominal safety factor to apply to R_n and R_c. Alternatively, if the designer wishes to use the Reliability-Based Design Approach, it will be necessary for the designer to establish a probabilistic strength reduction factor, ϕ_F. The following subsections discuss various methods of determining values of these parameters.

5.2 Deterministic Design Approach

Deterministic design procedures have been applied to the geotechnical design of foundations for many years. Typically this approach uses deterministic ultimate foundation design loads in conjunction with nominal ultimate design strengths (R_n, R_c) divided by a nominal factor of safety.

However, some design methods require the use of partial safety factors. For example, both Hansen [1961] and Meyerhof [1970] have advocated the use of partial safety factors for soil parameters, e.g. using a factor of 1.2 - 1.3 on the tangent value of the angle of shearing resistance of the soil (effective stress) and 1.5 - 2.5 on cohesion and this approach has been adopted by the Hong Kong Geotechnical Engineering Office [HKG 1993].

In determining a single safety factor or partial safety factor, the following issues should be considered:

- The consequences of the limit state being reached;
- Reliability of the geotechnical model;
- Uncertainties in the method of analysis and the application of the design calculation model;
- Differences in the strengths of the materials in the actual ground and the strengths derived from the geotechnical investigation;
- Level of supervision to be provided and the likely quality of workmanship.

One method of overcoming or at least minimising these uncertainties in establishing safety factors uses the results of full-scale foundation load tests to calibrate the theoretical design model and thereby establish a probabilistic strength reduction factor (ϕ_F) for the design model.

5.3 Reliability-Based Design Approach

The Reliability-Based Design (RBD) Approach has been under development for many years and has been implemented on an increasing basis over several years, especially in the design of foundations of structures such as bridges. As shown In Figure 1.1, the use of the RBD Approach requires the use of a strength reduction factor (ϕ_F) in order to determine the e^{th} percent exclusion limit foundation strength (R_e).

The e% exclusion limit strength (R_e) of the foundation takes into account the variability of the design / analysis method being used. The relationship between the e% exclusion limit strength (R_e) and the mean strength ($\overline{R}$) computed using the selected design / analysis method is given by the relationship:

$$R_e = \bar{R}(1 - k.V_r) \quad \text{. . . . . Eq. 5.1}$$

where k is a factor depending on the exclusion limit strength adopted, and the type of probability density function (ie. normal or log-normal) and V_r is the coefficient of variation of strength for the foundation design model used. The exclusion limit strength, R_e, corresponds to a defined exclusion limit, which is taken as 10% (R_{10}) by IEC 60826 [IEC 1991] or 5% by ASCE Manual 74 [ASCE 1991].

Efforts have been made in the past years to evaluate the coefficient of variation (V_r), for various foundation design models by collecting and analysing full-scale foundation load tests. Details of the procedure for undertaking full-scale foundation load test, are given IEC 61773 [IEC 1996] and Cigré Special Publication No.81 [Cigre 1994]. Where the foundations are tested with the support in-situ reference should be made to Cigré Special Publication No. 141 [Cigre 1999]. Figure 5.1 presents a schematic representation of a probability density function fitted to strength test data for a specific type of foundation. The terms R_{TEST} and R_n, are the test measured capacity of the foundation and the nominal ultimate strength of the foundation predicted by the selected design model, respectively. The predicted nominal ultimate strength (R_n), is based on the selected design model, the subsurface geotechnical parameters and the foundation parameters at each test site.

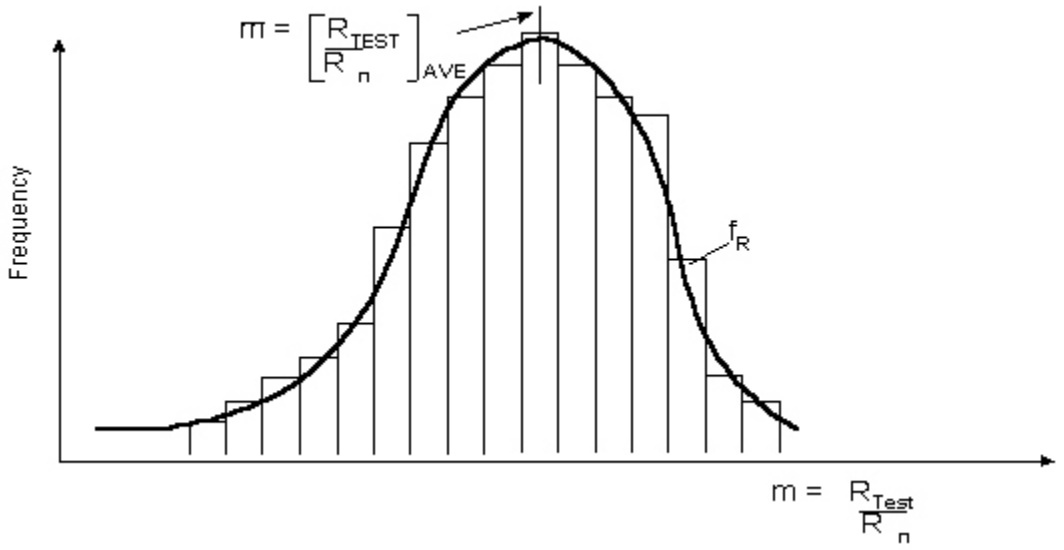


Figure 5.1 Probability Density Function for Strength Test Data

If the average value of the ratio of R_{TEST} / R_n is denoted as $\bar{m}$, then the expected (mean value) $\bar{R}$ of the nominal ultimate foundation strength can be estimated as:

$$\bar{R} = R_n.\bar{m} \quad \text{. . . . . Eq. 5.2}$$

Substituting Equation 5.2 into Equation 5.1 gives:

$$R_e = R_n \,.\, \bar{m}\,(1 - k.V_r) \quad \text{. . . . . Eq. 5.3}$$

In addition, assuming that V_m (the coefficient of variation of m) is a good measure of V_r, then Equation 5.3 becomes:

$$R_e = R_n \,.\, \bar{m}\,(1 - k.V_m) \quad \text{. . . . . Eq. 5.4}$$

For ease of use, Equation 5.4 can be simplified as follows:

$$R_e = R_n \,.\, \bar{m}\,(1 - k.V_m) = \phi_F\, R_n \;.\;.\;.\

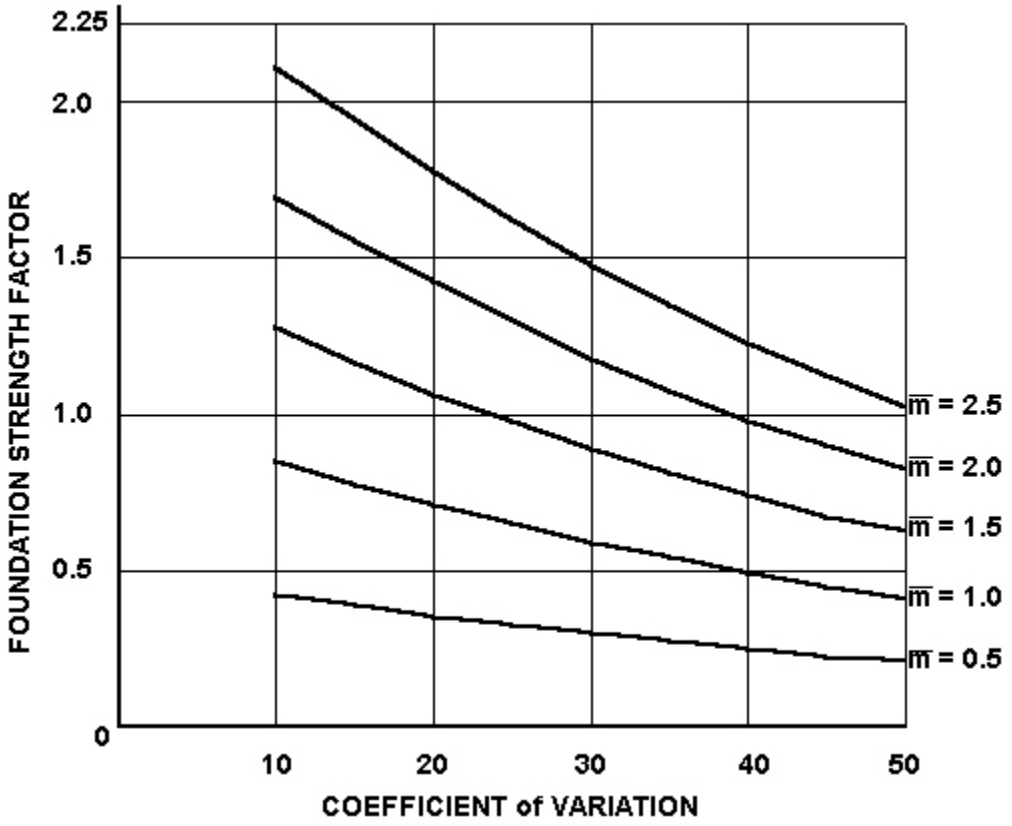


Figure 5.3 - Relationship between the foundation strength factor (ϕ_F) and the coefficient of variation (V_r) for log-normal probability density function at a 5% exclusion limit

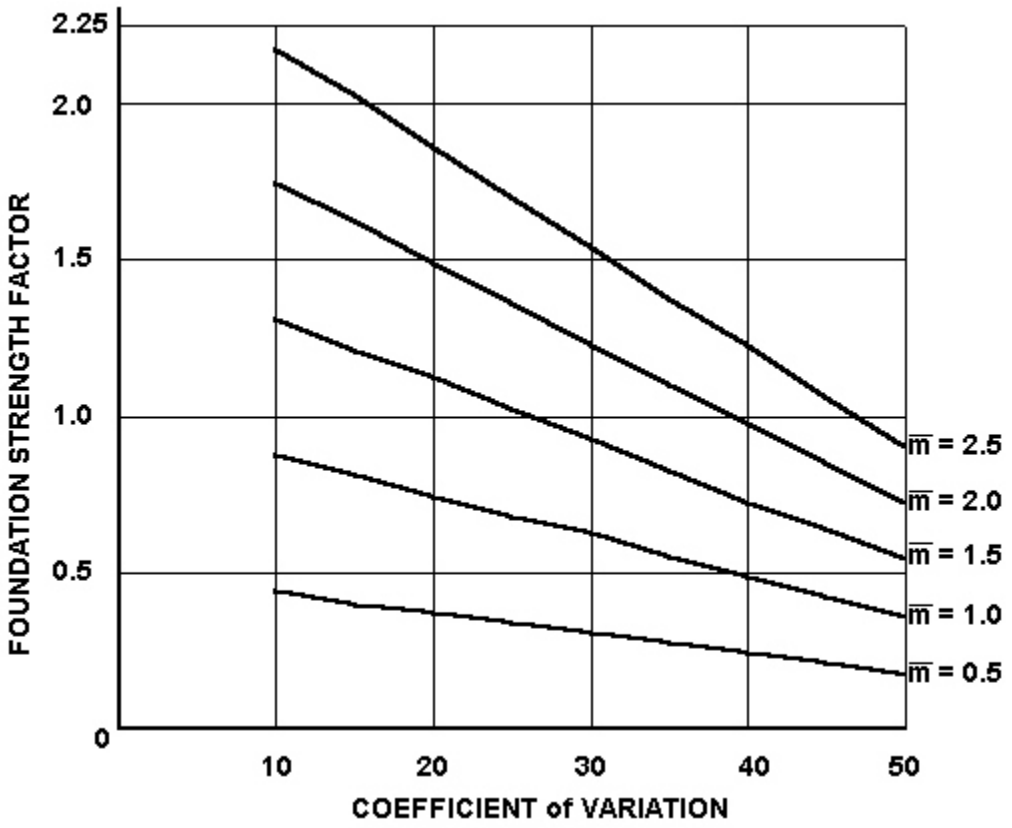


Figure 5.4 - Relationship between the foundation strength factor (ϕ_F) and the coefficient of variation (V_r) for normal probability density function at a 10% exclusion limit

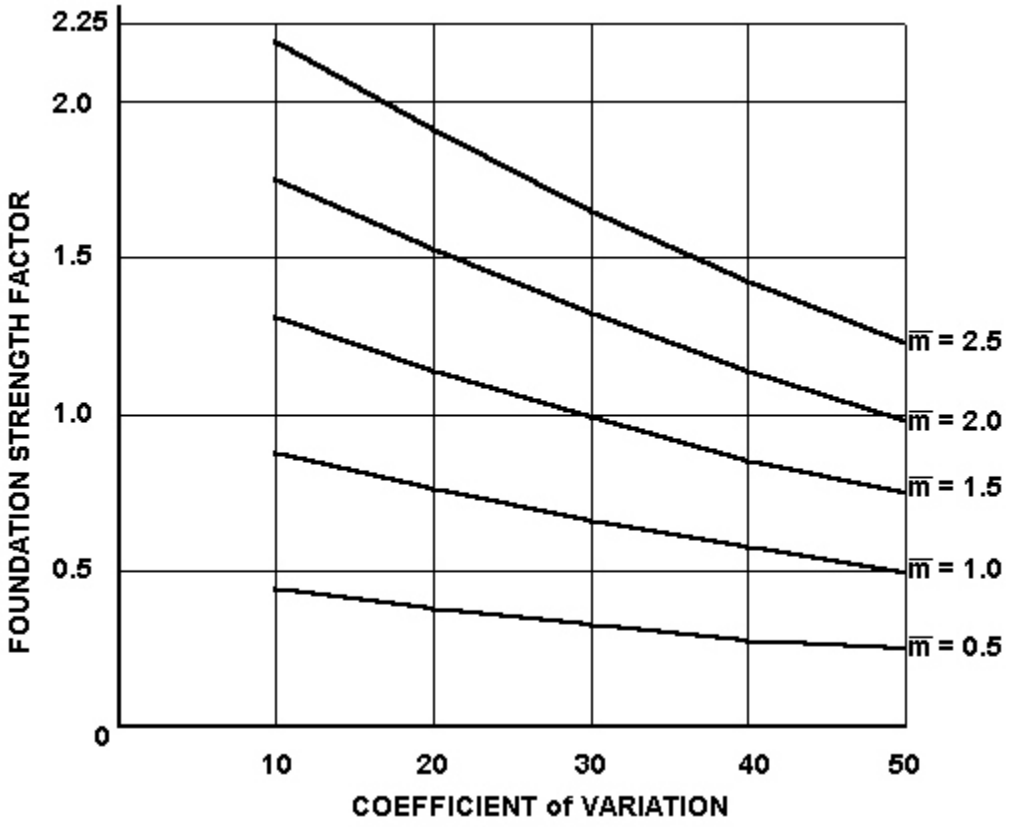


Figure 5.5 - Relationship between the foundation strength factor (ϕ_F) and the coefficient of variation (V_r) for log-normal probability density function at a 10% exclusion limit

Footnote:

1. The relationship between the foundation strength reduction factor (ϕ_F) and the coefficient of variation (V_m) can be determined from expression $\phi_F = \bar{m}\ (1 - k.V_m)$.

   For a Normal PDF at a 5% exclusion limit, k = 1.65, therefore $\phi_F = \bar{m}\ (1 - 1.65V_m)$, where V_m is in decimal form.

   For a Normal PDF at a 10% exclusion limit, k = 1.28, therefore $\phi_F = \bar{m}\ (1 - 1.28V_m)$, where V_m is in decimal form.

   For a Log-Normal PDF, the value of k is a function of V_m and can be taken from the following table for 5% and 10% exclusion limits:

Value of k for various V_m values [Ref ASCE Manual 74 Appendix C. Table C3-1]

Exclusion limit	5%	10%	20%	30%	40%	50%
5%	1.60	1.55	1.46	1.36	1.27	1.18
10%	1.26	1.24	1.19	1.14	1.08	1.02

6 SUMMARY

Transmission line foundations provide the interlinking component between the support and the in-situ soil and/ or rock. Many issues have to be considered in the design of the foundations, e.g. support type, applied loads, strength requirements, deflection limitations, safety factors or strength reduction factors, etc. These considerations explain the number and diversity of the available methods of design and although this not an exhaustive report, an extensive literature review of technical publications available in the public domain has been undertaken and is provided in Annex A. Since the report is purely an overview and not a technical guide / text book, no attempt has been made to include details of the corresponding design methods. Reference should always be made back to the original source literature.

Section 2 of the report has demonstrated that the type of foundation used at any particular location is a function of both the support type and hence the applied loading and the geotechnical conditions present. Wide base lattice towers generate predominately vertical uplift and compression foundation loads and are constructed on separate foundations. Single poles and narrow base lattice towers generate predominately overturning moments and hence can be supported by compact foundations. The geotechnical conditions will influence whether the foundation is a conventional spread footing, a drilled shaft or a pole directly embedded for normal soil conditions or whether piled or raft foundations are required for weaker soils.

Various design methods for determining the nominal ultimate design strengths are presented in Sections 3 and 4 for separate and compact foundations, respectively. Within both of these categories the principal types of foundations commonly used in transmission have been reviewed, i.e. for separate foundations: spread footings, drilled shafts, piled, anchor and H-frame footings and for compact foundations: monoblocks, drilled shafts, direct embedment, piled and raft foundations. Where possible, an indication has been given as to whether the geotechnical design model has been calibrated against reduced scale (model) laboratory tests or the results of full-scale loading tests. In addition, the influence of construction techniques on the foundation design has also been described, since this, can have a significant influence on the successful outcome on any transmission line project.

Section 5 describes, in general terms, Deterministic and Reliability-Based (RBD) Approaches. In utilizing the Deterministic Approach, a great deal of engineering judgement is needed in establishing safety factors. On the other hand, the RBD Approach requires the use of Strength Reduction Factors, which, in turn are based on the results of full-scale foundation load tests. The selection of the design Strength Reduction Factors for a specific design model also requires engineering judgement since they may be based on a limited number of full-scale foundation load tests.

Limited information on the effects of sustained loading or dynamic loading on support foundations has been published. The results of these investigations, should however, be treated with caution, since the dynamic loadings considered may not be representative of the conditions experienced in the field.

ANNEX A

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