Bauforschung

Mitarbeit bei der internationalen Vereinheitlichung von technischen Baubestimmungen im Grundbau

T 1706

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11.17

DEUTSCHE GESELLSCHAFT FÜR ERD- UND GRUNDBAU e.V.

Abschlußbericht zum Forschungsvorhaben:

MITARBEIT BEI DER INTERNATIONALEN VEREINHEITLICHUNG VON TECHNISCHEN BAUBESTIMMUNGEN IM GRUNDBAU

Verfasser: o.Prof.Dr.-Ing.U.Smoltczyk, Stuttgart
Dr. W.Sadgorski, München

Dezember 1985

INHALTSÜBERSICHT

EINLEITUNG

Die "Deutsche Gesellschaft für Erd- und Grundbau e.V." ist eine 1950 gegründete Ingenieurvereinigung, deren Zweck der Austausch von Informationen zu geotechnischen Fragen und die Förderung der Arbeit an einschlägigen Problemlösungen ist. Sie versteht sich als Mittlerin zwischen Forschung und Praxis, indem sie einerseits die wissenschaftlichen Arbeitsergebnisse für die Baupraxis in einer anwenderfreundlichen Form aufbereitet und andererseits die Erfahrungen und Innovationen der Baupraxis zur wissenschaftlichen Vertiefung und Systematisierung an geeignete Forschungseinrichtungen vermittelt.

Die Gesellschaft ist durch die Person ihres Vorsitzenden mit dem Deutschen Normenausschuß verbunden. Er leitet als Vorsitzender des Fachbereichs V "Baugrund" die Arbeit der mit geotechnischen Normen befaßten Arbeitsausschüsse, wobei ihm seit 1984 ein Lenkungsausschuß behilflich ist.

Da die Gesellschaft die deutschen Interessen in den internationalen Vereinigungen "International Society of Soil Mechanics and Foundation Engineering", "International Society of Rock Mechanics" und "International Association of Engineering Geology" vertritt sowie die Belange des deutschen Grundbaus in der Europäischen Gemeinschaft und in der ISO, obliegt ihr die Aufgabe, an allen für den deutschen Tiefbau relevanten außerdeutschen technischen Regelungen in Form von Normen, Codes, Standards, Empfehlungen usw. aktiv teilzunehmen.
Da solche Arbeiten nur aus eigenen Mitteln finanziert werden können, war es außerordentlich fördernd, daß die Gesellschaft seit 1979 vom Institut für Bautechnik durch Forschungsaufträge in diesen Verpflichtungen unterstützt wurde. So wurden 1979/80 erstmals Mittel für die Erarbeitung eines "Vergleichs nationaler Richtlinien für die Berechnung von Fundamenten" durch K.Malcharek und U.Smoltczyk (veröffentlicht als Mitteilung Nr.16 des Baugrundinstituts Stuttgart) zur Verfügung gestellt.

Der nachstehende Abschlußbericht bezieht sich auf die 1980 anschließende Förderung, wobei insbesondere die Mitwirkung der DGEG bei der Erarbeitung des Eurocodes 7 "Gründungen" unterstützt wurde.

Der Bericht gliedert sich daher in einen Teil 1, der die Aktivitäten der Gesellschaft bei der Erstellung internationaler geotechnischer Regelwerke darlegt; einen Teil 2, der die spezielle Arbeit am EC 7 betrifft, und einen kurzen Schlußteil 3, der die bisherigen Erfahrungen im Hinblick auf die weitere Arbeit wertet.

TEIL 1

MITWIRKUNG DER DGEG AN REGELWERKEN AUßERHALB DER NORMEN

Die Deutsche Gesellschaft für Erd- und Grundbau ist, wie Bild 1 ausweist, in ein weit verzweigtes Raster von Institutionen international eingebunden. Dementsprechend hat sich die Mitwirkung an übernationalen geotechnischen Normen und Empfehlungen auf verschiedenen Ebenen entwickelt:

(a) durch Mitarbeit in Technischen Komitees der internationalen geotechnischen Gesellschaften, wahrgenommen von Mitgliedern der DGEG ohne unmittelbare Kostenbelastung für die Gesellschaft, wenn auch mittelbar durch den damit verbundenen Verwaltungsaufwand;

(b) Übernahme der Federführung in folgenden Technischen Komitees der Internationalen Gesellschaft für Grundbau und Bodenmechanik (im folgenden ISSMFE abgekürzt):
- Field and Laboratory Testing of Soils (Obmann: Smoltczyk Tätigkeit 1985 abgeschlossen);
- Information Advisory Committee (Obmann: Nendza);
- Ground Freezing Committee (Obmann: Jessberger);
- European Technical Committee on Piling (Obmann:Franke).

(c) Beteiligung am ISO Technical Committee 182 "Geotechnics".

(d) Dokumentationsdienst für das gesamte geotechnische Schrifttum.

Wesentliche Kostenbelastungen entstehen dabei aus (b) und (d), die ganz überwiegend aus Mitgliederbeiträgen finanziert werden müssen.

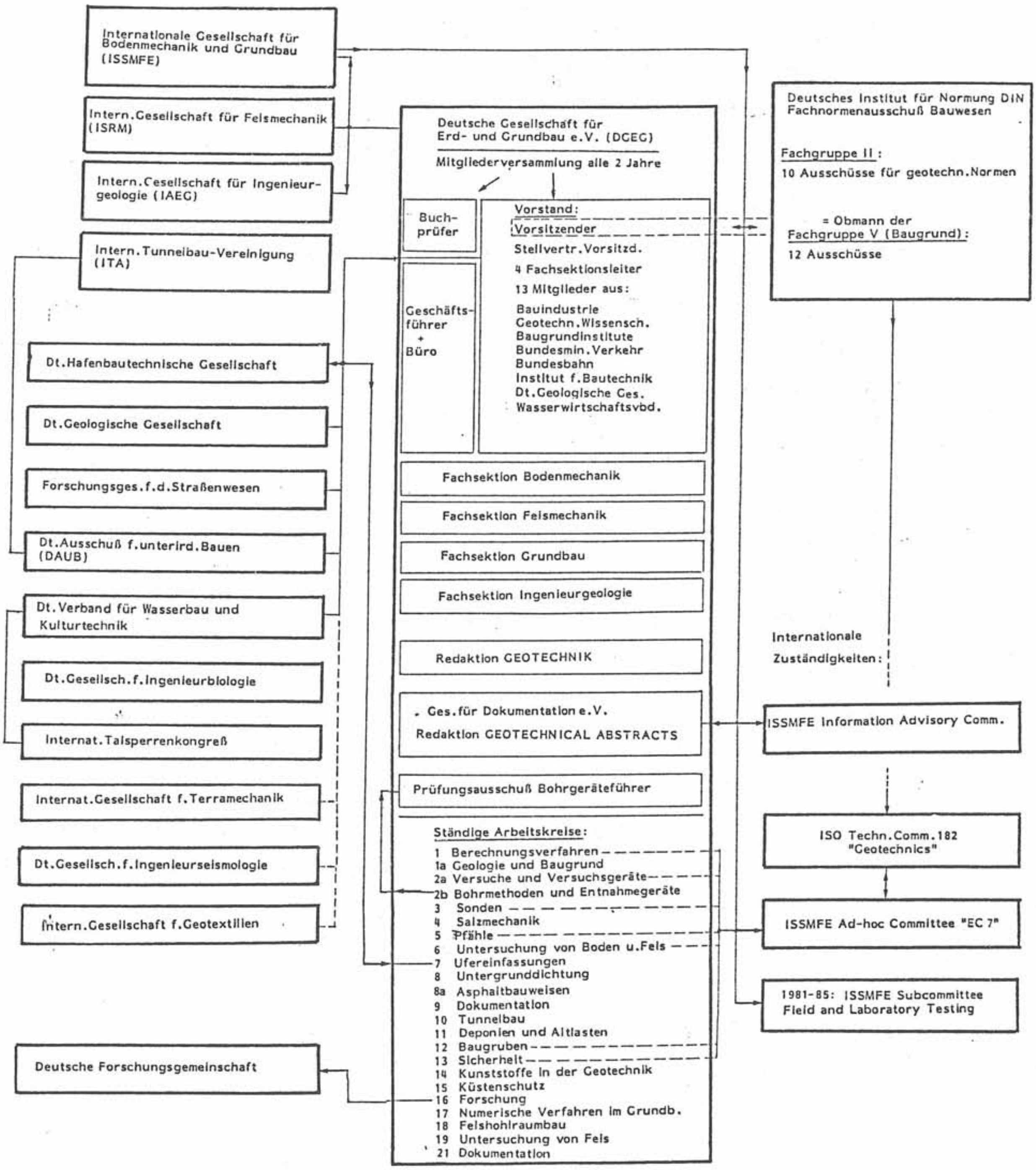

Bild 1 Aufbau und Verknüpfungen der Deutschen Gesellschaft für Erd- und Grundbau e.V.

1.1 VEREINHEITLICHUNG DER SONDIERVERFAHREN

Bei der internationalen Tagung der ISSMFE 1957 in London wurde auf holländische Anregung hin ein europäischer Sondenausschuß gegründet, in den 1961 Herr Dr.-Ing.Zweck als deutscher Vertreter berufen wurde; ihm wurde in der Folge auch die Leitung der Untergruppe übertragen, die die Ramm- und Drucksondierungen behandeln sollte. Die amerikanische Gruppe wollte sich dagegen mit dem Standard Penetration Test befassen. Die europäische Gruppe konnte 1965 einen Entwurf vorlegen, während die amerikanische aus Mangel an Einigung ihre Arbeit einstellte.

1974 kam es zu einem ersten europäischen Symposium in Stockholm (ESOPT 1), 1982 folgte ESOPT 2 in Amsterdam. Während die Gruppe zunächst nur aus Vertretern der Schweiz, der Niederlande, Schwedens und der Bundesrepublik Deutschland bestand, wurde sie beim ESOPT 1 um Mitglieder aus Bulgarien, der UdSSR, Großbritanniens und Belgiens erweitert. Dieses Komitee leitete Dr.Zweck bis 1976, dann B.Broms (Schweden). Die von diesem Kreis erarbeitete Empfehlung umfaßte die Rammsonde, die Drucksonde, den Standard Penetration Test und die schwedische Gewichtssonde. Sie wurde 1977 in Tokio vorgelegt und genehmigt. Entgegen deutschen Wünschen wurde eine schwere Rammsonde mit 63,5 kg Gewicht empfohlen, um dasselbe Fallgewicht wie beim SPT zu haben. Glücklicherweise konnte in einem Nachtrag 1981 noch die leichte 10 kg Sonde aufgenommen werden.

Die ursprünglich nur europäische Empfehlung ist inzwischen (San Francisco 1985) auch als international verbindliche Regel akzeptiert worden; das damit befaßte Komitee (deutscher Vertreter: Dr.-Ing.Melzer, Frankfurt) ist ein Technisches Komitee der ISSMFE. Zum Berichtszeitpunkt liegen folgende Empfehlungen anwendungsreif vor:

- (Recommended) Reference Test Procedure for the Cone Penetration Test CPT;
- International Reference Test Procedure on the Weight Sounding Test (WST);
- International Reference Test Procedure on the Standard Penetration Test (SPT);

- International Reference Test Procedure on the Dynamic Probing Test (DP).

Insbesondere der erst- und der letztgenannte Sondentyp sind für die deutsche Normung von Bedeutung; die betreffenden internationalen Referenztexte werden zur Zeit von dem mit der DIN 4094 befaßten NABau-Arbeitsausschuß (= Arbeitskreis 3 der DGEG) in die deutschen Normen eingearbeitet. Eine deutsche Übersetzung erübrigt sich deswegen zur Zeit.
Die ISSMFE-Gruppe betrachtet ihre Arbeit als vorerst beendet. Die 4 Dokumente werden vom Schwedischen Geotechnischen Institut zu einem Gesamtbericht zusammengefaßt und bis August 1986 an alle Mitgliedsstaaten der ISSMFE zur Kenntnis- und Stellungnahme verschickt. Danach kann 1987 darüber formell und abschließend Beschluß gefaßt werden. Erst danach ist eine Abgabe des Dokuments an z.B. die ISO formal zulässig.

1.2 VEREINHEITLICHUNG DER BAUGRUND-UNTERSUCHUNGSVERFAHREN

Zu der besonders dringend zu vereinheitlichenden Thematik der Baugrunduntersuchungen wurde 1978 ein ISSMFE-Komitee gegründet, in dem Herr Prof.Sommer, Kassel, deutscher Vertreter ist. Es begann seine Arbeit mit der Sammlung von Angaben über die in den verschiedenen Ländern üblichen Erkundungsverfahren. Auf Grund des so gewonnenen Materials wurde 1983 ein Handbuch der Baugrunderkundung vorgelegt, das vor allem demjenigen nützt, der sich über die Praktiken eines Auslandes informieren möchte, wenn ihm anläßlich eines Auslands-Bauvorhabens ein Baugrundgutachten eines örtlichen Baugrundinstituts vorgelegt wird. Von einer Harmonisierung der Untersuchungsverfahren kann jedoch vorerst keine Rede sein; sie kann wohl auch weniger von dieser Gruppe als von denjenigen geleistet werden, die sich mit den einzelnen Versuchen in Feld und Labor mit dem Ziel einer internationalen Abstimmung befassen.

1.3 VERFAHREN DER PROBEN-GEWINNUNG

Zu den Verfahren der Proben-Gewinnung besteht unter japanischer Leitung seit 1978 ein ISSMFE-Komitee, in dem die deutschen Belange von Herrn Prof.Kany, Nürnberg, wahrgenommen werden.

In der ersten Bearbeitungsrunde bis 1981 wurde das "International Manual for Sampling of Soft Cohesive Soils" als Entwurf (s.a. NABau V 11,Nr.2-80) zusammengestellt. Es dokumentiert die Vielzahl der regional angewendeten Verfahren, siehe dazu auch die im Auftrag des Instituts für Bautechnik 1981 von R.Herrmann gefertigte Querschnittsstudie (FA IV/1-5-265/80).
1981 erfolgte eine organisatorische Umordnung bei der ISSMFE, wonach es fortan zwei Technische Komitees gab: "Sampling and Testing of Residual Soils" und "Undisturbed Sampling and Laboratory Testing of Soft Rocks and Indurated Soils". Die erste Gruppe unter dem Vorsitz von Dr.Brand (Singapur) begann ihre Arbeit mit einer Materialsammlung und veröffentlichte 1984: "Sampling and Testing of Residual Soils: A Review of International Practice".
Auch die zweite Gruppe unter australischer Federführung ist noch ganz in den Anfängen und bemüht sich insbesondere um die gebotene Abstimmung mit der Felsmechanik.
Für die deutschen Norm-Interessen ergeben sich bei diesen beiden Gruppen in naher Zukunft wohl noch keine Ansätze.

1.4 GEOMECHANISCHE SOFTWARE

Das Komitee wurde 1973 in Moskau gegründet und steht unter kanadischer Leitung. Deutscher Vertreter ist Herr Dr.Semprich, Mannheim. Die Hauptaufgabe des Komitees ist die Entwicklung eines Programm-Dokumentationsdienstes auf der Basis der Geotechnical Abstracts. Die Bilder 2 und 3 zeigen die inzwischen akzeptierten Dokumentationsformen an einem Beispiel.
Auch die Internationale Gesellschaft für Felsmechanik hat ein dieses Thema bearbeitendes Komitee, das inzwischen bereits einen Grundstock an Software erfaßt hat und im Nachweisdienst anbietet.
Eine direkte Bedeutung für die internationale Normung haben diese Arbeiten noch nicht; man wird aber auf sie Bezug zu nehmen haben, wenn das Thema der EDV-gestützten Standsicherheitsnachweise zur Harmonisierung anstehen wird. Es wird sich dann die Frage stellen, ob nur solche Verfahren als anerkannte Verfahren der Geotechnik zu gelten haben, die in dieser Weise international archiviert sind.

GEOTECHNICAL ABSTRACTS (program abstract)

GA | IGC

Name of the program and short designation (max. 220 characters):

Three-dimensional calculation of stresses and displacements for the analysis of the stability of caverns, slopes and foundations in soil and rock (FEST 03)

Year of first installation: 1979

Year of installation of the version currently offered: 1980

Available from (max. 160 characters):

Institut für Grundbau, Bodenmechanik, Felsmechanik und Verkehrswasserbau der RWTH Aachen, Mies-van-der-Rohe-Str. 1, 5100 Aachen, Fed. Rep. of Germany

Capability (max. 1050 characters):

The method of calculation is based upon the Finite Element Method. The program implements element types of an isoparametric three-dimensional element with 8 - 21 nodal points, as well as bar, spring and joint elements. A linearly elastic viscoplastic stress-strain behaviour is assumed for the ground. In the elastic range a transversal isotropy is described by the constants of elasticity E_1, E_2, ν_1, ν_2, G_2. The transition from elastic to viscoplastic displacements is described by the Mohr-Coulomb failure criterion for isotropic strength as well as for planes of reduced strength of any given spatial orientation (e.g. discontinuities and schistosities in rock masses). The simulation of the different construction stages and the associated states of stress and displacement can be determined very economically by means of an iterative method. The stresses and displacements can be plotted perspectively with the aid of a plotter program.

Program language: Fortran

Number of statements: about 8000

Type of program: batch

Computer upon which the program is installed: IBM 3033

Descriptors:

program, cavern, slope, foundation, stability, Finite Element Method

Name and address of contributor:

....

Signature

Bild 2 International vorgeschlagene Kurzform der Programmbeschreibung für geotechnische Software

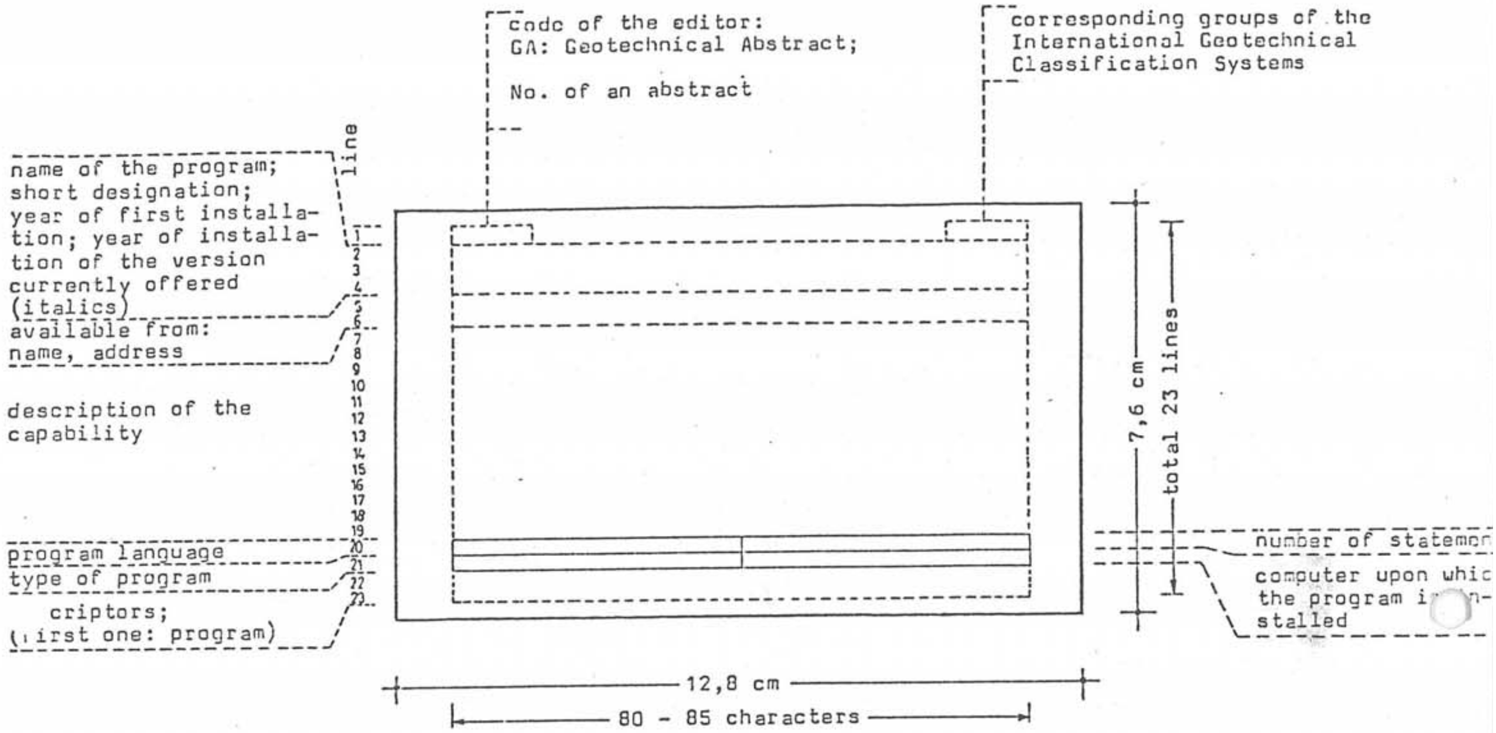

Bild 3 Format der Nachweiskarte für die Information nach Bild 2

1.5 VEREINHEITLICHUNG DER BEGRIFFE

Eine der wesentlichen Voraussetzungen für die Harmonisierung technischer Regeln ist eine international vereinbarte und eindeutige Zuordnung von Bezeichnungen und Bedeutungsgehalten in verschiedenen Sprachen, wobei heute die englische Sprache in der Geotechnik als Referenzsprache benutzt wird.

Zu diesem Thema haben alle internationalen geotechnischen Gesellschaften Studiengruppen, die ISSMFE bereits seit über 30 Jahren. Aus Anlaß der internationalen Konferenzen wird jeweils ein Glossar veröffentlicht, das die inzwischen eingebürgerten Fachausdrücke und Symbole zu den bereits früher vereinbarten ergänzt und dann verbindlich ist für alle wissenschaftlichen Publikationen auf geotechnischem Gebiet.

Die Felsmechanik wird in den Komitees der ISSMFE und der IAEG mit vertreten.

Die Verbindung zur Normung ist einmal dadurch gegeben, daß in der DIN 1080 grundsätzlich von den international festgelegten Symbolen und Bezeichnungen ausgegangen wird; zum anderen durch den Umstand, daß die ISO ein Komitee zur Klassifizierung von Boden und Fels und zur Auflistung der Symbole unter schwedischer Federführung eingerichtet hat.

1.6 FELD- UND LABORVERSUCHE

Die international einheitliche Festlegung von Anforderungen an boden- und felsmechanische Untersuchungsverfahren ist in der Geotechnik noch mehr als in anderen Sparten des Bauingenieurwesens eine Voraussetzung für jede andere Harmonisierungsbemühung.

Die Internationale Gesellschaft für Felsmechanik berief bereits 1967 eine Kommission zur Vereinheitlichung felsmechanischer Untersuchungsmethoden, die seitdem folgende Empfehlungen veröffentlichte:

- "Suggested methods for determining shear strength"(1974);
- "Suggested methods for rock bolt testing"(1974);
- "Suggested methods for the quantitative description of discontinuities in rock masses"(1977);
- "Suggested methods for determining hardness and abrasiveness of rocks"(1977);
- "Suggested methods for petrographic description of rocks"(1977);
- "Suggested methods for determining the strength of rock materials in triaxial compression"(1977);
- "Suggested methods for monitoring rock movements with borehole extensometers"(1977);
- "Suggested methods for determining uni-axial compressive strength and deformability of rock materials" (1978);
- "Suggested methods for determining water content, porosity, density, absorption and related properties and swelling and slake-durability index properties" (1978);
- "Suggested methods for pressure monitoring using hydraulic cells"(1979).

Diese Empfehlungen haben allerdings in der Regel eher einen qualitativ beschreibenden als quantitativ festlegenden Charakter. Eine Standardisierung der Versuche ist damit noch nicht geleistet.

Die ISSMFE ist hier erst relativ spät aktiv geworden, nachdem man im westlichen Ausland und in den Entwicklungsländern jahrzehntelang vorwiegend nach den ASTM Standards verfuhr, wo eigene nationale Regeln nicht vorhanden waren.

Erst in den Jahren 1979 bis 1985 wurde ein unter deutscher Leitung stehendes Technisches Komitee hierfür berufen, das aus 24 Mitgliedern bestand (Leitung: Smoltczyk) und sich demgemäß nur wenige Male treffen konnte, im übrigen aber versuchte, auf dem Korrespondenzweg weiterzukommen. Inzwischen hat man die Konsequenz gezogen und die Aktivitäten auf diesem Gebiet regionalisiert. So gibt es seit 1985 ein europäisches Komitee für bodenmechanische Versuche (speziell: Triaxialversuch), in dem die DGEG mitarbeitet, das aber unter niederländischer Federführung steht.

Die bisher publizierten Entwürfe stellen Zwischenstadien dar:

- "Suggested Procedure on the Compression and Swelling Test" (Verfasser: Frydman, Haifa, und Calabresi,Rom. Veröffentlicht als Mitteilung des Techneion Haifa 1984);
- "Suggested Procedure on the Triaxial Shear Test" (Verfasser: Berre,Oslo. Veröffentlicht als Mitteilung des Norwegischen Geotechnischen Instituts 1985).
- "Recommended Procedure on the Axial Pile Loading Test" (Veröffentlicht durch den Obmann Smoltczyk 1985 im ASTM Journal Juni 1985).

Auch das gegenwärtig stark diskutierte Verfahren der dynamischen Pfahlprobebelastung wurde in eine "Suggested Method" gefaßt, konnte aber nicht bis zur Veröffentlichungsreife gebracht werden.

Bei der Durchführung dieses recht aufwendigen Verfahrens wurde ein im Rahmen dieses Forschungsvorhabens beschäftigter englischer Diplomingenieur zu Hilfe genommen.

Die o.g. drei Arbeitsdokumente bilden die Grundlage für die Überarbeitung der entsprechenden deutschen Normen in den Arbeitskreisen für Versuche und Versuchsgeräte (v.Soos) und für Bauart und Tragfähigkeit der Pfähle (Franke); sie ergänzen außerdem die im Teil 2 geschilderten Bemühungen um eine europäische Norm an der Stelle, wo dort die Versuche angesprochen werden.

TEIL 2

BERICHT ÜBER DIE AUSARBEITUNG EINES "MODEL CODE" FÜR DEN EUROCODE 7

2.1 KONSTITUIERUNG UND ZUSAMMENSETZUNG DER ARBEITSGRUPPE

Die Kommission der Europäischen Gemeinschaften (KEG) beabsichtigt, europäische Regelwerke - die EUROCODES - für den Entwurf, die Bemessung und die Ausführung von Gebäuden und Ingenieurbauwerken aufzulegen. Mit Hilfe dieser Regelwerke sollen einheitliche Regeln bereitgestellt werden als Alternative zu den geltenden, differierenden Regeln in den verschiedenen Mitgliedstaaten. Ferner soll gleichzeitig mit der Aufstellung der EUROCODES eine freiwillige Harmonisierung der nationalen Normung angestrebt werden.

Das Programm der Kommission zur Vereinheitlichung von Bestimmungen, Gesetzen und Verwaltungsvorschriften der Mitgliedstaaten auf den Gebieten der Sicherheit, der Gebrauchsfähigkeit und der Dauerhaftigkeit der verschiedenen Bauarten und Baustoffe sieht anfänglich acht EUROCODES vor; davon betrifft der EUROCODE Nr. 7 die Gründungen sowie Erd- und Stützbauwerke. Es ist vorgesehen, als Grundlage bei der Abfassung der EUROCODES sog. "model codes", d.h. bestehende Richtlinien übernationaler Gremien, zu verwenden. Solche bestehen z.B. auf dem Gebiet des Stahl- und Stahlbetonbaues ("Recommendations for steel structures" des EKS, "Common Unified Rules" des CEB/FIP), nicht jedoch auf dem Gebiet des Grundbaues.

Deshalb nahm die bei der Kommission der EG zuständige Stelle, die Generaldirektion für den Binnenmarkt und gewerbliche Wirtschaft, Verbindung zum damaligen Generalsekretär der Internationalen Gesellschaft für Bodenmechanik und Grundbau (ISSMFE), dem inzwischen verstorbenen Prof. Nash, auf.

Die ISSMFE erklärte sich bereit, die Aufstellung eines "model code" für die von der KEG angestrebte Grundbaunorm EC 7 zu übernehmen. Die ISSMFE konnte Herrn Prof. Krebs Ovesen aus Dänemark für die Leitung dieser Arbeiten gewinnen und einen Arbeitsausschuß aus Vertretern der bisher neun nationalen Gesellschaften der EG-Länder bilden.

Ursprünglich war vorgesehen, diesem Komitee den Status eines technischen Komitees der ISSMFE zu geben. Einige nationale Gesellschaften der ISSMFE meldeten jedoch angesichts des Umstandes, daß es sich nicht um ein wirklich internationales, sondern nur auf einen europäischen Teilbereich beschränktes Gremium handelt,Bedenken an. Daher wurde vereinbart, die Arbeitsgruppe als gemeinsames Ad-hoc-Komitee der Mitgliedsgesellschaften der ISSMFE in den neun EG-Ländern anzusehen. Die EG-Kommission erklärte sich mit dieser Organisationsform einverstanden.

Der Arbeitsgruppe gehört also je ein Mitglied aus einem jeden EG-Land mit Ausnahme Luxemburgs an. Das französische und das griechische Mitglied werden aus haushaltstechnischen Gründen bei den Arbeitssitzungen häufig durch ständige Vertreter ersetzt, die inoffiziell als Vollmitglieder betrachtet werden. Die DGEG wurde bei den ersten beiden Sitzungen von Prof. Smoltczyk vertreten. Auf der Sitzung des NABau-Arbeitsausschusses V4 "Baugrund-Berechungsverfahren" am 24.9.1981 in Sindelfingen hat Prof. Smoltczyk angeregt, einen deutschen Spiegelausschuß für die Arbeit des Ad-hoc-Ausschusses zu bilden, wozu vor allem der Arbeitsausschuß V4 als in der Sache Hauptbeteiligter in Frage komme. Der Arbeitsausschuß hat dann wiederum sein Mitglied, Dr. Sadgorski, Landesamt für Wasserwirtschaft in München, um Übernahme der deutschen Vertretung im Ad-hoc-Komitee gebeten.

Die DGEG beauftragte ferner zur Unterstützung von Dr. Sadgorski ihren befristet eingestellten englischen Mitarbeiter Dipl.-Ing. Thorp mit der geschäftsmäßigen Betreuung des englischen Schriftverkehrs. Die Tätigkeit von Herrn Thorp endete am 30.6.1984.

Die Arbeitsgruppe besteht aus folgenden weiteren Mitgliedern:

- Belgien: Prof. E. Lousberg, Université Catholique de Louvain; z. Zt. Präsident der Belgischen Geotechnischen Gesellschaft;
- Dänemark: Prof.N.Krebs Ovesen, Ingenieurakademie in Lyngby; z. Zt. Vizepräsident der ISSMFE für Europa;
- Frankreich: Prof.F.Baguelin, LCPC in Nantes, und S.Amar, LCPC Paris;
- Griechenland: Prof.A. Anagnostopoulos, TU Athen, und Dr.D. Coumoulos, Beratender Ingenieur in Athen;
- Großbritannien: Dr.B.Simpson, Fa.Ove Arup & Partners in London;
- Irland: Dr.T.Orr, Trinity College in Dublin;
- Italien: Prof.R.Japelli, früher Universitá di Palermo, jetzt II Universitá di Roma;
- Niederlande: Herr W.J.Heijnen, Laboratorium voor Grondmechanica Delft;

Herr Heijnen hat dankenswerterweise das Sekretariat der Arbeitsgruppe übernommen und wird dabei von Herrn H.Nelissen unterstützt.

2.2. SITZUNGEN DER ARBEITSGRUPPE

Die konstituierende Sitzung der EC 7-Arbeitsgruppe fand am 2./3.04.81 in Brüssel unter dem Vorsitz von Prof. Krebs Ovesen statt. Zeitweilig anwesend waren die Herren Dr. Gray und Dr. Ehrentreich von der o.g. Generaldirektion der KEG. Als weitere Arbeitssitzungen folgten:

Nr.	Ort	Datum
2	Stockholm	15.+17.06.81
3	Paris	15./16.10.81
4	London	14./15.01.82
5	München	01./02.04.82
6	Athen	10./11.06.82
7	Kopenhagen	30.09./01.10.82
8	Dublin	14./15.01.83
9	Helsinki	27.05.83
10	Rom	29./30.09.83
11	Delft	19./21.01.84
12	Louvain-la-Neuve/Belg.	17./19.05.84
13	Athen	13./14.09.84
14	Paris	17./18.01.85
15	London	22./23.05.85
16	München	16./17.09.85

Die Organisation der Sitzungen übernahmen die Mitglieder der Arbeitsgruppe aus dem jeweiligen Gastgeberland.
Bei den ordentlichen Sitzungen war die Gruppe fast stets vollzählig, wobei die Mitglieder aus Frankreich und Griechenland jeweils alternativ an den Sitzungen teilnahmen. Häufig stoßen 1 bis max. 3 Kollegen aus dem Land, in dem die jeweilige Sitzung stattfindet, zur Beratung einzelner Abschnitte hinzu.
Die Sitzungs-Niederschriften werden von Herrn Heijnen unter Mitwirkung von Herrn Nelissen gefertigt und den Mitgliedern zugesandt.

Zur laufenden Information werden sie von Herrn Sadgorski mit den meisten Unterlagen an die Herren Dr. Hanisch, Institut für Bautechnik, Prof. Smoltczyk und Prof. Horn als Obmann des Spiegelausschusses weitergeleitet. Ausarbeitung und Versand der offiziellen Niederschriften nehmen jedoch meist mehrere Wochen Zeit in Anspruch. Um aber die Erfüllung der Aufgaben, die die deutsche Vertretung übernommen oder zugewiesen bekommen hat, nicht unnötig zu verzögern und zur schnelleren Information wurden von Herrn Thorp und später von Dr. Sadgorski nur wenige Tage nach jeder Sitzung gesonderte Sitzungsnotizen (rough personal notes) aufgestellt und nach dem vereinbarten Verteiler versandt.

Für 1986 sind folgende drei weitere Sitzungen vorgesehen:

17.	Rom	23./25.1.85
18.	Kopenhagen	12./13.6.85
19.	(Ort noch nicht bestimmt)	Sept. 1985

Angesichts der bevorstehenden Aufnahme ihrer Länder in die EG lud der Obmann Vertreter der Spanischen und der Portugiesischen Gesellschaft als Gäste zu diesen Sitzungen ein.

2.3 GEGENWÄRTIGE SITUATION IN DEN EINZELNEN EG-LÄNDERN

Die Arbeitsgruppe nahm ihre Arbeit mit einer Bestandsaufnahme der Nationalen Richtlinien auf dem Gebiet der Geotechnik in den einzelnen Ländern auf. Hierzu fertigte jedes Mitglied der Gruppe eine Übersicht des Normungsstandes in seinem Land im Frühjahr 1981. Spätere Änderungen dieses Standes wurden den Mitgliedern der Gruppe mitgeteilt und neuerschienene Richtlinien wurden ihnen meistens überreicht. Nachfolgend einige Ergebnisse der Bestandsaufnahme:

2.3.1 Belgien

Zur Zeit existieren in Belgien noch keine Normen oder andere Richtlinien auf dem Gebiet der Geotechnik.

Für Labor- und Feldversuche werden die DIN- oder ASTM-Normen oder aber Festlegungen des "Rijksinstituut voor Grondmechanica" verwendet.
1977 wurde eine Kommission für Pfahlgründungen unter dem Vorsitz von Prof. de Beer mit folgenden 5 Unterausschüssen gebildet:

1. Baugrunduntersuchung (Vorsitz Prof. de Beer, 12 Mitglieder)
2. Eigenschaften des Pfahlmaterials
3. Systematik der in Belgien verwendeten Pfahltypen - Beschreibung, Bauausführung, Baukontrolle.
4. Bestimmung der Tragfähigkeit und der Setzungen von Pfahlgründungen
5. Bauüberwachung und Probebelastungen (Vorsitz Prof. Lousberg, 15 Mitgl.)

Die von den Gruppen 1 und 5 ausgearbeiteten Empfehlungen wurden im Heft 3/1984 der "Annales des Travaux Publiques de Belgique" veröffentlicht (insges. 33 S., zweisprachig) und zur Diskussion gestellt. Die Empfehlungen der anderen 3 Gruppen sollen in nächster Zukunft ebenfalls veröffentlicht werden.

2.3.2 Dänemark

Eine dänische Grundbaunorm (Dansk Ingeniørforenings Norm for Fundering, DS 415) existiert seit Ende der 50-er Jahre; sie war von Prof. Brinch Hansen maßgeblich geprägt. 1977 erschien eine zweite und 1984 eine dritte Ausgabe, die auch in englischer Sprache vorliegen. Die Norm besteht aus Text und Guide, die sukzessiv aufeinander folgen. Die recht kompakte, übersichtliche und knapp gehaltene Norm schreibt für Standsicherheitsnachweise die Methode der Partialsicherheiten vor; es werden jedoch keinerlei Details für Bodenuntersuchungen, Standsicherheitsnachweise o.ä. gegeben.

Mit ihrer klaren Konzeption und übersichtlichen Systematik dient die DS 415 weitgehend als Vorbild bei der Ausarbeitung von EC 7.

2.3.3 Frankreich

Die in Frankreich existierenden Regelwerke können folgenden 4 Gruppen zugeordnet werden:

a) "Documents Techniques Unifiés" (DTU)
b) "Cahiers des Prescriptions Communes" (CPC) der Straßenbauverwaltung
c) Empfehlungen und Erläuterungen
d) Andere Regelwerke

Die DTU's werden von paritätisch besetzten Ausschüssen unter der technischen Betreuung des "Centre Scientifique et Technique du Batiment" verfaßt. Sie können aus: "Cahiers des charges" mit grundsätzlichen technischen Bedingungen, "Regles des calcul" und "Cahiers de clauses specielles" mit Vertragsbedingungen bestehen.
Die DTU's sind für öffentliche Bauvorhaben verbindlich; bei privaten Bauvorhaben können sie als Vertragsbestandteil vereinbart werden.
Zur Zeit liegen auf dem Gebiet der Geotechnik zwei gültige DTU's vor:

- DTU Nr. 13.1 für Flachgründungen
- DTU Nr. 13.2 für Tiefgründungen (2. Ausgabe 1978)

Diese sind von relativ niedrigem Ausführlichkeitsgrad und enthalten keine Bemessungsregeln (bis auf Festlegungen zu den zul. Spannungen im Pfahlmaterial). Für Baugrunduntersuchungen ist das DTU 11.1 maßgeblich, das sich in Überarbeitung befindet. Die im Zuge der Überarbeitung erstellten Beiträge wurden 1983 in einem durchaus konsistenten Werk mit ca. 150 S. unter der Bezeichnung "Etude géotechnique et reconnaissance des sols" - Project de DTU herausgegeben.

Anders als die vorerwähnten DTU's sind in diesem Werk verbindlicher Text und Erläuterungen konsequent getrennt.

Zu den CPC gehört das "Fascicule 68 - Exécution des travaux de fondation d'ouvrages", in Kraft seit 1967 (2. Ausgabe 1982), das für staatliche Bauvorhaben verbindlich ist. Dort werden vorwiegend Vertragsbedingungen behandelt. Rein fachtechnische Fragen der Bemessung von Gründungen sind im "Fascicule Special 79-12" zusammengefaßt.

Das Laboratoire Central des Ponts et Chaussées (LCPC) und das Centre d'Etudes Techniques des Routes et Alloroutes (SETRA) geben gemeinsam eine Reihe von ausführlichen Empfehlungen (Documents LCPC - SETRA) heraus, auf die in Streitfällen durchaus Bezug genommen wird. Hierzu gehören: FOND 72 für Bodenerkundung sowie bezüglich Bemessung von Flach- und Tiefgründungen; MUR (1973) für Stützbauwerke; "Les Pieux Forés (1978)" für die Ausführung von Bohrpfählen; "Les ouvrages en terre armée" (1979) u.a.
Für seinen eigenen Bedarf hat das LCPC verschiedene Richtlinien für Versuchsdurchführungen und Berechnungsverfahren, die auch von anderen Behörden verwendet werden, ohne daß sie allgemeinverbindlichen Charakter hätten. Hierzu gehören die Richtlinien für Vorspannanker (1979) und für die Bemessung von Pfählen für Horizontallasten.

Alles in allem ist festzustellen, daß in Frankreich zur Zeit kein einheitliches und überschaubares System von Normen und anderen verbindlichen Richtlinien besteht.

2.3.4 Griechenland

Das Ministerium für öffentliche Arbeiten hat 1966 eine Richtlinie für Baugrunduntersuchungen für öffentliche Bauvorhaben herausgegeben, die sich in ihrem Versuchsteil an ASTM anlehnt.

Für Flach- und Tiefgründungen liegen Empfehlungen des gleichen Ministeriums vom Jahr 1958 vor, die sich an die DIN 1054 anlehnen und auch Mindestanforderungen für die Baugrunduntersuchungen enthalten. Die Anwendung der beiden Werke bei privaten Bauvorhaben ist üblich, jedoch nicht obligatorisch. Weitere DIN wie z.B. die DIN 4017, 4019 und 4125 werden auf optioneller Basis inoffiziell verwendet.
Das genannte Ministerium strebt die baldige Ausarbeitung und Einführung eines neuen Codes für das Gebiet der Geotechnik an.

2.3.5 Großbritannien

Die British Standards Institution gibt sowohl Codes of Practice (CP) als auch British Standards (BS) heraus. Laut Präambel ist in den CP's in der Form von Empfehlungen der Stand der Technik ("Good practice") niedergeschrieben. Die wichtigsten CP's auf dem Gebiet der Geotechnik sind

CP 2004 - Foundation, letzte Ausgabe 1972
CP 2003 - Earthworks, letzte Ausgabe 1981
CP 2 - Earth Retaining Structures, letzte Ausgabe 1982

Der CP 2001 - Site Investigation - befindet sich in Überarbeitung. Für Stahlbeton existieren zwei Codes - CP 110 (auf der Grundlage der Grenzzustände, mit Partialsicherheiten) und CP 114 (mit zul. Spannungen). Im CP 2004 wird für Gründungen die Bemessung nach zul. Spannungen empfohlen.
Die drei genannten Codes mit einem Gesamtumfang von fast 500 S. decken ziemlich das ganze Gebiet der Geotechnik ab, ohne die Sonderprobleme. Sie sind als Entscheidungshilfen für kompetente qualifizierte Ingenieure beim Entwurf und Bau entsprechender Grundbauwerke gedacht und sind in Inhalt und Form mehr deskriptiv als imperativ gehalten.

Formeln werden im CP 2 praktisch nicht, in CP 2003 und 2004 nur in begrenztem Umfang angegeben. Zahlenwerte kommen meistens als Richtwerte und nicht als verbindliche Forderungen vor; die Wahl des Sicherheitsbeiwertes bleibt praktisch dem Bearbeiter überlassen. Die Codes enthalten jedoch umfangreiche konstruktive Vorschläge und Details.

Obwohl die britischen Codes of Practice einen ähnlichen Status besitzen wie die DIN, spielen sie durch ihre allgemein gehaltenen Aussagen und Forderungen doch nicht die gleiche regulative und vereinheitlichende Rolle.

Für Labor- und Feldversuche liegt eine große Anzahl von British Standards vor.

2.3.6 Irland

Auf dem Gebeit der Geotechnik sind keine Regelwerke vorhanden. Die neuen "Draft Building Regulations" des Department of the Environment fordern für Gründungen die Befolgung der britischen Codes of Practice.

2.3.7 Italien

In Italien besteht kein einheitliches Normungssystem. Die Grundregeln für Baugrunduntersuchung, Entwurf, Bemessung und Bauausführung von Grundbauwerken sind*) im Ministerialerlaß (Decreto Ministeriale) LL.PP.DM, 1981 des Ministeriums für Öffentliche Arbeiten festgelegt.

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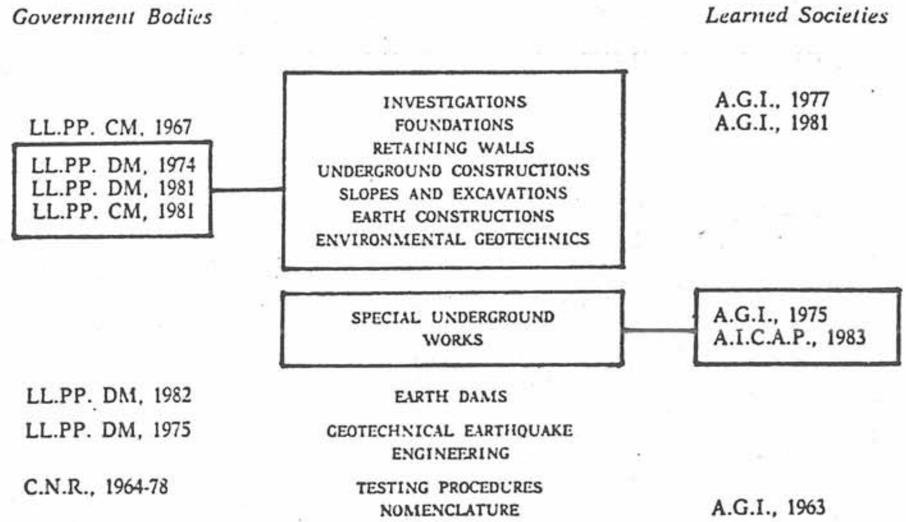
*) DECRETO MINISTERIALE 21 gennaio 1981
Norme tecniche riguardanti le indagini sui terreni e sulle rocce, la stabilità dei pendii naturali e delle scarpate, i criteri generali e le prescrizioni per la progettazione, l'esecuzione e il collaudo delle opere di sostegno delle terre e delle opere di fondazione.
Gazzetta Ufficiale R.I., S.O. n° 37, 7/2/1981.

Dieser Erlaß ist in der "Gazzetta Ufficiale" der Italienischen Republik veröffentlicht; er besitzt gesetzliche Kraft und gilt sowohl für öffentliche als auch für private Bauvorhaben. Der Geltungsbereich der Norm erstreckt sich fast auf das gesamte Gebiet der Geotechnik (s. auch Tab. 2.1) ohne Stauanlagen, für die es eine gesonderte Norm gibt. Die genannte Grundnorm strebt keine weitgehende Vereinheitlichung von Baugrunduntersuchung und Berechnungsverfahren an. Sie gibt vielmehr nur die Grundsätze wieder und beläßt dem Ingenieur ziemlich viel Spielraum. Es werden zwar Mindestwerte für die Sicherheitsfaktoren gefordert, die sich von den Werten der DIN 1054 kaum unterscheiden; da jedoch die Berechnungsverfahren und die Festlegung von Bodenkennwerten kaum behandelt werden, ist die Norm LL.PP.DM 1981 doch recht großzügig und unverbindlich. Das gleiche Ministerium gibt auch ministerielle Rundschreiben (LL.PP.CM) heraus, die keinen gesetzlichen Charakter haben, jedoch als Empfehlungen landesweit befolgt werden.
Das "Consiglio Nazionale di Ricerche" (CNR) hat zahlreiche Normen für bodenmechanische Versuche herausgegeben. Die Italienische Geotechnische Gesellschaft (Associazione Italiana di Geotechnica - AGI) hat Empfehlungen über Bodenbezeichnungen, Baugrunduntersuchungen (1977), Pfahlgründungen und Anker (Entwürfe 1981) herausgegeben.
Einige Regionen Italiens sind ebenfalls befugt - aufgrund besonderer natürlicher Gegebenheiten - eigene, zusätzliche Richtlinien auf geotechnischem Gebiet zu erlassen.
Tab.2.1 gibt eine grobe Übersicht des geotechnischen Regelwerkes in Italien.

TOPICS CONSIDERED IN ITALIAN CODES CONCERNING GEOTECHNICAL ENGINEERING



A.G.I.	Italian National Society of Soil Mechanics and Foundation Engineering and for Rock Mechanics, Viale Regina Margherita, 183 - 00198 Roma
A.I.C.A.P.	Italian Society for Prestressed Reinforced Concrete, Via G. Antonelli, 41 - 00197 Roma
C.N.R.	National Research Council, Piazzale A. Moro, 7 00185 Roma
LL.PP.	Ministry of Public Works, Porta Pia, 00100 Roma
DM	Decree, with a force of law
CM	Circular letter, with instructions

Tab. 2.1: Das italienische geotechnische Regelwerk - Übersicht

2.3.8 Niederlande

In den Niederlanden ist das Niederländische Normungsinstitut*) zuständig. Bisher sind noch keine nennenswerten Normen für das Gebiet der Geotechnik erschienen; eine Norm für Drucksondierungen - NEN 3680 wird jedoch demnächst veröffentlicht, eine Pfahlrichtlinie ist in Bearbeitung.

Vorgesehen ist auch eine grundlegende Norm für Gründungen, Erd- und Stützbauwerke, als Teil eines umfassenden Normenwerkes für das gesamte Bauwesen.

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*) auch für das geotechnische Regelwerk

2.3.9 Bundesrepublik Deutschland

Auf die Normungs- und Richtlinienverhältnisse in der BRD wird hier nicht eingegangen.

2.3.10 Zusammenfassung

Die Bestandsaufnahme der Regelwerke und des Richtlinienwesens der einzelnen EG-Länder hat eine außerordentliche Vielfalt sowohl hinsichtlich des Umfanges, Inhalts und Ausführlichkeitsgrades als auch hinsichtlich des Gewichtes und der praktischen Bedeutung der Regelwerke gezeigt. Dies ist umso verwunderlicher, als die Grenzen der EG-Länder untereinander für die wissenschaftlichen Erkenntnisse und Kontakte gar nicht existieren und auch für die Bauindustrie recht durchlässig geworden sind. Auch sind mehrere Kollegen Mitglieder von nationalen Normungs- und Empfehlungsausschüssen in jeweils anderen Ländern.

Da die unterschiedlichen Verhältnisse in ihren Ländern die Einstellung einiger Mitglieder der Arbeitsgruppe zu verschiedenen Problemen beim EUROCODE 7 offensichtlich und wohl zwangsläufig beeinflussen, war es erforderlich, diesem Phänomen nachzugehen. Es zeigte sich bald, daß hinsichtlich der technischen Kompetenzen und der Genehmigungsverfahren, also auf dem Gebiet des Baurechtes, ebenso große Unterschiede bestehen, die wiederum das Richtlinienwesen kausal beeinflussen. Um einen vorläufigen Überblick über die rechtliche Situation in den einzelnen Ländern zu erhalten, wurde unter den Mitgliedern der Arbeits-Gruppe eine Umfrage per Fragebogen durchgeführt, deren Ergebnisse in Anlage 1 zusammengestellt sind.

2.4 GLIEDERUNG UND AUFBAU VON EC 7

EC 7 gliedert sich in 10 Abschnitte, die in einen allgemeinen (Abschnitte 1 bis 5 und 10) und einen speziellen (Abschnitte 6 bis 9) Teil eingruppiert werden können. Tabelle 2.2 gibt eine Übersicht der Titel (in englischer und deutscher Sprache), der zuständigen Bearbeiter sowie des Bearbeitungsstandes der einzelnen Abschnitte.

Ursprünglich sollten im Abschnitt 5 die geometrischen Parameter behandelt werden. Später hat die Arbeitsgruppe jedoch beschlossen, diesem wohl recht begrenzten Problemkreis keinen gesonderten Abschnitt zu widmen. Dafür wurde als Abschnitt 5 in knapper Form die Behandlung von Erdbauten und Bodenverbesserungen im Kontext des EC 7 aufgenommen.

EC 7 setzt sich zusammen aus dem eigentlichen Code und aus Beilagen (annexes), (s. Tab. 2.3). Der Code wiederum besteht aus verbindlichem Text, Erläuterungen (guide) und Abbildungen bzw. Tafeln.

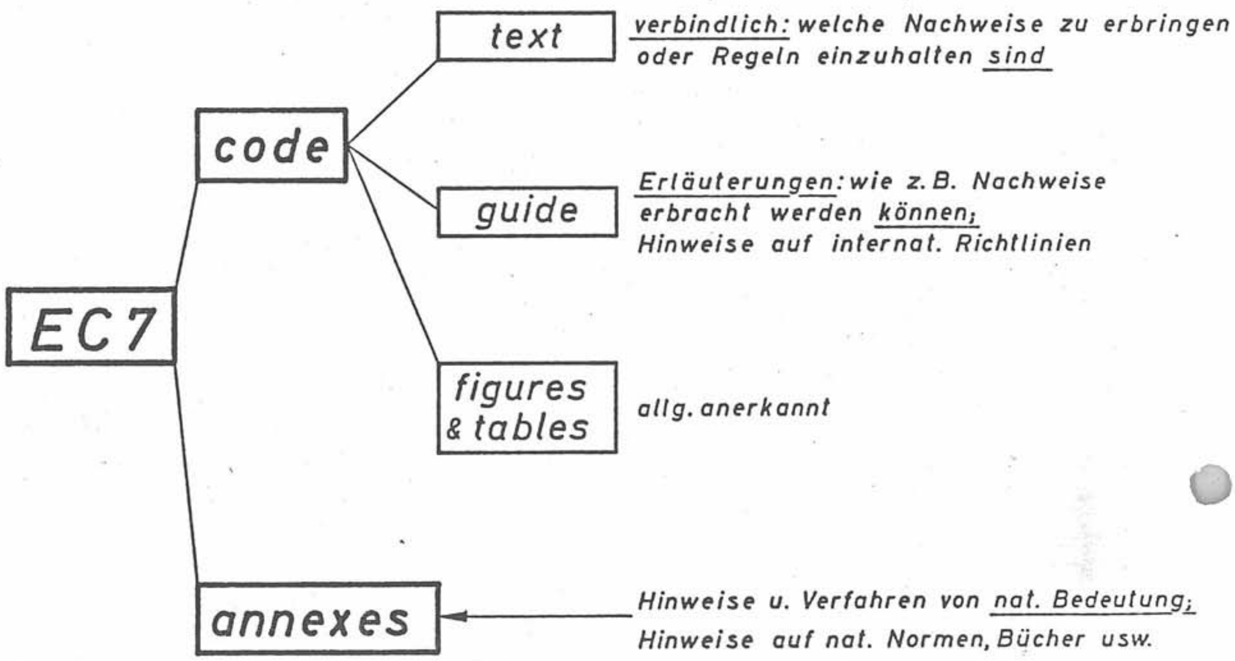
Im verbindlichen Text wird festgelegt, welche Anforderungen erfüllt bzw. Nachweise erbracht werden müssen.

Der Guide enthält Hinweise auf Verfahren, Vorgehensweisen u.ä. zur Erfüllung der Forderungen und Führung der Nachweise, die allgemein anerkannt sind und empfohlen werden, sowie weitere klärende Erläuterungen (Beispiel-Grundbruchformel). Allgemein anerkannte, klärende Darstellungen, Kurventafeln und Tabellen werden in begrenztem Umfang im Code im Rahmen der Erläuterungen (Guides) aufgenommen.

Nr.	Titel		Bearbeiter	Letzter Entwurf	
				Nr.	von
1.	General Principles	Grundsätze	K. Ovesen	F	4.84
2.	Verification of Safety and Serviceability	Nachweise der Standsicherheit u. Gebrauchsfähigkeit	K. Ovesen	F	4.84
3.	Actions	Einwirkungen	Amar/Baguelin Simpson	4	3.85
4.	Geotechnical Data	Geotechn. Eingangsdaten			
	4.1 Geotechnical Investigations	Baugrunderkundung	K. Ovesen	F	5.83
	4.2 Field Investigations	Feldversuche	Lousberg	~F	3.85
	4.3 Laboratory Tests	Laborversuche	Anagn./Coum.	~F, 6	8.84
	4.4 Evaluation of geotechnical Data	Auswertung von geotechnischen Parametern	Simpson	2	3.85
	4.5 Geotechnical Reporting	Baugrundgutachten	Anagn./Coum.	F	12.83
5.	Earthworks & Ground Improvement	Erdbau- u. Bodenverbesserung	Orr/Simpson	1	7.85
	Verification Procedures for:	Nachweise für:			
6.	Spread Foundations	Flächengründungen	Orr	F, 10	6.84
7.	Piles	Pfahlgründungen	Heijnen	4	9.85
8.	Retaining Structures	Stützbauwerke	Sadg.	~F, 6	7.84
9.	Slopes	Böschungen	Sadg.	~F, 3	12.84
10.	Construction Control	Bauüberwachung	Japelli	6	9.85

TAB. 2.2: EUROCODE 7 - GLIEDERUNG UND BEARBEITUNGSSTAND ZUM 1.11.1985

Weitere Schilderungen von oder Hinweise auf rechnerische bzw. experimentelle Verfahren, Zahlen- und Kurventafeln, die aufgrund ihres geringeren Bekanntheitsgrades oder begrenzter, bodenspezifischer Gültigkeit nur in einigen Ländern (oder Regionen) der EC gebräuchlich oder anerkannt sind, können in begrenztem Umfang in den annexes beigefügt werden. (Beispiel: Pressiometer-Verfahren zur Bestimmung der Grundbruchlast).



Tab. 2.3: Schematische Gliederung von EC 7

2.5 GEGENWÄRTIGER BEARBEITUNGSSTAND UND WEITERE BEHANDLUNG

Die ersten Abschnittsentwürfe des EC 7 trugen naturgemäß den Stempel der Normungsgepflogenheiten und der Praxis im Lande des jeweiligen Bearbeiters (s. Tabelle 2.2)*). Im Zuge der weiteren Bearbeitung wurde bis zur 16. Sitzung in München eine weitgehende Harmonisierung sowohl in den Aussagen als auch hinsichtlich des Ausführlichkeitsgrades erreicht. Dabei mußte aber in Kauf genommen werden, daß die Ausführungen, vor allem der Spezialabschnitte 6. bis 9., zu wenig konkret wurden und in der Praxis in vielen Fällen kaum direkt und ohne weitere Festlegungen anwendbar sein dürften.

Tab. 2.2 gibt auch den Bearbeitungsstand der einzelnen Abschnitte zum 1.11.1985. Daraus ist zu ersehen, daß die meisten Abschnitte, zumindest was ihre Aussagen anbetrifft, einen Reifezustand erreicht haben, der zunächst keine weitere Diskussion innerhalb der Arbeitsgruppe erfordert (Zeichen "F" in Tab. 2.2). Lediglich an den Abschnitten 3, 7 und vor allem 5 sind noch weitere Inhaltskorrekturen notwendig.

Um im Frühjahr 1986 entsprechend den Wünschen der EGK einen 1. Gesamtentwurf für EC 7 verabschieden zu können, hat sich der Obmann entschlossen, einen britischen Ingenieur, Herrn H. Roscoe von Fa. Ove Arup & Partners, mit der sprachlichen und formalen Überarbeitung der inhaltlich fertigen Abschnitte zu beauftragen. Obwohl gegen dieses Vorgehen verschiedene Bedenken geäußert wurden, gab die Arbeitsgruppe während der 16. Sitzung ihre Zustimmung dazu. Herrn Roscoe Vorschläge unterliegen der Beurteilung und evtl. Annahme durch den

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*) so. z.B. waren die ersten Fassungen der Abschn. 8 und 9 sowie der Unterabschnitt 6.6, bearbeitet von Dr. Sadgorski, stark an die DIN 4019, 4084 und 4085 angelehnt.

zuständigen Bearbeiter bzw. durch die ganze Arbeitsgruppe.

H. Roscoe nahm seine Arbeit im Oktober/November 1985 mit den Überarbeitungsvorschlägen zu den Abschnitten 8 und 9 auf. Bis Ende 1985 sollen die meisten weiteren Abschnitte folgen, so daß zur 17. Sitzung der Arbeitsgruppe am 23./25.1.86 in Rom ein kompletter Entwurf in Reinschrift *) vorliegen kann. Nach der Diskussion dieses Entwurfes in Rom sollen vereinbarte Änderungen so rasch eingebracht werden, daß ein aktualisierter Entwurf zum 1.3.86 den (zu diesem Zeitpunkt schon 11) nationalen Gesellschaften der EG-Länder zugesandt werden kann. Dabei werden die Gesellschaften um erste, vorläufige und noch inoffizielle Stellungnahmen bis 1.6.86 gebeten, in denen vorwiegend auf grundsätzliche Aspekte eingegangen werden soll.

Diese Stellungnahmen sollen während der 18. Sitzung der Arbeitsgruppe am 12./13.6.86 in Kopenhagen diskutiert werden, wobei zu entscheiden sein wird, wie weit sie berücksichtigt werden. Daraufhin werden die einzelnen Abschnitte erneut von den zuständigen Bearbeitern revidiert und der in einem Schreibautomaten in Kopenhagen gespeicherte Text des Entwurfes des EC 7 wird erneut aktualisiert. Diese Fassung wird dann als erster offizieller Entwurf Anfang August 1986 der EGK vorgelegt.

Dieses ziemlich gedrungene Arbeitsprogramm für das Jahr 1986 ergibt sich aus dem Wunsch der EGK, den offiziellen Entwurf für den EUROCODE 7 möglichst bald zu erhalten, um ihn einer Behandlung zu unterziehen, wie dies bereits mit EC 1, EC 2, EC 3 und EC 8, Teil 1 geschehen ist bzw. gegenwärtig geschieht.

*) der Entwurf wird nach seinem Erscheinen diesem Abschlußbericht als Anlage 2 beigefügt.

2.6 BESONDERHEITEN IM INHALT DES EC 7

Der Entwurf des EC 7 unterscheidet sich inhaltlich vom traditionellen Normenwerk der deutschsprachigen Länder in folgenden wichtigen Punkten:

1. Klare Formulierung von Grenzzuständen
2. Ableitung von Bemessungswerten der Basisvariablen nach der Methode der Teilsicherheitsbeiwerte. Zu den Basisvariablen zählen:
   a) Einwirkungen, die entweder Nutzlasten oder aufgezwungene Verschiebungen sein können,
   b) Eigenschaften des Bodens und anderer Baustoffe,
   c) geometrische Parameter,
   d) Randbedingungen, z.B. Verformungsbedingungen.
3. Einführung der "Geotechnischen Kategorien".

Die Formulierungen der Grenzzustände der Tragfähigkeit und der Gebrauchsfähigkeit in EC 7 entsprechen allgemein den Grundsätzen der GruSiBau (1981), Abschn. 5.1 und 5.2 sowie des EC 1. Eine Besonderheit von EC 7 ist jedoch die Unterteilung des Grenzzustandes der Tragfähigkeit in zwei Typen und zwar:

Typ 1A - bei dem ein Bruchmechanismus im Baugrund entsteht, und

Typ 1B - bei dem ein Bruchmechanismus im Bauwerk entsteht auf Grund von Bewegungen (ohne Bruchmechanismus) im Baugrund.

In den Spezialabschnitten 6. bis 9. sind die Mechanismen herausgestellt, die beim jeweiligen Grundbauwerk zu einem der Grenzzustände führen können.

Die Grundgleichungen der geotechnischen Bemessung in EC 7 und die Ableitung der Bemessungswerte aus charakteristischen Werten mit Hilfe von Teilsicherheitsbeiwerten entsprechen ebenfalls den Grundsätzen des EC 1 und der GruSiBau (1981). Zahlenwerte für die Teilsicherheitsbeiwerte, die das Sicherheitsniveau festlegen würden, werden in EC 7 nicht angegeben. Ihre Festlegung bleibt den einzelnen Ländern vorbehalten; die für jedes EG-Land gültigen Werte werden in "nationalen Anhängen" zu EC 7 aufgeführt.

Um bestimmte Mindestanforderungen für den Umfang und die Qualität geotechnischer Untersuchungen, Berechnungen und Bauausführungsüberwachungen aufstellen zu können, ist es sehr zweckmäßig, zunächst Schwierigkeitsgrad und Komplexität jedes geotechnischen Problems klar herauszustellen. Zu diesem Zweck werden in EC 7 entsprechend der dänischen Praxis drei "geotechnische Kategorien" festgelegt.

Die folgenden Faktoren sind zu berücksichtigen, wenn für eine bestimmte Situation die entsprechende geotechnische Kategorie festgesetzt wird:

a) Art und Größe des Bauwerkes und seiner Teile

b) besondere Verhältnisse auf der Baustelle und ihrer Umgebung (benachbarte Bauwerke, Verkehr, öffentliche Einrichtungen usw.)

c) Baugrundverhältnisse

d) Grundwasserverhältnisse

e) regionale Erdbebentätigkeit

f) Einfluß der Umgebung (Hydrologie, Oberflächenwasser, Senkungen usw.).

Der Grundgedanke der geotechnischen Kategorien ist an sich nicht neu. Auch bisher war es üblich, den Umfang und die Qualität der Baugrunduntersuchungen, der rechnerischen Analysen und der Bauüberwachung dem Schwierigkeitsgrad des Problems anzupassen. Dies geschah jedoch meistens intuitiv oder nach der Erfahrung und der Meinung des Bearbeiters im Rahmen der örtlichen Gepflogenheiten, ohne eine zwingende Objektivierung der maßgebenden Aspekte. Die Einführung der geotechnischen Kategorien zwingt zu einer Quantifizierung der Situation unter Berücksichtigung aller Randbedingungen und schafft damit eine solide Grundlage für die richtige Festlegung der Maßnahmen für Untersuchungen und Baubegleitung. Dabei ist freilich nicht an ein starres Schema gedacht; vielmehr sind die geotechnischen Kategorien als Koordinatenmarkierungen im Koordinatensystem U = f (S) zu verstehen (mit U = Untersuchungsumfang und S = Schwierigkeitsgrad). Interpolationen in diesem Koordinatensystem werden sich häufig als angebracht oder notwendig erweisen.

2.7 VERGLEICH MIT DEUTSCHEN REGELWERKEN

Schon bei einer ersten, flüchtigen Gegenüberstellung der in der Bundesrepublik auf dem Gebiet der Geotechnik geltenden Normen, Empfehlungen und weiteren Regelwerke stellt man erhebliche Unterschiede sowohl in der Ausdehnung des Geltungsbereiches als auch im Grad der Ausführlichkeit und der Verbindlichkeit fest. Um den Vergleich von EC 7 mit den deutschen Regelwerken zu erleichtern, wird vorgeschlagen, den Begriff "Normungsstufe" einzuführen und damit auch die drei Normungsstufen:

1. Rahmennorm
2. Anforderungsnorm
3. Durchführungsnorm.

In Tabelle 2.4 sind Vorschläge für die Definitionen dieser Normungsstufen gemacht. Dort wird auch eine Zuordnung des deutschen geotechnischen Normenwerkes sowie einiger anerkannter Empfehlungen in ein übersichtliches Schema vorgenommen,welches so gut wie alle gängigen Probleme der Geotechnik abdeckt. In der letzten Spalte der Tabelle 2.4 sind die entsprechenden Abschnitte des EC 7 aufgeführt.

Es ist evident, daß das historisch, im Laufe von fast vier Jahrzehnten entstandene DIN-Normenwerk auf dem einschlägigen Gebiet vor allem bei den Anforderungsnormen kein systematisches Bild präsentiert. Zudem ist die Grenze zwischen Anforderungs- und Durchführungsnormen häufig unscharf. Dies kann in der Praxis zur irrigen Vorstellung führen, daß die existierenden Durchführungsnormen (Stufe 3) alle Aspekte ansprechen oder gar gründlich behandeln, die für das entsprechende Grundbauwerk (Punkte E bis F der Spalte 3) relevant sind und dabei mitunter fehlende Anforderungsnormen ersetzen.

Noch unsicherer ist die Situation bei den Bauverfahren (Punkte G und H der Spalte 3) und der Baukontrolle (Punkt I), für die eher unzureichende oder gar keine verbindlichen normativen Festlegungen vorliegen.

Demgegenüber deckt EC 7 systematisch wohl den ganzen Bereich der gängigen geotechnischen Problematik auf der Ebene der Anforderungsnormen ab und liefert damit ein überzeugendes und methodisch einwandfreies Konzept. Die Ebene der Durchführungsnormen wird dagegen vom EC 7 in seinem gegenwärtigen Entwurfsstadium entweder nur sehr flüchtig oder gar nicht verfolgt. Vorschläge des Berichters, wenigstens völlig unstrittige Verfahren in die Erläuterungen (guide) aufzunehmen, fanden nicht die Zustimmung der Arbeitsgruppe. Der häufigste Einwand war, daß EC 7 kein Lehrbuch sei und der Ingenieur, der einen Nachweis zu erbringen hat, wissen müsse, wie dies zu geschehen hat. In dieser Einstellung spiegeln sich freilich die Gepflogenheiten im Normungswesen und hinsichtlich der technischen Kompetenzen der meisten EG-Länder wieder.

Normungsstufe (Detaillierungsgrad)	Inhalt	Gliederung	Behandlung in DIN	Behandlung in weit. Regelwerke	Behandlung in EC 7
1. Rahmen-Norm	Grundlagen mit allgemeinen, jedoch sehr verbindlichen Festlegungen für die gesamte Geotechnik		1054		Ch. 1 + 2
2. "Anforderungsnormen"	Einigermaßen detaillierte Anforderungen an Baugrunderkundung, Standsicherheits- und Gebrauchsfähigkeitsnachweise, sowie Bauausführung für die verschiedenen Grundbauwerksarten	A) Baugrunderkundung	4020		4
		B) Lastannahmen	1055	EAU	3
		C) Flachgründungen	(1054/4)		6
		D) Pfahlgründungen	4014,4026,4128		7
		E) Stützbauwerke	– (4124,4126,4125)	EAU, EAB	8
		F) Böschungen	–	Empfehlungen Böschgn EAU, Küste	9
		G) Erdbau	–	ZTVE	5
		H) Bodenverbesserung	– (4093)		5
		I) Baukontrolle und Aufsicht	–		10
3. "Durchführungs-Normen"	Sehr detaillierte Festlegungen für einzelne Versuchsarten und rechnerische Nachweise	A)	4021,4022,18196,VN		
		B)	– (1055)		
		C)	4017,4018, 4019		–
		D)	4014		
		E)	4084, 4085		–
		F)	4084		–
		G)			–
		H)	–		–
					–

VN = Versuchsnormen

Tab. 2.4 : Vergleichende Darstellung von Regelwerken

2.8 KOORDINIERUNGSBEMÜHUNGEN IN DER BUNDESREPUBLIK DEUTSCHLAND UND DIE ERWEITERTE EC 7-SITZUNG IN HELSINKI

In der Bundesrepublik Deutschland sind viele Ausschüsse mit einer sehr großen Anzahl von Mitgliedern auf dem Gebiet der Geotechnik tätig. Zwar bestand ursprünglich Einigkeit darüber, daß der Ausschuß Baugrund/Berechnungsverfahren als Spiegelausschuß zu EC 7 bei der Ausarbeitung von Textvorschlägen und bei der Beurteilung der von anderen Mitgliedern der Arbeitsgruppe ausgearbeiteten Entwürfe mitwirkt. Zur zweckmäßigen Behandlung einzelner Abschnitte war es jedoch angebracht und sogar erforderlich, auch die dafür zuständigen weiteren DIN-Ausschüsse und Arbeitskreise der DGEG mit einzubeziehen. Dies waren zunächst der Ausschuß für DIN 4020 und der Ausschuß für Pfähle; ferner die Ausschüsse Ufereinfassungen und Baugruben. Seitens der Obmänner und Mitglieder der genannten Ausschüsse kamen mehrere weitere wertvolle Hinweise und Hilfeleistungen.

Zur beschleunigten und wirksameren Abwicklung der Koordinierung von EC 7 wurde im Herbst 1983 innerhalb des Ausschusses Baugrund/Berechnungsverfahren ein Unterausschuß gebildet. Ihm gehören die Herren Dr. Sadgorski (Obmann), Dr. Demharter, Prof. Franke, Dr. J. Hanisch, Dr. H. Schulz und Dr. K. Weiß an. Dieser Unterausschuß kam am 4.4.1984, vor einer Sitzung des Gesamtausschusses, in Lübeck zusammen. Die weitere Koordinierung geschah auf schriftlichem Wege.

Die Fachöffentlichkeit in der Bundesrepublik Deutschland wurde durch mehrere Aufsätze im Organ der DGEG GEOTECHNIK über die Bearbeitung des EC 7 und über seine Besonderheiten informiert (SMOLTCZYK 1979 und 1980, SADGORSKI 1983 und 1984).

Im Rahmen der 8. Europ. Konferenz für Bodenmechanik und Grundbau in Helsinki war für den Nachmittag des 25.5.1983 eine Sitzung der Arbeitsgruppe des EC 7 mit einer größeren Anzahl (ca. 25) weiterer Fachleute aus den EG-Ländern vorgesehen.

Zur Vorbereitung dieser Sitzung fand am 27. April 1983 in München eine Besprechung mit Teilnahme der Herren Prof. Smoltczyk, Prof. Horn, Prof. Gudehus, Prof. Kany, Dr. Hanisch, Dr. Weiß, von Soos und Dr. Sadgorski statt. Dabei wurden die damals gültigen Entwürfe der Abschnitte 1,2,4 und 6 kommentiert und die Haltung der deutschen Vertreter bei der erweiterten Sitzung in Helsinki vorbesprochen.

Die letztgenannte Sitzung wurde unter Vorsitz von Herrn Prof. Smoltczyk abgehalten. Nach Einführungen von ihm und dem Obmann der EC 7-Arbeitsgruppe Prof. N. Krebs Ovesen haben Sprecher aus den EG-Ländern*) Stellung zu den vorgelegten vier Abschnitten genommen. Dabei und bei der anschließenden allgemeinen Diskussion wurde die durchaus positive Bewertung des eingeschlagenen Weges und der von der Arbeitsgruppe geleisteten Arbeit seitens der meisten nationalen Gesellschaften offenkundig. Allgemein war allerdings der Wunsch, den Umfang des Werkes durch Beschränkung auf das Wesentliche nicht ausufern zu lassen und sowohl auf triviale Hinweise als auch weitgehend auf Einzelheiten über Berechnungsverfahren zu verzichten.

Am 17.9.84 fand, unmittelbar vor der Baugrundtagung in Düsseldorf, eine Sondersitzung der DGEG über EC 7 statt. Auf dieser Sitzung wurden Stellungnahmen der Obmänner der jeweils zuständigen Ausschüsse (bzw. ihrer Vertreter) zu den damals vorliegenden Fassungen der einzelnen Abschnitte vorgetragen. Diese Stellungnahmen waren, soweit möglich, in den Ausschüssen bereits diskutiert worden.

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*) Die DGEG war durch die Herren Prof. Franke, Prof. Gudehus, Prof. Kany, Dr. Hanisch, von Soos und Dr. Weiß vertreten.

Zur Vorbereitung der Sondersitzung hatte die DGEG die deutsche Übersetzung der Abschnitte 1 und 2 wie auch vorläufige Kurzfassungen der Vortragsmanuskripte in Form einer Broschüre als "Materialien ..." (DGEG 1984) allen Teilnehmern übersandt bzw. überreicht.

An der Sondersitzung beteiligten sich 106 Teilnehmer; über den Ablauf und über die vorgetragenen Einwände und Empfehlungen haben HANISCH/SADGORSKI (1984) ausführlich berichtet. Dieser Aufsatz wurde von der Britischen Geotechnischen Gesellschaft in die englische Sprache übersetzt und denjenigen von ihren Mitgliedern, die dafür Interesse bekundet hatten, zugestellt. Auch alle Mitglieder der Arbeitsgruppe für EC 7 erhielten entweder den Originalaufsatz oder seine englische Übersetzung.

2.9 KOORDINIERUNGSBEMÜHUNGEN IN DEN ANDEREN EG-LÄNDERN

In den weiteren acht EG-Ländern vollziehen sich die Schritte zur Information der Fachöffentlichkeit bzw. zur Koordinierung zwischen der nationalen Normung und EC 7 auf sehr unterschiedliche Weise.

In Belgien, Griechenland und den Niederlanden sind Ausschüsse zur Begleitung der Tätigkeit für EC 7 gegründet worden, die mit unterschiedlicher Intensität die vorliegenden Entwürfe durchsehen und kommentieren. Besonders aktiv war bisher der belgische Ausschuß mit 8 Mitgliedern und Prof. de Beer als Obmann. Dieser Ausschuß hatte am 17.5.1984 in Louvain-la-Neuve eine gemeinsame Sitzung mit der Arbeitsgruppe für EC 7. Dem Berichter ist nicht genau bekannt, welche Breitenwirkung die Tätigkeit der Begleitausschüsse in diesen drei Ländern erreicht hat und in welcher Form eine Übernahme von EC 7 dort beabsichtigt ist.

In Italien hat Prof. Japelli bei verschiedenen Tagungen über die Entwicklung des EC 7 berichtet (z.B. JAPELLI 1983 und JAPELLI/VALORE 1983). Nach den Vorträgen haben stets Diskussionen stattgefunden. Während der Inhalt von EC 7 in diesem Lande im großen und ganzen positiv aufgenommen wird, scheint es noch unklar zu sein, auf welche Weise der Code in das italienische Richtlinienwesen implementiert werden kann.

Die British Geotechnical Society (BGS) bemühte sich sehr intensiv um die frühzeitige Unterrichtung ihrer Mitglieder über die Arbeiten für EC 7 und organisierte bereits am 12.5.83 in London eine Sitzung über die Abschnitte 1, 2, 4 und 6. Mehr als 100 Mitglieder nahmen daran teil und eine Sitzungsnotiz wurde den Mitgliedern der Arbeitsgruppe für EC 7 überreicht.

Ferner wurde in mehreren Aufsätzen in der Zeitschrift "Ground Engineering " über EC 7 berichtet und zu verschiedenen Punkten Stellung genommen (BOLTON 1983, SIMPSON 1983).

Eine weitere Sitzung der BGS fand am 22.5.85, unmittelbar vor der EC 7-Sitzung in London statt. Dazu hatte man vielen Mitgliedern auf Anforderung den derzeitigen Entwurf von EC 7 zur Verfügung gestellt und insgesamt 7 Kollegen aufgefordert, nach Studium entsprechender Abschnitte bei der Sitzung Stellung zu nehmen. Die Mitglieder der Arbeitsgruppe für EC 7 waren zu dieser Sitzung eingeladen und aufgefordert, sich zu der Kritik aus den Reihen der BGS zu äußern.

An der Sitzung, die in der Institution of Civil Engineers stattfand, haben etwa 50 britische Kollegen teilgenommen. Herr Driscoll vom Building Research Establishment berichtet über Ablauf und Ergebnisse dieser Sitzung (DRISCOLL 1985). Obwohl die britischen Kollegen generell jeder Normung recht reserviert gegenüberstehen*), scheint EC 7 in seiner jetzigen Form allmählich akzeptiert zu werden. Lediglich der Widerstand gegen festgelegte Sicherheitsbeiwerte ist noch ziemlich ungebrochen.

Da bei der Bearbeitung von EC 7 die dänische Grundbaunorm Pate stand, ist eine besondere Koordinierung in Dänemark nicht erforderlich. Nach Äußerungen dänischer Kollegen ist beabsichtigt, die dänische Norm durch EC 7 nach seiner Einführung zu ersetzen.

Über Öffentlichkeitsarbeit für EC 7 und Koordinierungsbemühungen in Frankreich und Irland ist dem Berichter nichts bekannt.

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*) Außerhalb des offiziellen Berichtes ein Zitat aus FAREBROTHER (1983), der angeblich die Stimmung unter den Kollegen vom Stahlbetonfach wiedergibt:

" Our Campaign, however, is not directed against CP110 alone. It is directed against all Codes, and if it is successful (as we are sure it will be) it will result in considerable rethinking for all Codes. ...

...Remember, Euro Codes are just around the corner, and unless the brake is applied hard now we will find ourselves committed to even greater complexity in the future than that which exists at present. "

2.10 AUSBLICK UND ANREGUNGEN FÜR DIE DEUTSCHE NORMUNG

Mit der Verabschiedung eines kompletten Entwurfes, die für die 18. Sitzung der Arbeitsgruppe am 12./13.6.86 in Kopenhagen vorgesehen ist, wird die erste Bearbeitungsphase des EUROCODE 7 abgeschlossen sein (s. Abschnitt 2.5). Darauf wird eine Periode des Kennenlernens dieses Codes und der Auseinandersetzung mit seinem Inhalt in den einzelnen Ländern der EG folgen. Nach den bisherigen Reaktionen in der Kollegenschaft ist wohl mit einer grundsätzlichen Zustimmung zum Geiste und zu den Einzelheiten des EC 7 in den meisten Ländern zu rechnen, wobei allerdings nach den Beobachtungen des Berichters die Methode der Teilsicherheitsbeiwerte am ehesten auf Widerstand stoßen würde.

Es ist zunächst sehr schwer vorauszusagen, welcher Stellenwert dem EC 7 als Gesamtwerk in den einzelnen Ländern zugewiesen wird. Dabei dürften die jeweiligen Normungsgepflogenheiten und die rechtlichen Verhältnisse eine nicht unerhebliche Rolle spielen. In Dänemark und höchstwahrscheinlich auch in Griechenland sowie Irland werden die nationalen Fassungen, d.h. die Übersetzungen des EC 7 in die Nationalsprachen, versehen mit nationalen Anlagen (Annexes s. Tab. 2.3) zu alleinigen Grundbaunormen erhoben. Ähnliches Vorgehen wäre z.T. auch in Belgien und Holland denkbar, wobei hier sicher durch zusätzliche nationale Normen oder nationale Anlagen den geologischen Besonderheiten und der Tiefbautradition (Pfahlgründungen) Rechnung getragen wird. Unter Umständen wird man auch in Portugal ähnlich vorgehen.

In der Bundesrepublik Deutschland, Frankreich, Großbritannien und Italien bestehen bereits feste aber (bedauerlicherweise) sehr unterschiedliche Normungstraditionen (s. Abschn. 2.3). Dabei scheint das jetzige DIN-Normenwerk einer Einführung des EC 7 noch am wenigsten entgegenzukommen (vgl. Abschn. 2.7).

Die EG-Kommission strebt in einer ersten Harmonisierungsphase eine "optionelle Harmonisierung" der Bauvorschriften an, d.h. die Anwendung der EUROCODES als Alternativen zu den bestehenden nationalen Regelwerken (s. z.B. STILLER/LITZNER 1984). Diese Lösung ist für die Bundesrepublik Deutschland auf dem Gebiet der Geotechnik nach Meinung des Berichters kaum praktikabel. Wohl aber böte sich die Möglichkeit, eine deutsche Fassung des EC 7 in der BRD als eine Art Anforderungsnorm einzuführen, eine günstige Gelegenheit, aufeinander wohlabgestimmte, konsequente und übersichtliche Kriterien für die Standsicherheitsbeurteilung aller Arten von Grundbauwerken zu etablieren. Vorhandene oder in Bearbeitung befindliche DIN mit Anforderungscharakter (z.B. DIN 4014, 4026 und 4028 für Pfähle und anderes) könnten nach der allenfalls erforderlichen inhaltlichen Anpassung als verbindliche Supplementa (Annexes) diesem "DIN-EC 7" beigefügt werden. Es wäre zu überlegen, ob man nicht auch die DIN 1054 dadurch ersetzen könnte, die im Zuge der Einführung von Teilsicherheitsbeiwerten ohnehin gründlich zu überarbeiten wäre.

Ob dann die bestehenden Durchführungsnormen für Berechnungen (Tab. 2.4, Buchst. C) bis F)) eine Statusänderung erfahren und evtl. in 3 oder 4 Normen zusammengefaßt werden sollen, wird noch zu überlegen sein.

Als eine weitere Möglichkeit bietet sich die schrittweise Harmonisierung zwischen EC 7 und einigen DIN, die sich in Neu- oder Überarbeitung befinden, an. Als ein gutes Beispiel sei die gute Übereinstimmung zwischen der DIN 4020 (Entwurf 1985) und dem Abschnitt 4. des EC 7 erwähnt.

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Teil 3

FINANZIERUNG UND ABWICKLUNG DES FORSCHUNGSAUFTRAGS

Die Durchführung des Forschungsvorhabens "Internationale Vereinheitlichung technischer Baubestimmungen - Grundbau" wurde vom Institut für Bautechnik auf Grund des Vertrages vom 14.5.1980 Az.: IV/1-5-236/80 der Deutschen Gesellschaft für Erd- und Grundbau (DGEG) übertragen. Das Institut für Bautechnik übernahm die Finanzierung, die zunächst auf 1 Jahr begrenzt war, jedoch auf Antrag der DGEG mehrmals erweitert wurde (Erweiterungsbewilligungen vom 26.3.81, 30.11.81 und 17.5.83).

Im Rahmen des Forschungsvorhabens beschäftigte die DGEG etwa vier Jahre lang den britischen Ingenieur Herrn T.N. Thorp, der mit der Vorbereitung und Abwicklung der Leistungen gemäß der Leistungsbeschreibung zum o.g. Vertrag betraut war.

Die direkte Vertretung und Mitarbeit in der Arbeitsgruppe für EC 7 übernahm Dr. W. Sadgorski vom Bayer. Landesamt für Wasserwirtschaft, München (s.a. Abschnitt 2.1). Diese Tätigkeit von Dr. Sadgorski wurde von der vorgesetzten Behörde unter der Bedingung genehmigt, daß dem Landesamt keine Kosten für Dienstreisen u.ä. entstehen. Daher wurden alle Reisekosten für seine Teilnahme an den Sitzungen der Arbeitsgruppe für EC 7 von der DGEG im Rahmen der Abwicklung des Forschungsauftrages bestritten.

An dieser Stelle wird dem Institut für Bautechnik für die Übernahme der Finanzierung, die eine überdurchschnittlich aktive Beteiligung der DGEG an den internationalen Harmonisierungsbemühungen auf dem Gebiet der Geotechnik ermöglichte, verbindlich gedankt.

SUMMARY

The "Deutsche Gesellschaft für Erd- und Grundbau e.V." (=German Society for Soil Mechanics and Foundation Engineering, abbrev. DGEG) is the professional organisation of the engineers in the Federal Republic of Germany, working in the field of geotechnical engineering. It belongs to the activities of this society to participate in the elaboration of multinational and international regulations in this field.

The first part of the report deals with the DGEG's collaboration in the work of the committees of the ISSMFE, such as for Penetration Testing, Subsoil Exploration, Sampling, Geomechanical Software and Terminology, further of the many committees of the ISRM.

The object of the second part of the report is the work on a "model code" for EUROCODE 7 for foundations, retaining structures and earthworks by an ad-hoc committee of the nine member societies of the ISSMFE for the EC-countries. Professor N.Krebs Ovesen of Denmark is chairman of this committee, which was established in 1981 and consists of other 8 official representatives, 2 deputy (alternative) members and a secretary. Up to the end of 1985 16 meetings of the committee took place and 3 further meetings are to be held in 1986. A survey of the national regulations of the 9 member countries was done which showed, that the situation d ffers significantly from one country to the others, the main points of the survey's results are given.

In the report a review is made over the headlines and the present stage of elaboration of EC 7. Then the main specialities of the code's content-limit states, partial safety factors and geotechnical categories-are discussed and a comparison between EC 7 and the present German geotechnical regulations is done. Finally, the propagation of the contents and the ideas of EC 7 in the countries of the community and the discussion of it are reported and some recommendations for its implementation in the Federal Republic of Germany are made.

RESUMÉE

La "Deutsche Gesellschaft für Erd- und Grundbau e.V." (= Société Allemande de Mécanique des Sols et des Travaux de Fondation, abbrév. DGEG) est la société professionelle des ingénieurs allemands travaillante dans la domaine de la géotechnique. Une des activitées les plus importantes de cette société est la participation et élaboration des régles et documents techniques internationaux dans cette domaine.

La première partie de ce rapport traite le travail de la DGEG dans les comitées de la ISSMFE, ce sont les comitées pour: les essais pénétrometriques, la reconnaissance des sols, le prélévement des échantillons, la software dans la géomécanique et la terminologie, aussi que dans tous les comités de la ISRM.

L'objet de la seconde partie du rapport est l'élaboration du EUROCODE 7 pour des fondations, des ouvrages de soutènement et des travaux de terrassement par un comitée "ad-hoc" des 9 sociétés membres de la ISSMFE des pays des Communautés Européennes. Le président du comitée, etabli en 1981, est le professeur N. Krebs Ovesen de Danemark, le comité se compose d'autres 8 membres officiaux, de 2 membres alternatifs et d'un secretaire. Jusqu'â la fin de 1985 le comité a eu 16 réunions; autres 3 réunions seront réalisées en 1986.

Une recherche des réglements normatifs officiaux des 9 pays était procurée, que demontre des différences significantes. Les résultats les plus importants de cette recherche sont rapportés.

Le rapport traite les éléments les plus importants et la situation présente d'élaboration du EC 7. Il y suive l'éxplication des particularités du contenu du code - états limites, coéfficients de sécurité partiels et catégories géotechniques - et un parallele entre EC 7 et les normes géotechniques allemands. En fin, la publication du contenue de EC 7 et la coordination avec des comités de normalisation dans les pays des CE sont rapportés et quelques idées pour sa future implementation dans la RF d'Allemagne sont proposées.

ZUSAMMENFASSUNG

Die Deutsche Gesellschaft für Erd- und Grundbau e.V. (DGEG) ist der Berufsverband der auf dem Gebiet der Geotechnik tätigen Ingenieure in der Bundesrepublik Deutschland. Zu den Aufgaben der DGEG gehört auch die Mitwirkung bei der Ausarbeitung internationaler Richtlinien auf dem einschlägigen Fachgebiet.

Der erste Teil des Berichtes behandelt die Mitwirkung der DGEG in den Ausschüssen der Internationalen Gesellschaft für Bodenmechanik und Grundbau (ISSMFE) für Sondierungen, Baugrunderkundung, Probenentnahme , Software in der Geotechnik und Terminologie, ebenso wie die Mitwirkung in einer großen Anzahl von Arbeitsgruppen der Internationalen Gesellschaft für Felsmechanik.

Der Gegenstand des zweiten Teiles des Berichtes ist die Ausarbeitung eines "model code" für den EUROCODE 7 für Gründungen, Stützbauwerke und Erdarbeiten durch eine gemeinsame Arbeitsgruppe der 9 nationalen Gesellschaften für Bodenmechanik und Grundbau der Mitgliedsländer der Europäischen Gemeinschaft. Obmann dieser Arbeitsgruppe ist Prof. N. Krebs Ovesen aus Dänemark; der Gruppe gehören weitere 8 offizielle Vertreter, 2 stellvertretende Mitglieder und ein Sekretär an. Bis Ende 1985 hat die Arbeitsgruppe 16 Sitzungen abgehalten; 3 weitere Sitzungen sind für das Jahr 1986 vorgesehen.

Zunächst führte die Arbeitsgruppe eine Bestandsaufnahme der nationalen Richtlinien der 9 Mitgliedsländer durch, welche ergab, daß die Verhältnisse außerordentlich vielfältig sind. Die wichtigsten Ergebnisse der Bestandsaufnahme sind im Bericht zusammengefaßt. Ferner wird ein Überblick der Struktur, der Unterteilung und, des gegenwärtigen Bearbeitungsstandes von EC 7 angegeben.

Danach werden die wichtigsten Besonderheiten des Inhalts von EC 7 behandelt und mit den Bestimmungen der deutschen Regelwerke verglichen. Es handelt sich um die Grenzzustände, die Methode der Teilsicherheitsbeiwerte und die Geotechnischen Kategorien. Zum Schluß wird über die Bemühungen um Bekanntgabe der Grundzüge von EC 7 und um die Koordinierungsbemühungen in den einzelnen Ländern der Europäischen Gemeinschaft berichtet und es werden einige Empfehlungen in Verbindung mit einer künftigen Einführung von EC 7 in der Bundesrepublik Deutschland gemacht.

zu 11,17

Anlage 1 zum Forschungsbericht vom Dezember 1985

ad hoc-commitee

SOME QUESTIONS ON THE LEGAL SITUATION RELATED TO VERIFICATION OF SAFETY OF GEOTECHNICAL STRUCTURES

in _______________

	Answers
1. Is a verification (calculation) of the safety*) of geotechnical structures (GS) necessary (required) before begin of construction works ? a) for every GS b) only in following cases:	
2. What is the main criterion for the reliability of a verification? a) the qualification and "credibility" of its autor, b) the following (observing) of the provisions of codes, requirements etc. or c) the positive result of an examination (control, check) by a specially licenced control engineer	
3. Persons, abled to carry out verifications (calculations) of GS's: a) everybody b) only graduates by technical schools or universities c) as b), but only after a supplementary licencing	
4. If the verification is carried out by an employee of an engineering company, who is responsible: the company as a "juridical person" or the engineer personally?	
5. Is an examination (checking) of the verification by an autorized person or institution legally (or by a code) prescribed?	
6. Who has the final responsibility: the person or company, that carried out the verification (autor), the examination engineer or both?	
7. What is the legal relevance of the codes, dealing with the safety verification of GS's?	
8. Does the strict observing of the code make the autor of a safety verification free of responsibility in the case of damage or incident?	
9. Which are the criteria for a licencing a) of civil engineers generally and b) of geotechnical engineers?	
10. Further remarks:	

*) Standsicherheitsnachweis, Calcul de stabilité
Stability analisis

Munich, 16.09.1985

Evaluation of the answers on legal situation

Question \ Country	BELGIUM	DENMARK	FRANCE	GERMANY	GREAT BRITAIN	GREECE	IRLAND	ITALY	NEDERLAND	PORTUGAL
see p. 1. 1a)	de jure: no	yes	gen.: yes	yes	yes	priv.-yes	no	yes	yes	yes
1b)	R.O.	specially GC2+3	R.1	-	-	publ.-GC2+3	buildings and ret.str.in towns			
" 2a)	de jure; yes	combination	def.: no	no	maybe	yes	-	combin.	yes R.4	for GC3
2b)	no	a)+b)+c)	gen.: yes	yes	for GC 1+2	maybe	yes	a+b+c	R.5	gen. yes
2c)	de jure; no	a)+b)+c)	yes for GC3	yes	R.2	-	-	R.3		-
" 3a)	de jure; yes	de jure; yes	no	de jure; yes	-	no	de jure: yes	no	d.f.: no	gen.: yes?
3b)	def.: 3b	-	yes? R.6	-	yes	-	def. only 3b	-	yes	-
3c)	-	-	-	control: yes	-	yes	-	yes	-	yes for dams
" 4)	civil: comp. penal: eng.	comp.	civil: comp. penal: eng.?	civil: comp. penal: depends	comp.	comp.	comp.	eng.	?	eng.
" 5)	de jure; no for ins. d.f.yes	no some by local autor.	public: yes priv.: d.j. no d.f. yes	yes, by licenced	yes by local autority (basic control)	?	no some by local autority	gen.: no R.7	gen.: no building by loc. autority	yes, by local autority
" 6)	gen.: autor sometimes; contr.	autor	both	civil.: autor penal: both	gen.: both	both	d.f.: both	gen.: autority also control	prob. autor	autor
" 7)	-	"good practice"	different, mostly requir. (contractual)	state of art (good practice)	good practice	-	"good practice"	law, but very general	-	R.8
" 8)	- (no)	gen.: yes	no, but it depends	mostly yes	mostly no	-	gen.: yes	mostly yes	-	-
" 9a)	no lic.	no lic.	graduation	gen. no licencing only for control	gen. no lic. graduation, chartering by ICE	exams after graduation	gen. no lic.	practice & exams after graduation suppl. exams	no lic. -	no lic. -
9b)	-	-	-	-	-	suppl. qualif.				

Key to p. 2

- mostly means: "does not apply"

R.0 For private buildings the insurance company mostly requires a verification

R.1 Not required for medium and small size shallow foundations

R.2 In GB all designs must be submitted to the local (or county) autority and will be checked against very basic criteria by non-speciallist engineers.

R.3 In Italy public and publically financed works must be checked by the involved autority.

R.4 Designs made by well known institutes are generally accepted in NL.

R.5 In NL designs are checked against the regulations of local autorities.

R.6 Answers not completely clear.

R.7 For private owned earth structures no checking. For concrete and steel structures "collaudo" (control engineer) must be involved.

de j. = d.j. = de jure = legally
de f. = d.f. = de facto

Summary

1a	8 x yes; limitations in DK, F, Gr 2 x no; with limitation (B, Irl.)
2a	2 x yes (B, Gr); 2 x comb. a+b+c (DK,I); meanly (NL); yes for GC3 (P)
2b	3 x yes (GB,Irl,P); 2 x comb. b+c (F,D)
2c	exists only in D (important!), F (for insurance), DK+GB+NL (by loc.aut.)
3a	5 x yes de j. (B,DK,D,Irl.,P), but in B,Irl. de f. 3b
3b	3 x yes (F?, GB, NL),
3c	2 x yes (Gr, I); P for control; (P) for dams
4)	4 x comp.; 2 x eng.; 3 x civil→comp., but penal→engineer (Dk, GB, Gr, Irl) (I, P) (B, F, D)
5)	generally yes: 2 x by specialists (D,I); 2 x loc. aut. (GB,P); 2 x by insurance for privates (B, P); 2 x only for buildings (Irl.,NL); DK?
6)	4 x autor (B, DK, NL, P); 3 x both (F, Gr, Irl.) 2 x civil-autor, penal-both (D, I)
7)	"good practice" (state-of-the-art), in all countries, if available; in (I) law, but very general
8)	licencing only in (Gr + I) → practice + exams in (D) licencing for control

Evaluation by W. Sadgorski,

Munich, 9.6.1986 Sadgorski

Zu 11.7

Anlage 2 zum Forschungsbericht vom Dezember 1985

Draft model of March 86

EUROCODE 7 : FOUNDATIONS

Report prepared for the Commission of the European Communities

by Representatives of the Geotechnical Societies within the European Communities

2. Version of a model for:

EUROCODE No. 7 - Foundations

prepared by an ad-hoc committee established in 1981 by the following Representatives of nine European national Geotechnical Societies:

Belgium:	Prof. E. Lousberg
Denmark:	Prof. N. Krebs Ovesen (chairman)
France:	Mr. F. Baquelin assisted by Mr. S. Amar
Germany:	Dr. W. Sadgorski
Greece:	Dr. A. G. Anagnostopoulos assisted by Dr. D. Coumoulos
Ireland:	Dr. Trevor Orr
Italy:	Prof. R. Japelli
The Netherlands:	Mr. W. Heijnen (secretary) assisted by Mr. H. Nelissen
United Kingdom:	Dr. B. Simpson

In 1985 the ad-hoc committee was joined by:

Portugal: Mr. E. Maranha das Neves

and in 1986 by:

Spain: N.N

2. version

March 1986

PREFACE

The present document is the second draft of a code to be presented by eleven European National geotechnical Societies to the Commission of the European Communities (CEC) to be used as a model for Eurocode No. 7 - Foundations (EC 7).

During 1984 draft versions of four Eurocodes (EC 1, 2, 3 and 4) were published for discussion; the following is a quotation from the preface of these draft versions:

> "1.1 The objectives of the Eurocodes
>
> The Commission of the European Communities (CEC) intends to issue European Codes - the Eurocodes - for the design and execution of buildings and civil engineering structures. These codes are intended to establish a set of common rules as an alternative to the differing rules in force in the various Member States.
>
> The Commission's programme for aligning the regulations, laws and administrative provisions of the Member States concerning the safety, serviceability and durability of the different types of construction and materials provides initially for the following eight Eurocodes:
>
> - Eurocode No. 1 - common unified rules for different types of construction and material
> - Eurocode No. 2 - for concrete structures
> - Eurocode No. 3 - for steel structures
> - Eurocode No. 4 - for composite steel and concrete structures
> - Eurocode No. 5 - for timber structures
> - Eurocode No. 6 - for masonry structures
> - Eurocode No. 7 - for foundations
> - Eurocode No. 8 - for structures in seismic zones.
>
> The objectives of the Eurocodes are to:
>
> - promote functioning of the Common Market by removing obstacles arising from differing rules

- provide common technical rules for an efficient application of the Council Directive 71/305 on the coordination of procedures for the award of public contracts, which can be applied as an alternative to the national rules

- reinforce the competitive position of the European Construction Industry and allied professions in countries outside the Community

- establish a harmonized basis for the intended common rules for building products.

1.2 The application of the Eurocodes

The Eurocodes will provide an optional set of design rules which can be applied within the Community as an alternative to the corresponding national rules covering the same technical matters. EC 1 is not intended as an operational document. It provides the general philosophy and fundamental considerations from which unique solutions have been developed for practical use in EC 2, 3, 4, and 8 and will be used as a base document by those preparing future draft Eurocodes.
Adaptation of the common rules to the respective national safety level, by specification of appropriate values for safety coefficients, will be subject to national responsibility. The application of the Eurocodes and the continuation of the harmonization effort will permit the provision of the gradual establishment of common values.

The control of design and execution and any approval procedure of structues will remain subject to national regulations. The same applies to technical supplements with regard to aspects which are not yet comprehensively covered by the Eurocodes or which cannot be covered in terms of generally applicable rules".

In 1980 an agreement was reached between the CEC and the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) according to which the Society should undertake to survey existing codes of practice for foundations within the Members States and to draft a model code which may be adopted as EUROCODE No. 7 for Foundations.

In 1981 the ISSMFE established an ad-hoc Committee for this task; the committee consisted of one member from each of the then nine member countries of the EEC: Belgium (prof. E. Lousberg), Denmark (Prof. N. Krebs Ovesen, Chairman), France (Mr. F. Baguelin assisted by Mr. S. Amar), FRG (Dr. W. Sadgorski), Greece (Dr. A. G. Anagnostopoulos assisted by Dr. D. Coumoulos), Ireland (Dr. T. Orr), Italy (Prof. R. Japelli), the Netherlands (Mr. W. Heijnen, Secretary assisted by Mr. H. Nelissen) and United Kingdom (Dr. B. Simpson); Luxemburg has had no member. The ad-hoc committee was extended to include Portugal (Mr. E. Maranha das Neves) in 1985 and Spain (N.N.) in 1986.

The Committee has met a total of 17 times in sessions lasting normally two full working days: Brussels (April 1981), Stockholm (June 1981), Paris (October 1981), London (January 1982), Munich (April 1982), Athens (June 1982), Copenhagen (September-October 1982), Dublin (January 1983), Helsinki (May 1983), Rome (September 1983), Delft (January 1984), Louvain la Neuve (May 1984), Athens (September 1984), Paris (January 1985), London (May 1985) Münich (September 1985) and Rome (January 1986).

On the occasion of the Eight European Conference on Soil Mechanics and Foundation Engineering in Helsinki in may 1983 the Committee met with about 50 representatives from the nine National geotechnical Societies to discuss preliminary versions of chapters 1, 2, 4, and 6 of the model code.

At the end of 1985 a contract was given to the Committee by the Steering Committee for the Eurocode System. According to the contract the Committee is obliged to deliver a preliminary draft of a Model Code for EUROCODE 7 Foundations in July 1986.

It is a clear understanding between the Steering Committee and the Eurocode 7 Committee that the contract may be extended over a period of one more year.

At the committee's meeting in Rome in January 1986 it was decided to ask all member societies within the European Community to discuss, comment and make recommendations concerning the draft Model Code before it is delivered to the Steering Committee in July 1986. The Committee decided to set up the following scheme for consulations with the national geotechnical societies:

1. In March 1986 a copy of the present draft Model Code for Eurocode 7 Foundations is mailed to the eleven national geotechnical societies within the European Community.

2. In the period March to May 1986 the national societies are invited to comment and make recommendations on basis of the draft Model Code. It is left to the national societies to decide in which way they will persue discussions among members to collect such comments and recommendations. However, it is recommended that the national societies divide their comments and recommendations into a rather short document (2 to max. 5 pages) containing general comments and recommendations and another document containing detailed comments and recommendations in relation to specific paragraphs etc.

3. The next meeting of the Committee will take place in Copenhagen on June 12-13, 1986. The Committe will not be able at this meeting - due to the lack of time - to take into account all the comments and recommendations received from the national societies before the draft Model Code is delivered to the Steering Committee in June 1986. However, the comments and recommendations will form the basis of the work that is foreseen under next year's contract. All general comments and recommendations received from national geotechnical societies will be forwarded to the Steering Committee together with the Model Code and a covering letter outlining the proposal of the Committee.

Copenhagen March 1986

Niels Krebs Ovesen

Chairman, ISSMFE regional technical committee on EUROCODE 7 - Foundations

CONTENTS

CHAPTER 1 - GENERAL PRINCIPLES

1 GENERAL PRINCIPLES

1.1 Purpose and Scope

This code of practice comprises a set of principles and procedures intended to ensure an adequate technical quality for foundations, retaining structures and earthworks.

The code is applicable to geotechnical engineering which is defined as that branch of civil engineering which deals with the design and construction of structures and parts of structures whose performance or influence on their surroundings are substantially dependent on the properties of the ground. Throughout the code, the term 'structures' is taken to include earth structures and the term 'ground' is taken to include both soil and rock.

guide: Examples include the following: Shallow and deep foundations for buildings, bridges and other structures, excavations, retaining walls, embankments, cofferdams, dykes and small dams.

The use of the code affects, but is not limited to the following:

- site evalutation
- field and laboratory investigations
- design of foundations, retaining structures and earth works
- observations and evaluations during and after construction
- evaluation of material sources for earth structures.

1.2 Use of the Code in Engineering Works

1.2.1 Basic Geotechnical Considerations

In applying the provisions of this code, the special characteristics of geotechnical design must be considered. These are:

- soils and rocks display a far greater range of material properties and of heterogeneity than do manufactured materials such as steel and concrete. The properties needed for design are therefore difficult to assess and the relationship between measured parameters and field behaviour requires careful consideration for individual situations,

- consideration of geological and other background information is an essential part of geotechnical design, together with a study of the observed behaviour of similar structures in similar ground conditions,
- because soils and rocks display a large range of material behaviour, many different test techniques are appropriate in order to measure or infer the required material parameters,
- geometrical parameters, especially the interfaces between strata, water levels, and ground levels may be major uncertainties in the design,
- water pressures in the ground are of major importance and are often significant uncertainties,
- geotechnical design is frequently concerned with the foundations of structures. In many cases, the structure could be seriously damaged by deformations which are too small to constitute a significant disturbance or failure of the ground itself,
- it is necessary to consider all the ground which affects the structure under consideration, and not just the ground in contact with it or immediately adjoining it,
- conventional practice includes the testing of full scale elements such as piles or anchors,
- it is sometimes appropriate to use the observational method of design.

guide: The items listed above distinguish geotechnical design from conventional structural design. Their significance is developed further in appropriate sections of the code.

1.2.2 Personnel

It is a requirement of the code that the project must be supervised at all stages by personel with geotechnical knowledge appropriate to the project in hand and that adequate supervision, skill and experience will be avilable during all site works.

1.3 Performance Criteria and Limit States

1.3.1 Performance Criteria

Each structure or part of a structure is required to fulfill certain fundamental requirements of stability, rigidity, etc. during construction and throughout their designlife. The fundamental requirements are expressed in specific terms as performance criteria.

1.3.2 Design Considerations

In chapters 6 to 9 of the code, the performance criteria which must be considered in geotechnical design are indicated for each type of structure. In sections marked "guide", calculation models and/or prescripitive measures which may normally be used to ensure that the performance criteria will be satisfied are indicated, but alternative approaches are permitted if they can be justified. Such alternatives will usually necessitate additional geotechnical analyses and calculations or additional supervision and monitoring of site works. The design results should always be checked against local experience with the same type of structure in the same ground under similar geological conditions.

1.3.3 Limit State Method

Whenever a structure or part of a structure fails to satisfy one of its performance criteria it is said to have reached a "limit state". This Code is based on the "limit state method" in which each possible limit state is considered separately in the design and its occurrence is either eliminated or shown to be sufficiently improbable.

guide: In structural engineering design it is general practice to distinguish between "ultimate" and "serviceability" limit states. Ultimate limit states involve loss of static equilibrium or rupture of a critical section of the structure. Serviceability limit states involve failure to satisfy the required standards of utility, appearance, comfort, etc. Often, the main performance criteria can be statisfied by demonstrating that the structures will at all times have the necessary margins of safety against reaching ultimate limit states and are also unlikely to reach their serviceability limit states.

guide: In geotechnical design it is normal practice to consider the possible formation of a mechanism in the ground. However it is also necessary to consider the possibility that serious damage could occur in the structure due to deformation in the ground without the mobilization of a mechanism in the ground.

Two main classes of limit states are considered in this code:

- Type 1: an ultimate limit state at which either
  - (A) a mechanism is formed in the ground, or
  - (B) a mechanism is formed in the structure or severe structural damage occurs due to movements in the ground.

- Type 2: a serviceability limit state at which deformation in the ground will cause loss of serviceability in the structure.

A detailed analysis of the problems of interaction between structure and ground is sometime required in order to demonstrate that the structure and the ground will have the necessary margins of safety against reaching ultimate limit states and are also unlikely to reach their serviceability limit states.

In practice experience will often show which type of limit state will govern the design, and other analyses may be omitted completely or be limited to rough control checks.

1.4 Durability

Durability of the structure during its entire, intended lifespan must be considered when selecting the design parameters.

guide: Durability should not be considered a serviceability limit state as such. Durability can also be secured by paying attention to the detailed aspects of design with provision for protection and maintenance, etc.

1.5 Geotechnical Categories

1.5.1 Basic Concepts

In order to establish minimum requirements for the extent and quality of geotechnical investigations, calculations and construction control checks, the difficulty and complexity of each geotechnical design must be clearly identified. To facilitate this, three 'Geotechnical Categories' are defined.

The following factors should be taken into consideration when determining which Geotechnical Category is appropriate to each particular design situation:

- nature and size of the structure and its elements, including any special fundamental requirements,
- special conditions with regard to its surroundings (neighbouring structures, traffic, utilies, hazardous chemicals, etc.),
- ground conditions,
- groundwater situation,
- regional seismicity,
- influence of the environment (hydrology, surface water, subsidence, etc.).

Classification of a structure according to geotechnical category must be performed prior to the geotechnical investigations. The category may later be changed; it is important, however, that it remains well defined throughout the design and construction control process.

Classification according to the structure and its neighbouring structures (a, b, e and f above) can often be performed prior to the geotechnical analyses. However, the final geotechnical category determined by the ground conditions (c and d above) will generally be established later in the design process. It will sometimes be as late as the construction control check before it is found necessary to classify a design in a higher category than hitherto envisaged.

Checks of the design or construction in accordance with the specifications given for a geotechnical category higher than that required for the structure by the code may, if desired, be applied to any structure.

The procedures of higher categories may sometimes be used to justify more economic designs, or where a suitably qualified and experienced engineer considers them to be appropriate.

1.5.2 Geotechnical Category 1

This only includes small and relatively simple structures for which it is possible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations.

1.5.2.1 Nature and Size of the Construction. Geotechnical Category 1 procedures will normally not be sufficient in the case of foundations subjected to an inclined loading except in the case of foundations for small retaining walls listed below.

The following are examples of Geotechnical Category 1 structures:

1. Light buildings with a maximum design column load of 250 kN and 100 kN/m for walls, with no special requirements as regards settlement conditions, etc., and using conventional types of foundations.
2. Retaining walls and excavation supports where the difference in ground levels does not exceed 2 m, and the ground is not subject to significant surcharges.
3. Earthworks involving not more than 3 m of fill below trafficked areas, etc., or not more than 1 m of compacted fill below ground bearing floor slabs.
4. Ground bearing slabs which can be designed using empirical principles without detailed analyses.
5. 1 and 2 storey houses and agricultural buildings on conventional piled foundations.
6. Small excavations for drainage works, pipe-laying, etc.

Additional examples are given in the national appendices.

1.5.2.2 Surroundings. Geotechnical Category 1 procedures will only be sufficient when there is no risk of damage to neighbouring buildings, utilies, public areas etc.

1.5.2.3 Ground Conditions. Geotechnical Category 1 procedures will only be sufficient where the ground is not sloping significantly and in ground conditions which are known from extensive local experience to be sufficiently straightforward that routine methods may be used for foundation design and construction. Geotechnical Category 1 procedures will not normally be adequate for foundations bearing on slopes, refuse, uncompacted fill, fissured, swelling clay, or soft, loose or highly compressible soils.

1.5.2.4 Groundwater Situation. Geotechnical Category 1 procedures will be sufficient only if there is no excavation below the water table or if extensive local experience indicates that a proposed excavation below the water table will be straightforward.

1.5.2.5 Regional Seismicity. In seismically active areas, Geotechnical Category 1 procedures will be sufficient only for insensitive structures.

1.5.2.6 Influence of the Environment. Geotechnical Category 1 procedures will not be sufficient if problems involving hydrology, vegetation, surface water, subsidence or other environmental factors could reasonably be suspected.

1.5.3 Geotechnical Category 2

This category includes structures for which quantitative geotechnical data are necessary to ensure that the functional requirements will be satisfied, but for which conventional procedures of design and construction may be used. These necessitate the involvement of qualified engineers with relevant experience.

1.5.3.1 Nature and Size of the Construction. Geotechnical Category 2 procedures are sufficient only for conventional types of structures and foundations with no abnormal loading and no abnormal risks. The following are examples of Geotechnical Category 2 foundations and structures:

Conventional types of

1. Spread footings
2. Raft foundations
3. Piled foundations
4. Walls and other structures retaining soil or water
5. Excavations
6. Bridge piers and abutments
7. Embankments and earthworks
8. Ground anchors and other tie-back systems.

1.5.3.2 Surroundings. Where a project involves a risk of damage to neighbouring structures by excavation, pile driving or lowering of the groundwater table, for example, the geotechnical investigations and calculations performed with regard to the conditions for these neighbouring structures must correspond at least to Geotechnical Category 2, and should be related to the nature, size and foundations of the neighbouring structures. Geotechnical Category 2 procedures will not necessarily be sufficient in situations where either the risk or the effects of damage to surrounding structures or utilities would be extremely severe.

1.5.3.3 Ground Conditions. Geotechnical Category 2 procedures will be sufficient only for ground conditions for which the properties needed for design can be obtained using routine procedures for field and laboratory testing.

1.5.3.4 Groundwater Problems. Geotechnical Category 2 procedures will be sufficient only if the nature of the groundwater conditions is such that lasting damage cannot be caused to structures or load-bearing strata without prior warning due to the absence or failure of groundwater lowering or drainage systems. For example, Geotechnical Category 2 procedures may be insufficient for excavations considerably below the groundwater table in strata whose permeability increases with depth.

1.5.3.5 Regional Seismicity. Geotechnical Category 2 procedures, used in conjunction with national seismic codes, will normally provide an adequate basis for a seismic design. This approach may not be adequate in areas of exceptionally high seismic activity or for very sensitive structures.

1.5.3.6 Influence of the Environment. Geotechnical Category 2 procedures will be sufficient only when routine procedures exist to deal with environmental problems which could arise.

1.5.4 Geotechnical Category 3

Structures which do not fall within the limits of Geotechnical Category 1 and 2 are included in Geotechnical Category 3. The involvment of experienced engineers with relevant geotechnical experience, will be necessary in these projects.

Geotechnical Category 3 includes very large or unusual structures, structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions and structures in highly seismic areas. The code specifications for Geotechnical Category 2 form the lower limits for the extent and quality of the necessary investigations and calculations, but apart from this no detailed code requirements have been formulated for Geotechnical Category 3. No attempt has been made to establish a fixed boundary between categories 2 and 3.

Examples of structures which require Geotechnical Category 3 procedures include:

1. Buildings with exceptional loads
2. Multi-storey basements
3. Retaining dams and other structures acted upon by great differential water pressures
4. Facilities for temporary or permanent lowering of the level of the groundwater table and which involve a risk of serious earth movement and/or structural damage
5. Earthworks and pavements below traffic systems acted upon by abnormally heavy loads
6. Large bridges and tunnels
7. Machine footings with heavy dynamic loads
8. Power stations
9. Offshore structures
10. Chemical plants treating hazardous chemicals.
11. Structures which are very sensitive to seismic activity or structures in areas of exceptionally high seicmic activity.

1.6 Local Experience

In Geotechnical Designs previous experience of the construction and performance of similar structures in similar conditions is frequently quoted. In this code reference is made to "Local experience".

The term "local experience" refers to documented, or other clearly established, information related to the geological strata being considered in design, involving the same soil types and for which similar geotechnical behaviour are expected.

Chapter 2 VERIFICATION OF SAFETY AND SERVICEABILITY

Contents

2 VERIFICATION OF SAFETY AND SERVICEABILITY

2.1 Limit States

The approach which must be used to check the adequacy of designs is described in this chapter. This consists of compiling a list of required performance criteria and determining the limit states at which these criteria would be infringed. It must then be demonstrated that the limit states are unlikely to occur.

When compiling limit states for design of geotechnical structures, it is necessary to consider various situations which will occur during their construction and use, and to derive appropriate design situations. Chapters 6 to 9 specify for each type of geotechnical structure limit states which should be considered.

It must be shown in the design that the occurrence of limit states is sufficiently improbable provided that the construction and loading are generally in accordance with the design. This may be achieved either by the adoption of prescriptive measures, as described in 2.2, by study of calculation or experimental models incorporating appropriate basic variables, as described in 2.3 and 2.4 respectively, or by an observational method, as described in 2.5.

2.2 Prescriptive Measures

For certain limit states calculation models are either not available or unnecessary. Instead, the limit state can be avoided by the adoption of conventional and generally conservative details in the design, and by attention to specification and control of materials, workmanship, protection and maintenance procedures. These will be referred to as "prescriptive measures" and they are considered further in Chapters 6 to 9.

guide: Prescriptive measures are often used to ensure durability to frost
: action and chemical or biological attack. They may sometimes also be
: used to avoid unnecessary calculation in very familiar design situa-
: tions. For example, conservative presumed bearing pressures might be
: adopted for some foundations without calculation.

2.3 Calculation Models

2.3.1 Available Approaches

Design calculations should follow one of the three approaches described below. Reference is made to the two main types of limit state defined in section 1.3, and to chapters 6 to 9. Chapters 6 to 9 and the national appendices indicate where each of the approaches described here is applicable.

a) Each limit state may be studied directly by considering design values of parameters and other conditions, for which the calculations indicate that the limit state would be imminent.

guide: Limit states involving the formation of a collapse mechanism in the ground (type 1 A) are readily checked using this approach. For limit states defined by displacement considerations (type 1 B or 2), the displacements must be calculated or otherwise assessed if this approach is used. In some cases, especially those concerning ultimate limit states in the structure (type 1 B), this will require calculations using non-linear models of deformation in the soil.

b) For limit states defined by displacement considerations calculation models for the direct approach may not be readily available. In these cases the limit state may sometimes be checked by limiting the proportion of the strength of the soil which can be mobilised.

guide: This approach may be used for either ultimate (type 1 B) or serviceability (type 2) limit states. It is important to identify for each limit state whether the strength terms used in the calculations refer to limits of ultimate strength or mobilised strength.

c) In some cases it can be shown that one particular limit state governs the design and is always more likely to occur than others which might be considered. In these cases only the governing limit state need by analysed and the others may be deemed to be satisfied.

For cases belonging to geotechnical categories 1 and 2 the design of the supported structure should normally result in a specification giving

- the loads acting on the foundation in the various desing situations
- the allowable settlement of the foundation.

The design of the foundation for such cases may normally be done in accordance with the principles and guidelines given in this code.

For more complicated cases interaction may occur between the supported structure and the soil; such cases belong to geotechnical category 3.

2.3.2 Design Values of Variables

The following basic variables will be involved in most calculation models:

- actions, which may be either imposed loads or imposed displacements
- properties of soils and other materials
- geometrical parameters
- constraints, which are design requirements such as acceptable deformations

The values of the variables entered into calculations are called 'design values'; these are indicated by a subscript d. For structures of a conventional type for which there is experience of successful designs which are generally considered to be economic, the design values should be chosen so as to lead to conventional designs.

In the analysis of any limit state, the set of design values adopted in the calculations should be such as to ensure that the occurrence of a more adverse set of values is, in practice, sufficiently unlikely. The values may also be chosen such that the design may be deemed to satisfy other selected limit states which should be identified explicitly. Guidance on the selection of design values is given in this code, but the designer must always check that, in his opinion, the selected design values will achieve the aims stated here.
Special attention must be paid to exceptional cases, particularly those involving uncertainty in water levels, geology or stratification. Further, the accuracy of the calculation model and the significance of the level of workmanship and control should be considered.

guide: In choosing design values, major uncertainties will genereally be covered explicitly by adopting pessimistic values for the correponding basic variables. If other, more minor uncertainties are not considered explicitly, they must be taken into account in the selection of the design values of the basic variables. Uncertainty in the calculation method itself may, depending on the circumstances, be regarded as either a basic variable or a more minor uncertainty.

The selection of design values for the basic variables must in general take account of:

- consequences of the occurence of the limit state
- the possibility of unfavourable variations of the parameters
- the independence or interdependence of the various parameters involved in the calculation.

The selection of design values for actions (F_d) must further take account of:

- uncertainty in the loading model, if this is not accounted for within the model itself
- time of loading.

The selection of design values for soil properties (f_d) must take account of:

- uncertainties in the relation between soil properties in the geotechnical structure and those measured by field or laboratory tests
- the influence of workmanship on artificially placed or improved soils
- the brittleness or ductility of the soils involved
- time effects
- possible inaccurate assessment of the resistance of sections or load-carrying capacity of the soil or the structure, unless this is allowed for in the resistance model
- uncertainties in geometrical parameters, unless they are accounted for directly.

The selection of design values for geometrical parameters (a_d) should take account of:

- the specified tolerances on the geometrical parameters.

guide: The most important geometrical parameters in geotechnical design are usually the level and slope of the ground surface, the levels of the water table and interfaces between strata, and the levels of excavations for basements, service trenches, etc.

In cases where variations of the geometrical parameters are not important, they may be allowed for in the selection of design values for material properties or actions. In other cases it is generally advisable to allow for these uncertainties directly. For limit states with servere consequences, design values for geometric parameters should represent the most adverse values which could occur in practice.

The selection of the design values for constraints (C_d) must take account of:

- the confidence with which the acceptable value of the constraint can be specified

2.3.3 Design Calculation Models

The design calculation model will generally consist of two elements:

- a method of analysis, often based on a theoretical approach including simplifications
- if needed, a modification to the results of the analysis to ensure that the results of the design calculation model are generally accurate or conservative.

guide: Whenever possible, the method of analysis should be calibrated against field observations of previous designs, model tests or more reliable analyses.

The selection of the design calculation model should take account of the following factors:

- the range of uncertainty in the results of ~~the method of analysis~~ on which the design calculation model is based;
- any systematic errors known to be associated with the method of analysis.

guide: The design calculation model may be represented by a mathematical function θ_d. The design requirement may then be stated in the form:

$$\theta_d\ (F_d,\ f_d,\ a_d,\ C_d) \geq 0 \qquad 2.1$$

<u>or</u> $$\Theta_d\ (F_d,\ f_d,\ a_d) \leq C_d \qquad 2.2$$

An example of requirements of this form are equations 6.1, in connection with 6.2 and 6.3.

It is sometimes possible to divide this calculation into two steps in which the design resistance effect, R_d, and the design disturbance effect, S_d, are calculated separately. In this case the design requirement may be stated in the form:

$$R_d\ (f_d,\ a_d) \geq S_d\ (F_d,\ a_d) \qquad 2.3$$

Examples of equations of this form are equations 6.2 and 6.9.

Most of the methods of analysis described in chapters 6 to 9 will give conservative results in normal situations. Unless otherwise stated, they may be used without modification as design calculation models.

2.3.4 Derivation of Design Values by the Method of Partial Coefficients

Design values may be derived using the method of partial coefficients. In this approach, representative or specified actions (i.e. loads and imposed displacements) and characteristic material parameters are first selected. Design values are derived from these by applying the partial coefficients. Each coefficient may be decomposed into several different factors, which each take account of one or more uncertainties. Values are given for the partial coefficients in the National Appendices, and these have been selected so as to ensure that the design values will comply with section 2.3.2.

Representative or specified actions, F_r, are multiplied by partial coefficients, γ_f, and load combination factors, ψ, thus

$$F_d = \gamma_f \psi F_r \qquad 2.4$$

Representative and specified actions are defined in Chapter 3.

Characteristic values of material properties, f_k, are divided by partial coefficients, γ_m, thus

$$f_d = f_k/\gamma_m \qquad 2.5$$

Characteristic values of the soil parameters should be based on a careful assessment of the range of values which might be encountered in the field. This assessment should take account of geological and other background information, and the results of laboratory and field measurements. For parameters for which the relevant values in the field are well established with little uncertainty, the characteristic value may be taken as the best estimate of the value in the field. Where there is greater uncertainty, the characteristic value is somewhat more conservative, and constitutes a "conservative best estimate".

guide: Characteristic values should be selected such that, in the opinion of the designer, the probability of a more adverse value occurring in the field is not greater than about 5%.

The choice of characteristic values is not dependent on the serverity of the limit state under consideration. However, the choice is often dependent on the mechanism or mode of deformation being considered. For example, if the avoidance of a limit state is dependent on the behaviour of a small zone of soil (as in an end-bearing pile) a more pessimistic assessment of strength is required than for a limit state related to the average strength of a larger amount of soil (as in a long friction pile). Similarly, different characteristic strengths would be required for a shear failure in a fissured material, depending on whether the shear surface is free to follow the fissures or constrained to intersect intact material.

It might sometimes be helpful to carry out a statistical analysis of measured data. However, it is emphasised that this will rarely lead directly to characteristic values since these depend on the designer's assessment of the field situation.

Characteristic geometrical parameters a_k are modified by additive coefficients Δ_a, thus

$$a_d = a_k \pm \Delta_a \qquad 2.6$$

To allow for uncertainty in the method of analysis, the design calculation model may incorporate a further coefficient γ_d. Unless otherwise stated, this factor may normally be taken as unity for the methods of analysis presented i chapters 6 to 9.

guide: If the method of partial factors is used, inequalities (2.1) to (2.3) can be expressed as follows:

guide:

$$\Theta\ (F_k,\ \gamma_f,\ f_k,\ \gamma_m,\ a_k,\ \Delta_a,\ C_d,\ \gamma_d) \geq 0 \qquad 2.7$$

<u>or</u>

$$\Theta\ (F_k,\ \gamma_f,\ f_k,\ \gamma_m,\ a_k,\ \Delta_a,\ \gamma_d) \leq C_d \qquad 2.8$$

$$R\ (F_k,\ \gamma_f,\ a_k,\ \Delta_a,\ \gamma_{dR}) \geq S\ (f_k,\ \gamma_m,\ a_k,\ \Delta_a,\ \gamma_{dS}) \qquad 2.9$$

in which Θ, R and S are functions which give conservative results when used with the partial coefficients γ_d, γ_{dR} and γ_{dS}.

2.3.5 Derivation of Design Values by Other Methods

Design values may be derived by methods other than the use of partial coefficients provided that they comply with Section 2.3.2.

The sets of partial coefficients specified in the national appendices indicate the levels of safety required for the various limit states in conventional situations. Similar levels of safety are required when design values are selected by other means.

guide: If design values are selected directly, it is recommended that they are based on assessments of the most adverse value of each parameter which could occur in practice. Alternative methods of deriving design values are discussed extensively in current literature.

2.4 Experimental Models and Load Tests

In some cases it is possible to demonstrate that limit states will be avoided by carrying out tests, either on full scale or smaller scale models, or on a sample of the final construction.

When test results are used to justify a design, the following features must be considered and allowed for

- variations in the soil conditions between the test(s) and the working construction(s)
- time effects, especially if the duration of the test is much less than the duration of loading of the working construction(s)
- scale effects, especially if small models are used. The effect of stress levels on soil behaviour must be considered, together with the effects of soil particle size.

2.5 Use of the Observational Method

Because prediction of geotechnical behaviour is often very difficult, it is sometimes appropriate to adopt the approach known as "the observational method". When this approach is used, the following requirements must all be met before construction is started.

- The limits of behaviour which are acceptable must be established.
- The range of possible behaviour must be assessed and it must be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits.
- A plan of monitoring must be devised which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring must make this clear at a sufficiently early stage to allow contingency actions to be undertaken successfully.
- A plan of contingency actions must be devised which may be adopted if the monitoring reveals behaviour outside acceptable limits.

During construction, the monitoring must be carried out as planned, and additional or replacement monitoring must be undertaken if this becomes necessary. The results of the monitoring must be assessed at appropriate stages and the planned contingency actions must be put into operation if this becomes necessary.

guide: The observational method is often used in the design of geotechnical construction such as deep excavations and embankments, including dams. The parameters which are most frequently observed are ground movements and water pressures. Contingency actions may include regarding the surface of the natural ground or fill, installation of structural support or installation of drainage. In some cases the action necessary may simply be a modification of the time scale for continued construction.

2.6 Design Report

The assumptions, data, calculations and results of the verification of safety and serviceability must be recorded in a Design Report. Items which require checking during construction or which require maintenance after construction must be clearly identified in this report. When the required checks have been carrried out during construction, they must be recorded in an addendum to the report.

guide: The complexity of Design Reports will vary greatly, depending on the type of design. For simple designs, a single handwritten sheet may be sufficient.

The report will normally include the following items, with cross-references to other documents which contain more detail:

- a description of the site and surroundings,
- a description of the ground conditions,
- a description of the proposed construction, including loads,
- assumed values of soil and rock parameters, including justification, as appropriate,
- a statement of the design, including calculations and other justification as appropriate,
- a note of items to be checked during construction or requiring maintenance.

CHAPTER 3 - DESIGN SITUATIONS AND ACTIONS

3 DESIGN SITUATIONS AND ACTIONS

3.1 Definitions

3.1.1 Design Situations

Design situations are those sets of physical conditions for which it must be demonstrated that limit states will not occur. The selected design situations must be sufficiently severe and varied as to encompass all reasonable conditions which can be foreseen to occur during the construction and use of the proposed structures.

The detailed specifications of design situations must include, as appropriate:

- the disposition and classification of the various zones of soil, rock and elements of construction which are involved in the calculation model,
- the actions, as defined in 3.1.2,
- the nature of the environment within which the design is set, including the following:

  - effects of scour, erosion and excavation, leading to changes in the geometry of the ground surface;
  - effects of chemical corrosion;
  - effects of weathering, including freezing;
  - variations in groundwater levels, including the effects of possible flooding, failure of drainage systems, etc.;
  - other effects of time and environment on the strength and other properties of materials.

3.1.2 Actions

An action is a group:

- of concentrated or distributed forces, acting on the structure (direct action)

or

- of deformations imposed on or contained in the structure (indirect actions).

Before any calculation is carried out, the designer must choose the forces and imposed displacements which will be treated as actions in that calculation.

guide: In geotechnical analyses, any of the following may be treated as actions:

- the weights of soil, rock and water,
- in situ stresses in the ground,
- free water pressures,
- ground water pressures,
- seepage forces,
- dead, imposed and environmental loads from structures,
- surcharge,
- removal of load or excavation of ground,
- traffic loads,
- movements caused my mining,
- swelling and shrinking caused by vegetation or climate,
- movements due to degradation, decomposition, self-compaction and solution,
- movements and accelerations caused by earthquakes, explosions, vibrations and dynamic loads,
- temperature effects, including frost heave,
- ice loading,
- imposed prestress in ground anchors or struts.

Actions are constants for the calculation model being considered. They are not unkowns in the calculation model.

guide: Some forces and imposed displacements are treated as action i certain calculations, and not in others. For example, in the design of sheet piled walls, the tie force is often treated in two different ways:

- when calculating the sheet pile section which is required, the tie force may be treated as a variable which depends on the stiffness of the sheet piles. It is not an action, guide: - when calculating the size of the tie rod, and the anchorage which is required, the tie force may be treated as a constant force, and is an action for those calculations.

Earth pressures are treated as actions in some design situations, but not in others.

In the analysis of simple earth retaining structures (walls), the retained soil is often considered to be in an active state. The pressure which it exerts on the structure is independent of the response of the system and is treated as an action.

In more complex earth retaining structures, such as an anchored cast in situ wall, the pressures exerted often depend upon the soil-structure interactions and are unknowns in the calculations. The are not actions.

Forces due to ties and ground anchors are considered as actions if they are independent of the response of the system being analysed. The component of force caused by controlled prestressing operations may always be regarded as an action.

Scour and erosion which cause a change in geometry, for example, by removing material at the toe of slope, are not actions. Their possible effects must be considered when selecting design situations (see section 3.1.1).

Actions which, when they are present, depend on one another and attain upper values at the same time, must be considered as a single action. Only those actions which have negligible dependence on one another can be considered as independent.

3.2 Derivation of Design Values for Actions

3.2.1 General

In deriving design values for actions for geotechnical calculations, attention must be paid to the following points.

a) For loads which act in combination, it must be considered whether all loads might attain their most adverse values coincidentally in position and/or simultaneously in time. The sets of design values adopted must be sufficiently adverse to conform to sections 2.3.2 and 2.3.3. For each design situation, it may be necessary to check several different load cases and combinations of actions.

guide: In Eurocode EC 2 and EC 3, load combination factors, ψ, are specified for use in deriving sets of loads acting in combination. This format can be used to calculate loads derived from structures but is not recommended for the geotechnical aspects of the design such as earth and ground water pressures since the values of the factors cannot be prescribed, and must be selected by the designer in relation to each particular design situation.

b) The duration of the loads must be considered with reference to time effects in the material properties of the soil, especially the drainage properties of fine grained soils.

guide: Depending of the type of soil being considered, it may be helpful to distinguish between:

- transient loads (for example wind loads) which act for a very short time during which the soil may display enhanced strength and stiffness;
- short term loads (for example, construction loads) which act for a period during which drainage of the soil will be negligible;
- long term loads.

c) Loads which are applied repeatedly must be identified for special consideration with regard to continued movements, liquefaction of soils, etc.

d) Loads which are applied cyclically with high frequency must be identified for special consideration with regard to dynamic effects.

e) Extreme loads which may be applied accidentally must be considered. It is normally appropriate to use these in combination with moderate values of other loads.

3.2.2 Dead Loads due to Soil and Rock

For soil and rock it is often unnecessary to allow for uncertainty in density and its distribution. Uncertainty in geometric parameters must be considered, however, as discussed in 2.3.2.

For soil which is known to be of very variable density, or for proposed fills which have not been designed in detail, a range of possible densities and distributions of density must be considered in design.

Where earth pressures are treated as actions, they must be evaluated according to the principles set out in Chapter 8.

3.2.3 Pressures due to Water

For limit states with servere consequences (generally ultimate limit states), design values for water pressures must represent the most adverse values which could occur in extreme circumstances. For limit states with less severe consequences (generally serviceability limit states), design values must be the most adverse which could occur in normal circumstances.

The following features which may effect the water pressures must be considered:

- the level of the free water surface or the groundwater table,
- the beneficial or adverse effects of drainage, both natural and artificial, taking account of its future maintenance,
- the supply of water by rain, flood, hydrological conditions, burst water mains or other means,
- changes of water pressure due to the growth or removal of vegetation.

guide: The risk of adverse water levels due to change in the water catchment, and reduced drainage possibilities (owing to blockage or freezing), etc. must be considered.

Unless the adequacy of the drainage system can be demonstrated and its maintenance ensured, it will often be necessary to assume that the groundwater table could rise to ground level in extreme circumstances. In some cases this could be considered as an accidental action.

3.2.4 Dead Loads due to Supported Structures

In many cases the design values for both serviceability and ultimate limit state calculations may be calculated from nominal design dimensions and mean unit weights.

The possible absence of part of the dead load, for example during construction, may sometimes be particularly adverse and must be considered as a separate design situation.

guide: Where it is significant, an allowance may be included for uncertainty in design dimensions and unit weights. Consideration of the ductile mode of failure of foundations suggests that if an individual foundation element supporting a redundant structure were to approach an ultimate limit state, the load aplied to the element by the structure would usually be reduced. It is therefore unnecessary in most cases to allow for adverse patterns of load transfer within redundant structures.

3.2.5 Imposed and Environmental Loads due to Supported Structures

For both the serviceability and ultimate limit states, imposed and environmental loads on foundations must be calculated as for the structures supported.

guide: With regard to wind loads, see section 3.2.1(b) above.

3.2.6 Other Actions

It is usually appropriate to derive design values for other actions either from national loading regulations or directly by consideration of the likely values and possible extreme values of the actions.

3.3 Deriviation of Design Values using the Method of Partial Coefficients

3.3.1 General

For the derivation of design values of actions due to supported structures the method of partial coefficients is appropriate in some cases. Use of this method does not, however, relieve the designer of the responsibility to check that the design values adopted are consistent with sections 3.1 and 3.2.

TYPE OF ACTION	CLASSIFICATION										
						Clay site			Sand site		
	P	V	A	FX	FR	L	S	T	L	S	T
Weight of soil	*			*		*			*		
Weight of structures	*			*		*			*		
Imposed loads in structure		*		?	?	?	?		*		
Wind loads		*			*			*			*
Snow loads		*			*		*		*		
Normal maximum water pressures	*			*		*			*		
Flood water pressures		?	?	?			*		*		
Seismic loads		?	?		?			*			*
Trafic loads		*			*	?	*		?		
Construction loads		*		?	?		*		?	?	?
Collision loads			*		*			*			
Temperature loads		*	?	?	?		?		?		*

Key: P = permanent, V = variable, A = accidental, FX = fixed, FR = free
L = long term, S = short term, T = transient, * = likely, ? = possible

Table 3.a. Typical Classification of Actions for Sites on Clay and Sand Soils

3.3.2 Classification of Actions

The actions are classified as defined in the following clauses.

guide: Examples of the use of these classfications are given in Table 3.a.

3.3.2.1 Permanent, Variable and Accidental Actions. This classification results from variation of each action with the time during the design situation under consideration.

Permanent actions are those which vary only infrequently (but with times of action which are probably long) or which vary in a negligible way from their mean value; or those which vary only in one direction tending towards an adverse limit.

Variable actions are those which are not likely to act throughout the duration of a design situation in a given project, or for which the variations in magnitude as a function of time are neither negligible in comparison to the mean value, nor monotonic.

Accidental actions are those for which the occurrence in a given structure and at a significant value is improbable.

3.3.2.2 Fixed Actions and Free Actions. This classification relates to the variation in space of each action.

An action is termed a fixed action if its magnitude and direction at every point in the structure is determined by defining the action at one point.

Actions are termed free actions if they can have an arbitrary spatial distribution over the structure, within certain limits.

Actions which cannot be defined as belonging to either of these groups can be considered as made up of a fixed part and a free part. This may apply for snow or wind loads on roofs.

guide: This classification is in principle independent of the classification given in 3.3.2.1; however, in practice, most of the free actions are variable, and many variable actions are free.

: Water pressure and permanent earth pressure may generally be considered as fixed.

3.3.2.3 Static and Dynamic Actions. This classification arises from the response of the structure or ground to the action.

Static actions are those which do not give rise to significant acceleration of the structure or of members of the structure.

Dynamic actions are those which can give rise to significant acceleration of the structure.

guide: In most cases the dynamic actions can be treated as static actions, taking into account the dynamic effects by an appropriate increase in the magnitude of the static actions. When this is not the case, a special treatment of safety is necessary in order to take account of the dynamic response of the structure.

3.3.2.4 Transient, Short Term and Long Term Actions. This classification arises from the duration of each action and is dependent upon the rate of response of the ground to the action.

Transient actions are those which act for a very short time during which the soil may display enhanced stiffness or strength.

Short term actions are those which act for a period during which drainage of the soil will be negligible.

Long term actions are those which act for a period during which drainage of the soil will be significant.

3.3.2.5 Constant and Repeated Actions. The classification arises from the influence of repeated or cyclic loading on the ground or structure.

Constant actions are those which do not fluctuate or vary cyclically in a manner which could have a cumulative effect on the ground or structure.

Repeated actions are those which fluctuate or vary cyclically in a manner which could have a cumulative effect on the ground or structure.

3.3.3 Design Values and Representative Values of Actions

3.3.3.1 General. Design values of actions, F_d, may be derived from representative values, F_r, using the equation

$$F_d = \gamma_F F_r \qquad 3.1$$

The same action can have different representative values for different design situations, according to the probability, frequency or duration of each situation.

The representative values must be derived by one of the following alternative approaches:

- they may be nominal values fixed by codes, standards or conctracts,
- they may be characteristic values determined by judgment. In this case an effort must be made to choose values such that the probability of being exceeded in an adverse sense is of the order of 5%,
- for variable actions, other representative values may be derived from the characteristic values as described in 3.3.3.2.

The magnitude and direction of earth pressure depend on the material properties of the soil. In calculations of earth pressure, therefore, partial coefficients are used for the material properties in contrast to other actions.

guide: The magnitude of the earth pressure in the serviceability limit state and the ultimate limit state is determined from two fundamentally different calculations. Consequently, when expressed as an action, earth pressure cannot be characterized by a single characteristic value.

3.3.3.2 Representative Values of Permanent Actions. The symbol G is used to represent either the characteristic or a nominal value of a permanent action, as defined in 3.3.3.1.

guide: For permanent actions, a unique representative value may normally be used. This occurs if the effect on the design of likely variations in the value is small, or if there is only one adverse characteristic value to be considered.

3.3.3.3 Representative Values of Variable Actions. The symbol Q_k is used to represent either the characteristic or a nominal value of a variable action, as defined in 3.3.3.1. The notation ψ_i Q_k is used to represent other representative values of the same action. In some cases the symbol ψ may represent multiplicative factores whose values are specified in national appendices. In other cases this is not so, and the combined notation ψ_i Q_k represents a characteristic or nominal value for a particular design situation.

The following representative values are defined for variable actions.

Characteristic values, Q_k. In some cases these may be replaced by nominal values taken from codes, standards or contracts.

Combination values, ψ_0 Q_k. In some cases these may be replaced by nominal values taken from codes, standards or contracts.

guide: These values are associated with the use of combinations of actions (see 3.3.4). They permit the assessment of the effects of actions taking account of the fact that the simultaneous attainment of their characteristic values by several actions is highly improbable.

Frequent values, ψ_1, Q_k. These are determined such that the total time during which they are exceeded is only a small part of the reference period, or that the frequency of their exceedence is limited.

guide: These values are principally associated with serviceability limit states whose attainment is connected with repeated load applications.

Quasi-permanent values, $\psi_2 Q_k$. These values are determined such that the total period during which they are exceeded is a large part of the reference period.

guide: A common example is the proportion of total imposed load taken to be relevant to long-term settlement calculations.

3.3.3.4 Representative Values of Accidental Actions. Each accidental action is generally represented by a single value F_A.

guide: This value will correspond directly to the degree of safety required; it represents the value of the action beyond which safety is not assured. In some cases, this value will be a nominal value fixed by a code, standard or contract.

3.3.4 Load Cases and Combinations of Actions

Load cases are the arrangements of free actions which are introduced in the calulations. They take account of the variation in location of the free actions.

Combinations of actions are collections of design values which are introduced into the calculations when several actions are to be considered simultaneously. They take account of the variation in magnitude of actions which may act simultaneously.

Most of the permanent and variable actions are included in most combinations, the more unfavourable value being used for those with upper and lower representative values. Only one accidental action may occur in a combination.

The following combinations are defined:

For ultimate limit states:

- fundamental combinations
- accidental combinations

For serviceability limit states:

- rare combinations
- frequent combinations
- quasi-permanent combinations.

		Representative values of actions					
				Variable Actions Q			
Limit State	Combinations	Permanent actions G	Accidental actions F_A	Characteristic value Q_k	Combination value $\psi_0 Q_k$	Frequent value $\psi_1 Q_k$	Quasi-permanent value $\psi_2 Q_k$
ultimate	fundamental	$\gamma_g G$	-	$(\gamma_q Q_k)_1$	$(\gamma_q \psi_0 Q_k)_{i>1}$	-	-
	accidental	$\gamma_{ga} G$	F_A	-	-	$(\psi_1 Q_k)_1$	$(\psi_2 Q_k)_{i>1}$
serviceability	rare	G	-	$(Q_k)_1$	$(\psi_0 Q_k)_{i>1}$	-	-
	frequent	G	-	-	-	$(\psi_1 Q_k)_1$	$(\psi_2 Q_k)_{i>1}$
	quasi-permanent	G	-	-	-	-	$(\psi_2 Q_k)_{i>0}$

Table 3.b. Symbolically Representation of Combinations of Actions

These combinations are defined in 3.3.4.1 and 3.3.4.2 and represented symbolically in Table 3.b.

For some design situations it will be necessary to define additional combinations.

guide: It will usually be necessary to check the fundamental combinations.
: Judgment is required to decide which other combinations must be
: checked.

3.3.4.1 Combinations for the Ultimate Limit State

The fundmental combination of actions may be represented by

$$\gamma_G G + \gamma_{Q1} Q_{1K} + \sum_{i>1} \gamma_{Qi} \psi_{oi} Q_{ik} \qquad 3.2$$

where

G	is the collection of permanent actions
γ's	are the partial load factors taken from the national appendices
Q_1	is the "basic" variable action, selected to give the most critical combination of loads; if necessary several alternative actions must be tested to find the most critical
$Q_i(i>0)$	are the other variable actions
ψ_0	is as defined in 3.3.3.3.

The accidental combinations may be represented by

$$\gamma_{GA}\, G + F_A + \psi_1\, Q_{1k} + \sum_{i>1} \psi_{2i}\, Q_{ik} \qquad 3.3$$

guide: : It is usualy appropriate to consider only one accidental action in this formula. In some cases the accidental situation may not itself be an action, but it may represent conditions immediately after an accidental event such as an explosion.

In geotechnical design it is normally considered advisable to chose the nummerical values of the partial coefficients in such a way that γ_G and γ_{GA} equal unity.

guide: : By chosing $\gamma_G = \gamma_{GA} = 1$ the problem of identifying the part of a soil mass that acts as a stabilizing force and the part that acts as a driving force is avoided. In this way formal conflicts between geometry (ground water table) and actions (water pressures) are also avoided.

3.3.4.2 Combinations for the Serviceability Limit State. The rare combinations may be represented by

$$G + Q_{ik} + \sum_{i>1} \psi_{oi}\, Q_{ik} \qquad 3.4$$

guide: These combinations are concerned with the short term limit states, concerned with one simple attainment of a certain value of the effect being studied. Such a combination might be used, for example, to study the effects on serviceability of flooding in servere storm conditions.

The frequent combinations may be represented by

$$G + \psi_1 Q_{1k} + \sum_{i>1} \psi_{2i} Q_{ik} \qquad 3.5$$

guide: These combinations are to be considered for actions with medium term durations or which repeat at intervals.

The quasi-permanent combination may be represented by

$$G + \sum_{i>o} \psi_{2i} Q_{ik} \qquad 3.6$$

guide: This combination should be taken into account when considering long term effects such as consolidation settlement.

In particular cases, partial coefficients γ_F different from unity may be required in the combinations of actions for serviceability limit states.

3.4 Derivation of Design Values when the Method of Partial Coefficients is not Used

When the method of partial coefficients is not used, design values must be consistent with the principles outlined in section 3.2.

guide: In cases where several independent actions each have a significant influence on the design, it is often appropriate to carry out a parametric study. For this purpose, it is recommended that an approach using the concept of a "lead variable" should be considered. This requires that each action in turn is set to an extremely adverse value, whilst less severe values are adopted for other actions.

The partial coefficients set out in national appendices indicate

guide: the level of safety considered appropriate for conventional designs.
: These may be used as guidance to the required level of safety when
: the method of partial coefficients is not used.

CHAPTER 4 - GEOTECHNICAL DATA

4 GEOTECHNICAL DATA

4.1 General

Careful collection, recording and interpretation of geotechnical data are an essential part of geotechnical design. It is necessary to study the geology, morphology, hydrology and history of the site as well as to evaluate the parameters which are to be used in calculations. It is often necessary to involve geotechnical specialists in this work.

4.2 Geotechnical Investigations

4.2.1 Introduction

The aim of a Geotechnical investigation is to obtain adequate and reliable data on the soil and ground water conditions in order to verify that the performance criteria for the geotechnical structure are satisfied.

The geotechnical category of the structure determines the character and extent of the investigations. The ground conditions may determine the geotechnical category and are to be established as early as possible in the investigation.

guide: Geotechnical investigations can be classified into three phases:
:
: - preliminary investigations see Section 4.2.2,
: - design investigations see Section 4.2.3,
: - control investigations see Chapter 10.

4.2.2 Preliminary Investigations

Preliminary investigations are carried out:

- to assess the general suitability of the site,
- to compare alternative sites,
- to determine the changes which may be caused by the proposed works.

A preliminary investigation must provide the advance information which is needed to plan any further investigation that is required. Preliminary investigations must include:

- topography,
- hydrology,
- examination of neighbouring structures and excavations,
- geological records,
- previous site investigations in the vicinity,
- aerial photographs,
- old maps,
- any other relevant information.

The site must be inspected at an early stage of the investigation. Obvious topographic, historic or geological features must be recorded.

4.2.3 Design Investigations

4.2.3.1 General. Design investigations are carried out:

- to provide the information required for an adequate and economic design of the permanent and temporary works,
- to provide the information required to plan the method of construction,
- to identify any difficulties that may arise during construction.

A design investigtion must adequately identify the disposition and properties of all relevant soil strata. The parameters which affect the capacity of the structure to satisfy its performance criteria must be established before final design commences.

Investigations techniques include:

- geophysical surveys,
- boring with sampling,
- trial pits with sampling,
- in-situ tests,
- determination of ground water levels,
- pore pressure measurements,
- pumping tests,
- laboratory tests.

A geological evaluation must be made in order to ensure that the investigation covers all relevant soil formations. Investigation must normally be carried out at least down to strata which the engineer responsible for the investigation can classify geologically, and beyond which the strata can have no substantial influence on the behaviour of the structure. Particular attention must be paid to the follwoing:

- solution cavities,
- secondary consolidation,
- settlement due to degradation,
- soil creep,
- hydrological effects.

guide: Where soundings are made it is often necessary to carry out borings in order to identify the soil in which the soundings are made. If the geology of the site is well known, these may be omitted.

The ground water pressures acting during the investigation must be established. The extreme levels of any free water which might influence the ground water pressures must be established and the free water levels during the investigation must be recorded. The location and capacities of any dewatering or water abstraction wells in the vicinity of the site must be established.

4.2.3.2 Geotechnical Category 1. For structures in Geotechnical Category 1 no distinction is made between preliminary, design and control investigations.

The site and the upper layers of soil must be inspected, by means of shallow test pits, hand operated penetrometers or auger borings.

Ground water conditions must be assessed from inspections of the site made before and during construction. If an appreciable flow of water, or incipient erosion, is discovered during an inspection, then the structure must be treated in Geotechnical Category 2. Reference must be made to local experience and general knowledge of the ground conditions in the vicinity of the site.

For pile foundations the investigation should, as a minimum, aim at determining the depth of the bearing stratum. The investigation must confirm that negative skin friction derived from strata above the bearing stratum will not be significant. If this is not confirmed, the structure must be treated in Geotechnical Category 2.

4.2.3.3 Geotechnical Category 2. For structures belonging to this category the investigation must normally include in-situ tests, borings, and laboratory tests.

No general minimum requirements can be specified for this category. The distance between the exploration points is dependent on the geology of the area, ground conditions, and size of site. In uniform soil conditions the borings or excavation pits may partially be replaced by geotechnical or geophysical soundings.

guide: For footings the minimum depth of in-situ tests or borings below anticipated foundation level is normally between 1 and 3 times the width of the foundation. Greater depths must usually be investigated in some of the exploration points to ascertain settlement conditions and ground water problems. The strength and deformation parameters of the load-bearing soil strata must be established either by direct measurement or empirically.

: For rafts, filled areas and embankments the minimum depth of in-situ tests or borings is normally equal to or less than the foundation width.

: For piled foundations, borings, soundings, or in-situ tests must normally be performed to explore the soil conditions to a depth at least 10 times the width of the shaft of the pile below the anticipated level of the pile point.

The investigation of ground water problems must normallly include:

- observations of the water levels in boring and standpipes and of their fluctuations with time,
- an evaluation of the hydrology of the site.

guide: For excavations, the pore water pressures to a depth below the excavation which equals the depth of the excavation below ground water level should be established, in order to evaluate uplift.

4.2.3.4 Geotechnical Category 3. The extent of the investigation should at least be sufficient to meet the requirements for Geotechnical Category 2. Additional investigations of a more specialized nature will often be necessary.

guide: These may include:

- special geological investigations,
- special geophysical investigations,
- special laboratory tests,
- special in-situ tests,
- load tests on piles,
- load tests on anchors,
- plate loading tests,
- trial embankments with settlement observations,
- deformation measurements,
- measurements of pore water pressure,
- special borings,
- pumping tests,
- surveys of seismic conditions.

4.3 Field Investigations

Field investigations must be carried out and reported generally in accordance with published international or national standards. Deviations from these standards and additional test requirements must be specified by engineers with experience in geotechnical testing who will be responsible for the interpretation of the results.

guide: In the following subparagraphs the main points of the most frequently used investigations are given.

Field investigations may be grouped as follows:

- testpits, deep shafts, borings, and sampling,
- in situ tests,
- geophysical tests.

The geotechnical engineer has a wide assortment of field tests at his disposal.

Some of these tests are aimed at a direct in situ determination of basic geotechnical parameters. This may for example be the case with pressuremeter tests, vane tests, pumping and other in situ permeability tests, in situ density tests, and plate load tests.

More often complex soil properties are determined which are indirectly related to the basic soil mechanical parameters. Such tests are for example Cone Penetrometer tests, Standard Penetration Tests, Dynamic Probing, and plate and pile load tests. Sometimes the results of these tests are used directly in calculations. Otherwise, the results are either used in a purely empirical way, or basic soil parameters are derived from them by theoretical methods.

Boring and sampling of undisturbed soil cores require great care and special attention.

In some cases the field investigations are initiated by geophysical tests.

4.3.1 <u>Testpits, Deep Shafts, Borings, and Sampling</u>

The following data must be recorded for every testpit, deep shaft, or boring:

- the type of boring,
- the position of the boring on the site,
- the accurate groundlevel,

- the type of sampling method,
- a complete and accurate log of the boring including exact data on the depth and length of the samples,
- the level of water in the borehole and outside and its variations with time,
- the data on the in situ tests made in the boring,
- all detailed observations made during the execution of the boring.

guide: It may be necessary to apply methods deviating from international or national standards because of local experience. The selection of the type of boring and sampling, the number of borings and quantity and the depth of the samples must take account of the geotechnical problems under investigation.

Disturbed and undisturbed samples may be taken in a boring or testpit for the determination of the soil characteristics and parameters described in the section 4.3.

Some in situ tests require the execution of a boring for example the pressuremeter test, the permeability test, the Standard Penetration Test, etc....

In some cases boring is required only for the execution of in-situ tests, but sometimes it is also possible to take undisturbed cores.

4.3.2 In Situ Tests

Although other in situ tests exist, the code is restricted to the following tests:

- static cone penetrometer test,
- standard penetration test,
- dynamic probing,
- pressuremeter test,
- vane test,
- piezometer test,
- permeability test.

The following general information must be included in the test report:

- the position of the test on the site,
- the dates of execution,
- the groundlevel at the location of the test,
- the material and procedures used during the test,
- all particulars experienced during the performance of the test which may be of importance for the geotechnical problem at hand,
- the data obtained by the test.

4.3.2.1 Cone Penetrometer Test. The Cone Penetrometer Test is normally used for the following purposes:

- to obtain a contiunous picture of soil strength with depth,
- to complete the overall picture of the soil profile at the building site (in addition to geophysical investigation and borings),
- to get information on the soil type of the soil layers penetrated,
- to get qualitative data about the compressibility and deformation parameters of the soil,
- to derive shear strength parameters of the various soil layers.

Furthermore, the results are applied directly for the prediction of the ultimate bearing capacity (see chapter 7) of piles.

The results must be presented in the form of a diagram in which the measured soil resistance is plotted with depth. The type of cone must be clearly indicated in the diagram.

guide: Standard scales must be respected for the presentation of the diagrams.
: A Standard penetrometer is recommended, but divergences from the
: standard may also be used.

guide: The results of the measurements with the various cones may exhibit differences which are attributed to the shape of the cone. It is therefore very important to record the type of cone used on the same sheet as the test results.

The cone penetrometer test is stopped when the cone resistance is too high (rock, gravel, very dense sand,) or when the side friction resistance becomes to high with depth. Procedures or equipment are avilable to perform the test at greater depth in some particular soil conditions.

Soil type can be deduced from the diagram by experience. There are also graphs which may be used to derive the soil type from the ratio between the local side friction and the cone resistance.

It is unreliable to derive the angle of internal friction for sand or sandy layers from the cone resistance.

If a relation between cone penetrometer results and compressibility or deformation exists, such a relation is unreliable. Very global empirical relations are sometimes used for a first approximation of the settlement and deformation behaviour of foundations, dikes and embankments. It is necessary to calibrate the results against computations with compression or deformation moduli obtained from laboratory tests on undisturbed samples or with settlement data of existing structures in the neighbourhood.

4.3.2.2 Standard Penetration Test. The Standard Penetration Test is used in cohesionless soils for the following purposes:

- the estimation of soil mechanics characteristics such as relative density, strength and deformability for cohesionless soils
- the direct calculation of bearing capacity for shallow or deep foundations.

For cohesive soils and soft rock the test may be used to obtain some information about the mechanical characteristics of the soil.

The Standard Penetration Test can be performed in all kinds of soil and weak rock but the sampler is equipped with a solid driving shoe in gravelly soil.

During the performance of Standard Penetration Test it is important to maintain a constant water level in the borehole. This water level should be recorded.

To avoid hydraulic disturbance when boring in sand, the water pressure must correspond to ambient water level: special attention must be paid when boring in artesian conditions.

guide: The results are presented in a table giving the penetration resistance
: (number of blows required for a penetration from 0.15 to 0.45 m from
: the bottom of the borehole) in function of depth.

4.3.2.3 Dynamic Penetration Test). The Dynamic Penetration Test is normally used for a qualitative investigation of soils with the aim of:

- the control of the homogeneity of a building site,
- the determination of the thickness of ground layers and more especially with the presence of dense layers which cannot be penetrated in any other way,
- to locate holes or other discontinuities,
- to locate bed-rock.

It gives also useful information for the prediction of the driving conditions of piles or sheet piles.

Empirical rules have also been put foreward for the use of Dynamic Penetration results for the calculation of foundations, but they must be used with great care.

guide: The results of dynamic probing test may be presented in the form of
: a diagram giving the numer of blows for a given penetration and the
: resistance values deduced from an energy formule plotted with depth.
: The method (type A - elimination of friction along the rods or
: type B - presence of friction along the rods) must be clearly indicated
: on this diagram.

guide: Sometimes light dynamic probing is used for the same purpose as heavy dynamic probing methods A and B, but in less dense soil, to a shallower depth, and when the site is less accessible to heavy apparatus.

4.3.2.4 Pressuremeter test. The pressuremeter test is normally used to determine the soils parameters, pressuremeter modulus and limit pressure which are subsequently used for the following purposes:

- to obtain a overall picture of the soil profile at the building site (completed by the data coming from the borings)
- to calculate the ultimate bearing capacity and to estimate the settlement of shallow foundation (see chapter 6)
- to calculate the ultimate bearing capacity (base resistance and side friction resistance) of piles (see chapter 7)
- to estimate the ultimate bearing capacity of soil anchors
- to estimate the horizontal modulus of subgrade reaction and the ultimate reaction pressure used in design calculations for piles subject to lateral loads,
- to estimate the horizontal modulus of subgrade reaction used in deformation analyses of retaining structures such as cast in-situ diaphragm walls or anchored bulkheads.

guide: The pressuremeter test is performed in a boring by means of a cylindrical rubber bladder - the measuring cell - which is radially inflated into the ground at a given depth. A diagram giving the volume changes versus the pressure in the cell is obtained. From this curve, it is possible to obtain a strain-stress relation for the soil in plane strain conditions from which the pressuremeter modulus E_M and the limit pressure p_1 may be deduced.

There are different types of pressuremeter. The choice of equipment depends on the nature and the soil conditions (soft, dense,..). Special equipment is available to protect the bladder or to maintain the borehole open when necessary.

4.3.2.5 Vane Test. The vane test is normally used to assess the undrained shear strength of saturated clay and silt. The ratio between the peak and the residual values of the shear strength gives an estimate of the sensitivity of the clay. If used in fissured clay or clay with a relatively high organic content, the results of vane tests must only be used after evaluation on the basis of extensive local experience.

guide: The vane test is performed by measuring the torque to be applied at a given rate of time to move a vane in the ground. A vertical log can be obtained by repeating the test at several depths.

This test is not standardized at international level but the following recommendations are to be made:

- the height-diameter ratio of the vane is 2
- the maximum torque is to be reached after around 2 minutes
- care should be taken to eliminate the friction between the vane rod and the surrounding soil.

4.3.2.6 Piezometer Test. Piezometer tests are normally used to asses

- the water pressure at a given point,
- the presence of different layers and especially of artesian conditions
- the variation of the water pressure with time due to seasonal or tidal conditions or following works executed in the neighbourhood (pumping, recharging, injections, loading or unloading of the ground, etc.....).

The following piezometers are normally used

- open standpipe piezometers
- closed standpipe piezometers
- constant volume piezometers.

Special care must be taken in installing a piezometer to obtain a perfect seal between layers subjected to different water pressures.

guide: When the permeability is high (sand, gravel) an open piezometer may be used; for less permeable soils (clay, silt) piezometers with constant volume are to be used.

In interpretation of the results of piezometer tests account should be taken of the fact that the response time of a piezometer is a function of the permeability of the layer considered.

4.3.2.7 Permeability Test. Permeability tests are normally used to assess the value of the coefficient of permeability and its variation in the ground.

The following permeability tests are normally used

- constant head type (Lefranc test or permeameter test)
- variable head type.

guide: The Lefranc test is performed by measuring the rate of discharge at the bottom of a borehole submitted to a hydraulic head by pumping. The coefficient of permeability of the surrounding ground and its variation with depth may be deduced from the test results.

The permeameter test is performed in a borehole by means of equipment which injects water under a given pressure in a section of the borehole. The permeability may be deduced by measuring the rate of discharge.

In interpreting the results of these tests consideration must be given to the following:

- the tests concerns only a small volume of the ground and do not give the overall permeability of a site; nevertheless, by performing a number of tests on a site, it is possible to obtain valuable information about the structure and the hydraulic heterogeneity of the ground
- the execution of the tests must be controlled carefully. When water is injected the drain may clog, and when a borehole is pumped fines may collect in the borehole,
- the calculation of permeability depends on hypotheses about the soil profile (heterogeneity, anisotropy, confined or unconfined water table, ...) which are difficult to assess.

The best way to determine the overall permeability of a site is normally to perform a pumping test in which water is pumped from a borehole at constant rate, and several piezometers are installed at increasing distance of the borehole.

From observations of the rate of discharge and the water pressure in the piezometers, the overall coefficient of permeability and the transmitivity of the tested layer may be deduced.
The test may be performed in permanent or in transient conditions.

4.3.3 Geophysical Tests

The aim of geophysical tests is to give a quantitative account of the properties of the ground.

The interpretation of the results of geophysical tests should be done by an expert in geophysical tests having geotechnical knowledge. The results of such tests should be checked against existing geotechnical knowledge and experience.

Geophysical tests are classified as
- seismic and/or sonic tests
- geo-electrical tests.

guide: In some cases, geophysical tests preceed the borings and in-situ
: tests and provide useful information for the programme and the planning of the borings and of the other field tests.
: Geophysical tests are often used when a soft layer is resting
: on a more dense layer (for example a dense sand layer, a rock layer)
: in order to estimate the thickness and the extent of the soft
: layer.
: To investigate the presence of a less dense zone or a hole (for
: example in karstic zones), the use of gravimetric tests is re-
: commended.
: In seismic and sonic tests, the velocity of shear and compression
: waves in the ground is measured in such a way that data concerning
: thickness, slope and quality of the soil layers may be derived from
: the results.
: In geo-electrical tests, the electrical resistance of the soil
: is measured in order to establish the thickness and extent of
: soil layers in the ground.

4.4 Laboratory Investigations

Laboratory tests must be carried out and reported generally in accordance with published international or national standards. Deviations from these standards and additional test requirements must be specified by engineers with experience in geotechnical testing who will be responsible for the interpretation of the test results. The procedures used for sampling, transportation and storage must be reported and considered in interpretating of test results.

guide: In the following subparagraphs the main points of the most frequently used tests are given. For the purpose of establishing a unified presentation and performance of tests, the requirements about reporting of test results are outlined with particular emphasis on the consolidation and triaxial test.

Laboratory tests on soils may be grouped as follows:

- identification,
- compressibility and strength,
- compaction,
- chemical tests on soils and ground water,
- other tests.

4.4.1 Tests on Soils for Identification Purpose

This group includes, but is not limited to, the following determinations: moisture content, particle size distribution, Atterberg limits, dry unit weight, specific gravity of solid particles, and relative density.

Classification of soils is based on their particle size distribution and plasticity characteristics.

On the basis of the results from the above tests it is possible to obtain, with the aid of empirical correlations, indications about strength, compressibility, swelling potential, collapsing properties, dispersivity, etc.. Such correlations should be used with precaution.

In the following, the main points of the most important tests are given.

(a) Moisture Content

Moisture content of a soil is determined as a percentage of its dry mass.

(b) Particle Size Distribution

Gradation is determined by sieving and sedimentation.

(c) Atterberg Limits

Atterberg limits of a soil sample should be accompanied by natural water content determinations and particle size distribution curves. Atterberg limits may be determined also on samples of soil at their natural state.

(d) Dry Unit Weight

Density determinations can be made by the water displacement method, or on the basis of geometrical characteristics of the samples.

(e) Specific Gravity of Solid Particles

The specific gravity of solid particles of a soil sample can be determined on oven dried samples with the aid of calibrated pycnometers.

(f) Relative Density

Relative density expresses the degree of compactness of a cohesionless soil with the respect to the loosest and the densest conditions that can be attained by specific laboratory procedures, for which a complete description must be given.

4.4.2 Compressibility and Strength Tests on Soils

These tests are performed on undisturbed samples or on laboratory prepared specimens for the purpose of determining the compressibility and strength characteristics of soil.

This group includes, but is not limited to, the following tests: one-dimensional consolidation, unconfined compression and all types of triaxial and direct shear tests, plus the California Bearing Ratio test.

In the following the basic requirements of each type of test are given.

(a) One Dimensional Consolidation Test

The one dimensional consolidation test is used to determine the compressibility and rate of consolidation of soils when they are restrained laterally, subjected to vertical axial pressure, and allowed to drain freely from the top and bottom. Secondary consolidation effects and history of the sample can also be studied.

guide: Loading of the specimen can be achieved either by weights through a lever arm system or hydraulically. Each load will be maintained on the specimen until the slope of the characteristic linear secondary portion of the thickness vs. log of time plot is apparent. Special loading and unloading schedules may be specified to suit the requirements of a particular project.

Reporting of results of one dimensional consolidation tests must include the following:

- sample size
- plot of voids ratio (or strain) vs. log of applied pressure
- time curves
- plot of coefficient of consolidation vs. log of vertical stress
- plot of coefficient of volume compressibility (or constrained modulus) vs. consolidation pressure
- tabulation of all relevant data
- complete identification of the sample and its physical properties
- equipment and test procedures used.

(b) Unconfined Compression Test

The primary purpose of the unconfined compression test is to obtain quickly approximate quantitative values of the undrained compressive strength of soil possessing sufficient coherence to permit testing in the unconfined condition. The test is restricted predominantly to nonfissured soils.

(c) Triaxial Tests

The triaxial test is used to determine shear strength parameters of cylindrical soil specimens in the triaxial apparatus on the basis of the stress-strain behaviour of the soil under controlled drainage conditions. Pore pressure parameters as well as deformability characteristics, can also be assessed.

The main types of triaxial tests are the following:

- Unconsolidated Undrained (UU) tests. These are usually carried out without pore pressure measurements. If pore pressure measurements are required, the tests are designated as UUPP.

- Consolidated Undrained tests with Pore Pressure measurements (CUPP). A suitable back pressure is applied prior or after the consolidation stage, to saturate the specimens and maintained through the duration of the test. Specimens are consolidated either isotropically or anisotropically and then sheared. The rate of strain during shearing is determined on the basis of the consolidation of the specimens.

- Consolidated Drained (CD) tests. Specimens are consolidated either isotropically or anisotropically and then sheared by load increments sufficiently small and applied at sufficient time intervals that no significant pore pressures develop. Rate of strain during shearing is determined on the basis of the consolidation of the specimens.

Reporting of results of triaxial tests must include the following:

- plots of deviator stress vs. strain curves,
- plots of pore pressure vs. strain (for UUPP and CUPP tests),
- plots of principle stress ratio vs. strain,
- plots of volume change vs. strain,
- Mohr circles,
- stress path diagrams,
- tabulation of all relevant data,
- complete identification of the sample and its physical properties,
- equipment and test procedures used.

Other types of triaxial tests, e.g. controlled stress path tests, extension tests, cyclic loading tests, constant volume tests, may be specified to suit the particular requirements of a project.

(d) Direct Shear Test

The purpose of the direct shear test is to determine the shearing resistance along a predetermined plane within a circular or square soil specimen in the shear box.

The direct shear test is suited to a consolidated drained test. Rate of strain during shearing is determined on the basis of the consolidation of the specimens. "Quick" consolidated or unconsolidated undrained direct shear tests should be avoided.

Direct shear tests of the consolidated drained type with multiple reversals of the shear stress are suitable for residual strength determinations particularly when made along weak planes within the soil material.

The test is not suited to the development of exact stress-strain relationships within the test specimen because of the non-uniform distribution of shearing stresses and displacements.

(e) California Bearing Ratio (CBR) Test

The CBR test is designed to give an evaluation of the bearing capacity of soil for flexible pavement design. The results of tests on natural or recompacted soils, in soaked or unsoaked conditions can be compared with standard test results curves.

4.4.3 Compaction Tests on Soils

Compaction tests are performed in order to study the moisture-density relation of a soil for specific compaction procedures.

guide: Molds for compaction tests can vary depending on the gradation of the materials, provided compactive effort remains unchanged.

4.4.4 Chemical Tests on Soil and Ground Water

The purpose of these tests is to assess the possibility of deterioration of buried steel and concrete foundation structures, and to investigate hazards arising from toxic waste.

Organic content determinations, together with the results of other classification tests can be used to assess the degree of of compaction that can be achieved with organic soils, and the long-term behaviour of such soils under structural loads.

Standard methods should be used, as far as possible, for chemical determinations on soils, aqueous soil extracts and groundwater.

This group includes, but is not limited to, the following chemical determinations: pH values, sulphate, carbonate and organic content. Determinations of pH can be made either with the aid of electrically operated pH meters or with the colorimetric method using a suitable chemical indicator.

Organic content determinations should be confirmed by more than one standard method and if necessary on the basis of Atterberg limits determinations on air-dried and oven-dried samples.

4.4.5 Other Tests

Other common laboratory tests on soils are the shrinkage limit and permeability tests. In addition to these, special tests are performed in order to study other properties such as swelling potential, dispersibility etc.

In the following supparagraphs the main points of the first two tests are outlined.

(a) Shrinkage Limit

The object of this test is to determine the water content level at which any further reduction in the water content will not cause a corresponding decrease in the volume of the soil mass.

guide: Volume measurements can be made on the basis of geometrical characteristics, or with the aid of the mercury displacement method.

(b) Permeability Tests

Constant head or falling head permeability tests can be carried out in permeameter cylinders, sampling tubes, or in the triaxial apparatus and in specially modified oedometers.

The preferred method is using the triaxial compression chamber, because this minimizes leakages along the boundaries of the specimen.

4.5 Evaluation of Geotechnical Parameters

4.5.1 General

In order to carry out designs it is often necessary to express the characteristics of soils and rocks in a quantitative manner using geotechnical parameters. Design values of geotechnical parameters are often derived from laboratory or field tests for use in analytical calculations.

Parameters derived from field tests, such as Standard penetration test, cone penetrometer and pressuremeter tests, may also be used directly in design calculations based on empirical relationships. These are sometimes found to be more reliable than analytical calculations, especially in familiar ground conditions for which the empirical relationships are well established.

Design values of geotechnical parameters must be based on a careful assessment of the range of values which might be encountered in the field. This assessment must take account of all available information, including geological and other background information, and the results of laboratory and field tests. Where information is found to be in conflict, an explanation of the discrepancy must be sought.

The values which are selected for the parameters must be appropriate to the particular limit mode (or method of calculation) under consideration.

guide: Many material parameters are not true constants and it will sometimes be necessary to adopt different values for one parameter for different limit states in the same ground.

It will be rarely possible to establish design values of geotechnical parameters with sufficient confidence solely on the basis of a single type of test.

For the value of each parameter relevant published data must be considered, together with local and general experience. Published correlations between parameters must also be considered when relevant.

In interpreting test results, published information relevant to the use of each type of test in the appropriate ground conditions must be considered.

Testing schedules must include sufficient tests to provide results representative of the variation of material properties relevant to the design.

Whenever possible, the results of large scale field trials and measurements from full scale constructions should be analysed in order to check values of parameters.

4.5.2 Identification of Soil Type

The character and basic constituents of the soil or rock must be identified before the results of other tests can be interpreted. The material must be inspected visually and described in accordance with a recognised nomenclature.

guide: The main tests used for identification purposes are grading analyses to determine the particle size distribution, Natural moisture content and Atterberg tests to determine plasticity characteristics.

4.5.3 Unit Weight and In Situ Density

The unit weight of the soil must be measured with sufficient accuracy to determine design values of the actions which derive from it. Design values are to be derived as given in Section 3.2.

guide: For saturated cohesive soils the saturated density or unit weight may be measured in the laboratory using undisturbed samples. For other soils, the bulk density may be measured in situ, usually by removing a measured weight of soil from the ground and filling the void which is left by a measured volume of another material.

guide: In-situ densities may often be estimated with sufficient accuracy on the basis of the soil type, grading and tests or observations which indicate the strength of the soil such as penetration tests.

4.5.4 In-Situ Density

Natural or man-made variations or layering must be considered in the use of tests to measure in-situ density.

guide: In order to obtain a direct measure of the state of compaction or relative density of a soil, an accurate measurement of its in situ density is required. This is compared with laboratory values of its density after standard amounts of compaction.
The in-situ density may often be assessed for a particular soil type and grading on the basis of tests or observations which indicate the strength of the soil, such as penetration tests.

4.5.5 Strength

4.5.5.1 Undrained Shear Strength of Soils. It is conventional to express the strength of soil in terms of total stresses by the undrained shear strength c_u.

In assessing the undrained shear strength parameter, the following features must be considered:

- differences between the stress situations in situ and in a test,
- sample disturbance, especially for laboratory tests on samples obtained from boreholes,
- anisotropy of strength, especially in clays of low plasticity,
- fissures, especially in stiff clays. Test results may represent the strength either of the fissures or of the intact clay, and either of these may govern field behaviour. Sample size may be important,
- rate effects. Tests carried out quickly tend to yield higher strengths,

- large strain effects. Most clays exhibit a loss of strength at very large strain and on preformed slip surfaces,
- time effects. The period for which a soil will be effectively undrained depends on its permeability, the availability of free water and the geometry of the situation. Some soils exhibit enhanced strength for loading of very short duration,
- inhomogeneity of samples, such as inclusions of gravel or sand within a sample of clay,
- degree of saturation, especially in undrained tests,
- the level of confidence in the theory used to derive undrained shear strength from the test results, especially for in situ tests.

guide: Methods which may be used to assess undrained shear strength are listed in table 4.5 a. The methods are not listed in order of preference.

guide:

LABORATORY TESTS	
1. Penetration and moulding tests carried out in the hand 2. Unconfined compression tests	Only suitable for very approximate assessment of strength
3. Triaxial tests on "undisturbed" specimens 4. Plane strain tests in suitable laboratory apparatus	The features listed in Section 4.5.5.1 must be considered when assessing the test results
5. Hand held penetrometer (Laboratory or in situ) 6. Hand held shear vane (Laboratory samples or in situ) 7. Correlation with moisture content or liquidity index 8. Correlation with CBR	Suitable for an approximate assessment of strength, and to establish reliable design values if a correlation with other measurements has been established for the soil in question
IN-SITU TESTS	
9. Field shear vane of diameter not less than .... mm 10. Pressuremeter test 11. Plate bearing test	The features listed in Section 4.5.5.1 must be considered when assessing the test results
12. Cone penetrometer 13. Standard penetration test	Suitable for an approximate assessment of strength, and to establish reliable design values if a correlation with other measurements has been established for the soil in question
14. Correlation with overburden pressure, established in laboratory tests	May be used to establish a lower bound in normally consolidated clays

Table 4.5 a. Some of the common Field and Laboratory Tests from which undrained Shear Strength (c_u) can be assessed

4.5.5.2 Effective Stress Parameters. It is conventional to represent the strength of soil in terms of effective stresses by the effective cohesion (c') and the angle of shearing resistance (ϕ'). In assessing the drained shear strength parameters, the following features must be considered:

- the values of c' and ϕ' must only be assumed constant within the range of stresses for which they have been evaluated; at low stresses c' may tend to zero and at high stresses ϕ' may have a reduced value,

- the value of φ' consists of two components, one depending on the intrinsic frictional properties of the soil (the critical state angle of friction) and an additional component depending mainly on density and stress level. The former component may be considered constant whilst the latter will change if the soil dilates or compacts. The value of c' also depends on density and stress level,
- the value of φ' depends on the density and the packing of the soil particles. These are readily altered by disturbance during sampling, and this must be considered in analysing test results
- in plane strain, soils generally exhibit a slightly higher value of φ' than in triaxial tests.

guide: Some of the common field and laboratory tests which may be used to assess effective stress parameters are listed in table 4.5 b. The methods are not listed in order of preference.

1. Penetration of in situ material with hand tools	Only suitable for very approximate assessment of φ'
2. Correlation with results of compaction tests, relative density, grading and angularity	For φ' only. Suitable for an approximate assessment of φ', and to establish reliable design values if correlation with other measurements has been established for the soil in question
3. Triaxial test 4. Plane strain tests in suitable laboratory apparatus 5. Shear box test	For φ' and c' The features listed in Section 4.5.5.2 must be considered when assessing the test results. The critical state angle of shearing resistance may be measured by reconstituting the material in a loose state
6. Correlations based on results of in situ penetration tests including cone penetrometers, Standard Penetration Test and others. 7. Pressuremeter tests	For φ' only. Suitable for an approximate assessment of φ', and to establish reliable design values if correlation with other measurements has been established for the soil in question

<u>Table 4.5 b. Some of the common field and laboratory tests from which effective strength parameters (c', φ') can be assessed</u>

4.5.5.3 Strength of Rocks. If it is necessary to determine the strength of undisturbed rock in order to design a structure, then the structure must be clarified as Geotechnical Category 3.

guide: The strength of an undisturbed rock mass often depends on the nature of the jointing. Consideration should be given to the following characteristics of the joints:

- spacing
- inclination
- continuity
- tightness
- roughness, including the effects of previous movements on the joints
- infill material
- water pressures
- pronounced variations in properties between different layers.

Intact sections of some rocks, particularly porous carbonate deposits, may be very sensitive to disturbance and will rapidly degrade to a soil of low strength if overstressed.

4.5.6 Stiffness

Evaluation of the stiffness of soil deposits must take account of the following factors:

- observations of settlements and other ground movements for similar situations in the same stratrum,
- the effect of stress level and water content, particularly in relation to preconsolidation pressures,
- the effect of rate of strain with time, particularly in relation to drainage of the soil,
- the significance of the shear stresses in the soil as a proportion of the shear strength,
- the effect of the order of magnitude of strain involved in the deformations.

guide: Reliable measurements of the stiffness of the ground are often very difficult to obtain from field or laboratory tests. In particular, owing to sample disturbance and other effects, measurements obtained from laboratory specimens often underestimate the stiffness of the soil in situ. Analysis of observations of the behaviour of previous constructions is therefore very valuable.

It is sometimes convenient to assume a linear or log-linear relationship between stress and strain for a limited range of the soil's behaviour. However, this must always be adopted with caution since the actual behaviour of soil is generally significantly non-linear.

Some of the common field and laboratory tests which may be used to assess stiffness of soil are listed in table 4.5 c. The methods are not listed in order of preference.

1. Correlation with results of compaction tests and relative density or water content 2. Correlation with laboratory measurements of shear strength	Suitable for an approximate assessment of stiffness, and to establish reliable design values if correlation with other measurements has been established for the soil in question
3. Oedometer tests 4. Triaxial tests 5. Plane strain tests in suitable laboratory	Guidance given in Section 4.5.6 must be considered when assessing the test results. For stiff clays and cohesionless soils problems of sample disturbance must be considered very carefully
6. Correlations with penetration tests	Suitable for an approximate assessment of stiffness, and to establish reliable design values if correlation with other measurements has been established for the soil in question
7. Pressuremeter tests 8. Plate bearing tests	Guidance given in Section 4.5.6 must be considered when assessing the test results
9. Seismic tests	These give a measure for stiffness at very small strain from which stiffness at larger strains can be assessed approximately on the basis of estabished correlations

Table 4.5 c. Some of the common field and laboratory tests from which stiffness parameters can be assessed

4.5.7 Permeability and Consolidation Parameters

The assessment of permeability and consolidation parameters must take into account the nature and disposition of strata within and beyond the project site, including grading, inhomogeneity and layering.

If it is necessary to measure permeability in order to design a structure, then the structure must be classified as Geotechnical Category 3.

guide: Permeability may be measured in situ using pumping tests or by testing laboratory specimens. Most soil deposits are not uniform in permeability and large variations can be expected; permeability is often also strongly anisotropic. In situ tests which measure the average properties of a large volume of soil are therefore to be preferred whenever possible. In assessing permeability parameters, the fact that in situ tests often indicate the horizontal permeability of the ground, whilst laboratory tests usually measure the vertical permeability (unless special procedures are adopted) should be taken into account.

The bulk permeability of rock deposits is often governed by jointing and can only be measured by large scale field tests.

Coefficients of consolidation may be calculated from stiffness and permeability values or may be derived directly from oedometer tests.

Tests which may be used to assess permeability are listed in table 4.5 d. The methods are not listed in order of preference.

guide:

1. Field pumping trials	These may take many forms, generally involving pumping at a measured rate at one or more locations and observing water levels in the surrounding ground. Careful attention must be paid to the design and construction of filters in boreholes. In many situations pumping trials are the most reliable means of assessing the bulk permeability of the ground, yelding an average value for a large volume of ground. Careful analysis is required, often including consideration of several different distributions of permeability
2. Borehole permeability tests	These involve pumping into or out of a single borehole. Pumping-out tests generally provide reliable results unless the borehole collapses during the test. Pumping-in tests often lead to an underestimate of permeability because the sides of the borehole become clogged with fine particles
3. Laboratory permeameter tests	Normally performed on cohesionless soils. Guidance given in Section 4.5.7 must be considered when assessing the test results
4. Laboratory oedometer tests	Normally performed on soils of low permeability. Guidance given in Section 4.5.7 must be considered when assessing the test results
5. Flow tests in the triaxial apparatus	Normally performed on soils of intermediate permeability, where seepage along the boundaries of the specimen could invalidate permeability tests. Guidance given in Section 4.5.7 must be considered when assessing the test results

Table 4.5 d. Some of the common field and laboratory tests from which permeability parameters can be assessed

4.5.8 Other Geotechnical Parameters

4.5.8.1 Cone Resistance. In assessing design values of the cone resistance, q_c, the following items must be considered:

1) the detailed design of the cone and friction sleeve may affect the results significantly. Allowance must therefore be made for the type of cone in use,

2) the results can only be interpreted with confidence when the soil succession is established. In many situations borings will therefore be needed in conjunction with the penetration tests,
3) in inhomogeneous soils for which widely fluctuating results are recorded, the penetration values which represent the part of the soil matrix relevant to the design in hand must be considered. Design values of cone resistance must be derived as given in Chapter 2,
4) established correlations with other test results, such as density measurements and other forms of penetration testing, should be considered when available.

guide: The penetration resistance may be used directly in the design of piles and other elements as described in chapter 7 and elsewhere. Alternatively the resistance to penetration measured in a static cone penetration test may be used to assess the strength and stiffness parameters of the ground as discussed in Sections 4.5.5.1, 4.5.5.2 and 4.5.6

For soils in which reliable, conservative correlations are available, values of q_c may be assessed from the results of other forms of penetrations tests.

4.5.8.2 Blow Count (N) from Standard Penetration Test. In assessing blowcounts (N) the following points must be considered:

- detailed description of the performance of the test (lifting method, etc),
- ground water conditions,
- the influence of the overburden,
- the stress history of the site,
- the nature of the ground particularly when cobbles or coarse gravel are encountered.

guide: Cohensionless soils: With the aid of the Standard Penetration Test a measure of the relative density is obtained. Indirectly, bearing capacity and settlements of shallow and deep foundations can be assessed.

guide: The accuracy of ϕ values based on Standard Penetration Tests is affected by several factors such the gradation of the material and the grain shape.

Cohesive soils: Estimation of the undrained shear strength is strongly affected by the plasticity of the soil. It may only be used when previous experience exists from comparisons with laboratory tests.

Stiffness may be estimated from Standard Penetration tests with caution, as it is strongly affected by local conditions.

4.5.8.3 Pressuremeter Limit Pressure. In assessing the design values of the Limit Pressure (P_L) the following items must be considered:

- the type of equipment and, most importantly, the procedure used to install the pressuremeter in the ground may have a significant effect on the pressuremeter curve. Curves which exhibit more than a moderate degree of disturbance may not be used,
- where the Limit Pressure is not reached during the test a moderate and conservative extrapolation of the curve may be used to estimate it,
- for tests in which only the initial part of the pressuremeter curve is determined general correlations or, preferably, local correlations from the same site, may be used conservatively to estimate the Limit Pressure (P_l) from the pressuremeter modulus (E_M),
- in interpreting the test the civil conditions determined from geological conditions, the rest of the site investigation and the results of the boring in which the test is performed.

guide: The Limit Pressure may be used directly in the design of spread foundations, piles and other elements, as described in Chapters 6 and 7 and elsewhere. Alternatively, the Limit Pressure may be used to assess the strength parameters of the ground as discussed in Sections 4.5.5.1 and 4.5.5.2.

4.6 Reporting Geotechnical Data

The results of a geotechnical investigation should be compiled in a Geotechnical Report which is complementary to, or part of, the Design Report described in Section 2.6. The Geotechnical Report should normally consist of the following three parts:

- presentation of available geotechnical information and relevant data,
- geotechnical evaluation of information,
- conclusions and recommendations.

These parts may be combined into one report or divided between several reports. They are discussed in Sections 4.6.1 to 4.6.3.

The Geotechnical Report must state the assumed geotechnical conditions and parameters. For structures complying with Geotechnical Category 1 this statement may be very brief. For structures belonging to categories 2 or 3 a more comprehensive statement will be necessary.

4.6.1 Presentation of Geotechnical Information

The presentation of geotechnical information will include a factual account of field and laboratory work and detailed description of methods used to carry out the field investigations and the laboratory testing.

guide: In addition to the above, the factual report may include the following information:

- purpose and scope of the geotechnical investigation,
- authorization to carry out the geotechnical investigation,
- brief description of the project for which the geotechnical report is being compiled giving information about the location of the project, its size and geometry, anticipated loads, structural elements, materials of construction, etc,
- a statement of the anticipated geotechnical category of the structure,
- dates between which field and laboratory work were performed
- types of field equipment used,

guide:

- names of specialized field personnel responsible for the continuous follow-up of the field work, the visual description of the samples and their handling for storage and transportation to the testing laboratory
- field reconnaissance of the general area of the project noting particularly:

  (i) evidence of ground water,
  (ii) behaviour of neighbouring structures,
  (iii) faulting,
  (iv) exposures in quarries and borrow areas,
  (v) areas of instability,
  (vi) difficulties during excavation.

  - history of the site,
  - geology of the site,
  - information from aerial photographs,
  - local experience in the area,
  - information about he seismicity of the area.

- tabulation of quantities of executed field and laboratory work Presentation of field observations which were made by the supervising field personnel during the execution of the subsurface explorations,
- data on fluctuations of ground water table with time in the boreholes during the performance of the field work and in piezometers after the completion of the field work,
- compilation of boring logs with descriptions of subsurface formations based on field descriptions and on the results of the laboratory tests,
- grouping and presentation of field and laboratory test results in appendices.

4.6.2 Evaluation of Geotechnical Information

The evaluation of the geotechnical information will include:

- review of the field and laboratory work by the Geotechnical Engineer. In cases where there are limited or partial data, the Geotechnical Engineer should state it. If, in the Geotechnical Engineer's opinion, the data are defective, irrelevant, insufficient, or inaccurate, he can and should point this out and qualify his comments accordingly. Any particularly adverse test results should be considered carefully in order to determine whether they are misleading or represent a real phenomenon that must be accounted for in the design,
- submission of proposal(s) for further field and laboratory work, if deemed necessary, with comments justifying the need of this extra work, This proposal should be accompanied by a detailed programme for the types of the extra investigations to be carried out with specific reference to the points which have to be answered.

guide: In addition to the above, the evaluation of the geotechnical data may include the following:

- tabulation and graphical presentation of the results of the field and laboratory work in relation to the requirements of the project and, if deemed necessary, histograms illustrating the range of variation of the most relevant data and their distribution,
- determination of the depth of the ground water table and its seasonal fluctuations,
- subsurface profile(s) showing the differentiation of the various strata. Detailed description of all strata including their physical properties and their compressibility and strength characteristics. Comments on irregularities such as pockets and cavities,
- grouping and presentation of the range of variation of the geotechnical data for each stratum. This presentation must be in a comprehensible form which enables the most appropriate soil parameters to be selected for the design.

4.6.3 Conclusions and Recommendations

The conclusions and recommendations of a geotechnical report will include the following:

- review of the Geotechnical Category of the structure,
- differentiation between strata and selection of suitable design parameters for the calculations required for the design,
- recommendations for the easiest and cheapest foundation solutions based on experience or on simplified computations,
- recommended solutions for any problems which are anticipated during construction, including:

  (i) excavations,
  (ii) pumping operations,
  (iii) retaining structures,
  (iv) ground anchors,
  (v) placement of fill.

Chapter 5 ARTIFICIALLY PLACED SOIL AND IMPROVED GROUND

Contents

5 ARTIFICIALLY PLACED SOIL AND IMPROVED GROUND

5.1 Scope

The provisions in this chapter apply in situations where:

- soil is placed for engineering construction,
- existing ground is treated to improve its engineering properties.

Existing ground which is treated to improve its properties may be either natural ground or artificially placed fill.

guide: Examples of situations where soil is placed for engineering construction include:

- embankments for roads, dykes and small dams,
- fills beneath foundations and ground slabs,
- backfill to excavations and retaining structures,
- general landfill including hydraulic fill, landscape mounds and spoil heeps.

Examples of situations where existing ground may need to be improved include:

- foundation or embankments on soft natural ground or loose fill
- excavations below the groundwater table.

5.2 Performance Criteria

The performance criteria to be satisfied in the case of both artificially placed soil and improved ground are that the ground must:

- be capable of carrying the design loads without failure or excessive deformations,
- remain stable under rain, frost, seeping water, vibrations, etc.

5.3 Artificially Placed Soil

5.3.1 Selection

The source material for use as a fill must be identified to ensure that it is suitable for its intended purpose. The effect of transportation and placing must also be considered.

Identification must include a determination of the particle size distribution, natural water content, Atterberg Limits and specific gravity. The number and frequency of identification tests must be selected according to the heterogeneity of the material and the nature of the project.

Materials selected for use as fill must not be:

- organic,
- susceptible to frost,
- chemically agressive,
- soluble, or
- collapssible.

guide: If suitable natural material is not available locally it may be
: necessary to mixe the selected material with cement, lime,
: etc. in order to satisfy the performance criteria.

5.3.2 Compaction

When soil is placed for engineering construction it must be compacted so that its properties after compaction satisfy the performance criteria.

guide: Various methods may be used to compact the ground and these include:
:
: - ramming and rolling which is suitable for shallow compaction,
: - dropping heavy weights,
: - using a vibrator.

Supervision of the compaction procedure for fills will depend on the purpose of the fill and will include, as appropriate, checking the following:

- the placement method,
- the characteristics of the compaction equipment and its velocity,
- the number of panes,
- the initial and final thicknesses of the lift,
- possible variations in the water content of the material,
- sluicing,
- the air temperature and humidity,
- the features of the ground surface after compaction.

guide: This kind of external checking is usually purposive. In the case of large fills involving large volumes of soil statistical procedures may be adopted.

For large fills in Geotechnical Category 1 checks should be carried out at least once during each working day. For large fills in Geotechnical Category 2 the thickness of the lift, the count of the roller coverages, and the features of the ground surface after compaction should be checked for every three lifts.

Design documents must include instructions on how to check the compaction. These instructions must specify:

- the sampling procedure,
- the sampling frequency,
- the geotechnical properties to be checked,
- the range of acceptable results and rejection criteria.

Checking the geotechnical properties of the compacted fill must aim at identifying possible zones where the design specifications are not met. When the characteristics of the compacted fill do not fall within the acceptability range assumed in the design calculation, the layers concerned must be replaced or recompacted.

For large fills checks must be performed:

- at least daily,
- when changes in the source material is suspected,
- when appreciable changes in the weather condition occur.

For each of the above circumstances at least 3 tests must be carried out.

guide: Checking of compaction normally includes direct measurements of the in situ density by the sand replacement method or by comparable reliable methods. Direct density measurements may be replaced by direct measurements of related material characteristics such as penetration resistance, shear strength or deformation parameters, provided that calibration of the latter is reliably performed.

Indirect methods of checking compaction which may permit an almost continuous control of density during the compaction process may prove to be advantageous but must be calibrated against direct tests.

For Geotechnical Category 1 fills a visual assessment of the suitability of the compaction is often sufficient.

For Geotechnical Category 2 fills purposive or random sampling procedures may be selected.

5.4 Improved Ground

5.4.1 Choice of Improvement Process

Before any ground improvement process is chosen or used, a careful design investigation, as described in Section 4.2.3 must be carried out to obtain an adequate knowledge of the initial ground conditions.

guide: Depending on the particular situation a design investigation would normally include an investigation of the following:

- the ground profile,
- the groundwater conditions,
- the soil particle size distribution,
- the soil shear strength properties,
- the soil compressibility.

The most suitable ground improvements process for a particular situation must be chosen taking into account the following factors where appropriate:

- the thickness and properties of the in situ soil strata,
- the thickness and properties of the fill material,
- the magnitude of the water pressure in the various strata,
- the nature, size and position of the structure to be supported by the ground,
- the prevention of damage to adjacent structures or services,
- whether the proposed ground improvement is temporary or permanent,
- the relationship between the ground improvement process and the construction sequence.

guide: The processes for improving the ground include:

- dewatering,
- surcharging,
- geotechnical processes.

After implementation of a ground improvement process, a control investigation must be carried out to check the effectiveness of the improvement process by determining the changes in the appropriate ground properties or condition resulting from the improvement process.

5.4.2 Dewatering

When construction is to take place on poor ground where the groundwater level is high, the first consideration for improving the strength of the ground must be to lower the groundwater by draining the ground.

When lowering the groundwater level to improve the properties of the ground, the following conditions where applicable should be fulfilled:

- the dewatering system should be so designed, arranged and installed as to maintain the water levels and pore pressures anticipated in design without significant fluctuations,

- the system adopted should not lead to excessive settlements or damage to nearby structures,
- in the case of excavations, the effect of groundwater lowering should be that the sides of the excavation remain stable at all times and that excessive heaving of the base does not occur,
- the system adopted should avoid excessive loss of ground by seepage from the side or base of the excavation,
- except in the case of fairly uniformly graded material which can establish itself as a filter material, adequate filters should be provided around the sumps or wells to ensure that there is no significant transportation of soil with the pumped water,
- there should be an adequate margin of pumping capacity and stand-by plant should be available in the case of breakdown to facilitate maintenance,
- water removed from an excavation should be discharged well clear of the excavated area,
- when allowing the groundwater to return to its original level, care should be taken to do it slowly enough to prevent problems such as the collapse of soils having a sensitive structure, e.g. loose sand.

The effectiveness of a dewatering scheme must be checked by monitoring the groundwater level, the pore pressures and the ground movements. Collected data must be reviewed and interpreted frequently to determine the effects of dewatering on the ground conditions and on the behaviour of partially completed and nearby structures.

If a pumping operation is to extend over a long period of time, the groundwater must be checked for the presence of dissolved salts and gasses which could either result in corrosion of the well screen or cause plugging of the screens by the precipitation of salts.

5.4.3 Surcharging

When using surcharge to improve the properties of in situ ground or fill by increasing the density, the follwoing factors must be taken into account:

- the nature and variability of the ground,
- the position of the groundwater level.

When using surcharging on saturated soft soils adequate drainage must be provided to permit the removal of excess water and allow consolidation.

5.4.4 Geotechnical Processes

A number of geotechnical processes are available for improving the properties of the ground and these include:

- ground injection,
- stone columns,
- dynamic compaction,
- soil reinforcement.

When geotextiles are used to reinforce artificially placed soil the geotextiles must not be exposed to:

- the air any longer than is necessary for the placing operation,
- agressive soils.

Geotextiles used to reinforce artificially placed soil must be correctly orientated.

Steel or geotextile members used to reinforce soil must be sufficiently durable so that they retain their strength for the design life of the structure.

Chapter 6 VERIFICATION PROCEDURES FOR SPREAD FOUNDATIONS

Contents

6 SPREAD FOUNDATIONS

6.1 Scope

The provisions in this chapter apply to spread foundations for buildings and retaining walls. Spread foundations include pads, strips and rafts for which the strength of the soil above foundation level does not contribute significantly to the bearing capacity. The provisions in this chapter do not apply to foundations consisting of multi-storey basements founded on rafts or pads or to piled foundations.

6.2 Limit States

In order to satisfy the performance criteria related to stability, limited deformations, durability and limitation of damage to nearby structures or services the following limit states must be prevented:

Type 1A Ultimate Limit States

- the formation of a mechanism in the ground mass containing the foundation corresponding to a loss of overall stability
- the formation of a mechanism in the ground corresponding to a bearing capacity failure
- the formation of a mechanism in the interface between the foundation and the ground corresponding to a failure by sliding
- overturning of a foundation
- the formation of a mechanism in the structural materials of the foundation itself

Type 1B Ultimate Limit States

- the formation of an ultimate limit state involving loss of static equilibrium or rupture of a critical section of the supported structure due to movement of the foundation.

Type 2 Serviceability Limit States

- the occurrence of settlements or other movements of the foundation which affect the appearance or efficient use of the structure or cause damage to finishes or non-structural elements
- the occurrence of excessive vibrations in the structure due, for example, to resonance in the soil/structure system.
- the seepage of water through a raft foundation.

guide: For many lightly loaded structures, the critical limit state governing the design of the foundations may result from, frost, vegetation or soil wetting or drying.

The design of building foundations is often governed by a serviceability limit state involving foundation movements. It may be necessary to limit foundation settlements in order to prevent unacceptable damage such as cracking of plaster or jamming of doors. To achieve risk, the bearing pressure may be reduced below the value giving an adequate margin of safety against a bearing capacity failure.

6.3 Actions and Design Situations

6.3.1 Actions

In selecting the actions for any calculation, the forces and displacements listed in Section 3.1.2 must be considered.

Design values for the actions must be derived in accordance with the principles stated in Section 3.2.

6.3.2 Design Situations, Load Cases and Loading Combinations

When designing a spread foundation it is normally necessary to check that no limit states will occur for a number of different design situations. Design situations must be chosen in accordance with the principles given in Section 3.1.1. Examples of design situations which have commonly caused failures include:

- a prolonged drought,
- the growth of a tree,
- a burst water main.

Load cases and combinations must be chosen in accordance with Section 3.3.4.

guide: For fine grained soils the rate at which actions are applied must be considered. Actions applied faster than the soil's capacity to drain will cause pore water pressures to develop. The design must take account of these. Conditions following a fast transfer of load (for example, at the end of construction) are termed undrained, and must be considered separately from long term, or drained, conditions. Separate soil parameters are normally used for drained and undrained conditions.

When designing a foundation resting on or close to rock, design situations involving factors such as:

- dipping bedding planes
- interbedded hard and soft strata
- faults, joints, and fissures
- weathering
- solution cavities such as swallow holes or fissures filled with soft material
- mine workings, caves or other underground cavities

must be considered and the influence of these factors on the stability and performance of the structure must be taken into account.

The design groundwater table must normally be assumed to be at the ground surface unless a system of drains is installed around the foundation to ensure a lower groundwater level.

6.4 Design and Construction Considerations

6.4.1 Choice of Spread Foundation

The following must be considered when choosing the type of spread foundation:

- the magnitude and disposition of the loads,
- the tolerance of the structure to settlements,
- nearby excavations,
- erosion or scour,
- earthquakes,
- mining subsidence,
- the effect of the new structure on existning structures or services.

guide: Excavations for a new foundation adjacent to an existing foundation can reduce its stability even when the depth of excavation is less than the depth of the existing foundation.

The increase in load due to a new structure may cause adjacent structures or services to settle.

6.4.2 Foundation Depth

When choosing the depth of a spread foundation the following must be considered:

- reaching an adequate bearing stratum,
- for clay soils, the depth above which shrinkage and swelling due to seasonal weather changes, or to trees and shrubs, may cause appreciable movements,
- the depth above which frost damage may occur,
- for inclined loads, the possibility of failure by sliding,
- the level of the water table in the ground and the problem which may occur if excavation for the foundation is required below this leve
- possible ground movements
- high or low temperatures transmitted from the building.

guide: On sloping sites strip foundations must normally be on a horizontal bearing surface, stepped where necessary to maintain adequate depth.

The foundation depth required to safeguard against movements due to ground freezing depends on the susceptibility of the soil to frost heaving. This depth can be reduced by heating, insulation or drainage.

6.4.3 Foundation Width

The foundation width must be designed taking account of the following factors:

1) the bearing pressure must be low enough to prevent the occurrence of a limit state,
2) practical considerations related to economic excavation, setting out tolerances, working space requirements and the dimensions of the wall or column supported by the foundation.

6.4.4 Design Methods

The design method adopted must ensure that both ultimate and serviceability limit states are sufficiently improbable.

guide: The following methods may be used:

- a direct method, in which separate analyses are carried out for each limit state using calculation models and appropriate values for the actions and the soil parameters,
- an indirect method, in which a single ultimate limit state analysis is carried out using factors to ensure that other limit states are sufficiently improbable,
- empirically obtained presumed bearing pressures.

6.4.4.1 Direct Method. In this method each limit state is considered explicitly, following approach 'a' of Section 2.3.1. When checking against a type 1A ultimate limit state the calculation must model the failure mechanism which is envisaged as closely as possible. When checking against a type 1B ultimate limit state or a serviceability limit state, a deformation analysis must be used.

A thorough investigation of a type 1B ultimate limit state requires a complex non-linear analysis involving soil-structure interaction, and is rarely undertaken.

Experience of similar structures and soil conditions will often indicate that settlement will be acceptable and that settlement calculations are not necessary. In other cases settlements may be estimated from a deformation analysis or by correlation with relevant previous experience.

6.4.4.2 Indirect methods. Although in many cases the serviceability limit state governs the design, the foundation of structures are often designed using only a type 1A ultimate limit state analysis. This is because settlement calculations are relatively complex, cumbersome and often unreliable. In this method the foundations are designed against a type 1B ultimate limit state or a serviceability limit state on the basis of local experience, following approach 'b' given in Section 2.3.1. In the calculation models used to check the type 1A ultimate limit states, design values of the soil properties are selected, which provide a suitable margin of safety and which prevent unacceptable ground movements.

This method does not take into account:

- the deformation in behaviour of the soil,
- the influence of the size of the building,
- the type of building.

Also, no estimate of the settlement is obtained. Because of these chawbacks, there are situations for which the method is unsuitable and others, such as the design of very wide spread foundations, for which it may be very conservative.

6.4.4.3 Presumed bearing pressures. The presumed bearing pressure is a conservative value for the bearing capacity of a soil stratum estimated empirically using local experience and the results of field or laboratory measurements or observations and chosen so that the performance criteria are fulfilled for serviceability limit state loads.

guide: When using this method the alternative limit states need not be considered explicitly.

Factors affecting the presumed bearing pressure include:

- the soil type,
- the width of the foundation,
- the serviceability of the structure to settlement,
- local experience.

Foundation on cohesionless soils may be designed using a presumed bearing pressure estimated from the results of in-situ tests, such as the standard penetration tests, cone penetrometer or pressuremeter, and empirical relationships based on local experience.

Presented presumed bearing pressures may only be used to design foundations not exceeding 2 m in width for structures belonging to Geotechnical Category 1.

Presumed bearing pressures which are adopted to design foundations without unnecessary calculations are prescriptive measures, as described in Section 2.2.

6.5 Ultimate Limit State Design

6.5.1 Loss of Overall Stability

The procedures given in chapter 9 must be used to demonstrate that a slope stability failure of the soil mass containing the foundation is sufficiently improbable.

Failure due to loss of overall stability must be checked in particular for foundations in the following situations:

- on an inclined site or close to a natural slope
- close to an embankment or a cutting
- close to a river or a canal
- close to a lake, a reservoir or the sea shore
- close to mine workings
- close to a retaining wall.

6.5.2 Bearing Capacity Failure

6.5.2.1 Failure Condition. To demonstrate that a foundation will support the design load with adequate safety against bearing capacity failure the following inequality must be satisfied:

$$V_d \leq Q_d \qquad 6.1$$

where

V_d is the ultimate limit state design vertical load on the foundation including the weight of the foundation and of any backfill material

Q_d is the ultimate limit state design vertical bearing resistance of the foundation, taking into account the effect of any horizontal or eccentric load.

Q_d must be calculated from ultimate limit state design values of the relevant parameters chosen in accordance with Section 2.3.2.
In calculating V_d and Q_d the effect of the groundwater table must be considered.

guide: The design bearing resistance of a spread foundation must preferably be estimated using an analytical approach based on soil shear strength parameters and a bearing capacity equation. When reliable soil shear parameters are not available the design bearing resistance may be estimated using an empirical approach based on the results of in-situ tests. It is often valuable to use both approaches and to compare the results.

6.5.2.2 Evaluation Based on Soil Shear Strength Parameters. The ultimate limit state design vertical bearing resistance of a spread foundation, Q_d may be evaluated analytically. The strength of the soil depends on:

- the design situation,
- in situ stresses,
- soil density,
- soil deformation,
- mode of failure.

guide: The governing ultimate limit state design situation for most foundations on saturated, normally or lightly overconsolidated fine-grained soils is the undrained condition. The design bearing resistance is then calculated using a total stress analysis. The appropriate soil shear strength parameter is the undrained shear strength, c_u.

For foundations on heavily overconsolidated clays both the initial and the long-term design situations may need to be checked. The initial undrained bearing resistance may be determined as described. The long-term drained bearing resistance may be calculated using an effective stress analysis. The appropriate soil shear strength parameters are the effective cohesion, c' and the effective angle of shearing resistance, ø. It is difficult to measure 'c reliably and the values obtained from tests should be used with caution.

A conservative estimate of the bearing resistance may be calculated assuming c' is equal to zero and adopting the critical state value of ø' at constant volume obtained from laboratory tests.

In the case of silty soils a decrease in water content during construction leading to an increase in shear strength may often be taken account when selecting the appropriate shear strength parameters for design.

For foundations on highly permeable non-cohesive soils the critical design situation is usually the drained condition. The bearing resistance is calculated using an effective stress analysis and the appropriate shear strength parameter is ø', with c' = 0. When it is not possible to obtain undisturbed samples of sands or gravels, the critical state value for ø' obtained at constant volume from laboratory tests may be used.

The design bearing resistance of a spread foundation may be calculated using the following approximate equations based on plasticity theory which take into account the shape and depth of the foundation and the inclination of the loading. For undrained conditions the design bearing resistance is:

$$Q_d = Ac_uN_cs_ci_c + Aq \qquad 6.2$$

and for drained conditions

$$Q_d = Ac'N_cs_ci_c + Aq'N_qs_qi_q + 1/2A\gamma'BN_\gamma\ s_\gamma i_\gamma \qquad 6.3$$

guide:

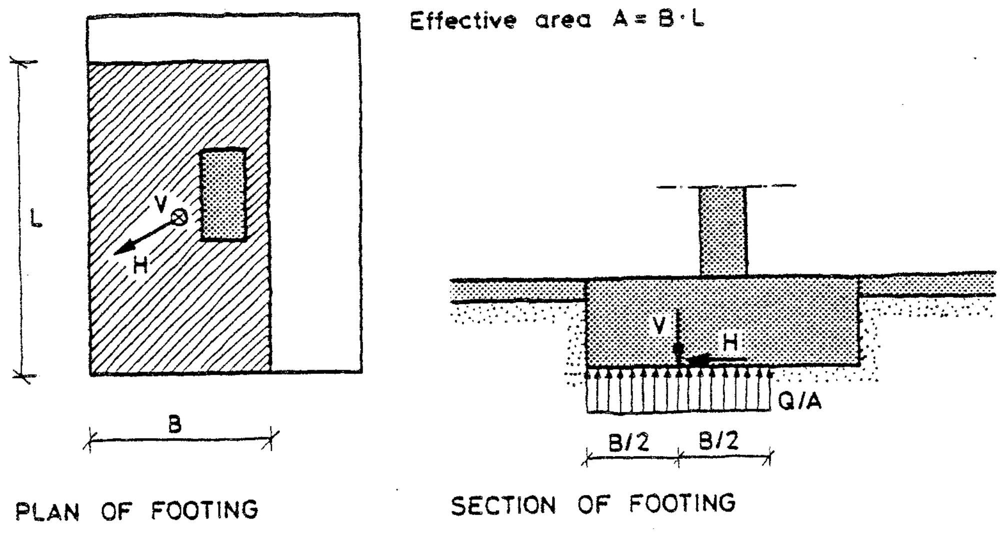


Figure 6.5 a. Eccentrically loaded footing

where:

- A is the design effective foundation area, defined as the area of the foundation base or, in the case of an eccentric load, the reduced area of the foundation whose centroid is the point through which the vertical component of the load acts as illustrated in Figur 6.5 a
- c_u, c' is the design undrained shear strength and drained cohesion of the soil
- q, q' is the design minimum total and effective vertical stresses at the foundation level due either to the embedment depth or a surcharge ($q' = q - u$)
- γ' is the design effective unit weight of the soil below the foundation level, reduced in the case of an upward hydraulic gradient, i to $\gamma' = \gamma - \gamma_w (1 + i)$
- B is the foundation width
- N, s, i is the design values of the dimensionless factors for the bearing capacity, the shape of the foundation and the inclination of the load, respectively. The subscripts c, q and γ indicate the influences due to cohesion, the surcharge and the weight of the soil. These coefficients are only valid when the shear parameters are independent of direction. Note that the part of Equation 6.3 concerned with the soil weight includes a factor of 1/2 and this should be taken into account when choosing values for N_γ.
- u is the pore water pressure at the foundation level

guide: Additional factors which allow for embedment depth, inclination of the base of the foundation and the ground surface may also be included, but are not considered here.

Because the bearing capacity factors N_c, N_q and N_γ increase very rapidly as the angle of friction increases careful consideration must be given to the value adopted for $\emptyset'$.

When the soil unit weight and shear strength parameters vary only slightly with depth below the foundation, the design values used to calculate the design bearing resistance may be assigned values corresponding to a depth below the foundation level equal to half the effective foundation width.

When the soil or rock mass beneath a foundation presents a definite structural pattern of layering or discontinuities in general, the assumed rupture mechanism and the selected shear stength and deformation parameters must take into account the structural characteristics of the ground.

When calculating the design bearing resistance of a foundation on highly layered deposits, the characteristic values of the soil parameters for each layer must be determined. Use of the bearing capacity equation and average soil parameter values is only permissible if the characteristic angle of friction of the individual strata does not vary by more than 3^o from the mean characteristic value. If the characteristic value of $\emptyset'$ varies by more than 3^o from the mean value then an alternative method such as a slip circle analysis may be required. Where a weak stratum underlies stronger strata the foundation load may be assumed to spread with depth at a rate of 1 in 2 with the vertical and the design bearing resistance calculations may be carried out using the shear strength parameters for the weaker stratum.

6.5.2.3 Evaluation Based on Semi-Empirical Methods. The design bearing resistance of a spread foundation may be estimated semi-empirically from the results of in-situ tests or by observing foundations on similar soils. The use of a particular in-situ test and the interpretation of the results must take account of local experience.

guide: To estimate the design bearing resistance of a foundation semi-empirically, the following types of in-situ test may be used:

- plate loading test,
- pressuremeter test.

Further details about these tests are given in Section 4.2.

i) Plate Loading Test

Plate loading tests are particularly useful in the case of weak jointed rocks or soils containing large gravel or boulders in which in-situ penetration tests cannot be carried out. If the plate size is roughly similar to the width of the proposed foundation the measured bearing pressure may be used directly in the design of the foundation. However, the measured ultimate pressure is not the design pressure. Instead a much more conservative assessment must be made. When extrapolating the results of small plate loading tests to design wide foundations, consideration must be given to the influence of foundation width and possible variations in the soil strength with depth on the bearing capacity of the proposed foundation.

ii) Pressuremet Test, Vertical Central Load

The design bearing resistance of a foundation subjected to a vertical central load is related to the limit pressure of the soil determined from a pressuremeter test by the linear function:

$$Q_d = Aq + Ak\overset{*}{p}_{le} \qquad 6.4$$

where:

- A is the design effective foundation area, taking into account eccentricity of the load as in Section 6.5.2.2
- q is the total design vertical stress at the foundation level after construction due either to the embedment or a surcharge
- k is the design bearing factor varying from 0.8 to 3.5 according to the embedment, the shape of the foundation and the soil category
- $\overset{*}{p}_{le}$ is the design net equivalent limit pressure

6.5.3 Failure by Sliding

When the loading is inclined to the vertical, foundations should be designed against failure by sliding. Sliding failure occurs when the horizontal component of the loading exceeds the horizontal resisting force. For safety against failure by sliding the following inequalities must be satisfied:

$$H_{sd} + E_{prd} \geq H_d \qquad 6.5$$

$$H_{rd} \geq H_d \qquad 6.6$$

where

H_d is the horizontal component of the design load

H_{sd} is the design shear resistance between the foundation and the ground

E_{prd} is part of the design passive resistance of the ground in contact with the vertical face of the foundation

H_{rd} is the design horizontal shear resistance of the ground.

guide: The value of E_{prd}, depends on:

- whether the foundation is cast against undisturbed soil or not,
- the density of the backfill, if any, between the foundation and the edge of the excavation,
- whether the foundation can, without danger, move sufficiently to mobilize the required passive resistance.

For foundations on clay soils bearing within the zone of seasonal movements, shrinkage may cause a gap between the soil and the foundation, and this must be considered. It is also important to ensure that the soil in front of the foundation will not be removed by erosion or human activity.

A value of 50% of the maximum passive resistance is acceptable in most cases.

Prescriptive Measures

In the case of inclined loads a shear key may be designed to prevent failure of the foundation by sliding.

guide: Calculation Models

For drained conditions the design horizontal shear resistance may be calculated using the following equation:

$$H_s = V_d' \tan \delta_s \qquad 6.6$$

where:

V_d' is the design vertical effective load and

δ_s is the design friction angle on the foundation base.

The friction angle, δ_s, may be assumed equal to ϕ' for cast-in-situ concrete foundations and equal to 2/3 ϕ' for smooth precast foundations. Any effective cohesion, c', is generally neglected.

For undrained conditions the design horizontal shearing resistance will be limited by:

$$H_s = Ac_u \qquad 6.7$$

and $$H_s = 0{,}4V_d' \qquad 6.8$$

where A is the base area through which V_d' acts, reduced if necessary to an effective area in the case of an eccentric load as described in Section 6.5.2.2. In some cases the area A used in Equation 6.7 may be the smallest contact area required to carry the design vertical load, V_d'. This may be significantly less than the total area of the footing.

6.5.4 Foundations with Highly Eccentric Loads

Foundations with highly eccentric loads, such as the foundations for retaining structures covered in Chapter 8, must be designed against the following situations:

- very high edge stresses causing a bearing capacity failure
- overturning.

To design against the above conditions the eccentricity of the line action of the load on the foundation must be restricted. Provided the maximum design bearing pressure at the edge of the foundation does not exceed the design bearing resistance the situations listed above are unlikely to occur. However, the foundation must be checked against bearing capacity failure due to the vertical component of the load acting on the reduced effective foundation area.

guide: The bearing capacity may be checked using Equation 6.2 or 6.3, depending on the design situation.

When designing foundations subjected to eccentric loads the possibility and consequences of water entering beneath the foundation due to the opening of a gap must be considered.

The passive resistance of the soil in contact with the sides of the foundation block must be considered as outlined in Section 6.5.3.

6.5.5 Structural Failure due to Foundation Movement

Differential settlements and horizontal displacements of the foundations for a structure under the ultimate limit state design loads and soil deformation parameters must be estimated to ensure that these do not lead to a Type 1B ultimate limit state occurring in the structure. The differential settlements for foundations which will cause structural failure depend on the type and the material of the superstructure and must take account of local experience.

guide: As a guide, structural damage of general buildings is to be feared if the angular distortion exceeds about 1/150. When a Type 1B failure occurs the system will generally be beyond the linear range and therefore the methods given in Section 6.6.3 etc. for calculating settlements may not be used.

To design against a Type 1B ultimate limit state the method outlined in Section 6.2.2 may be adopted. In this case design is based on local experience and the use of a Type 1A limit state calculation with appropriate partial coefficients chosen to limit the soil stresses to permissible values for which displacements will not be excessive.

6.6 Serviceability Limit State Design

6.6.1 Displacement Analyses

Foundation displacements can occur either as a total displacement of the entire foundation or as differential displacements of different parts of the foundation.

Three main types of foundation displacement should be considered:

- settlement
- horizontal displacement
- tilting

These usually take place simultaneously.

One of the methods described in Section 6.4.4 must be adopted to design for displacements. All the design situations which will arise during the construction and life of the structure must be considered.

If the magnitude of the settlement is calculated, both total and differential settlements must be quantified and taken into account.

The settlement behaviour of neighbouring structures which have similar conditions to the proposed structure must be studied whereever possible.

guide: In certain situations minimum loading conditions may be significant. For example the unloading of one foundation may cause differential heave with respect to its neighbours.

The serviceability limit state design loads must be used when calculating foundation displacements.

guide: Suitable serviceability limit state soil deformation parameters for use in soil deformation models to calculate foundation settlements may be assessed by evaluating the behaviour of neighbouring similar structures or on the basis of laboratory or field tests. Whenever possible it is preferable to use the first of these methods. For this reason it is very important to measure the deformations of structures and to evaluate them.

Foundation displacements may be calculated using the linear methods given in Section 6.6.3 only if the mobilized strength is low enough.

6.6.2 Settlement

6.6.2.1 General Considerations. When designing a foundation the settlements due to volumetric and shear deformations of the soil should be considered. To make reliable estimates of settlement, the values of the stiffness parameters to be used in the calculation models must be chosen carefully, as described in Section 4.5.6.

For saturated soils three components of settlement must be considered:

- undrained settlement due to shear deformations of the soil at constant volume, s_0,
- consolidation settlement, s_1,
- secondary (creep) settlement, s_2.

For unsaturated soils, additional components may be significant.

guide: For different soil types these three components may occur in very different proportions.

In some soils additional settlement may occur if the groundwater level varies or if the foundation or the soil is subject to vibrations. This Sections does not deal with these types of settlement. Particular care is needed in situations where settlemet may occur due to self compaction, for example on fill.

In some soils, such as organic soils or very sensitive clays, settlement may be prolonged almost indefinitely due to secondary consolidation or creep and will need special consideration.

The settlements of foundations on multi-layered soil is the sum of the vertical compression of each layer.

If the soil conditions are very complicated, they may be simplified by considering a few idealized soil layers with intermediate parameters.

The stress changes in the ground causing settlement are to be calculated from the total load on the footing due to the permanent loads, allowing for excavation of the foundation and deducting for any buoyancy. Live loads are to be considered where significant compared with the dead loads.

The total thickness, H, of the compressible soil layers to be taken into consideration depends on the size and shape of the foundation and on the variation in soil stiffness with depth.

Normally H should equal the depth at which the vertical stress due to the foundation load amounts to 20% of the overburden stress.

guide: For many cases the depth H may be roughly estimated as 1 to 2 times the foundation width, but may be reduced for lightly loaded wide foundation rafts. This approach is not valid for very soft clays.

6.6.2.2 Evaluation of Total Settlement. The total settlement of a foundation will include the three components listed in Section 6.6.3.1.

guide: The following methods may be used to evaluate total settlement:

i) stress-strain method,
ii) adjusted elasticity method,
iii) semi-empirical methods.

Experience of the chosen method applied to other foundations or the soils in the construction area is useful.

i) Stress-Strain Method

The total settlement of a foundation on cohesive or non-cohesive soils may be evaluated using the stress-strain method as follows:

- computing the stress distribution in the ground due to the loading from the foundation. This may be derived on the basis of elasticity theory, generally assuming homogeneous isotropic soil,

- computing the strain in the ground from the stresses using stiffmess moduli values or other stress-strain relationships determined from laboratory tests (preferably calibrated against field tests) or field tests, guide: - Integrating the vertical strains to find the settlements. To use the stress-strain method a number of points within the ground beneath the foundation may be selected and the stress and strains computed at these points.

Alternatively finite element or similar analyses may be used.

The stress-strain method may be used provided the soil's stiffness parameters are determined with confidence either from laboratory tests on good quality samples or from in-situ tests. This method is particularly useful in the case of layered soil deposits where the soil stiffness varies significantly with depth.

ii) <u>Adjusted Elasticity Method</u>

The total settlement of a foundation on cohesive or non-cohesive soil may be evaluated using elasticity theory and an equation of the form:

$$s = \frac{qBf}{E_m} \qquad 6.9$$

where:

- q is the average serviceability limit state bearing pressure on the base of the foundation, which for normally consolidated cohesive soils should be reduced by the weight of the excavated soil above the base. Buoyancy effects should also be taken into account
- E_m is a general stiffness parameter for the deformable soil stratum having units of stress
- f is a coefficient whose value depends on the shape and dimensions of the foundation area, the thickness of the compressible stratum and on Poisson's ratio, ν. Most published values of f only apply in the case of homogeneous instropic soil when E_m is constant with depth
- B is the width of the foundation.

guide: The general stiffness parameter, E_m, preferably should be obtained by evaluating (back analysing) the measured settlements of neighbouring similar structures using the inverse of Equation 6.9. Then the settlement values calculated using Equation 6.9 and the global E_m value obtained by back analysis will take account of local variations in the soil conditions and the possible increase in soil stiffness with depth which occurs in many soils.

If no useful settlement results are available to evaluate E_m, it may be obtained from the results of triaxial compression or other suitable tests carried out in the laboratory.

The adjusted elasticity method may only be used if the stresses in the soil are such that no significant yielding occurs and if the stress-strain behaviour of the soil may be considered to be linear. Great caution is required when using the adjusted elasticity method in the case of non-homogeneous ground.

iii) Semi-Empirical Methods

The total settlement of a foundation may be estimated from the results of a field test such as Cone Penetrometer Tests, Standard Penetration Tests or pressuremeter test using a semi-empirical relationship between the test results and the settlement.

When using a field test to calculate the total settlement it is important to take account of experience in the use of this test in the local soil or in similar types of soil.

For granular soil, settlement may be estimated using a semi-empirical method and interpolating from the results of in-situ tests.

This is because:

- sampling is difficult,
- the stiffness modulus varies significantly with stress level.

6.6.2.3 Evaluation of Settlement Components. To evaluate the settlement components, separate calculations are required for the undrained settlements and the consolidation settlements.

guide: Undrained Settlements

The undrained settlements of a foundation may be evaluated using the methods described in Section 6.6.3.2.

i) Stress-Strain Method

For layered soil, the undrained settlement may be estimated by summing the settlements for each layer calculated using the vertical and horizontal stress distribution in the soil and the appropriate tangents to the undrained stress-strain curve for each layer.

ii) Adjusted Elasticity Method

For materials which are approximately homogeneous in stiffness the soil may be assumed to behave as an ideal homogeneous isotropic elastic material. In this case the undrained settlement may be found from:

$$S_o = \frac{qB}{E_u} f_o \, f_u \qquad 6.10$$

where:

q is the average bearing pressure on the base of the foundation as in Equation 6.9

B is the width of the foundation

E_u is the mean value of Young's modulus for the deforming stratum for undrained conditions

f_o is the foundation depth factor

f_u is a settlement factor whose value depends on the shape and dimensions of the foundation area, the thickness of the deforming stratum and on Poisson's ratio for undrained conditions, ν_u.

E_u is assessed on the basis of either field tests (e.g. pressuremeter test) or laboratory tests (undrained or consolidated undrained triaxial compression tests). The selection of the E_u value is critical. A secant modulus at the approximate stress level should be used.

guide: Consolidation Settlements

To calculate the consolidation settlement a confined 1-dimensional deformation of the soil may be assumed and the compression test curve is then used. Disturbance of the specimen should be taken into account when considering the consolidation curve. The stress distribution in the soil due to the foundation loads must be estimated.

Any difference between the consolidation behaviour of the soil specimen in the test apparatus and the natural soil in-situ must also be taken into account.

The one-dimensional calculation methods given above tend to over-estimate the consolidation settlement and are not considered reliable for overconsolidated clays.

An alternative approach is to assume that the calculated one-dimensional settlement gives a good estimate of the total settlement. Then, if the consolidation settlement, s_1, is required separately, the undrained settlement, s_0, must be subtracted from the total settlement.

Time-settlement behaviour

With cohesive soils the rate of consolidation settlement before the end of primary consolidation depends on the soil type and on the in-situ drainage conditions. This can be estimated approximately using the time-settlement curve obtained from a compression test. However, allowance should also be made for the influence of soil fabric, fissuring etc. These features often lead to more rapid consolidation.

When estimating the rate of settlement for each layer, the time-settlement curve chosen is that obtained from the compression test for the load increment closest to the actual increase in stress at the centre of the layer due to the foundation load.

6.6.3 Tilting

Foundations subjected to an eccentric or an inclined central load for uniform soil conditions or foundations subjected to a vertical central load for non-uniform soil conditions should be designed against tilting. For foundations subjected to loads with large eccentricities the design must show that rounding of the soil surface beneath the foundation does not occur.

guide: Several methods may be used to estimate the tilting of an eccentrically loaded foundation. For example the tilting may be estimated by assuming a linear bearing pressure distribution and then calculating the settlement at the corner points of the foundation using the vertical stress distribution in the soil beneath each corner point and the settlement calculation methods described in Section 6.6.3.

The settlements of a structure due to shear stresses between the foundation base and the soil, caused mainly by horizontal loading, are normally insignificant and may usually be disregarded.

6.6.4 Differential Settlement

The differential settlements for foundation beams and rafts should be estimated to ensure that these do not lead to the occurrence of a serviceability limit state, such as unacceptable cracking or the jamming of doors, in the supported structure.

guide: The allowable differential settlements for foundation beams and rafts depend on the type and the material of the supported structure.

6.6.5 Vibration Analyses

Foundations for structures subjected to vibrations or with vibrating loads should be designed to ensure that resonance will not occur between the frequency of the pulsating load and a critical frequency in the foundation soil system and that the vibrations will not cause excessive settlements.

Even when resonance is avoided it is still necessary to limit the amplitudes of vibration of the system to levels which can be tolerated by the structure, its occupants, any machinery and the foundation.

If vibration is likely to be significant or cause problems then Geotechnical Category 3 procedures will be required.

guide: Settlements due to vibrations will be most marked in the case of very loose sandy soils and fill material because of compaction.

The preferred method of analysis of foundation block response to dynamic loads is based upon the theory of the elastic half space and requires the dynamic elastic moduli of the ground to be either measured or estimated.

6.7 Structural Design Considerations

6.7.1 Concrete Design

Except for minor structures, such as two storey dwelling houses and lightly framed structures, reinforced concrete pad or strip foundations should be used.

The thickness of concrete foundations should not be less than 150 mm and, following the requirements of Chapter 10, care should be taken with excavation levels to ensure that this minimum thickness is maintained.

To reduce the risk of tension developing on the underside of the base of unreinforced concrete strip foundations for minor structures, an adequate thickness of concrete must be used and the projecting portion should not be greater than the foundation thickness so that the angle of spread from the pier or base plate to the outer edge of the ground bearing does not exceed 1 vertical in 1 horizontal. At all changes in level unreinforced foundations should be lapped at the steps for a distance at least equal to the thickness of the foundations or twice the height of the step, whichever is greater. The steps should not be of greater height than the thickness of the foundation unless special precautions are taken.

6.7.2 Ultimate Limit State Design

When designing the concrete in a shallow foundation for an ultimate limit state failure of the foundation, the contact pressure should be in equilibrium with the ultimate limit state loads.

guide: To design the longitudinal reinforcement in a strip foundation
: supporting columns it is conservative to design the footing against
: the following two pressure distributions:
:
: - 1) where the parts of the strip beneath the columns act as pad
: foundations mobilizing the full bearing resistance of the
: soil, and the intermediate sections transfer no load to the
: soil,
: - 2) where a wider spread of load is assumed.

6.7.3 Serviceability Limit State Design

a) Design Against Structural Loads and Ground Reactions

Strip and raft foundations which support columns should be examined for the distribution of subgrade reactions corresponding to the deformations of the foundation and the soil in the serviceability limit state

guide: It is often difficult to determine the actual distribution of subgrade reactions even reasonably accurately. This may depend on the variation of loads on neighbouring areas, and changes may occur with time as the soil strata become compressed.

For strip footings it is generally on the safe side to design the footing against the following two pressure distributions:

- 1) uniform pressure (P) which excludes buoyancy and the weight of the excavated earth,
- 2) with a pressure of 1.5 x P on the outer quarters of the strip and a pressure of 0.5 x P on the inner quarters of the strip.

A similar procedure may be used for raft foundations but this approach is very conservative for large flexible rafts.

b) Protection Against Water

Raft foundations for structures should be protected against the penetration of groundwater or the transmission of vapour to the inner surface of the building by the use of a continous impervious membrane. Construction joints should be kept to a minimum so as avoid movements which could damage the impervious membrane.

guide: An impervious membrane may be provided by using mastic asphalt or some form of impervious sheeting such as bitumen sheeting. For structures where protection against visible penetration of water only is required and transmission in the form of vapour is acceptable, high quality concrete alone may be used.

6.7.4 Durability

Corrosion of the concrete in a foundation should be considered in relation to the ground conditions. Foundation concrete should be protected from attack by sulphate salts in the ground, acidic groundwater or other aggressive agents.

guide: Chemical attack does not take place if there is no groundwater and for the disintegration to continue there must be replenishment of the corrosive chemicals. Soil permeability therefore is an important factor in corrosive attack on foundation concrete. Other factors increasing the severity of attack are the porosity of the concrete and the presence of cracks.

: Prescriptive Measures

: Normally foundations are in a protected environment and no special precautions are required against corrosion. However to resist corrosion attack, the concrete should be dense and impermeable and of a high grade. In addition a waterproof membrane, such as polythene sheet, may be provided around the foundation.

6.8 Supervision of Construction

Supervision must be carried out during the construction of shallow foundations to check:

- that the actual ground conditions, including the groundwater conditions, and any other environmental features encountered during construction are not more adverse than those assumed for the design,
- that the foundation level is suitable geotechnically,
- that the position, depth and size of the foundation comply with the design specifications,
- that the quality of the construction components and materials is satisfactory and that the foundation pads, beams and slabs are not defective.

guide: The supervision to check the actual ground conditions will normally be based on visual inspection supplemented, as required, by specified tests to measure the strength, density, etc.

The suitability of a foundation may be checked by measuring the settlements and evaluating the distortion of the foundation during and after construction.

CHAPTER 7 - PILE FOUNDATIONS

7 PILE FOUNDATIONS

7.1 Scope

The provisions of this Chapter apply to end-bearing piles, friction piles, tension piles and horizontally loaded piles for buildings, structures and earth-retaining walls.

Piles may be vertical or inclined and may have enlarged bases. The pile material may be either concrete, wood or steel.

The piles may be installed by driving (with various types of hammers or vibrating techniques), by boring, by jacking or by screwing.

guide: Piles can be classified in three main categories, which depend on
: their effect of the soil during installation. The categories are
: shown in figure 7.1 a.

7.2 Limit States

In order to satisfy the performance criteria related to:

- stability,
- limited deformations,
- durability,
- limitation of damage to nearby structures or services,

the following limit states must be prevented.

Type 1 A Ultimate Limit States

These occur when a collapse mechanism forms in the ground due to:

- slope stability failure,
- uplift,
- shear failure of the soil beneath and around a single pile or pile group.

Structural failure of the pile will also cause a type 1 A Ultimate Limit State.

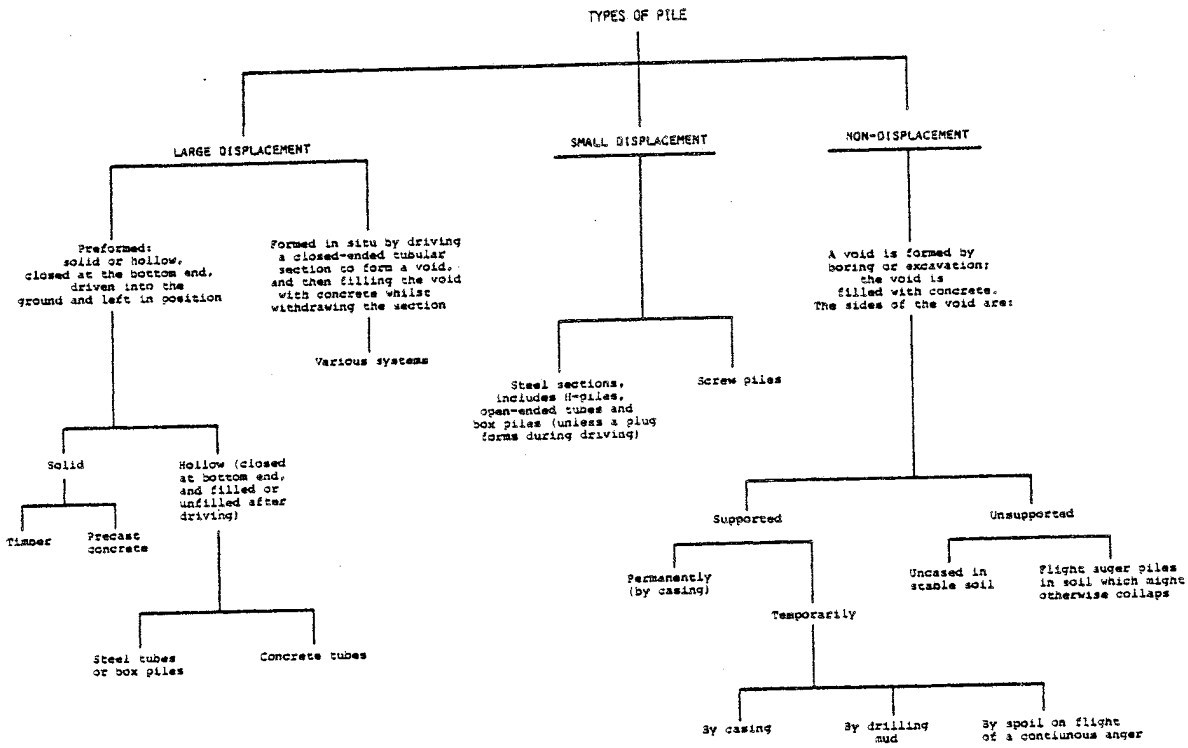


Figure 7.1 a. Pile Classification Related to their Effect on the Soil during Installation

guide: Slope stability failure may occur if a piled foundation is embedded in or near a slope, or in the vicinity of an earth retaining structure as illustrated in Figure 7.2 a.

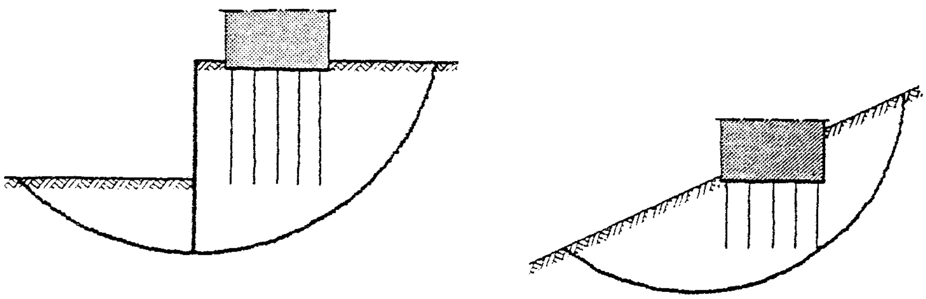


Figure 7.2 a. Examples of Loss of Overall Stability for a Piled Foundation

guide: For pile foundations in tension (for example beneath a dock or sluice), uplift of the structure and of the block of soil containing the piles may occur, as illustrated in Figure 7.2 b.

Shear failure of the soil may occur:

- under compression loading (bearing capacity failure),
- under tension loading,
- under transverse loading.

Under compression loading the soil surrounding the toe of a single pile or pile group yeilds due to shearing and compression. This is accompanied for a single pile by shear failure between the pile shaft and the soil.

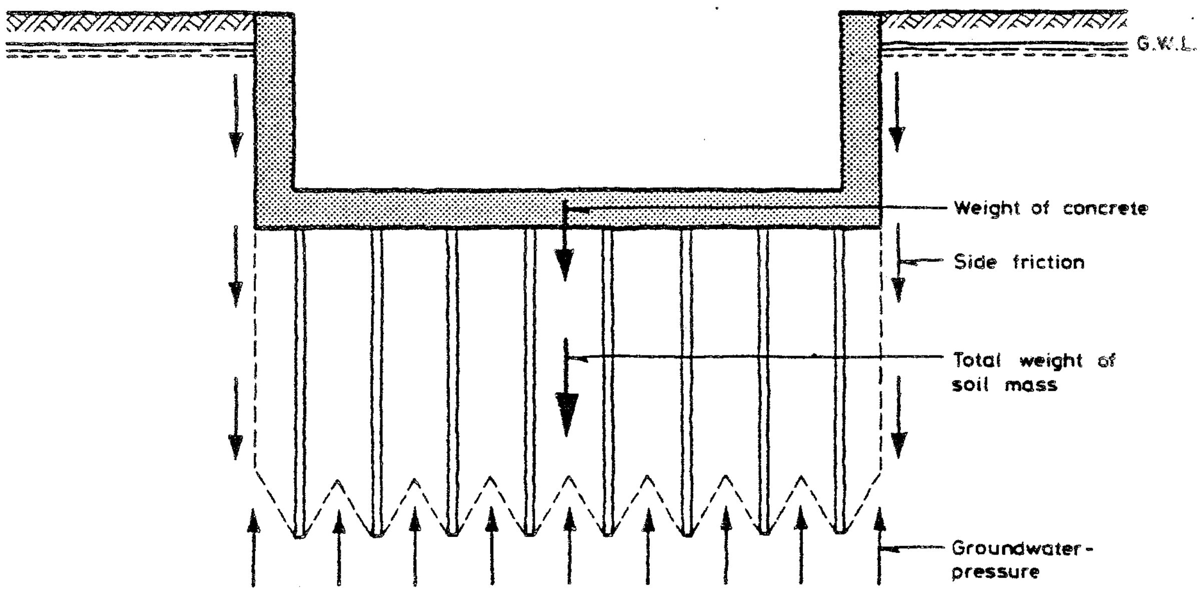


Figure 7.2 b. Forces Acting on Pile Foundation for Dry Dock Subjected to Uplift

guide: For a single pile under tension loading shear failure may occur between pile shaft and soil. For a pile group under tension shear failure may occur at the perimeter of the group.

A pile or group of piles may also fail in tension by pulling out a cone of soil originating from the toe of the pile as shown in Figure 7.2 b. This will occur if the combined resistance of the weight of the cone and the strength of the soil along its surface are exceeded.

The failure of piles under transverse loading is complicated. For short stiff piles failure of the soil may be represented near the surface by a 3-dimensional passive wedge, and at depth by a general shear failure (of the Prandtl type) in the horizontal plane. For long slender piles, ultimate failure due to transverse loads is normally accompanied by structural failure of the pile.

Structural failure of piles is a particular concern for long slender piles under tensile or transverse loads.

Type 1B - Ultimate Limit States

These occur when movements of the foundation load to servere structural damage in other parts of the structure. In structures with piled foundations they normally occur as a result of differential movements between piles, or between piles and other elements of a foundation.

guide: The design of piles in compression is often governed by a type 1B Ultimate Limit State (or by serviceability considerations), and it is necessary to determine the load-deformation behaviour of the pile. This is often established from the ultimate bearing capacity of the pile by relating settlement (normalised as a fraction of the ultimate bearing capactiy).

If this approach is not followed, and a direct estimate of settlement is made, then an independent check against a Type 1A Limit State due to shear failure of the soil is required.

Type 2 Serviceability Limit States

These occur:

- when movements of the piled foundation effect the appearance or efficient use of the structure or cause damage to finishes or non-structural elements,
- when the structure suffers excessive vibration, caused, for example, by resonnance in the soil/structure system.

guide: The serviceability limit state for buildings and structures is often connected with allowable distortion or relative rotations. Generally, deformations and differences between the movements of various parts of the foundations are to be investigated and the approach described for Ultimate Limit State 1B is often applicable.

7.3 Design Methods

The design must show that both ultimate and serviceability limit states are sufficiently improbable. Appropriate design values of loads, soil parameters and of measurements made in pile tests must be used in analyses.

The method of design, must describe the behaviour of the foundation at the limit state being considered. The method may be based on the results of load tests, empirical methods or on calculations and it is often helpful to use two or all of these approaches and to compare the results.

guide: To investigate Ultimate Limit States it is generally necessary to assess the design values of the ultimate bearing capacity and the movement of the foundation elements. The ultimate bearing capacity is generally estimated from the results of load tests on single piles, or from empirical methods based on results of in situ soil tests (for example, Potal cone Penetrometer, Standard Penetration Test or Pressuremeter) for which local experience is available. Displacement predictions are usually based on the results of load tests or on local experience with similar piles.

Group effects can have a significant adverse influence upon displacement behaviour and must be considered.

Settlement calculations may only be used if experience has shown that this approach is reliable.

Negative skin friction has a significant effect on the settlement of piles under compression and must be included in settlment calculations.

Structures which require piled foundations to resist tension must be classified in Geotechnical Category 3 if a failure of the piles will lead to severe damage of the structure.

guide: Pile foundations for structures classified in Category 1 may be designed from local experience provided that pile type and ground conditions remain within the area of experience, and that the site is controlled in accordance with the principles of Section 7-8 and of Chapter 10.

7.4 Actions

In selecting the actions for any calculation the designer must consider the forces and displacements listed in Section 3.1.2. The possible effects of negative skin friction, heave, and of horizontal movements of the ground, must also be considered.

Design values of the actions must be derived in accordance with the principles of Section 2.3.

7.4.1 Negative Skin Friction

In considering the effect of negative skin friction the settlement of the ground remote from the piled foundation must generally be treated as an action. In particular circumstances the force transmitted from the ground to the pile shaft must be treated as an action.

guide: Negative skin friction occurs when the soil moves downward along part of the pile shaft. It is caused by the the compression of layers of soft soil above the toe of the pile. Negative skin friction is generally evalutated by considering the relative stiffness of the piles and the soil in relation to the design values of the settlement of the ground.

guide: For piles founded in an unyeilding bearing stratum, for example, driven piles and bearing on rock, a small settlement of the ground may generate a high negative skin friction force. For structures in Geotechnical Categories 1 and 2 the effect of a small settlement of the ground must be considered when checking against a structural failure of the piles. When checking against other limit states, small settlements of the ground may be ignored.

Forces caused by negative skin friction may always be treated as actions, and must then be assigned the maximum values attainable. In many cases this approach is unreasonably severe, but it must be adopted if the settlement of the ground is much greater than the allowable settlement of the structure. Interaction between piles and soil has little beneficial effect in this case. Generally, this situation will arise when the expected compression of the soil above the toe of the pile exceeds 0,1 m.

guide: Negative skin friction acts in combination with other permanent loads. Live loads acting in combination with negative skin friction need not be fully taken into account.

7.4.2 Heave

Unloading, excavation, or removal of vegetation such as trees, may cause the soil surrounding the piles to expand or heave. Upward forces may be generated along the pile shaft. In considering this effect the movement of the ground is generally treated as an action.

guide: Heave may take place during construction, before the piles are loaded by the structure, and may cause unacceptable uplift or structural failure of the piles.

7.4.3 Horizontal Movements

Horizontal ground movements may exert pressures on piled foundations. They may be caused by any of the following:

- different amounts of surcharge on either side of the foundation,
- different levels of excavation on either side of the foundation,
- a foundation located at the edge of an embankment,
- a foundation constructed on a creeping slope.

The pressure on the piled foundation must normally be evaluated by treating the piles as bending elements in a deforming soil mass, and is not then an action. This approach is usual for strong soils, or for closely spaced piles.

In particular circumstances the pressure on the piled foundation may be treated as an action.

guide: If the soil near ground surface is weak, and the piles are widely spaced, interaction between the piles and the ground has little effect on the pressure exerted on the piles.

: In this situation the pressure is treated as an action and is evaluated by regarding the foundation as a stiff element within a mass of flowing soil. The pressure may be found approximately from the expression

$$p_L = N_L \, c_{ud} \qquad (7.1)$$

: where

: p_L is the pressure per unit area of the longitudinal cross section of the pile

: N_L is an empirical factor (between 8 and 9 for most soft soils)

: c_{ud} is the design undrained shear strength of the weak soil.

: Loads on piles due to horizontal ground movements may be evaluated by considering the equilibrium of a block of soil in which slidning is resisted by the reaction of the foundation, and is then an action.

7.5 Design Situations, Loads Cases and Loading Combinations

Design situations and related load cases and loading combination must be chosen in accordance with the principles of Sections 3.1.1 and 3.3.4

7.6 Design Considerations Related to Pile Installation

The design must ensure that the method of installation is appropriate for the ground conditions at the site and has no adverse effect on:

- the integrity of the pile being installed,
- piles which are already installed,
- adjacent structures.

The following aspects must be considered:

- pile-material quality,
- stresses in the pile during installation,
- sequence of pile installation, especially for cast-in-place piles,
- chemical attack,
- effect on adjacent structures or services.

guide: Items which requires attention include:

- the dynamic stresses in the pile during driving,
- the type of hammer to be used,
- the spacing of the piles in pile groups,
- seeking in cast-in-place piles,
- the retarding influence of chemicals in the soil on wet concrete in cast-in-place piles which are not permanently cased,
- local instability of a pile bore during concreting which may cause a soil inclusion within the shift.

For driven piles, a dangerous situation occurs when the pile enters a soft soil layer below a hard stratum. The compression stresswave is reflected at the pile base as a tension stresswave of nearly the same magnitude, and may fracture the pile. Defects in cast-in situ driven piles can be caused by driving successive piles to close together.

Necking may occur in a cast in situ pile at the boundary between an upper layer of stony soil and a lower layer of weak soil. The pile bore in the weaker layer may expand after the concrete is placed, allowing wet concrete to flow down the pile. If arching develops in the concrete higher up the pile, a neck is formed in the pile.

7.7 Structural Design of the Pile Foundation

The structural design of the piles must be in accordance with the requirements given in Eurocodes 2, 3 and 5.

guide: It may be necessary to add to EC 2, 3 and 5 if piles are not treated
: specially in these codes.

7.8 Limit State Design of Piles in Compression

7.8.1 Overall Stability

The procedures given in Chapter 9 must be used to demonstrate that a slope stability failure of the soil mass containing the foundation will not occur.

Axial and lateral loads acting on the piles must be included in the stability calculations.

guide: A check of the overall stability is not generally necessary for normal
: bearing pile foundations.
: Exceptions are:
:
: - pile supported earth-retaining structures,
: - pile foundations of abutments,
: - pile foundations in sloping ground.

7.8.2 Bearing Capacity Failure

Bearing capacity failure occurs when the piles are loaded to such an extent that rupture zones are formed in the ground beneath the pile base and at the pile/soil interface. In this condition the displacement of the pile foundation increase without significant increase in load.

To demonstrate that the foundation will support the design, load with adequate safety against this type of failure, the following inequality must be satisfied:

$$F_d \leq Q_d \tag{7.2}$$

where

F_d is the Ultimate Limit State axial design load

Q_d is the Ultimate Limit State design bearing capacity under the type of loading considered.

The values of F_d and Q_d must satisfy the requirements of Chapters 2 and 3.

Q_d must be obtained either from pile-loading tests or from calculations using soil-strength design values and/or pile-driving formulae.

The possible effects of pile installation and the type of pile (displacement or non-displacement) must be considered. If analytical and empirical design calculations are to be used they must be supported by evidence such as pile-loading tests carried out in similar conditions.

guide: Calculations in which design values of in situ test results are used empirically to represent the strength of the soil are generally preferred in practice. Methods based solely on bearing capacity calculations using design shear-strength parameters of the soil, are not reliable.

7.8.2.1 <u>Design Ultimate Limit State Bearing Capacity from Pile-Loading Tests</u>. Pile loading tests must be carried out in the manner specified by the designer. The designer must check that the test pile is installed in the same manner as the piles which will form the foundation.

In establishing the ultimate bearing capacity from pile-loading test results the following aspects must be considered:

- if negative skin friction is to be considered as an action (see Section 7.4.1), the ultimate pile resistance determined from the loading-test results must be corrected by subtracting the positive skin friction of the compressible stratum from measured ultimate resistance.

guide: Under pile-test loading conditions the test pile always moves downward with respect to the surrounding soil and the skin friction forces on the pile act upward in all layers. Because of this it is unsafe to use the measured ultimate pile resistance to obtain the design ultimate limit state bearing capacity in cases where negative skin friction is an action. The estimated value of the skin friction in the compressible soil layers must be subtracted from the measured failure load to obtain the design ultimate limit state bearing capacity of the pile. In equation 7.2 the anticipated design value of the negative skin friction must be added to the other actions.

The approach must also be adopted for end bearing piles if there is doubt that the contribution of shaft friction in the soil above the bearing stratum will continue to act throughout the life of the building.

guide: If the soil above the bearing stratum is soft, shaft friction in the soft layers will decrease with time, owing to compression and creep. In some cases it may be unreasonably conservative to ignore the shaft friction entirely, and the interaction between the pile and the ground may be analysed.

It must normally be located where the most adverse ground conditions are believed to occur. If this is not possible, an allowance must be made when deriving the design ultimate limit state bearing capacity.

If two or more pile loading tests are carried out, the test locations must represent the site of the piled foundations, and one of them must be located where the most adverse ground conditions are believed to occur.

The number of pile load tests carried out at the site and the range of the results must be considered when deriving the design ultimate limit state bearing capacity.

guide: Table 7.8 a shows a method of deriving the design ultimate limit state bearing capacity from the results of pile loading tests.

Number of pile-loading Tests N	Ultimate pile load from loading test Q_{max}	Design value of the ultimate pile-bearing capacity $Q_{max,d}$	Conditions
1	Q_{max}	$\frac{Q_{max}}{\gamma_m}$	
≥ 2	$Q_{1,max}$; $Q_{2,max}$; ...$Q_{N,max}$	$\frac{Q_{av,max}}{\gamma_m}$	if $\frac{Q_{N,max}}{Q_{1,max}} \leq 1,3$
	$Q_{av,max} = \frac{Q_{1max} + Q_{2max} + \ldots + Q_{Nmax}}{N}$	$\frac{Q_{1,max}}{\gamma_m}$	if $\frac{Q_{N,max}}{Q_{1,max}} > 1,3$
	$Q_{1,max}$ = lowest		
	$Q_{N,max}$ = highest		

Table 7.8 a. Design Ulitmate Limit State Bearing Capacity Derived on Basis of Pile Loading Tests

If the ultimate pile resistance cannot be reached during the loading test the ultimate bearing capacity of the pile must be set at the maximum applied test load.

Pile-loading tests must be carried out in the following cases:

- when using a piling system which is outside local experience and which has not been tested under similar soil and loading conditions,
- when using a pile system which is outside the experience of the operatives carrying out the work.

When the piles in the foundation will be subject to abnormal temporary loading conditions (e.g. heavy cyclic loading, including alternative compression and tension). The pile testing procedure then must contain similar loading cycles.

When the pile behaviour during installation is not as anticipated from the site investigation and previous experience.

7.8.2.2 <u>Design Ultimate Limit State Bearing Capacity from Soil Strength Parameters</u>. The calculation must comprise the following components:

- the ultimate end bearing resistance due to failure of the ground in the vicinity of the pile base,
- the ultimate shaft friction or adhesion forces.

The design ultimate limit state bearing capacity $Q_{ult,d}$ of a pile is the sum of the two component:

$$Q_{ult,d} = Q_{b,ult,d} + F_{s,ult,d} \qquad (7.3)$$

where

$Q_{b,ult,d}$ is the ultimate end bearing resistance calculated from design values of the soil strength parameters

$F_{s,ult,d}$ is the ultimate shaft friction calculated from design values of the shearing resistance between the soil and the pile shaft.

guide: In calculating the ultimate shaft friction where layers of soft soil are present above the stratum in which the pile is founded, the contribution of the soil above the bearing stratum must either be neglected, or reduced to a value which is obtained by considering the interaction of the pile and the soil.

For open-ended driven tube or box piles without special devices inside the tube or the box to induce plugging, the design ultimate end bearing resistence must be limited to the design ultimate friction between the soil plug and the inside face of the tube or box.

guide: Equation 7.3 can be transformed to:

$$Q_{ult,d} = q_{b,ult,d}\, A_b + \sum_{1}^{n} f_{s,ult,d}\, A_{s,i} \qquad (7.4)$$

where

A_b is the plan area of the base of the pile

$A_{s,i}$ is the surface of the pile shaft in soil layer i

$q_{b,ult,d}$ is the design value of the ultimate resistance per unit area of the base

$f_{s,ult,d}$ is the design value of the ultimate skin friction or adhesion per unit area of the pile shaft in layer i

The values of $q_{b,ult,d}$ and $f_{s,ult,d}$ must be derived from field tests (Cone Penetrometer test, Dynamic probing, Standard Penetration Tests, Vane tests, Pressuremeter tests) or from laboratory tests on undisturbed samples (triaxial tests, direct shear).

In cohesionless soils it is not normally possible to take undisturbed samples. In such cases the results of field tests must be used for the estimation of the values $q_{b,ult,d}$ and $f_{s,ult,d}$.

For piles which are completely embedded in the ground, failure by buckling is not likely to occur.

Slender piles passing through thick deposite of very weak soils must be checked against buckling.

7.8.2.3 <u>Calculation of the Design Ultimate Limit State End Bearing Resistance</u> ($q_{b,ult,d}$). The design values of the strength parameters of a zone of soil above and below the pile toe must be taken into account in calculating the ultimate bearing capacity of the pile base.

For non displacement piles the possible effect of installation on the strength of the surrounding soil must be considered.

If weak soil is present at a depth of less than 4 x the base-diameter below the toe of the pile, the possibility of a punching failure must be considered.

Variation of soil strength in plan must be taken into account in order to arrive at representative values of the soilstrength parameters.

If driven piles with oversized base plates are installed, the possible adverse effect of the oversized plate on the end bearing resistance of the pile must be taken into account.

guide: The zone of soil which influences the end bearing resistance extends for several diameters above and below the pile toe. Weak soil in this zone has a relatively large influence on end bearing resistance. This must be taken into account when the design values of the soil strength are assessed.

Assessments of this type are strongly empirical and it is necessary to follow local experience.

For non displacement piles such as bored piles the relief of the stress in the soil can be considerable, and the soil in the vicinity of the pile toe may be badly disturbed. Empirical correction factors are used to allow for these effects.

In calculating ultimate end bearing resistance from soil strength parameters, design values of the undrained shear strength, c_{ud}, or of the effective shear strength parameters, c'_d and ϕ'_d, are used, depending on type analysis which is appropriate for the design situation being considered.

If the ultimate end bearing resistance is obtained empirically from soil properties measured in in situ tests such as Dutch Cone Penetrometer, Standard Penetration Test or pressuremeter tests, the principles of Chapter 4 are to be adhered to in establishing the design values of soil properties.

Piles with enlarged base plates normally develop lower end bearing resistance than piles of uniform cross section and the same base area. At the protruding edges of the base plate, failure in the ground develops relatively easily. A reduction factor β, which depends on the ratio between the area of the base plate and the cross sectional area of the shaft, and on the length of pile which is enlarged, must be applied to allow for this.

7.8.2.4 Design Ultimate Shaft Friction ($f_{s,ult,d}$). To calculate the design ultimate shaft friction, design values of the relevant shear-strength parameters must be used.

For non-displacement piles, the possible effect of installation on the stress state and strength of the surrounding soil must be considered. Possible adverse effects of disturbance during installation must also be analysed. For all types of piles, the smoothness of the pile shaft must be considered and related to the installation procedure.

For piles with an oversized base plate, the possibility of a reduction in the shaft friction must be investigated.

guide: The design ultimate skin friction in a soil layer may be calculated by a simple analytical approach:

For drained conditions:

$$f_{s,ult,d} = a'_d + K_s \, \sigma'_v \tan \delta'_d \qquad (7.5)$$

where:

a'_d is the design value of the effective adhesion between pile shaft and soil

δ'_d is the design value of the effective angle of friction between pile shaft and soil

K_s is the earth-pressure coefficient at the pile shaft

σ'_v is the average effective vertical soil stress in the concerned soil layer

K_s depends on the type of pile, the method of installation and the length of the pile.

For short-term behaviour in cohesive soils:

$$f_{s,ult,d} = \alpha \, c_{u,d} \qquad (7.6)$$

where

$c_{u,d}$ is the design value of the undrained shear strength of the soil

α is the adhesion factor

guide: The adhesion factor α takes account of the disturbance of the soil caused by pile installation, and is evaluated from local experience.

The results of in situ tests may be used directly to assess the maximum skin friction in a soil layer, provided that the calculation method is based on locally established experience.

For cast in place concrete and bored piles the shaft roughness depends on the method of construction the pile shaft. In general, concrete piles cast in place without a casinq have a very rough surface. Piles bored under bentonite may have a betonite cake at the pile soil interface. This may affect the shaft friction which is developed.

Prefabricated piles and piles with a steel shaft are comparatively smooth.

The shaft friction developed by driven piles with oversized base plates may be reduced by the effect of the plate. The effect depends upon the way the pile is installed. If the concrete shaft is cast in the ground without a casing, the adverse effect of the protruding part of the base is negligible.

7.8.3 End Bearinq Resistance from Pile-Drivinq Formulae

If pile-driving formulae are used for the assessment of the design ultimate end bearing resistance of individual compression piles in a foundation, the validity of the formulae must have been demonstrated by static load tests on the same type of piles in the same ground conditions.

For structures in Geotechnical Categories 2 and 3, pile driving formulae may only be used in design if an adequate site investigation has been carried out.

The results of dynamic loading tests carried out with specialised loading and measuring equipment may only be used in design if an adequate site investigation has been carried out.

guide: Pile-driving formulae only give indicative values of the ultimate bearing capacity of piles which terminate in a layer of granular soil. The application of methods based on the wave-equation theory is recommended. These methods must be used with caution if the driving resistance decreases on redriving.

Dynamic loading tests are usually used to examine piles after doubts have been raised during the exceution of the piling work. These tests are also useful for types of pile (such as continuous flight auger piles) in which quality depends on installation procedures which are not easy to monitor.

7.8.4 Design Ultimate Limit State Bearing Capacity of Pile Groups

If the bearing stratum of a group of piles overlies a layer of weak soil, the effect of the weak layer on the bearing capacity of the group must be considered.

The design value of the ultimate limit state bearing capacity of the pile group may not exceed the sum of the design values of the ultimate limit state bearing capacities of the individual piles of the group. When deriving the design ultimate limit state bearing capacity of a pile group, structural connection between the piles in the group must be considered.

guide: If a group of piles is founded near to the bottom of a stratum which overlies soft soil, failure of the soft soil can occur due to a combination of punching through the bearing layer and squeezing of the soft soil.

If the piles are founded within a thick layer, or if the ground improves with depth below toe level, a group of driven piles which act together may benefit from compaction due to pile driving.

This effect may occasionally result in a block of stiffer soil containing the piles. The bearing capacity of the block may be greater than the sum of the individual bearing capacities. The stiffening effect depends upon changes in the soil during piling which are not certain and which are difficult to control. The bearing capacity of pile groups must not exceed the sum of the individual capacities of the piles in the group. The individual capacities may be determined by pile loading tests or in situ tests made in guide: the improved soil after the pile group is installed. Borings must not be made as they can disturb the soil between the piles considerably.

The ultimate bearing capacity of a group of piles depends upon the structural connection between them. Generally, the piles will have different individual capacities. If the piles support a rigid structure the capacity of the group is equal to the sum of the individual capacities. This is termed a parallel system. If the piles support a flexible structure they are unable to act together, and the failure of one pile may lead progressively to the failure of the whole foundation. This is termed a series system.

In a series system the bearing capacity of the foundation is determined by the bearing capacity of the weakest pile.

7.8.5 Settlement of the Pile Foundation

The assessment of settlement must include:

- the settlement of single piles,
- the additional settlement due to group action,
- compression of weak soil layers below the bearing stratum.

The settlment of the single piles must be estimated on the basis of:

- pile-load tests,
- empirical load-settlement curves obtained for similar soils and piles,
- calculations on the basis of soil-stiffness parameters. These methods must be calibrated against pile-load test results.

The compression of soil layers below the bearing stratum in which the piles are founded must be analysed in accordance with the principles given in Section 6.6.

The analysis must include an estimate of differential settlements of the foundation. If ground movements remote from the piles are small, and the force on the piles due to negative skin friction is not treated as an action, then the effect of the ground movements on the settlement of the piles must be considered.

guide: The settlement of a single pile is often based on empirical load-settlement curves for particular soil conditions and types of pile.

Generally these load-settlement curves are described by the non-dimensional ratios:

- load divided by the ultimate bearing capacity (Q/Q_{max}),
- settlement divided by pile-base diameter.

When checking against a type 1B Ultimate Limit State, the value of Q_{max} must equal the Ultimate Limit State design bearing capacity Q_d. This method is illustrated in Figure 7.8 a.

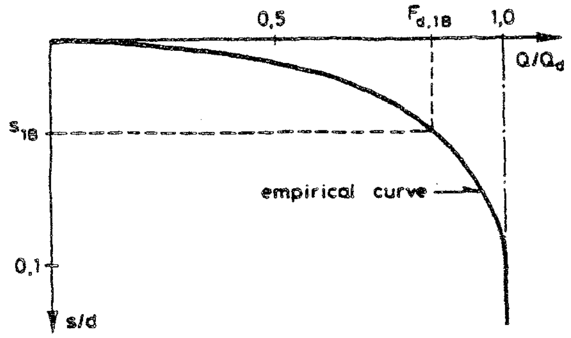


Figure 7.8 a. Design of a Pile against Type 1B Ultimate Limit State

The additional settlement caused by the interaction of the piles in a group may be assessed either from a simplified elastic calculation or from methods presented in Section 6.6.

The effect of negative skin friction on the settlement of a piled foundation may be assessed by considering the interaction process between the soil and the pile. The additional settlement cannot exceed the compression of the soil above pile toe level at a point remote from the foundation.

Provided that ground movements are small, and that forces caused by negative shin friction are not treated as actions, the additional settlement caused by negative skin friction is approximately equal to 0.5 x the settlement of the ground remote from the piles. If the settlement of the ground is only about 0.01 m, the effect of negative skin friction on pile settlement may be ignored.

7.8.6 Maximum Value of the Negative Skin Friction

The maximum value of the negative skin friction must be assessed by considering that the piles are fixed and the soil moves downwards.

If a bituminous coating is applied to the pile shaft above the bearing stratum, a residual shear force of 10 kN/m^2 must be assumed to calculate the maximum value of negative skin friction.

guide: The maximum value of the negative skin friction is defined as the smaller of:

- the total frictional resistance of the pile shaft in the soil layers above the stratum in which the piles are founded,
- the force $F_{n,k}$ which is notionally required to prevent further settlement of any fill which has been placed around the foundation, calculated as shown on Figures 7.8 b and 7.8 c. s_{gl} is the expected settlement of the ground level after installation of the piles. If a is the percentage of the expected ground settlement, which has already occured before the installation of the piles, then:

$$\Sigma F_{n,k} = \frac{100-a}{100} Ah\gamma'$$

Where $A = \frac{1}{4} \pi H^2$

The factor a may be established in the field by pore-pressure measurements or settlement measurements. For centre piles in a group consisting of a large number of piles the maximum negative skin friction $F_{n,k,max}$ will not exceed the value given by the following equation:

$$F_{n,k,max} \approx \frac{100-a}{100} hA\gamma'$$

Where A is defined in Figure 7.8 c.

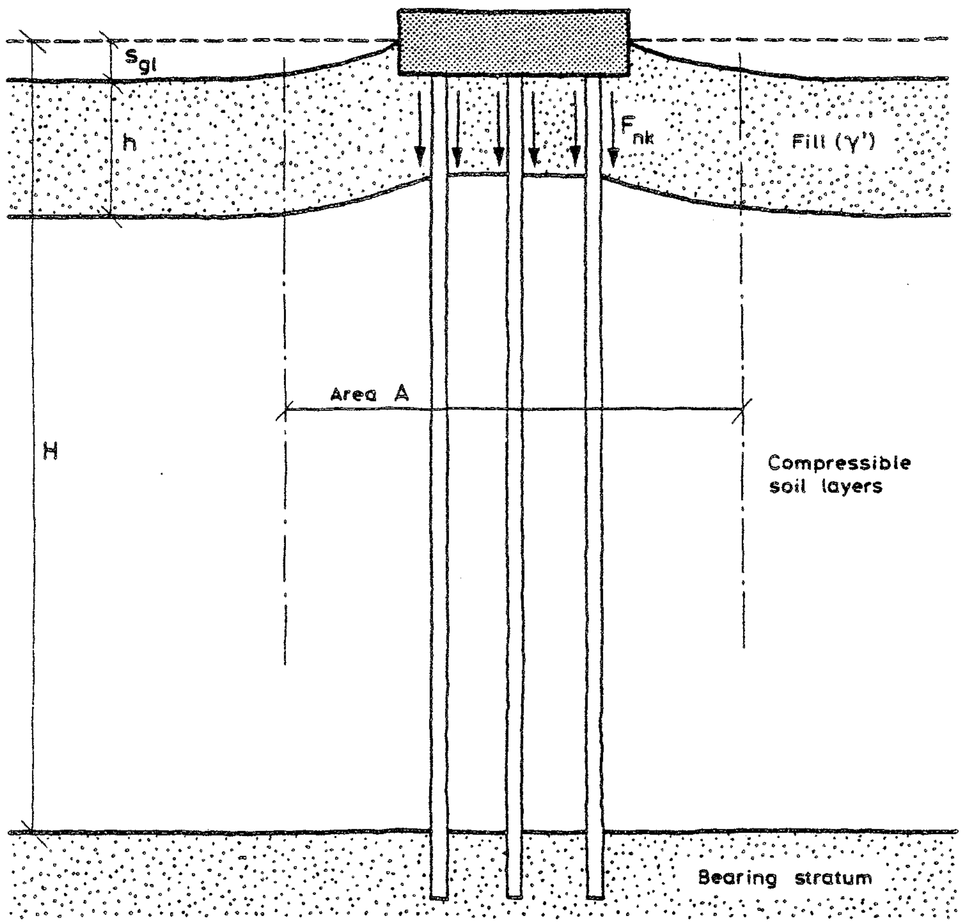


Figure 7.8 b. Calculation of Maximum Value of Negative Skin Friction for a Single Pile or a Pile Group Consisting of Few Piles

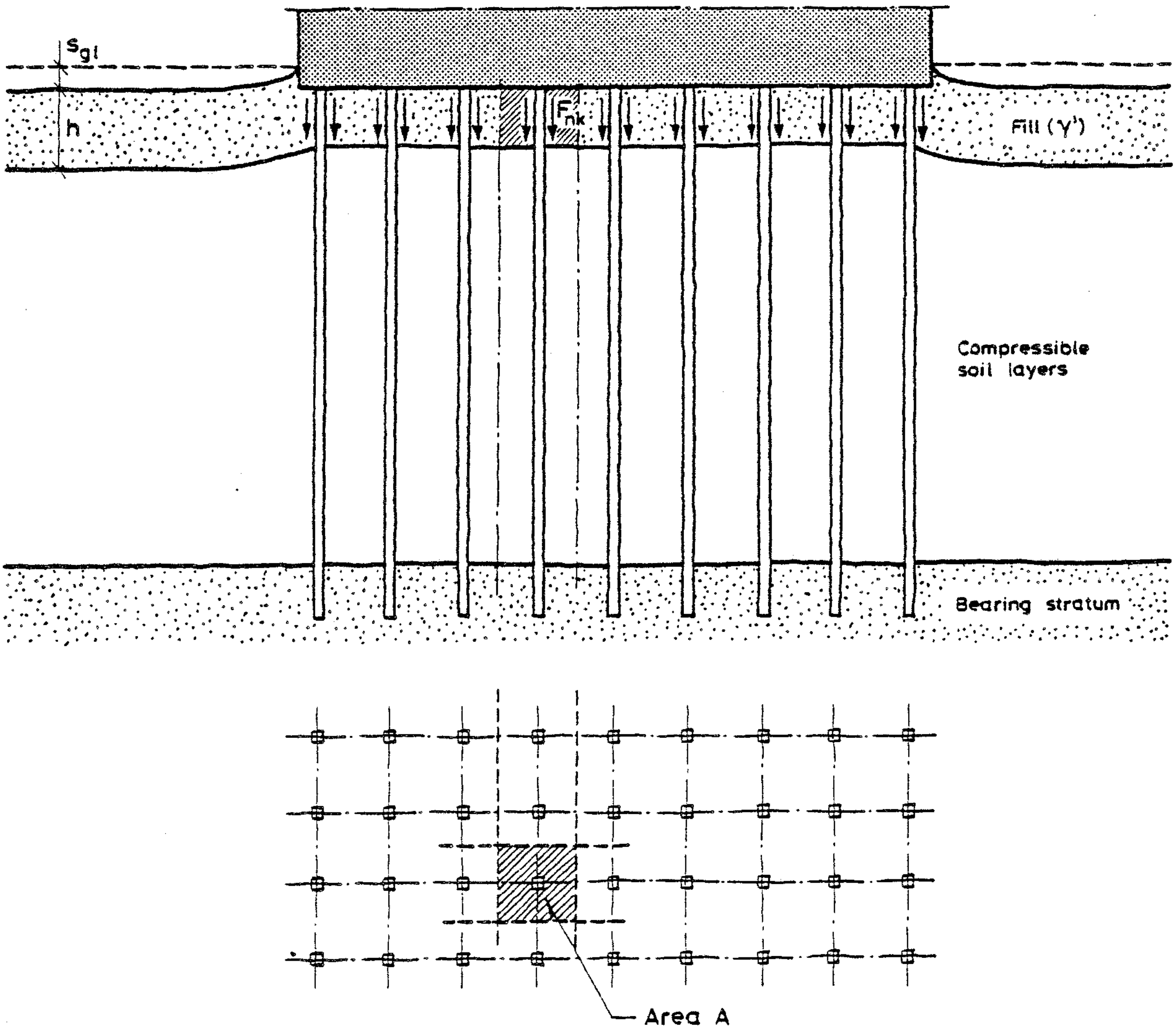


Figure 7.8 c. Calculation of Maximum Value of Negative Skin Friction for a Pile Group Consisting of a Large Number of Piles

If the pile shaft is coated with bitumen to reduce negative skin friction, the coating must not extend into the bearing stratum, as this can reduce the load carrying capacity of the pile considerably.

7.9 Limit State Design of Tension Piles

7.9.1 Overall Stability

The procedures given in Chapter 9 must be used to demonstrate that a slope stability failure of the soil mass containing the foundation will not occur. The tensions acting on the foundations must be included in the stability calculations.

7.9.2 Uplift

Groups of piles in tension may fail by uplift of the block of soil containing the piles as illustrated in Figure 7.9 a. To demonstrate that uplift failure is sufficiently remote, the following inequality must be satisfied:

$$\sum T_d \leq W'_d + \sum F_{pd} \tag{7.7}$$

where:

T_d is the design tension force acting on a pile

W'_d is the design effective weight of the soil block and piles

F_d is the shear resistance at the boundary of the block of soil.

guide: Uplift is generally the governing failure mechanism in closely spaced groups of tension piles in which the distance between the piles satisfies the condition:

$$\frac{D}{d} < \sqrt{\frac{L}{d}} \tag{7.8}$$

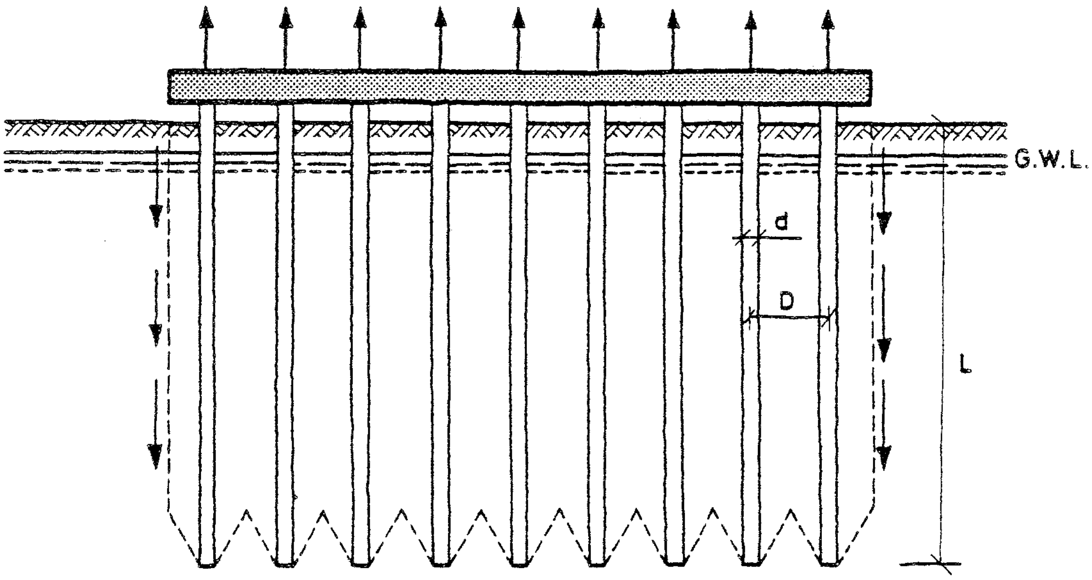


Figure 7.9 a. Group of Piles in Tension Failing by Uplift

7.9.3 Shear Failure in Tension

To demonstrate that shear failure of a pile foundation in tension is sufficiently improbable, the following inequality must be satisfied:

$$T_d \leq Q_{d.t} \qquad (7.9)$$

where:

T_d is the design value of the tension load on the pile foundation

$Q_{d,t}$ is the design value of the ultimate tensile capacity of the pile foundation

The values of T_d and $Q_{d,t}$ must be derived in accordance with the principles of Chapters 2 and 3.

$Q_{d,t}$ must be obtained either from pile loading tests or from calculations using design values of the shearing resistance between the pile and the soil.

The effect of pile installation and the type of pile must be considered.

The design of tension piles must normally be based on the results of load tests. The installation of certain types of pile can have a detrimental effect on the strength of the soil close to the pile shaft. Such effects are often erratic and may not be detected by a pile loading test. Piling systems which give wide differences in performance between a test pile and the piles used in the foundation must not be used as tension piles.

7.9.3.1 Design Ultimate Limit State Tensile Capacity from Pile-Loading Tests. Pile-loading tests must be carried out in the manner specified in the design and it must be checked that the test pile is installed in the same manner as the piles which will form the foundation.

If one pile-loading test is carried out it must normally be located where the most adverse ground conditions are likely to occur. If this is not possible, an allowance must be made when deriving the design ultimate limit state tensile capacity.

If two or more pile loading tests are carried out, the test locations must represent the site of the piled foundations, and one of them must be loacted where the most adverse ground conditions are believed to occur. The number of pile load tests carried out at the site and the range of the results must be considered when deriving the design ultimate limit state tensile capacity.

guide: The design ultimate limit state tensile capacity may be derived from the results of pile loading tests in accordance with the principles given in table 7.8 a for piles in compression. For pile groups, the effect of interaction should be allowed for when deriving the representative ultimate tensile load from the load-test results, and before applying the partial factors quoted in the table.

If the ultimate pile resistance in tension is not reached during the loading test, the ultimate tensile capacity must be set at the maximum applied test load.

Pile-loading tests must be carried out in the following cases:

- when using a pile system which is outside local experience and which has not been tested under similar soil and loading conditions,
- when using af piling system is outside the experience of the operatives carrying out the work,
- when the piles in the foundation will be subjected to abnormal temporary loading conditions (e.g. heavy cyclic loading, alternative tension and compression), the pile-testing procedure must then contain similar loading cycles,
- when the pile behaviour during installation is not as anticipated from the site investigation and previous experience.

7.9.3.2 Design Ultimate Limit State Tensile Capacity from Soil-Strength Parameters. The evalutation of the design ultimate limit state tensile capacity of isolated tension piles or of a group of tension piles from soilstrength parameters must include the following:

- the tensile strength of the pile itself,
- the ultimate shearing resistance between pile and soil in the strata which contribute to the tensile resistance of the pile,

- the possibility of failure by pulling out a cone of soil (especially for af pile with an oversized base or a rock socket).

The capacity of an isolated tension pile depends on the shearing resistance which can be developed at the interface of pile shaft and soil, or along a cone-shaped surface in the soil originating at the base of the pile. Progressive failure will invariably lead to a decrease of this resistance. Conservative values of strength parameters must therefore be used in design calculations for tension piles. The installation of piles in the ground requires thorough inspection.

The design ultimate tensile capacity of a pile may be assessed by simple calculations as follows:

$$f_{s,ult,d} = \alpha \cdot c_{ud} \qquad (7.10)$$

or:

$$f_{s,ult,d} = a_{u,d}$$

where

$f_{s,ult,d}$	is the design ultimate limit state resistance per unit area of the pile surface
$c_{u,d}$	is the design undrained shear strength of the soil
$a_{u,d}$	is the design undrained adhesion
α	is an adhesion factor, which is based on local experience and depends on the duration of loading

For long-term loading, and for short term loading in granular soils, the following relation may be applied:

$$f_{s,ult,d} = \sigma_n' \tan \delta_d' + a_d' \qquad (7.11)$$

which is often taken as:

$$f_{s,ult,d} = K_s\ \sigma_v' \tan \delta_d' + a_d' \qquad (7.12)$$

where

σ_n is the normal stress on the pile shaft

δ'_d is the design value of the angle of friction between pile and soil

a'_d is the design value of the effective adhesion between pile and soil

K_s is a coefficient of earth pressure at the pile shaft

σ'_v is the vertical effective stress in the soil.

Empirical methods based on local experience may also be applied. Some of these methods use experimentally obtained relationships between shear resistance and results of field tests, like Dutch Cone penetrometer, Standard Penetration Test and Pressuremeter tests; others use nominal shear-resistance or adhesion values for various kinds of soil.

In the assessment of the design ultimate tensile capacity the following factors must be considered:

- the effect of pile installation on soil properties and stress conditions. For non displacement piles, stress relief and possible disturbance must be considered,
- long term creep, which may reduce the horizontal in situ stress near to the foundation, and hence the shear resistance between the piles and the group,
- group action, which may reduce the effective vertical stress, and hence the ultimate tensile capacity of individual piles.

guide: For groups of tension piles, the tension forces applied to the piles
: cause upward forces in the soil mass between the piles. This decreases
: the effective vertical soil pressures, and may result in a substan-
: tial reduction of the ultimate tensile capacity of each of the piles
: in the group when compared with a pile loaded in isolation. This
: effect can be approximated by simple calculations.

The severe adverse effect on the ultimate tensile capacity in case of cyclic loading and reversals of load.

guide: If piles are subjected to substantial reversals of load for example in foundations for high-tension pylons and drydocks, the residual shear resistance between the piles and the ground may be considerably lower than the values occuring under quasi-static loading conditions. Local experience based on pile-loading tests in needed to appraise this effect.

7.9.4 Vertical Displacement (Lifting) of Pile Foundations under Tension

The assessment of the vertical displacement under tension must include.

- the vertical displacement of the single piles of the foundation,
- the additional vertical displacement due to group action,
- the expansion of underlying soil layers due to a decrease of the effective stresses in these layers.

The vertical displacement of single piles under tension must be estimated on the basis of:

- pile-load tests,
- empirical load-deflection curves obtained for similar piles and soils,
- calculations using soil-stiffness, which must have been calibrated against pile-loading test results. Calculations must include the interaction between the pile and the surrounding soil.

The expansion of the soil strata below the base of the piles must be calculated using soil-stiffness parameters.

The analysis must include an estimation of differential displacements in the foundation and an assessment of the deformation imposed on the structure.

guide: The vertical displacement or lifting of a group of piles under tension contains 4 main elements:

guide:
- the elongation of the piles, which is to be calculated from elastic theory,
- the movement of the pile with respect to the surrounding ground which is to be analysed on the basis either of the results of load tests or single piles, or of empirically obtained shear-stress/displacement curves for various pile and soil types,
- the expansion of the ground between the piles of the group, which is to be estimated in a pile/soil-interaction analysis in which soil-stiffness parameters established in expansion tests, are applied,
- the expansion of the underlying soil strata which is to be analysed by elasticity theory. Soil-stiffness parameters are usually obtained from measurements during excavations. The principles of Section 6.6 may be used.

For very large structures, like docks and sluices, the contribution of the expansion of the underlying soil strata to the upward movement of the structure can be considerable but uniform.

7.10 Ultimate Limit Design of Laterally Loaded Piles

7.10.1 Overall Stability

The procedures given in Chapter 9 must be used to demonstrate that a slope stability failure of the soil mass containing the foundation will not occur.

guide: A particular check on overall stability is to be made if the foundation:

- is in a slope,
- supports an abutment,
- supports an earth-retaining structure.

7.10.2 Ultimate Lateral Load Capacity

To demonstrate that the foundation will carry the design lateral load the following inequality must be satisfied:

$$F_{dh} \leq Q_{dh} \qquad (7.13)$$

where

F_{dh} is the design lateral load on the foundation

Q_{dh} is the design value of the ultimate lateral load capacity

The values of F_{dh} and Q_{dh} must be derived in accordance with the principles of Chapters 2 and 3.

Q_{dh} must be obtained either from pile-loading tests or be based on soil and pile-strength design values. The effect of pile installation on the quality of the soil must be taken into consideration when selecting soil-strength parameters.

Empirical or analytical design calculations must be supported by pile-loading tests carried out in similar soils.

guide: For normal buildings the horizontal loads caused only by wind are of minor importance and can easily be carried by the foundation. Special calculations are not required.

7.10.2.1 Design Ultimate Lateral Load Capacity from Pile-Loading Tests.

Pile-loading tests must be carried out in the manner specified in the design.

Pile tests must normally be located where the most adverse ground conditions are likely to occur. If this is not possible, an allowance must be made when deriving the design ultimate horizontal loading capacity.

The number of pile load tests carried out at the site and the range of results must be considered when deriving the design ultimate horizontal loading capacity.

guide: The design ultimate horizontal loading capacity may be derived from the results of pile loading tests in accordance with the principles given in table 7.8 a for piles in compression. For pile groups the effect of interaction should be allowed for when deriving the representative ultimate horizontal load from the load test results, and before applying the partial factors quoted in the table.

If the ultimate resistance is not reached during the loading test, the ultimate horizontal loading capacity must be set at the maximum applied load.

Pile-loading tests must be carried out in the following cases:

- when the horizontal load on the pile considerably exceeds normal practice,
- when the pile foundation is subjected to reversals in the direction of loading or to heavy cyclic loading.

7.10.2.2 Ultimate Lateral Load Capacity from Soil and Pile-Strength Parameters. If the ultimate horizontal load capacity of a pile or a group of piles is to be evaluated on the basis of soil and pile-strength parameters the piles must be treated either as short stiff piles or as long slender piles.

Short Stiff Piles

The pile is assumed to be a rigid body rotating around a point or translating until failure of the ground around the pile occurs.

guide: The failure mechanism in the ground changes with depth. Above a critical depth depending on the soil strength and the width of the pile, a wedge-shaped failure pattern may be assumed. Three-dimensional passive earth-pressure calculations may be used to assess the ultimate soil resistance.

Below the critical depth the failure mechanism is confined to a narrow area around the pile. The ultimate soil resistance may be calculated by adopting the methods of Section 6.5.2.2, to the situation of a vertical strip moving horizontally in the ground.

Long Slender Piles

This method normally applies only to steel piles. The pile is to be treated as a flexible beam in an elastic half space. The analysis must include the possibility of failure in the ground in the zone below ground surface. The ultimate horizontal loading capacity is determined by the flexural strength of the pile itself.

guide: The calculation may be carried out using the theory of a beam loaded at the end and supported by an elastic medium. The support may be simulated by a system of springs represented by moduli of subgrade reaction in the various soil layers.

The moduli must be assessed on the basis of results of empirical in situ tests.

The maximum value of the horizontal soil pressure in a restricted zone below ground surface may be calculated from three-dimensional earth-pressure theory.

The degree of freedom of rotation of the piles at the connection with the foundations must be taken into account.

7.10.3 Horizontal Displacement

The calculation must take account of the following:

- the stiffness of the soil,
- the bending stiffness of the pile itself,
- the degree of freedom of rotation of the pile at the connection with the foundation,
- the effects of load reversal or of cyclic loading.

guide: For short piles the bending stiffness of the pile can be omitted from the calculations.

7.11 Pile Installation

7.11.1 Pile Installation Procedures

The piling contractor must provide a statement of his capabilities, including his previous experience of forming the type of pile being considered in ground conditions which are similar to those at the site.

He must also provide a method statement in which all essential steps of the pile installation procedure are clearly described. The method statement must be approved by the designer of the piled foundation and must include the following:

- the type and power of the unit to be used to form the pile,
- details of the guiding structure,
- full details of the piling equipment.

A plan giving the location of each pile must be available on site. The plan must have been approved by the designer of the piled foundation and must include the following information:

- pile diameter,
- pile length,
- required load carrying capacity,
- pile toe level (with respect to a fixed level within or near the building site),
- installation sequence,
- obstructions,
- any other constraints on piling activities.

guide: For cast in place piles, the contractors experience of the piling system to be used, in ground conditions similar to those at the site, is of utmost importance. For non displacement piles, special attention must be paid to the installation procedure. Systems used for the removal of the soil can lead to extensive disturbance of the soil in the vicinity of the piles if not properly used. Continuous Flight Auger piles are very sensitive in this respect. The torque and the penetration must be compatible, in order to limit the amount of soil removed as the auger is screwed into the ground. The scraping factor, which is the reciprocal of the number of rotations needed to obtain a penetration of 1 x the pitch of the auger, must not be too high. The power of the drilling motor is a decisive factor in this respect.

The concrete or grout must be pumped through the stem of the auger and the rate of auger withdrawal must be so controlled that a continuous monolithic shaft of the full designed cross-section is formed.

For all types of bored piles the pressures of the fluid inside the bore must be kept at or above the pore pressure in the surrounding soil during boring.

7.11.2 Inspection of Construction

The pile construction operations must be inspected at least daily. For each pile a record must be kept. This record must be signed by the inspector if found to be correct.

The record for each pile must include the following, where appropriate:

- pile number,
- pile diameter and length,
- rake,
- concrete mix, volume and method of placing (for cast in place piles),
- specific weight of bentonits slurry (where used),
- pumping pressures of the grout or concrete (for continuous flight auger piles or other injection) piles,
- values of driving resistance measurements such as weight and drop of hammer, and number of blows for the last 0.25 m penetration (for driven piles),
- the power take-off of vibrators (where used),
- the torque applied to the drilling motor (where used),
- obstructions encountered during piling,
- interruptions to the construction process.

Records must be kept for at least a period of five years efter completion of the works, as they are the only source of reliable information in case of difficulties.

guide: These requirements for construction inspection apply for all three Geotechnical Categories.

7.11.3 Quality Control

If the inspection reveals uncertainties with respect to the quality of one or more installed piles, additional investigations must be carried out to establish the actual load-carrying capacity and deformation behaviour of these piles. These investigations must include either redriving, or pile-integrity tests in combination with soil mechanics field tests adjoining the suspected piles, and static pile loading tests.

If these investigations confirm the doubts, the safety of the pile foundation must be re-assessed on the basis of the principles of this code. Where these principles are not fulfilled, additional piles must be installed so that all requirements with respect to boths ultimate limit states and serviceability limit states are met. The implication for the superstructure must also be analysed.

guide: For cast in situ piles it is difficult to control pile quality in an reliable way during construction. Pile integrity tests can be useful. For structures in Geotechnical Categories 2 and 3 static or dynamic load tests on randomly selected piles are strongly recommended in addition to integrity tests.

7.11.4 Static Load Tests

The purpose of testinq piles is to determine the response of the pile and of the surrounding soil to load. It is necessary to know the pile characteristics and the mechanical properties of the soils and rocks in which the pile is installed.

The location of pile to be tested must be selected as described in Sections 7.8.2.1, 7.9.3.1, and 7.10.2.1.

Ground conditions of the test site must be investigated thoroughly an in detail. The depths of borings or soundings must be sufficient to asertain the nature of the ground both around and beneath the pile tip, including all strata likely to contribute significantly to settlement. Investigations must reach depths of at least 5 m beneath the pile tip, unless sound rock is found at a lesser depth.

The number of test piles must be selected in the design, taking into account the following:

- the soil conditions and their variability across the site,
- the geotechnical category of the structure,
- the methods used in design,
- previous documented evidence of the performance of the same type of pile in similar ground condition.

The minimum number of test piles must be given in the Design Report. The engineer responsible for construction may decide to increase the number of tests, for control purposes.

Where load tests are required, at least 2 load tests are normally specified for each geotechnically, comparable situation. Should the results of these tests lead to unclear or doubtful interpretations, then further loading tests must be carried out.

For larger works, when relevant previous experience is lacking, at least two load tests per 100 piles must be carried out up to a load not less than 1.5 times the working load.

When cavities are present in the subsoil, at least one test for each major grouping of piles must be carried out, and all piles which will act as single supports to a structure must be tested. This also applies to piles which, by failing would detrimentally effect the safety of the structure or seriously affect its serviceability.

Unless it is necessary to modify a test pile in order to install instruments, they must be of the same dimension, materials and reinforcement as the working piles, and must be installed by the same method.

Between the installation and the beginning of the test, adequate time must be allowed to ensure that the required quality of the pile material is achieved and that a state of equilibrium in the surrounding soil (with regard for instance to excess pore pressure) is established.

The method of installing of the test pile should be fully documented as described in Section 7.11.3

The designer must decide whether or not the test results meet the design requirements.

guide: In some instance it may be necessary to record excess pore pressure built up by pile driving and its subsequent dissipation For axially loaded pile a load test will normally establish the settlement of the pile head as a function of applied load and the ultimate bearing capacity of the pile (when the test is carried out to failure). The test, indirectly confirms the integrity and soundness of the pile.

guide: Special tests on instrumented piles are necessary in some cases.

The data determined in a loading test relates to an individual pile. The settlement and bearing capacity of a group of similar piles in the same ground do not necessarily have a direct relation to the settlement and bearing of an individual pile.

The test report for static loading test must include:

- a description of the site,
- the ground conditions,
- the pile type,
- a description of the loading and measuring apparatus,
- calibration certificates or the jacks and gauges,
- the installation record of the test piles,
- photographic records of the pile and the test sits,
- test results in numerical form,
- time settlement plots for each applied load when a step loading procedure is used,
- the measured load settlement function,
- a justification of the reasons for any departures from the above recommendations.

Chapter 8 VERIFICATION PROCEDURES FOR RETAINING STRUCTURES

RETAINING STRUCTURES

8.1 Scope

The provisions of this chapter apply to structures which retain soil or similar material. Material is retained if it stands at a slope steeper than the one it would eventually adopt if no structure were present. Retaining structures include all types of walls, and support systems in which structural elements are combined with soil.

8.2 Limit states

In order to satisfy the performance criteria for retaining structures of stability, limited deformation, durability and limitation of damage to other structural elements or to nearby structures or services the following limit states must be prevented:

Type 1A Ultimate limit states

These occur when a collapse mechanism forms in the ground due to:

1) slope stability failure,
2) bearing capacity failure,
3) base sliding,
4) structural failure,
5) subsurface erosion,
6) lack of passive resistance,
7) pull out failure of anchors,
8) combinations of these.

guide: Examples of structural failures include failure of an anchor in tension, and crushing of concrete in bending.

: For walls with inclined anchors, the effect of these on vertical equilibrium must be considered.

: For walls founded on rock or on soil of high strength, toppling failure must be considered.

Type 1B Ultimate limit state

These occur when movements of the retaining structure lead to severe structural damage in other parts of the structure or in nearby structures or services.

Type 2 Serviceability limit states

These occur:

1) when movements of the retaining structure affect the appearance or efficient use of:
   - the structure
   - nearby structures which rely on it
   - nearby services which rely on it,
2) in structures subject to an hydraulic gradient when unacceptable leakage or unacceptable interruption of flow occurs,
3) when the retained structure suffers excessive vibrations, caused, for example, by resonance under dynamic loading.

8.3 Actions and design situations

8.3.1 Actions

In selecting the actions for any calculation, the designer must consider the forces and displacements listed in Section 3.1.2. The principles and guidance contained in Sections 8.3.3 to 8.3.7 must also be taken into account.

Design values for the actions must be derived in accordance with the principles stated in Section 3.2.

guide: Earth pressures must be treated as actions in certain design situations, described in section 3.1.2. The way in which earth pressures are obtained is described in section 8.5. They may be due to:

- self weight of the ground,
- actions on the ground surface,
- compaction of the soil,
- seismic activity.

8.3.2 Design situations

Design situations must be chosen in accordance with the principles of Chapter 2. For retaining structures, the following situations are of particular importance:

- excavation in front of the retaining structure,
- backfilling behind the retaining structure,
- variations in soil properties in time or space,
- variations in pore water pressure,
- variations in loads and in the way they are combined,
- variations in water levels,
- the effects of new structures and of their construction, providing that the new structures are foreseen when the design is made
- mining subsidence.

8.3.3 Anchor Forces

For prestressed anchorages the anchor forces must be treated as independent actions.

For unstressed anchorages (deadmen, anchor piles, etc.) the anchor forces depend on the statical behaviour of the retaining structure and are not independent actions.

guide: Inclined anchors impose additional vertical loads upon the retaining structure.

8.3.4 Groundwater Pressures

In selecting design values for groundwater pressures, long term observations of groundwater levels in the vicinity of the structure must be considered, unless a reliable drainage system (Section 8.4.3) is installed.

Where the design assumes that drains are installed which permanently affect groundwater pressures, provision must be made for their maintenance and effective functioning throughout the life of the structure.

8.3.5 Ice and Wave Forces

For waterfront structures, ice forces and wave forces are alternatives.

guide: Ice forces occur in spring when temperatures increase, due to expansion of an ice sheet as temperature rises. The forces depend on:

- the initial temperature before warming begins,
- the rate at which temperature increases,
- the thickness of the ice.

When an ice floe collides with a structure, the impact load depends on the thickness and velocity of the floe and on the compressive strength of the ice. The strength of ice depends on its salinity and homogenity.

Design values for wave forces depend on the climatic and hydraulic conditions at the site of the structure.

8.3.6 Traffic Loads

Design values for traffic loads must be selected as described in chapter 3.

guide: When designing a retaining structure it is normally sufficient to represent dynamic actions by static actions of equal magnitude. Where crane rails are supported on a retaining wall, however, it is necessary to increase the magnitudes of the static actions.

Impact loads are normally evaluated by considering the energy absorbed by the structure. For lateral impacts on retaining walls it is necessary to consider the increased stiffness exhibited by the retained soil when resisting an impact on the face of the wall.

8.3.7 Temperature Effects

Temperature differences must be considered for walls which remain exposed to the atmosphere during their lifetime.

guide: Experience with lock walls indicates that the displacement of the walls of U-shape retaining structures depends on the seasonal average temperature rather than on extremes. These average values must be taken from observations made over a period of at least 10 years. Thermal expansion or contraction may cause significant changes in strut loads in braced excavations.

Artificial climates (e.g. at boiler houses or cold stores) may affect the loads to be carried by the retaining wall.

8.4 Design and Construction Considerations

8.4.1 General remarks on design principles

Alternative types of retaining structure are illustrated in Figure 8.4A. In selecting a retaining structure the following points must be considered:

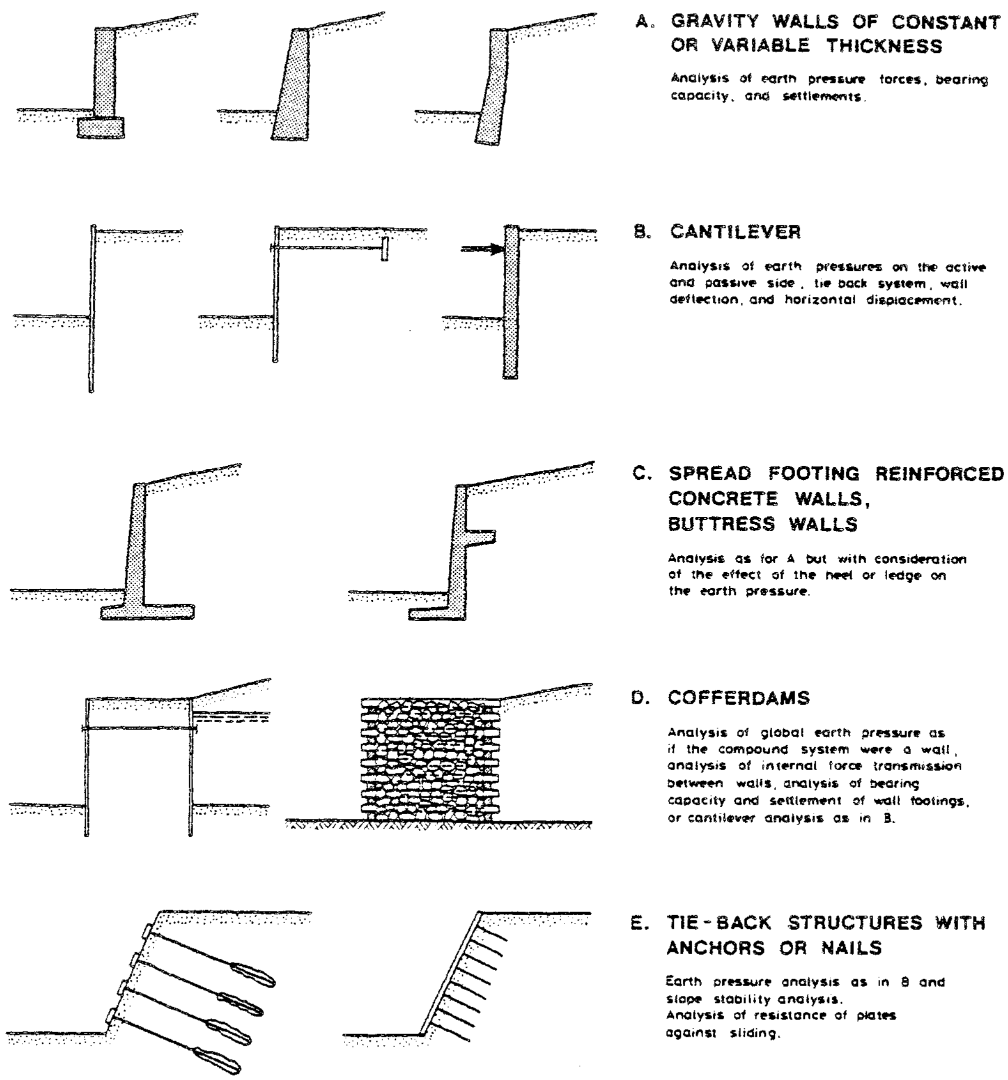


Figure 8.4 a. Alternative Types of retaining Structures

- topography. (Existing slope, cutting or embankment),
- groundwater conditions,
- existing drainage systems,
- shear strength of the soil on each side of the wall. (Does strength vary with time, or with movements of the structure ?)
- live loads on the retained ground,
- availability of backfill materials,
- suitability of the existing ground for ground anchors.

8.4.2 Ground anchors

Ground anchors may be temporary or permanent elements of a retaining structure. Anchor design must take into account all circumstances during the foreseeable design life of the anchor. The corrosion and creep of permanent anchors must be given special consideration.

guide: Structures in which permanent anchors are used are normally classified as Geotechnical Category 3.

The load carrying capacity of a prestressed anchor is normally evaluated from preliminary tests and from local experience before construction begins. Load tests may be carried out in situ on prestressed anchors as follows:

- suitability tests, which indicate the results that should be obtained from the working anchors. The anchors tested in this way must be identical to working anchors. The number of tests must be stated in the design report,

- routine acceptance tests, which check that the anchors behave at design load as the design report intended. Every working anchor must be tested in this way.

8.4.3 Backfill and drainage

The design of walls retaining soil of medium or low permeability (i.e. silts and clays) must assume that full hydrostatic pressure acts behind the wall unless a reliable drainage system is installed.

The drainage system must discharge either through weep holes or through porous land drains and pipes. Piped drainage must be located at the bottom of the wall and must outfall to sumps or sewers. Manholes must be provided from which the piped drainage can be cleaned.

guide: The quality of backfill is an important factor for the behaviour of retaining structures. Suitable procedures for compacting the backfill should be prescribed in the design.

8.5 Earth Pressure

8.5.1 General

Earth pressures which are independent of the response of the system adopted for the calculation model are actions. Earth pressures which depend on that response are not actions (see section 3.1.2).

The design value of an earth pressure at an ultimate limit state is generally different from its value at a serviceability limit state. They are to be calculated from the design values of soil parameters which are appropriate to the limit state being considered.

Calculations of the magnitudes and directions of earth pressures must take account of:

- density of the soil,
- shear strength of the soil,
- friction between wall and soil,
- slope of the ground surface on either side of the wall,
- the relative movement of wall and soil which may take place.

8.5.2 Wall friction

The mobilized wall friction angle, (δ), is the angle between the resultant force on the wall and the normal to the loaded wall.

guide: The angle of wall friction, (δ), is assessed from the following:

guide:
- roughness of the wall,
- inclination of the wall and of the ground surface behind the wall,
- the type of soil retained,
- the density (or consistency) of the soil retained,
- the amount and direction of the movement of the wall relative to the soil.

8.5.3 Earth Pressure at Rest

For a horizontal ground surface the earth pressure at rest is the horizontal stress which exists in the ground before it is displaced or disturbed. Its magnitude depends on the effective angle of internal friction ϕ' and the stress history of the soil. The cohesion of the soil must not be considered when calculating earth pressure at rest.

guide: Where a rigid wall is prevented from moving, the earth pressure on it may be assumed to equal earth pressure at rest.

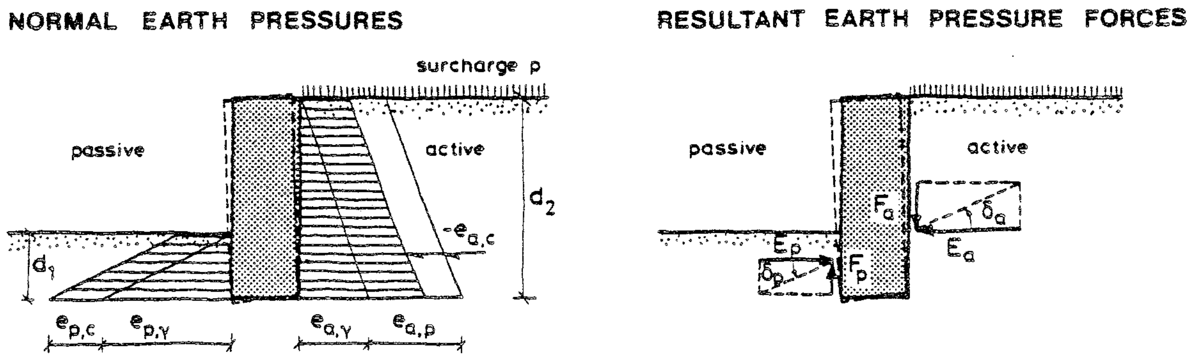
8.5.4 Limit Values of Earth Pressure

Limit (active or passive) values of earth pressure are produced when the strength of the soil is fully mobilised. They are calculated by considering the appropriate state of plastic equilibrium of the soil. The calculation must take account of the amount and direction of the movement of the wall relative to the soil.

guide: For a cantilever wall rotating at its base, Figure 8.5A illustrates the effect of wall movement on the state of plastic equilibrium and the direction of movement of the wall relative to the soil.

The movements most commonly required to mobilise active and passive states of plastic equilibrium in medium dense and dense granular soils and in stiff cohesive soils ($0,75 < I_c < 1,00$) are given in table 8.1.

guide:



$e_{p,c} = c \cdot K_{p,c}$ passive pressure, cohesion

$e_{p,\gamma} = \gamma d_1 \cdot K_{p,\gamma}$ passive pressure, self weight

$e_{a,\gamma} = \gamma d_2 \cdot K_{a,\gamma}$ active pressure, self weight

$e_{a,p} = p \cdot K_{a,p}$ active pressure, surcharge

$e_{a,c} = c \cdot K_{a,c}$ active pressure, cohesion

δ_a : wall friction angle, active

δ_p : wall friction angle, passive

E : normal resultant earth pressure force

F : tangential result. earth pressure force

Figure 8.5 a. Effect of Wall Movement on Plastic Equilibrium

MOVEMENT TO MOBILISE ACTIVE PRESSURE		MOVEMENT TO MOBILISE PASSIVE PRESSURE	
displacement	rotation (θ)	displacement	rotation (θ)
$0.001d_2$	Arctan 0.002 (at bottom of wall)	$0.05d_1$	Arctan 0.100 (at bottom of wall)
			Arctan 0.020 (at top of wall)

Notes: (1) Displacements are considered to take place without rotation.
(2) d_1 and d_2 are shown on figure 8.5 A
(3) Rotation is considered to take place about a fixed point at either the top or the bottom of the wall

Table 8.1. Movements Necessary to Mobilise Active and Passive Pressure

For very dense granular soils and for very stiff cohesive soils ($I_c > 1{,}00$) smaller movements than those given in table 8.1 are required. For loose granular soils and for soft cohesive soils larger movements than those given in table 8.1 are required.

In every case the movements required to mobilise passive pressure are much larger than those required to mobilise active pressure.

8.5.5 Mobilized Values of Earth Pressure

When the wall movements required to mobilise the limit values of earth pressure will not occur, or are prevented from occuring, then intermediate values must be used in design. Their magnitude depends upon the amount of wall movement and its direction relative to the soil.

8.5.6 Compaction Effects

If the wall is backfilled in layers and the fill is compacted, an additional earth pressure is incurred and must be taken into account.

guide: Measurements indicate that the additional earth pressure due to
: compaction is reduced when the next layer is placed and compacted.
: When backfilling is complete, the excess pressure acts only on
: the upper part of the wall, as illustrated in figure 8.5 B.

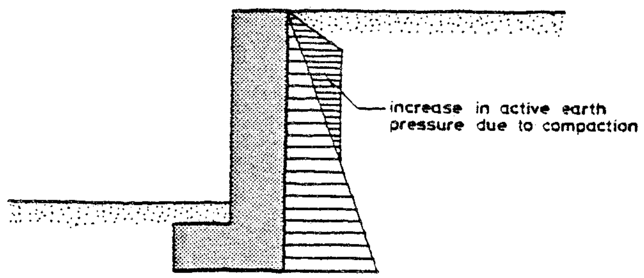


Figure 8.5 b. Compaction Effects on Active Earth Pressure

8.5.7 Earthquake effects on earth pressure

In seismic areas the influence of earthquakes on the behaviour of retaining structures must be taken into account in two ways:

1) reduced shear resistance of backfill and subsoil,
2) additional inertia forces that increase the earth pressure on the retaining structure.

guide: The magnitude of the shear resistance reduction may be estimated by laboratory tests or by use of available parameter studies.

The magnitude of the supplementary inertia forces may be assessed by using pseudo static analysis.

The characteristic values of the horizontal earthquake acceleration that must be considered are given in EC 8.

8.6 Ultimate limit state design

8.6.1 Limit states

The ultimate limit states given in section 8.2 must be considered in the design of retaining structures.

The type of limit state which governs the design depends upon:

- type of retaining structure,
- geometry of the soil and the structure,
- strength of the soil,
- groundwater.

guide: The limit states which most commonly govern different types of retaining structures are given in table 8.2.

ALL RETAINING STRUCTURES	GRAVITY RETAINING STRUCTURES	CANTILEVER WALLS
slope stability failure	base sliding	structural failure
subsurface erosion by piping	bearing capacity failure	lack of passive resistance

Table 8.2. Limit States for Retaining Structures

guide: For gravity retaining structures, it is normally permissible for a gap to form beneath the foundation. The gap may extend from one edge as far as the centroid of the foundation in plan. If no gap is permitted, the resultant force of the permanent actions should pass within the middle third of the foundation.

The design must take account of the possibility of subsurface erosion by piping.

guide: Piping is of particular importance in the design of waterfront retaining structures and excavations below groundwater level.

For structures which retain cohesive soil or which are founded on it, two analyses must be carried out:

- a short term analysis for undrained conditions,
- a long term analysis for the final drainage conditions.

8.6.2 <u>Overall Stability</u>

The procedures of Chapter 9 must be used to demonstrate that a slope stability failure of the soil mass containing the retaining structure is sufficiently improbable.

For anchored structures the overall stability of a soil mass containing both the wall and the anchor must be analysed. This often has a lower factor of safety than other soil masses.

guide: Figur 8.6 A illustrates examples of calculation models for loss of overall stability for retaining structures.

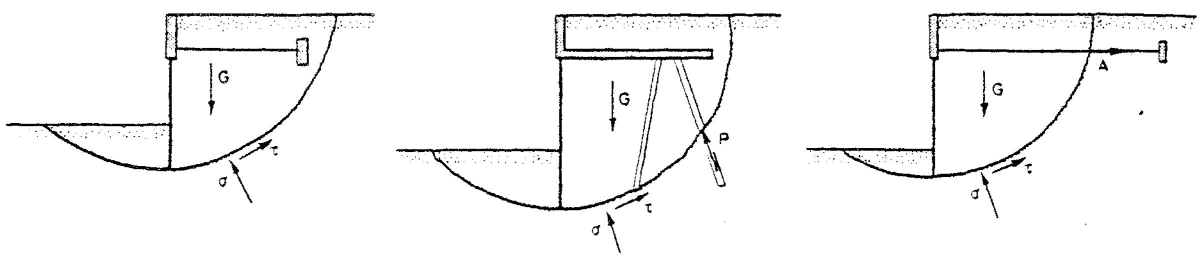


Figure 8.6 a. Examples of Calculation Models for Loss of Overall Stability for Retaining Structures

8.6.3 Foundation Failures

The procedures of Chapter 6 must be used to demonstrate that a foundation failure is sufficiently improbable.

Section 6.5.2 gives procedures for bearing capacity failure.

Section 6.5.3 gives procedures for base sliding.

guide: Figur 8.6 B illustrate an example of a calculation model for loss of bearing capacity of subsoil for a retaining structure.

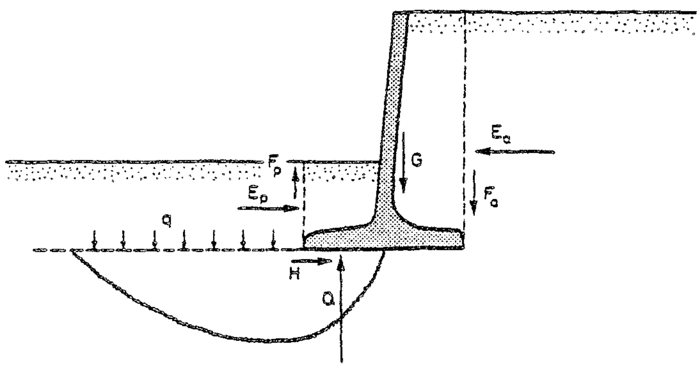


Figure 8.6 b. Example of a Calculation Model for Loss of Bearing Capacity for Retaining Structure

8.6.4 Subsurface Erosion

The design must show that failure by subsurface erosion (piping) will not occur.

guide: Piping may occur as shown in Figure 8.6 C. Water flows through a granular soil from one side of a retaining structure to the other. If the exit hydraulic gradient is too high, soil is eroded at the downstream surface of the soil and a channel is formed (piping). The channel causes a local increase in hydraulic gradient and more soil is removed. Eventually, a large volume of soil is removed, and foundation failure by subsurface erosion results.

Zones of disturbed or more permeable soil near the downstream surface can result in local erosion, from which a piping failure may develop.

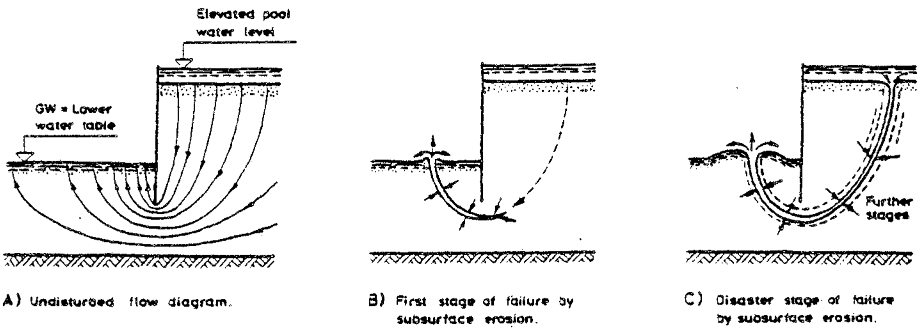


Figure 8.6 c. The Occurance of Piping

To eliminate the possibility that piping will occur that design must show that the hydraulic gradient at exit will not exceed generally accepted limit values.

guide: The measures most commonly used to ensure that piping does not occur are:

- seepage control,
- reduction of hydraulic gradient,
- protective filters.

8.6.5 Structural Failure

The design must show that no section of the structure will fail. The design of structural elements must comply with the provisions of the appropriate Eurocodes.

8.6.6 Failure due to Inadequate Passive Resistance

The design must show that the resistance of the soil in front of the wall is sufficient to prevent forward movements of the wall.

Where water flows beneath the wall, see figure 8.6 D, the effects of uplift and seepage forces on active passive earth pressures must be considered.

guide: Seepage effects may be considered by constructing a flow net, or in certain circumstances, by using accepted simplified methods.

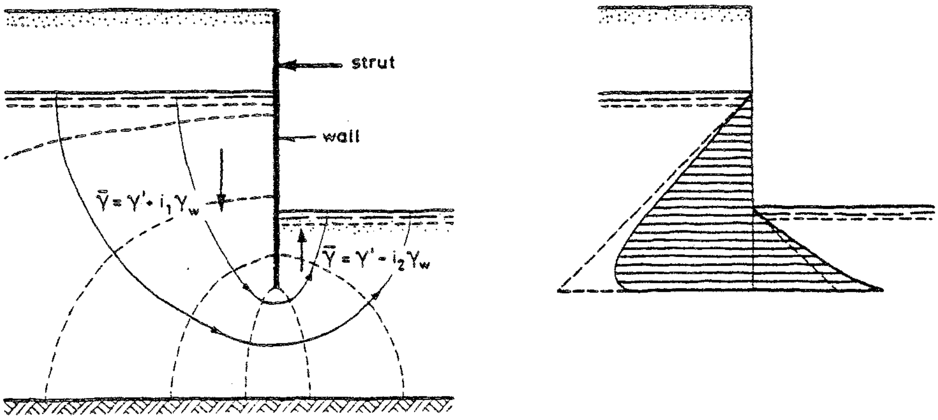


Figure 8.6 d. Effects of Flow on Wall Design: Increase, in Active Pressure, Decrease in Passive Pressure, and a Differential Water Pressure

8.7 Serviceability limit state

8.7.1 Displacement Analyses

The design of the retaining structure and support system must take into account the ability of the structure itself and of the nearby structures to accomodate displacement.

guide: Displacements may occur as:

- settlement,
- horizontal displacement,
- tilting.

These may take place simultaneously. The type and amount of displacement depends upon the foundations provided for the retaining structure and on the ground conditions.

For cantilever walls and cofferdams, displacements are predominantly horizontal. They may be evaluated iteratively by considering in turn the earth pressure, the displacement of the structure and the behaviour of the anchorage.

8.7.2 Vibration Analyses

The provisions of section 6.6.6 also apply to retaining structures.

8.8 Durability

8.8.1 Concrete Durability

The provisions of section 6.7.4 also apply to concrete retaining structures.

8.8.2 Corrosion

The thickness of the members of the retaining structure and the quality of the materials used must be sufficient for the intended life of the structure:

guide: Particular care is required to ensure that:

1) steel sheet piles are of sufficient thickness and are made of steel of adequate quality,
2) tension members such as anchors have either an adequate corrosion allowance or a protective coating. For permanent anchors, elaborate corrosion protection systems are often required.

Corrosion protection is usually needed for reinforcing steel in elements in bending, such as reinforced concrete sheet piles.

CHAPTER 9 - EMBANKMENTS AND SLOPES

9 EMBANKMENTS AND SLOPES

9.1 Scope

The provisions of this chapter principally apply to embankments and artificially slopes which are not reinforced or supported. They may also apply to embankments and slopes which are reinforced by vegetation, or artificially by means of piles, dowels, soil stabilisation, lime columns and the like.

The provisions of this chapter may apply to unstable natural slopes (landslides), if they are directly influenced, or could be influenced, by human activity.

Slopes which are supported by retaining structures are dealt with in chapter 8.

The behaviour of slopes and especially of slopes in natural materials depends significantly on the geological, morphological and ground water conditions of the site.

9.2 Limit States

In order to fulfill the fundamental requirements for embankments and slopes of stability, limited deformation, durability and limitation of damage to nearby structures or services the following limit states must be prevented:

Type 1A Ultimate Limit States

- Slope stability failure
- Bearing capacity failure of an embankment
- Seepage erosion or piping in a slope in soil
- Toppling failure in hard rocks.

Type 1B Ultimate Limit States

- Deformations of the embankment or slope which cause severe structural damage in structures, roads or services sited on or near the embankment or slope.

Type 2 Serviceability Limit States

- Deformations of the embankment or slope which cause loss of serviceability of structures, roads or services sited on or near the embankment or slope.

Where, for a slope, type 1B or type 2 limit states are possible, prefailure deformations of the slope must be considered. Measurements of slope movement and their evaluation and interpretation are an important part of this consideration.

9.3 Actions and Design Situations

9.3.1 Actions

In selecting the actions for any calculation, the forces and displacements listed in Section 3.1.2 must be considered.

Design values for the actions must be derived in accordance with the principles stated in Section 3.2.

9.3.2 Design Situations

It is necessary to derive appropriate design situations which cover the conditions which can be foreseen during the construction and the intended life of an embankment or slope, see Section 2.1. Each design situation is normally considered separately.

The following factors are to be considered when deriving design situations for slopes:

- construction processes, such as excavation in front of the slope or the placing of an embankment in layers,
- soil and rock properties and their variations in space or time,
- pore pressures and their variations,
- variations in loads and in the way they are combined,
- water levels and their variations,
- water pressures, and changes in pressure caused by the failure of drains, filters or seals, or by flooding,
- the effect of new structures, which may be placed on or near the embankment or slope after its completion,
- the effect of the new slope on existing work,
- earthquakes,
- meteorological factors such as rain or storms.

9.3.3 Dead and Live Loads

In considering the stability of a body of soil, its dead weight is to be determined from values of unit weight which take into account the position of the ground water level.

Live loads on the ground surface should only be considered if they act unfavourably on the stability of the embankment or slope.

9.3.4 Hydraulic Forces

The design free water level in front of the slope and the design groundwater level, or their combination, should be chosen from the available hydrological data to give the most adverse conditions that could occur in the design situation being considered.

guide: For water retaining embankments such as dams or dykes the most adverse hydraulic conditions are normally:

- For downstream slopes, steady seepage for the highest possible upstream water level
- For upstream slopes, rapid drawdown of the retained water level.

For steady seepage, the phreatic surface in soil and in isotropic or lightly anisotropic rock may normally be represented by a two dimensional parabolic surface.

In layered soil and in highly anisotropic rock the phreatic surface is not parabolic; its shape depends on the ratio of the horizontal and vertical permeabilities.

Where seepage is not steady, for example when rapid drawdown occurs, the change in the phreatic surface is related approximately to the ratio of the drawdown velocity to the coefficient of permeability.

The water pressures (u) should be treated as pressures acting on the sliding surface.

guide: The pore water pressures are normally obtained from flow nets. For gentle slopes it is permissible, and conservative, to approximate the water pressure (u) on the slip surface as:

$$u = h_s \gamma_w, \qquad (9.1)$$

where

h_s is the vertical distance between slip surface and phreatic surface

γ_w is the unit weight of water

9.3.5 Earthquake Effects

The design of slopes in seismic areas must take account of the following earthquake actions:

- in saturated soil there may be an increase in pore pressure due to cyclic shearing. This leads to a reduction of shear resistance and, in extreme cases, to liquifaction,
- supplementary inertia forces act on the sliding soil mass.

guide: The reduction in shear resistance for a given soil can be estimated by model laboratory tests or from the relative density of the soil, based on experience.

Approximate values of the supplementary inertia forces caused by horizontal accelerations may be obtained by the "pseudostatic method". In this method an additional horizontal force is considered to set through the centre of gravity of the soil mass with a magnitude of:

$$\frac{a_h}{g} W \qquad (9.2)$$

where
a_h is horizontal acceleration
g is vertical acceleration due to gravity
W is weight of the sliding mass.

9.4 Design and Construction Considerations

The design of slopes in soil or rock must take account of:

- the geomorphological and geological conditions of the site and of the surrounding area including relevant local variations in bedding, folding and jointing (stratigraphy and tectonics),

- the hydrology of the area, ground water levels measured over long periods of time, changes in water level at the bottom of a slope and changes in ground water level at the top af a slope,
- climatic conditions such as rainfall, sunshine and temperature.

Embankments and slopes must be designed and constructed in accordance with local experience.

guide: The behaviour of embankment slopes depends on the quality of fill, for example, the use of the material and methods specified in the design. Their construction should be carefully controlled in accordance with the principles given in Chapter 5 and 10.

Slopes should be sealed or planted, or protected artificially, in order to prevent surface erosion. For slopes with berms, a drainage system within the berm may be needed in order to prevent surface erosion.

9.5 Ultimate Limit State 1A Design

9.5.1 Failure due to Loss of Stability

9.5.1.1 Principles. In analysing the stability of a slope it is necessary to consider all the types of failure surface which could possible develop.

guide: Various types of failure surface are illustrated in figure 9.5 a. The mass of soil or rock bounded by the failure surface is normally treated as a rigid body, or as several rigid bodies moving simultaneously.

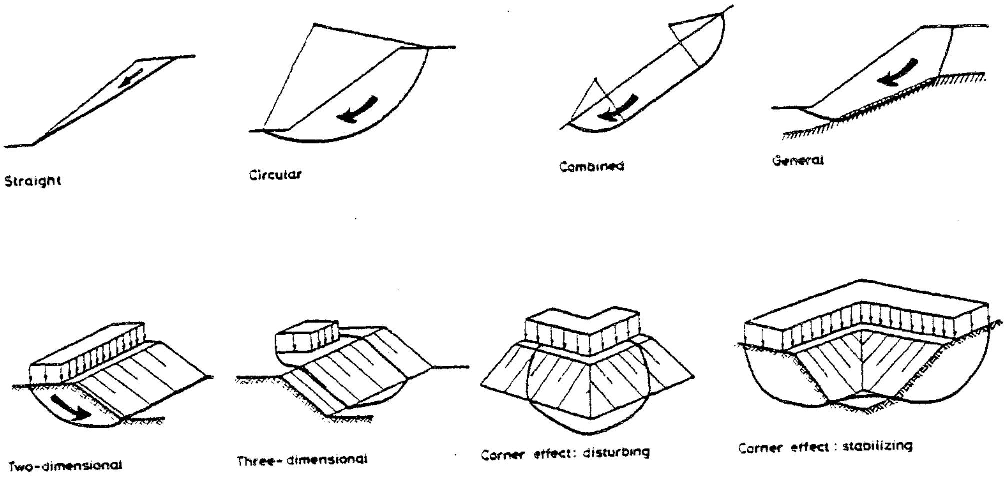


Figure 9.5 a. Types of Failure Surfaces Related to Loss of Stability of Slopes

guide: Because soils deform, actual failure surfaces usually deviate from the surfaces assumed for analysis. For slopes in jointed rock the material above the failure surface is treated as a number of rigid bodies. The effect of the internal shear forces between these bodies should be considered. This procedure is also followed for slips in soil where a combined slip surface has been located by observation or measurement.

In the analysis, the equilibrium of the body or bodies bounded by the failure surface is to be considered. The actions and the shear strength parameters of the soil are assigned with their design values. The most adverse slip surface is to be found by trial.

For slopes in Geotechnical category 1 and for some slopes in Geotechnical category 2, a numerical analysis on overall stability is not generally necessary. In these cases stability is ensured by prescriptive measures.

9.5.1.2 Selection of Shear Strength Parameters. When selecting the shear strength parameters for calculating the stability of a slope in soil or in soft rock, the influence of preconsolidation, weathering, fissuring and similar effects of geological history of the site, of actual and future time effects (such as decrease of cohesion and creep), of strain or deformation effects (such as reduction of angle of internal friction for large post-peak strains) for each ground layer and of transient, repeated or vibratory loading should be taken into account.

In jointed hard rock the potential failure surface may consist of a single plane or a complex path, mostly following discontinuities (joints) in the rock mass, as illustrated in figure 9.5 b. Therefore, the shear resistance must be estimated taking into account the orientation, roughness and filling of the discontinuities (joints) and not by using the shear strength of the intact rock mass. Dilation due to joint roughness and creep should also be taken into account.

Intact material failure. Stepped failure surface. Failure on discontinuity surface. Heavily jointed rock mass or rock fill.

Figure 9.5 b. Potentially Failure Surfaces for Slopes in Jointed Hard Rock

9.5.1.3 Calculation Methods for Slopes in Soil and in Soft Rock.

guide: For slopes in soils and soft rocks, which do not exhibit marked strength anisotropy, the simplified method of slices is recommended. The basic equation of this method is:

guide:

$$r \sum T_i = r \sum W_i \sin \nu_i + \sum M \qquad (9.3)$$

where

W_i is the design dead load of a single slice including surcharge,

M is the design moment about the centre of rotation of any load or force not included in W_i (disturbing moments are positive),

T_i is the design resisting shear force of a single slice, tangential to the slip surface (including lateral pile resistance),

ν_i is the angle between the horizontal and the tangent of the slip surface at the middle line of a slice,

r is radius of the slip circle.

For slopes in pronouncedly layered soils with considerable variations of shear strength, the most unsafe potential slip surface is normally non-circular and passes through the layer with the smallest shear strength.

9.5.1.4 Calculation Methods for Slopes in Jointed Hard Rock. In jointed hard rock, the shape of the slip surface depends on the discontinuities. Three types of slip surfaces, illustrated in Figure 9.5 c, are:

- plane surface,
- wedge surface,
- polygonal or circular surface.

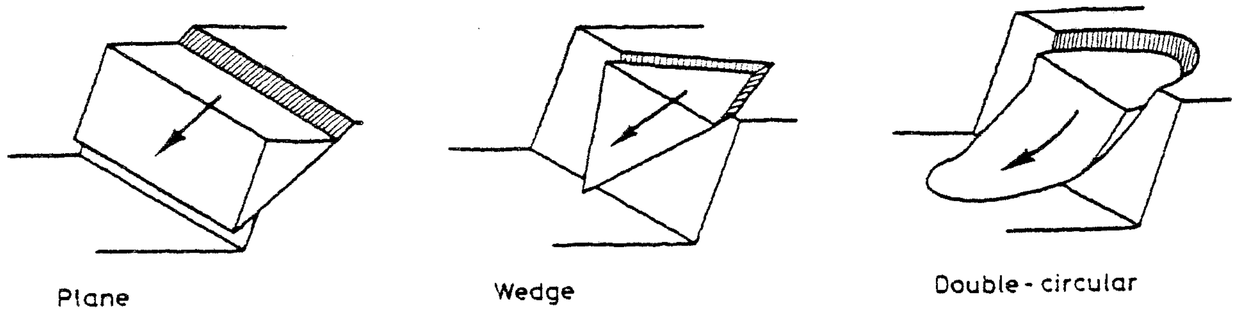


Figure 9.5 c. Three Types of Slip Surfaces for Slopes in Jointed Hard Rock

Analyses are to be carried out for two or all of these, if the information obtained about the discontinuities is not sufficient to indentify the most adverse.

9.5.2 Failure due to Loss of Bearing Capacity

For embankments constructed on soft soil the design must ensure that the soft soil will support the weight of the embankment with adequate safety against bearing capacity failure.

guide: The bearing capacity analysis may be performed by use of the
: principles of Section 6.5.2.

9.5.3 Failure due to Internal Erosion

If steady or temporary seepage of water is possible through a slope in erodable soil, the design must show that the slope will not fail or be endangered by internal erosion (piping).

To eliminate the possibility of piping, the design must show that the hydraulic gradient at exit will not exceed limit values which by experience have been proven sufficiently safe.

guide: The measures most commonly used to ensure that piping does not
: occur are:
:
: - seepage control,
: - reduction of hydraulic gradient,
: - protective filters.

In addition, observations of the phreatic surface and of the rate of seepage should be made to check that the slope is performing as intended.

9.5.4 Failure due to Toppling

In stiff jointed rock the design must show that slope failure by toppling will not occur.

guide: The conditions for simple toppling or toppling combined with sliding
: for a single block not subjected to water pressue are shown on figure
: 9.5 d.
: No simple method of designing a multiple block system against
: toppling exists. In such cases special consideration and collabo-
: ration of an experienced specialist in rock mechanics are required.

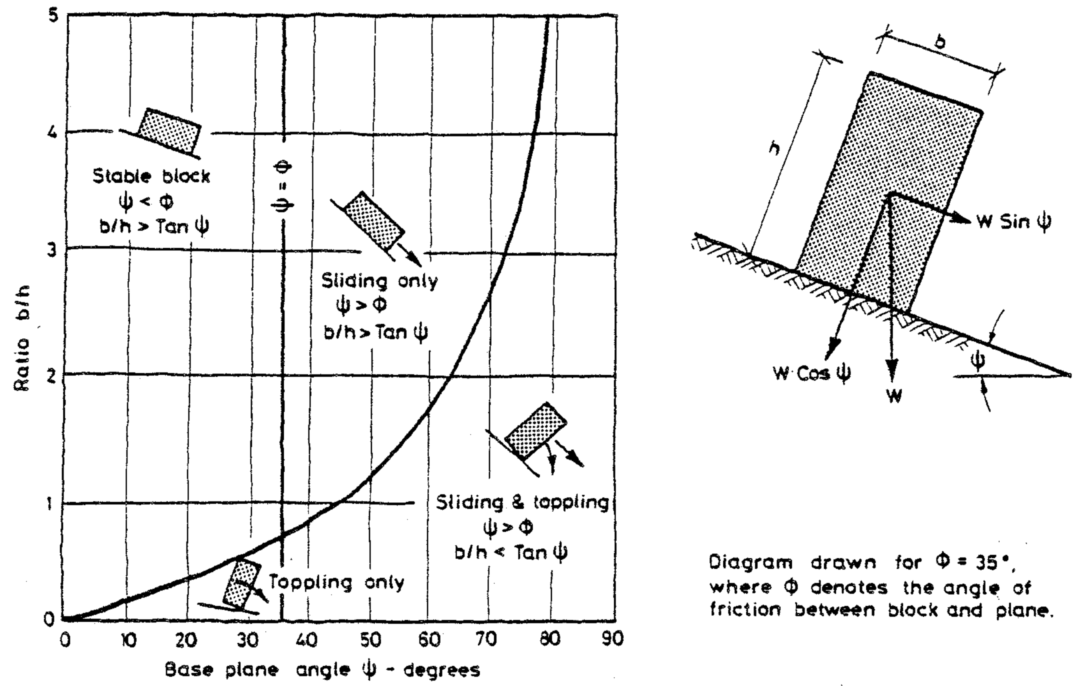


Figure 9.5 d. Conditions for Simple Toppling and/or Slidning for a Single Block

9.6 Ultimate Limit State 1B and Serviceability Limit Stat

The design must show that the expected deformation of the embankment or slope under the design actions will not cause severe structural damage (Type 1B Limit State), or loss of serviceability (Type 2 Limit State) in structures, roads or services sited on or near the embankment or slope.

guide: The settlement of an embankment on a compressible soil layer may be calculated using the principles of Section 6.6.3. Special consideration should be paid to the settlement-time relationship which includes both consolidation and secondary settlement. Attention should also be paid to the possibility of occurance of differential settlements.

The analytical and numerical methods available at present do not usually provide reliable predictions of pre-failure deformation of a slope. The use of the finite element method for this purpose is limited by the difficulty of evaluating the parameters that govern the stress-strain behaviour of the material from the results of field or laboratory tests. Therefore, the occurence of ultimate limit state 1B and serviceability limit state should be avoided either:

guide: - by limiting the mobilized shear strength (flattening the slope)

or:

- by observing the movements of the slope and taking action to control them if this proves necessary.

For slopes in rock above roads, buildings, trafficed areas, etc it must be ensured that rockfall (abrupt movements of loosened blocks) will not occur or will not involve the risk of life or cause substantial damage).

9.7 Monitoring

9.7.1 Slopes

The behaviour of a slope must be monitored using appropriate equipment if either:

- it is not possible to prove by calculation or by prescriptive measures that all of the limit states given in section 9.2 will not occur or

- the assumptions made in the calculations are not based on adequate reliable data.

Slopes which require monitoring will generally be classified as Geotechnical Category 3. A specialist with appropriate knowledge will normally design the monitoring system and will evaluate and interpret the results obtained. The evaluation and interpretation of the measurement results is an integral and important part of the supervision activity, described in Chapter 10.

guide: Monitoring may be required where:

- construction activity or precipitation may affect a natural slope or cutting,
- ground water levels or pore pressures in an unstable slope are measured so that an effective stress analysis can be carried out, or checked,
- lateral and vertical movements of a moving or sliding soil mass are measured in order to predict further movements,
- the depth and shape of the sliding surface in a developed slide are measured in order to derive the soil strength parameters and to design remedial works,
- rate of slidning are measured in order to give warning of impending danger. In such cases a remote diqital readout for the instruments or a remote alarm system may be appropriate.

9.7.2 Embankments

The construction of embankments on very soft impermeable soil must be monitored and controlled by means of porepressure measurements in the soft layers and settlement measurements of the fill. These measurements must be checked aqainst the results of the stability and settlement calculation made during the design for each phase of the construction of the embankment.

guide: Embankments on very soft soils are normally raised in layers. The thickness of these layers and the speed of construction should be determined durinq desiqn in order to prevent loss of stability of the slopes or bearing capacity of the subsoil during construction. Calculations of the expected consolidation time are unreliable. The rate of consolidation of the soft soil layers should therefore be measured durinq construction by means of porepressure measuring devices and settlement stations. As soon as the excees porepressure have fallen below safe values, which are to be stated in the design report, the next layer of fill may be placed. The results of the settlement measurements are to be used as a check on this procedure.

If vertical drains are installed to accelerate the consolidation, and hence the construction, special care must be taken with respect to the location of the porepressure measuring devices. They should be located in the centre of the qrid of vertical drains.

Chapter 10 SUPERVISION OF CONSTRUCTION

Contents

10 SUPERVISION OF CONSTRUCTION

10.1 Purpose and Role of Supervision

The purpose of supervision is to guarantee the safety and the quality of construction.

Supervision means the quality control, inspection, monitoring and evaluation which are needed to ensure that each limit state is sufficiently improbable.

Supervision includes, but is not limited to:

- checking the validity of design assumptions,
- identifying the differences between the actual ground conditions and those assumed in the design analyses,
- quality control and inspections to check that construction is carried out as specified in the design,
- observations and measurements for monitoring the performance of the structure and its surroundings, during and after construction,
- monitoring the behaviour of the structure during construction, so as to identify the need for remedial measures, alterations to the construction sequence and the like,
- evaluation of the performance of the completed structure.

Supervision should consider the structure and the surrounding ground. Aspects which are emphasised in this chapter are:

- the characteristics and response of the ground,
- structures in contact with the ground,
- design situations which occur during construction.

10.2 Principles of Supervision

- Inspection, control, field and laboratory testing during construction and performance monitoring are necessary parts of the design and must not be separated from it. The designer must be given the opportunity of inspecting the works at each stage of construction.

- The reliability and level of supervision during construction are to be taken into account in the selection of design parameters and factors of safety. Design decisions which are influenced by the reliability of supervision and monitoring are to be clearly identified.

10.2.1 Planning

A plan of supervision must be included in the design report, and must state acceptable limits for the results to be obtained by monitoring.

The plan must specify the type, quality and frequency of supervision, which must be commensurate with.

- the degree of uncertainty in the design assumptions,
- the complexity of the ground conditions,
- the geotechnical category of the structure,
- the feasibility of making design modifications or of implementing corrective measures during construction.

10.2.2 Inspection and Control

Visual inspection is normally the most important element of supervision of construction work. Instruments must be simple and reliable; complicated apparatus must only be used in special cases.

Control tests must be carried out by experienced personnel. All results must be evaluated qualitatively. Quantitative evaluations must be made wherever possible. Results must be made available to the designer before the decisions which are influenced by them are taken.

The installation and operation of instrumentation must be inspected and supervised by engineers who are familiar with the design requirements and with geotechnical problems. Details of the ground conditions must be detected carefully and recorded.

The suitability of the construction procedures and the sequence of operations must be reviewed against the ground conditions which are encountered.

Records must be maintained of the following,

- significant ground features,
- precise sequence of works,
- quality of materials,
- deviations from design,
- as-built drawings,
- results of measurements and of their interpretation,
- observations on the physical environmental conditions etc.

Records of temporary works must also be kept. Interruptions to the works, and their conditions on recommencement, must be recorded.

10.2.3 Assessment of Results

1) the design must be assessed on basis of the results, this assessment must include comparison of the predicted behaviour with the observed performance, if necessary the design must be re-evaluated,
2) the geotechnical category into which the structure has been placed must be re-assessed during construction. The most adverse conditions which occur during construction must be identified with regard to:

    1) ground conditions,
    2) groundwater conditions,
    3) actions on the structure,
    4) environmental impacts and changes including landslides and rockfalls.

guide:

Geotechnical Category 1

The supervision programme may be limited to visual inspection, rough quality controls and a qualitative assessment of the performance of the structure.

Geotechnical Category 2

Quantitative controls are normally required. The controls should be the responsibility of experienced professional engineers control measurements include:

- soil properties,
- pore water pressures,
- settlements,
- horizontal movements.

Geotechnical Category 3

Sets of measurements are normally made during each significant stage of construction, and are compared with the predicted behaviour of the structure. The comparisons are normally quantitative and made by specialists.

More detailed observations are often required, including:

- details of the ground conditions,
- variations in pore pressures,
- displacements.

10.3 Ground Conditions

10.3.1 Soil and Rock

The descriptions and geotechnical properties of the soils and rocks on which the structure is founded must be checked during construction. Deviations from the materials and properties assumed in the design must be reported to the engineer responsible for the project.

It is also necessary to check that the methods of analysis used in design are appropriate for the geological structure of the ground, and for any variations in ground conditions which are encountered.

Indirect evidence of the geotechnical properties of the soil (for example, pile driving records) must be recorded and used to assist in interpreting the ground conditions.

guide: Geotechnical Category 1

To check the descriptions of the soils and rocks it is necessary to:

1) inspect the site,
2) determine the types of soil and rock within the zone of influence of the structure,
3) record detailed descriptions of the soil and rock exposed in excavations.

Geotechnical Category 2

It is necessary in addition to check the geotechnical properties of the soil or rock on which the structure is founded. Additional site investigation may be carried out. Representative samples may be recovered and tested to determine the index properties, strength and stiffness.

Geotechnical Category 3

Additional requirements may include any of the following:

1) control surveys, of ground movements throughout the site and in the surrounding areas,
2) detailed examination of details of the ground conditions which may have important consequences for the design,
3) determination of soil or rock properties to take account of details of the ground conditions and of the pattern of discontinuities,
4) observations and further investigation work which determine the values of soil properties which were estimated but not measured during the design,
5) careful records of unexpected soil conditions, should these be encountered.

10.3.2 Groundwater

The groundwater levels, pore pressures and groundwater chemistry encountered during construction must be checked and compared with those assumed in the design. More thorough checks are needed for sites on which significant variations of soil type and permeability are known to exist.

guide: Geotechnical Category 1

Controls are usually based on previous documented experience in the area or on indirect evidence.

Geotechnical Categories 2 and 3

Direct observations are normally made of the groundwater conditions if these greatly affect either the method of construction or the performance of the structure.

Groundwater flow characteristics and pore pressure regime are - as a rule - ascertained by means of piezometers. Piezometers are often installed before the start of construction operations. This is necessary in order to establish steady state conditions against which changes can be monitored.

If pore pressures changes occur during construction which may affect the performance of the structure, piezometer readings will normally continue until construction is complete, or until the pore pressures dissipate to safe values.

For structures below groundwater level which may float, pore pressures are normally monitored until the weight of the structure is sufficient to rule out the possibility of floating.

The number, location and type of piezometers, and the duration of the monitoring period are influenced by soil properties, ground characteristics and the relevance of measured data to the project.

It is sometimes necessary to install piezometers at distances of up to several hundred metres from the site as part of the monitoring system. The need for this depends on the stratigraphy and on the pattern of groundwater movements, and normally only arises in built up areas.

The effect of construction (including processes such as de-watering, grouting and tunnelling) on the groundwater regime must be determined from piezometer readings.

Chemical analysis of circulating water must be performed when any part of the permanent or temporary works may be subject to chemical attack or corrosion.

10.4 Construction Schedule

The method of construction assumed in the design is stated in the design report. Site operations must be checked for compliance with the method assumed.

Subsequent variations must be explicitly and rationally considered and implemented.

guide: Geotechnical Category 1

: A formal construction schedule is not normally included in the design documents. The selection of the sequence of construction operations is decided by the contractor.

: Geotechnical Category 2

: The design documents may give the sequence of construction envisaged by the designer. Alternatively the design documents may state the sequence of construction is to be decided by the contractor.

: Geotechnical Category 3

: For these structures, and in other situations in which the behaviour of the works depends on the construction procedure, the design report includes the construction schedule envisaged by the designer. During construction the schedule should be assessed frequently and modified if necessary to take account of:

- the actual conditions encountered,
- the purpose and function of the structure,
- possible effects on nearby structures and services,
- possible disturbance of the ground or disruption of groundwater flows.

10.5 Monitoring

Construction must be supervised as specified in the design. The performance of the structure and of the surrounding ground must be evaluation during and after construction as specified in the design.

guide: The objects of monitoring are:

- to check the validity of predictions of performance made during the design,
- to ensure that the structure will continue to perform as required after completion.

Monitoring means measuring the performance of the structure directly or indirectly. Measurements may include the following:

- deformations of the ground affected by the structure,
- values of actions,
- values of contact pressure between soil and structure,
- pore water pressures and their variation with time,
- stresses and deformations (vertical or horizontal movements or rotations) in structural members.

Results of measurements may well be integrated with qualitative observations including architectural appearance.

For structures which may have an adverse effect on ground conditions or groundwater conditions, the possiblity of leakage or of alterations to the pattern of groundwater flow of fine grained soils, must be taken into account.

guide: Examples of this type of structure are:

- water retaining structures,
- structures intended to control seepage,
- tunnels,
- large underground structures,
- deep basements.

It is always necessary to evaluate and interpret the results which are obtained. This will normally be done in a quantitative manner. The collection of records does not in itself provide a sufficient indication of the safety of the structure.

The monitoring programme must be carried out in accordance with the design report. The design report must state:

1) the object of each set of observations or measurements,
2) the parts of the structure which are to be monitored and the stations at which observations are to be made,
3) the frequency with which readings are to be taken,
4) the way in which the results are to be used,
5) the range of values within which the results will be acceptable
6) the period of time for which monitoring is to continue after construction is complete,
7) the parties responsible for making measurements and observations, for interpreting the results obtained and for monitoring the instruments.

guide: The length of the post-construction monitoring period may be altered as a result of observations obtained during construction.

: The contract for the works should identify the organisation responsible for each of the elements of the monitoring programme given in the design documents.

: Records of the actual performance of structures are important to the development of the Geotechnical Engineering. Records of the performance of structures in Geotechnical Categories 2 and 3 should be collected and stored on a national basis. Full descriptions of the ground conditions and of the relevant geotechnical properties of the soil or rock influenced by the structure should accompany each record.

: For structures in Geotechnical Category 1, evaluation of performance may be simply qualitative and based on visual inspection.

: For structures in Geotechnical Category 2 it is advisable to undertake, at least, measurements of movements of selected points of the structure.

: For GC 3 structures, assessment of behaviour should be based on measurements of displacements, actions, deformation pattern and on their extrapolation, when feasible, to the service life of the structure. Performance control measurements should be made at intervals during construction. The behaviour of the completed structure should be assessed taking in due account the construction sequence and the associated stress and strain paths of significant soil elements.

guide: The instrumentation of structures in Geotechnical Categories 2 and 3 should be supervised by experienced geotechnical engineers.

For structures that may adversely impact on appreciable parts of the surrounding physical environment, and when failure of the structure may endanger human lives, monitoring will normally continue for more than ten years after construction is complete, or throughout the life of the structure.

10.6 Check List for Construction Supervision

This chapter sets out the factors which influence the scope of construction supervision. Their relative importance will vary from project to project.

The check which follows contains the most important construction controls. It is not exhaustive. Items which refer to specific aspects of geotechnical engineering have been reported in previous chapters of this code.

General Controls

guide:

1. Verification of ground conditions, and of the location and arrangement of the structure.

2. Groundwater flow and pore pressure regime; effects of dewatering operations on groundwater table; effectiveness of measures taken to control seepage inflow; internal erosion processes and piping; chemical composition of groundwater; corrosion potential.

3. Movements, yielding, stability of excavation walls and base; temporary support systems; effects on nearby buildings and utilities; measurement of soil pressures on retaining structures; measurement of pore pressure variation consequent to excavation.

4. Safety of workmen with the due consideration of geotechnical limit states.

Water Flow and Pore Pressures

guide: 5. Adequacy of system to ensure; control of pore-water pressures in all aquifers where excess pressures could affect stability of slopes or base of excavation, including atesian pressure in an aquifer beneath the excavation; disposal of water from dewatering systems; depression of groundwater table throughout entire excavation to prevent boiling or quick conditions, piping and disturbance of formation by construction equipment; diversion and removal of rainfall or other surface waters.

6. Efficient and effective operation of dewatering system throughout the entire construction period considering; encrusting of well screens, silting of wells or sumps; wear in pumps; clogging of pumps.

7. Control of dewatering to avoid disturbance of adjoining structures or areas; observations of piezometric levels; effectiveness, operation and maintenance of recharge systems if required.

8. Settlement of adjoining structures or areas

9. Geometry and effectiveness of subhorizontal borehole drains.

Performance of the Completed Structure

guide: 10. Settlement at established time intervals of buildings and other structures including those due to; effects of vibrations, metastable soils.
Settlement observations must be referred to a stable benchmark.

11. Lateral displacement, distorsions expecially those related to: fills and stockpiles; soil supported structures, such as buildings or large tanks; deep excavation channels.

guide: 12. Piezometric levels under buildings or in adjoining areas, especially if deep drainage or permanent dewatering systems are installed or if deep basements are constructed.

13. Deflection or displacement of retaining structures considering: normal backfill loadings; effects of stockpiles, fills or other surface loadings; water pressures.

14. Flow measurement from drains

15. Special problems. High temperature structures such as boilers, hot ducts, etc.: dessication of clay or silt soils; monitoring of temperatures; movements.
Low temperature structures, such as cryogenic installations or refrigerated areas: temperature monitoring; freezing of soil; frost heave, displacement; effects of subsequent thawing.

16. Watertightness