# *EVOLUTION OF SAFETY FACTORS AND GEOTECHNICAL LIMIT STATE DESIGN*

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***by***

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# Evolution of Safety Factors and Geotechnical Limit State Design

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## ABSTRACT

The historical development of limit state design in geotechnical engineering is reviewed. Total and partial factors of safety used for the design of land-based and offshore structures are compared. It is found that the factors of safety in different codes for the ultimate and serviceability limit states design of earthworks, earth retaining structures and land-based and offshore foundations are very similar.

Partial factors in the ultimate limit state design are linked to the variability of the loads and soil parameters, the design approximations and construction tolerances. They influence the nominal probability of failure of the type of structure considered and the seriousness of failure which differ for land-based and offshore structures. These probabilities are compared with human fatality risks of common experiences.

The serviceability limit states are governed by structural and operational constraints and the intended service life of the structure either land-based or offshore. The corresponding partial factors are generally taken as unity.


## FOREWORD

I should like to thank Dr. J. L. Briaud of Texas A&M University for the honour of inviting me to give the Buchanan Lecture, named in memory of the father of geotechnical engineering in Texas at this renowned University. I had known him since the fifties from meetings at several geotechnical conferences and was familiar with his well-known paper on the lower Mississippi levees, which I visited for the first time only this Spring!

## INTRODUCTION

About 50 years have passed since Terzaghi published in 1943 his classical book on "Theoretical Soil Mechanics." In it he divided geotechnical problems into stability and elasticity problems, which would now be called ultimate limit state and serviceability limit state problems, respectively. These problems are closely connected with the factors of safety used in geotechnical design in practice.

It is therefore timely to review here the historical development of this subject in the light of the original contributions of Terzaghi. This will be followed by a summary of more recent developments of the ultimate limit state design and corresponding partial factors in some codes of practice, including the Eurocode of 1993. The results of safety analyses of earthworks, earth retaining structures and foundations of land-based and offshore structures and their probability of stability failures will be discussed and compared with human lifetime risks of common experiences. Finally, some aspects of serviceability limit states are considered.

## HISTORICAL DEVELOPMENT

Some notable limit state problems in soil mechanics were already treated in the eighteenth and nineteenth centuries (Skempton, 1979), when theories of earth pressure and bearing capacity, the stability of slopes, seepage problems and elasticity problems in solids were analysed.

However, the classical geotechnical limit states approach developed only in the first half of this century when Terzaghi (1925) created modern soil mechanics. In 1943 he drew attention to the two principal groups of problems in this subject, namely, stability problems and elasticity problems. He indicated that stability problems (ultimate limit states) deal with conditions immediately before ultimate failure by plastic flow without consideration of strain effects. On the other hand, elasticity problems (serviceability limit states) are concerned with deformations of soil due to its own weight or external forces without consideration of stress conditions for failure.

Ultimate limit state estimates are based on plasticity methods for the stability of earthworks, earth retaining structures, foundations and other construction problems of ultimate resistance. Under the service conditions of these structures a factor of safety is applied on the ultimate resistance. The concept of a factor of safety in stability estimates was introduced in the eighteenth century by Bélidor (1729) and Coulomb (1773) who both suggested a value of 1.25 on the width of retaining walls determined from earth pressure theory. It is of interest to note that this factor of safety corresponds to a value of 1.5 against overturning of walls, which is still widely used in practice today. Later, Krey (1926) used a factor of safety of about 1.5 for the stability of slopes and retaining walls, and he recommended a range of 2 to 3 on the ultimate bearing capacity of foundations. Similar global or total factors of safety became customary for geotechnical design in Europe, North America and elsewhere during the first half of this century.

The total factor of safety may be defined as the ratio of the ultimate resistance of the earth structure or foundation to the applied loads or load effects to ensure freedom from danger, loss, or unacceptable risks. The ranges of the customary total factors of safety (Terzaghi and Peck 1948) are given in Table 1. The upper values of these factors apply to normal loads and service conditions, while the lower values are used for maximum loads and worst environmental conditions. The lower values have also been used in conjunction with performance observations, large field tests, analyses of failures of similar structures, at the end of the service life, and for temporary works.

On the other hand, Taylor (1948) introduced separate factors of safety on the cohesive and frictional components, c and tan $\phi$, respectively, of the shear strength parameters of soils in estimates of the stability of slopes. This approach was generalised by Brinch Hansen (1953 and 1956) when he proposed partial factors of safety on different types of loads, on the shear strength parameters of soils and on piles capacity for the ultimate limit state design of earth retaining structures and foundations. The principal numerical values of these partial factors are summarized in the first three columns of Table 2.

The partial factors were chosen to give about the same design estimates as conventional total factors of safety. This approach to geotechnical design has been used in Denmark ever since, and the first code of practice was issued there in 1965 (Table 2). These factors have been refined subsequently by semi-probabilistic methods on the basis of the variabilities of the loads, soil strength parameters and other design data in practice. Such methods have previously been used for failure analyses of structures (Pugsley 1955, Freudenthal 1956, Borges and Castanheta 1968).

Serviceability limit state estimates of elastic deformations and immediate settlements of earthworks and foundations are based on classical elastic theory, while long-term movements are predicted from consolidation theory (Terzaghi 1943) using average, conservative values of soil properties. Non-linear deformations can be estimated from recent finite element methods allowing for soil-structure interaction. These estimates are then compared with observed tolerances to movement of different types of structures to which attention was first drawn by Terzaghi (1935). Following settlement observations of buildings he suggested safe total and differential settlements for foundations (Terzaghi and Peck 1948). Safe distortion limits of floors had been given for some time in building codes to avoid cracking of finishes. These were extended to include structural elements by analyses of full-scale laboratory tests and settlement stresses in buildings (Meyerhof 1953), as well as using the results of extensive field observations on buildings (Skempton and MacDonald 1956, Bjerrum 1963). For the serviceability limit state design Brinch Hansen (1956) suggested a partial factor of unity on the loads and deformation properties of soils, and this value has been generally accepted in practice.

## ULTIMATE LIMIT STATES AND FACTORS OF SAFETY

The magnitudes of total and partial factors of safety used in ultimate limit state design are governed by the reliability of information, mainly loads and load effects for dead, live (occupancy, traffic) and environmental loads (such as wind, snow, and earthquake), soil resistance (test method, shear strength parameters, pore pressures, progressive failure and other factors), analysis (method, accuracy, assumed failure mechanism, simplified soil profile), construction (geometry, quality and control of materials and workmanship), economy and maintenance, and the probability and consequences of stability failure during service life.

The partial factors of safety are usually applied to specified or characteristic worst loads and load effects including uncertainties of the analysis, and to the characteristic conservatively assessed mean resistance of the soil including uncertainties of construction. These factors have been obtained mainly by calibration with conventional geotechnical design and analysis using total factors of safety to ensure the same margin of safety as that provided by present good practice and experience.

The values of these partial factors are given in the last four columns of Table 2 for the Eurocode (1993) and three North American design documents (CFEM 1992, NBCC 1995, ANSI 1980). In this Table the partial factors for loads given in brackets apply when their effect is beneficial, such as loads resisting instability by sliding, overturning or uplift.

It is of interest to note, as shown in Table 2, that corresponding partial factors for loads, and ultimate pile capacities in these codes are very similar and in remarkably good agreement with Brinch Hansen's original factors of about 40 years ago, except for dead loads. In the North American specifications the partial factors for deal loads, as for other loads, were chosen to be common to all materials of construction, including soil and water. Consequently, these dead load factors, while appropriate in structural design and comparable to the value of 1.35 proposed for it in the Eurocode (1993), are excessive when applied to geotechnical design, and they could well be reduced to about unity in this case. Also the partial factors for shear strength parameters in two of these documents (Eurocode 1993, CFEM 1992) support Brinch Hansen's original factors.

It should be noted that limit state design code's in Europe (including Eastern countries) are mostly based on partial factors for both loads and strength properties of materials, including soils, while corresponding code development in North America with some exceptions (such as CFEM 1992, CSA 1992) favours load factors combined with resistance factors. In these codes partial factors are used on the loads and resistance factors on the ultimate resistance computed from the unfactored strengths of the materials, including soils. These total resistance factors were derived by calibration with conventional total factors of safety or by semi-probabilistic studies of the resistance. This approach is intermediate between the design methods using either total or partial factors. The main advantages of using resistance factors in design are that they can, at least partly, incorporate not only variabilities in the soil properties, but also in the methods of site investigation, the as built dimensions and uncertainties in the type of failure and methods of analysis. As a result, the total resistance factors are expected to be somewhat greater than the partial factors on the shear strength parameters.

The design methods using load and resistance factors are represented by the new National Building Code of Canada (NBCC 1995) and the American National Building Standard (ANSI 1980), for which the partial factors on the loads and resistance factors on the ultimate resistance are given in the last two columns of Table 2. It is found that corresponding factors in these design documents are similar and they support the factors in the codes based on partial load and strength factors. Moreover, since partial factors on the shear strength parameters and total resistance factors are found to have the same range of 1.25 to 2.0, they make the same allowance not only for variabilities in the strength properties but also for such uncertainties as the geometry, type of failure and methods of analysis. Similarly, recent North American Highway Bridge Design Codes (AASHTO 1992, OHBDC 1992) are based on load and resistance factors. These factors are similar to those given in the last two columns of Table 2, since they were also derived by calibration with conventional total factors of safety and semi-probabilistic studies of the resistance.

Since partial load factors in European and North American codes are found to be very similar and since both shear strength factors and resistance factors have been obtained mainly by calibration with essentially the same total factors of safety, it may be expected that designs based on the factored shear strength parameter approach and the factored resistance approach will be similar. Accordingly, the choice between these two methods of geotechnical design in practice depends mainly on familiarity and convenience, and the determination of the in-situ soil strength remains the most important problem.

Since even the most recent partial factors were obtained mainly by comparison with designs based on total factors of safety (e.g., Meyerhof, 1984), it is of interest to compare the partial factors in the Eurocode (1993) with those derived from probability theory using a 90 percent reliability on the observed ranges of coefficients of variation of the loads (Allen 1975, Borges and Castanheta 1968, ANSI 1980, Maes 1986) and of the soil properties (Lumb 1966, Schultze 1982).

The results of this probabilistic analysis are given in Table 3 which indicates that the partial factors given in the Eurocode (1993) are supported by the present comparative estimates. It should be noted that the proposed code factors are to be used on specified or characteristic worst loads and on characteristic soil properties based on cautious estimates of mean values. Moreover, the variabilities of the soil properties were obtained mainly from laboratory tests on small samples from different sites. The lower ranges of the coefficients of variation are likely to govern the resistance of in-situ properties of large soil masses affecting the stability of earth structures and foundations on any one site in practice. The same has been found previously (Meyerhof 1982, Schultze 1982) in probabilistic analyses of other geotechnical data and their application to some limit states problems.

Since the above codes apply primarily to land-based engineering works, brief mention will be made of the safety margin of offshore foundations. As in the case of the design of foundations on land, offshore foundations were originally designed on the basis of total factors of safety. This approach is typified by the corresponding American (API 1991) and British (BS 1982) codes for offshore platforms, which specify for design a minimum total factor of safety of 2.0 under normal working or operating conditions and a total factor of 1.5 under extreme environmental conditions or under temporary loads. These factors of safety are the same as those used frequently for geotechnical design of earth retaining structures on land (Table 1).

The important new American Recommended Practice for offshore platforms (API 1993) is based on partial factors on the loads and resistance factors on the ultimate resistance using unfactored strengths of the materials. These factors are given in Table 4 and they are found to be similar to the corresponding factors for land-based foundations (Table 2), as would be expected from their calibration with total factors of safety.

On the other hand, both Canadian (CSA 1992) and Norwegian (DNV 1989, NPD 1992) codes for offshore platforms are based on partial factors on loads and shear strength parameters, which were also obtained by calibration studies in relation to total factors of safety. The values of the partial factors in the various codes are given in Table 4, which shows that the corresponding factors for loads and shear strength parameters are similar. These partial factors also compare well with a probabilistic analysis (last column of Table 4) using a 90 percent reliability on the observed lower ranges of the coefficients of variation of the loads and shear strength parameters given in Table 3. Moreover, in general, these factors for offshore structures are similar to those for land-based structures (Table 2), as would be expected from the calibration procedures used.

## PROBABILITY OF STABILITY FAILURES

In order to obtain an indication of the approximate nominal risks or probabilities of stability failures associated with geotechnical design based on conventional total factors of safety, safety analyses were made of earthworks, earth retaining structures and foundations on land (Meyerhof 1970) and subsequently for offshore foundations (Meyerhof 1976). Corresponding safety analyses based on partial factors have given similar results, as would be expected from the usual calibration of both types of factors.

The results are shown in Fig 1. It is found that the order of magnitude of lifetime probabilities of stability failures is about $10^{-2}$ for offshore foundations, about $10^{-3}$ for earthworks and earth retaining structures and about $10^{-4}$ for foundations on land. These values are consistent with a range of overall coefficients of variation of the loads and soil resistance between about 0.15 for earthworks and about 0.25 for both land-based and offshore foundations on the basis of the average between normal and lognormal distributions. These coefficients are in the lower range of those given in Table 3 for loads and shear strength parameters of soils.

The data shown for stability failures of earthworks and offshore foundations were obtained from the results of field observations, which have been confirmed by more recent failure records (Schnitter 1979, Whitman 1984, Young et al. 1984, Maes 1986). On the other hand, the ranges indicated for earth retaining structures and land-based foundations were semi-probabilistic estimates. The estimates for these foundations are at the upper end of the estimated lifetime failure probability of about $10^{-4}$ for concrete and steel structures (Freudenthal 1956, Allen 1975), and this nominal probability is also considered to be an acceptable human lifetime fatality risk (CIRIA 1977). Annual rates of probability of stability failures would be in the range of 1 to 2 percent of the lifetime values (service life of 50 to 100 years) for earthworks, earth retaining structures and land-based foundations and about 3 percent (life of about 30 years) for offshore foundations shown in Fig. 1.

This figure also indicates probabilistic values of the corresponding lifetime safety or reliability index $\beta$. This index varies from about 2.5 for offshore foundations, about 3.0 for earthworks and earth retaining structures to about 3.5 for land- based foundations, while it is estimated to be about 4.0 for concrete and steel structures. The rather low value of $\beta$ for offshore foundations represents an average reliability index (CSA 1992). This appears to be acceptable as a calculated risk in practice because of the special and restricted use and occupancy and the relative short economic life of offshore platforms. It is also of interest to note that a lifetime failure probability of about $10^{-3}$ has been estimated for merchant ships (Wheatcroft 1978).

The range of these probabilities of stability failures is somewhat smaller than the range of the probabilities of human lifetime fatality risks of common experiences (Flint and Baker, 1977; Kulkarni, 1981; Pugsley, 1955; Whitman, 1984), which are shown at the right-hand side of Fig. 1. These observed lifetime fatalities per person have probabilities in the order of about $10^{-1}$ (high risk) for natural and mining disasters, about $10^{-2}$ (medium risk) for accidents in motor vehicles, ships and fires, and about $10^{-4}$ (low risk) for railway and aircraft accidents, which are considered to be acceptable human fatality risks (CIRIA, 1977). The estimated fatality risk per person of roughly $5\times10^{-5}$ for gas pipelines and nuclear reactors (Collins and Hudson, 1981) would also represent a low risk. Again, annual probabilities of human fatality risks would be roughly 1 to 2 percent of the lifetime values shown.

## SERVICEABILITY LIMIT STATES

Allowable movements of foundations and structures, including earth structures, depend on soil-structure interaction, desired serviceability, harmful cracking, vibration, and distortion restricting the safety or use of the particular structure. Empirical damage criteria are generally related to relative rotation or angular distortion, deflection ratio, or tilt of the structure and other structural and operational constraints. For superstructures, these criteria differ for frame buildings (open or infilled), load-bearing walls (sagging or hogging) and other types of structures, depending on the relative settlement ratios after the end of the construction. The allowable movements and deformations of structures can only be determined in each particular case. This is especially true for large structures, which are usually designed to include the effects of anticipated foundation movements. For common types of buildings, however, and for some other types of engineering structures, tentative safe limits of the relative rotation may be suggested as a guide. Earlier suggestions (Skempton and MacDonald 1956, Bjerrum 1963, Burland et al. 1977) have been summarized (Meyerhof 1982) and are given in Table 5.

In general, the design of foundations and structures should include provisions for reducing or accommodating movements without damage, and suitable construction precautions should be taken to prevent excessive yield and movement of the ground. An adequate durability of the structures and foundations must also be ensured during their service life.

The serviceability limit states are checked by using a partial factor of unity on all specified or characteristic service loads and load effects. Similarly, the partial factors on the characteristic values of deformation and compressibility properties of soils using conservative mean values are frequently also taken as unity for serviceability limit states (Tables 2 and 4). The same applies when settlement estimates are based on the results of load tests or in-situ soil tests. This approach gives a nominal reliability of about 80 percent and a corresponding estimated lifetime reliability index $\beta$ of about unity, which should be adequate for serviceability limit state design in practice.

## CONCLUSIONS

The historical development of safety factors and limit state design in geotechnical engineering has been reviewed from the time of Terzaghi's classification in 1943 of the two main groups of problems, namely, stability problems and elasticity problems, representing ultimate and serviceability limit states, respectively. After earlier proposals of total factors of safety for the design of earth structures and foundations, Brinch Hansen in 1953 and 1956 introduced the principle of partial factors on loads and on shear strength parameters of soils for the ultimate limit state, and a partial factor of unity for the serviceability limit state.

The partial factors, which were chosen to give about the same design estimates as conventional total factors of safety, have remained practically unchanged in corresponding building codes during the past 40 years, including the Eurocode of 1993. Partial factors have also been introduced into some design standards for offshore structures. The values of partial factors are supported by comparative probabilistic estimates with a 90 percent reliability using the minimum range of observed coefficients of variation of loads and shear strength parameters of soils. While European codes follow the factored shear strength parameter approach, North American codes favour the factored resistance approach. Both methods lead to similar geotechnical designs in practice due to the calibration procedures used.

Field observations of stability failures of earthworks and offshore foundations show lifetime probabilities of failure of about $10^{-3}$ and $10^{-2}$, respectively, while semi-probabilistic estimates for earth retaining structures and foundations on land give corresponding values of about $10^{-3}$ and $10^{-4}$, respectively. Safety analyses based on conventional total factors of safety indicate for these probabilities a range of overall coefficients of variation of the loads and soil resistance between about 0.15 and 0.25, which are consistent with the observed lower range of the values for loads and shear strength parameters of soils. The range of these probabilities of stability failures is somewhat smaller than the range of the probabilities of human lifetime fatality risks of common experiences.

The serviceability limit states depend mainly on soil-structure interaction, harmful cracking and distortion. Empirical damage criteria related to relative rotation or angular distortion have been used to suggest tentative maximum safe limits for common types of buildings and some other types of engineering structures. Allowable movements of earth structures and foundations can usually be estimated from partial factors of unity on the characteristic loads and loads effects and on the characteristic deformation and compressibility properties of soils. This approach would give a corresponding estimated reliability of about 80 percent.

It is hoped that performance observations on different structures and foundations be actively continued to obtain further quantitative information on their actual safety and reliability for comparison with safety analyses, and thereby improve our engineering judgement and experience in the use of geotechnical limit state design in practice.

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Table 1. Values of total factors of safety.

| Failure type | Item | Factor of safety |
|---|---|---|
| Shearing | Earthworks | 1.3 - 1.5 |
| | Earth retaining structures, excavations,, offshore foundations | 1.5 - 2 |
| | Foundations on land | 2 - 3 |
| Seepage | Uplift, heave | 1.5 - 2 |
| | Exit gradient, piping | 2 - 3 |
| Ultimate pile loads | Load tests | 1.5 - 2 |
| | Dynamic formulas | 3 |

Table 2. Values of partial factors

| Item | Brinch Hansen (1953) | Brinch Hansen (1956) | Denmark DS 415 (1965) | Eurocode 7 (1993) | Canada CFEM (1992) | Canada NBCC (1995) | U.S.A. ANSI A58 (1980) |
|---|---|---|---|---|---|---|---|
| Loads | | | | | | | |
| Dead loads, soil weight | 1.0 | 1.0 | 1.0 | 1.1 (0.9) | 1.25 (0.8) | 1.25 (0.85) | 1.2 - 1.4 (0.9) |
| Live loads | 1.5 | 1.5 | 1.5 | 1.5 (0) | 1.5 (0) | 1.5 (0) | 0.5 - 1.6 (0) |
| Environmental loads | 1.5 | 1.5 | 1.5 | 1.5 (0) | 1.5 (0) | 1.5 (0) | 1.3 - 1.6 (0) |
| Water pressures | 1.0 | 1.0 | 1.0 | 1.0 (1.0) | 1.25 (0.8) | 1.25 (0) | |
| Accidental loads | | 1.0 | 1.0 | 1.0 (0) | | | |
| Shear strength | | | | | | | |
| Friction (tan $\phi$) | 1.25 | 1.2 | 1.25 | 1.25 | 1.25 | Resistance factor of 1.25-2.0 on ultimate resistance using unfactored strengths | Resistance factor of 1.2 - 1.5 on ultimate resistance using unfactored strengths |
| Cohesion (c) (slopes, earth pressures) | 1.5 | 1.5 | 1.5 | 1.4 - 1.6 | 1.5 | | |
| Cohesion (c) (spread foundations) | | 1.7 | 1.75 | 1.4 - 1.6 | 2.0 | | |
| (piles) | | 2.0 | 2.0 | 1.4 - 1.6 | 2.0 | | |
| Ultimate pile capacities | | | | | | | |
| Load tests | | 1.6 | 1.6 | 1.7 - 2.4 | 1.6 - 2.0 | 1.6 | |
| Dynamic formulas | | 2.0 | 2.0 | | 2.0 | 2.0 | |
| Penetration tests | | | | | 2.0 - 3.0 | 2.5 | |
| Deformations | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |

Table 3. Coefficients of variation and partial factors

| Item | Coefficient of variation | Partial factors: Analysis (90% reliability) | Partial factors: Eurocode 7 (1992) |
|---|---|---|---|
| Loads | | | |
| Dead loads, soil weight | 0.05 - 0.15 | 1.05 - 1.2 | 1.1 |
| Live loads | 0.2 - 0.6 | 1.3 - 1.8 | 1.5 |
| Environmental loads | 0.3 - 0.5 | 1.4 - 1.6 | 1.5 |
| Shear strength | | | |
| Friction (tan $\phi$) | 0.05 - 0.15 | 1.1 - 1.25 | 1.25 |
| Cohesion ($c_u$, $c'$) | 0.2 - 0.5 | 1.4 - (3) | 1.4 - 1.6 |
| Deformations | | | |
| Elastic modulus ($E_s$) | 0.2 - 0.5 | 1.4 - (3) | 1.0 |
| Compressibility modulus ($E_v$) | 0.2 - 0.4 | 1.4 - (2) | 1.0 |
| Compressibility ($C_c$) | 0.25 - 0.4 | 1.5 - (2) | 1.0 |
| In-Situ Properties | | | |
| Penetration resistance (N, $q_c$) | 0.3 - 0.5 | 1.4 - 1.6 | 1.5 |

Table 4. Values of partial factors of offshore structures

| Item | U.S.A. API (1993) RP2A-LRFD | Canada CSA (1992) S471, S472 | Norway DNV (1989) NPD (1992) | Analysis (90% reliability) |
|---|---|---|---|---|
| Loads | | | | |
| Dead loads | 1.3 (0.9) | 1.25 (0.9) | 1.3 (1.0) | 1.05 - 1.2 |
| Soil weight | 1.3 (0.9) | 1.25 (0.9) | 1.0 | 1.05 - 1.2 |
| Live loads | 1.5 (0.8) | 1.25 (0) | 1.3 (0) | 1.3 - 1.8 |
| Environment loads | 1.35 (0) | 1.35 (0) | 1.3 (0) | 1.4 - 1.6 |
| Water pressures | 1.3 (0.9) | 1.20 (0.9) | 1.0 (1.0) | 1.0 |
| Accidental loads | 1.0 (0) | 1.0 (0) | 1.0 (0) | 1.0 |
| Shear strength | | | | |
| Friction (tan $\phi$) | Resistance factor | | | |
| (slopes) | of 1.25-1.5 on | 1.5 | 1.2 | 1.1-1.25 |
| (passive earth pressures) | ultimate | 1.15-1.6 | 1.2 | 1.1-1.25 |
| (bearing capacities) | resistance using | 1.05-1.2 | 1.2 | 1.1-1.25 |
| Cohesion ($c_u$, $c'$) | unfactored | | | |
| (slopes) | strengths | 1.5 | 1.3 | 1.4 - (3) |
| (passive earth pressures) | | 1.5-2.0 | 1.3 | 1.4 - (3) |
| (bearing capacities) | | 1.5-2.0 | 1.3 | 1.4 - (3) |
| Ultimate pile capacities | 1.25 - 1.4 | 1.2 - 1.5 | | 1.4 - 1.6 |
| Deformations | 1.0 | 1.0 | 1.0 | 1.4 - (1.5) |

Table 5. Tentative safe rotation limits for structures.

| Relative rotation δ/1 | Type of structure |
|---|---|
| 1/150 | Statically determinate structures with flexible cladding and retaining walls |
| 1/250 | Open steel and reinforced concrete frames, offshore platforms, steel storage tanks and tilt of high, rigid structures |
| 1/500 | Panel walls of frame buildings and tilt of bridge abutments |
| 1/1000 | Sagging of unreinforced load-bearing walls |
| 1/2000 | Hogging of unreinforced load-bearing walls |

List of Figure



E = earthworks, $F_l$ = foundations on land, $F_o$ = offshore foundations, R = earth retaining structures, and v = coefficient of variation.

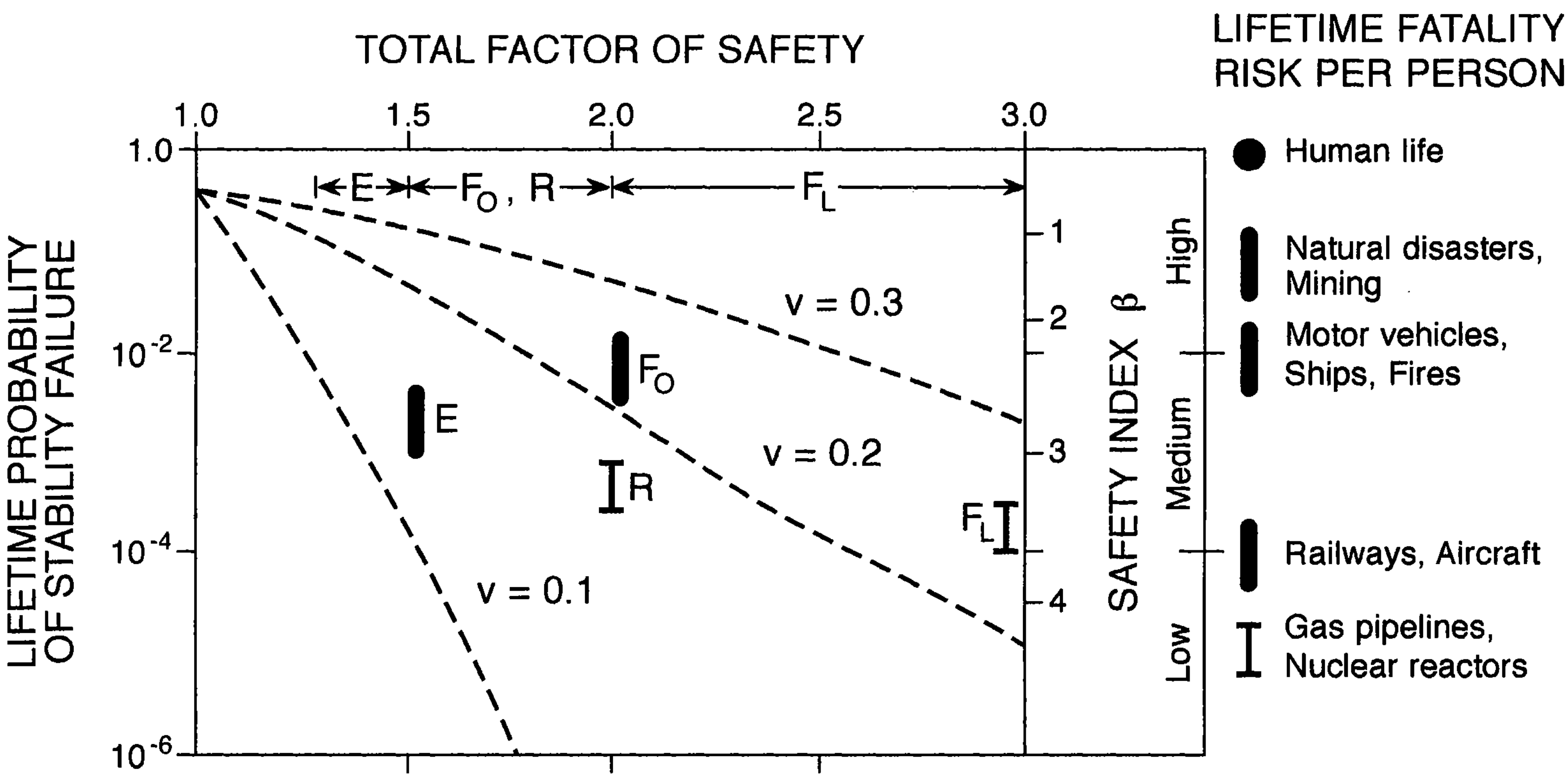


Fig. 1. Lifetime probabilities of stability failures and comparative human risks.

E = earthworks, $F_L$ = foundations on land, $F_O$ = offshore foundations,
R = earth retaining structures, v = theoretical coefficient of variation.

observed , theory - - - - -