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4 Reports

Subject Area

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AD HOC COMMITTEE ON FACTORS OF SAFETY IN SOIL ENGINEERING
(As of December 31, 1968)

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Foreword

The Symposium on Safety Factors in Soil Engineering brought together four excellent presentations on non-engineering elements, buried pipelines, trenches, excavations, and foundations. The informal discussions strengthened the presentations, calling attention to the importance of adequate, thoughtful consideration of soils problems in the preliminary planning stages. Many a site that had been selected through an assumed balance of cost, service, and convenience was later "wished" somewhere else because of neglect in obtaining a subsurface site investigation.

This RECORD should be of interest to planners, locators, designers, and constructors, as well as to student and professional soil engineers.

Keene uses 11 case histories to illustrate the various non-engineering factors that result in unsound designs, either unsafe or over-designed. Economic, sociological, psychological, and political elements are considered. Frequently these elements are mixed and hard to quantify. The humanistic element of public safety probably has been responsible for more over-design than any other factor, while corner-cutting, poor construction procedures, and false economy have produced the failures. The formal discussion by Moulton and Schaub substantiates Keene through additional case histories and comments.

Spangler presents the rationale for arriving at factors of safety to be used in the design of buried pipes or conduits. Spangler states that the structural performance of a buried pipeline depends nearly as much on the environment in which it is installed and the manner of its installation as it does on the inherent strength of the pipe itself. He recommends that the factors of safety to be used vary from 1.0 to 1.5 depending on the methods of test and design.

Mickle discusses the importance of realistic codes and practices to govern the design and construction of trenches. The loss of life due to trench cave-ins during construction continues to be a national tragedy. These deaths focus for a short time public attention on safety rules for excavations. The concerted efforts of soil engineers in producing reliable methods of trench design should be coordinated with the development of building codes drafted to satisfy minimum standards of performance and safety. These codes should not be so restrictive as to prohibit choice of system and must permit the innovator to create new systems. In his discussion, Hirst questions the use of cohesion rather than shearing strength for computation of factors of safety but otherwise supports Mickle's premise.

Sowers explores various facets of the factor of safety problem coupled as it is with the general problem of soil engineering design. Case histories illustrate the various facets leading to an overall view of the problem. He decries the fact that too often the safety factor is an illusion, an imaginary crutch, that helps designers over the difficult point of evaluating the unknown forces, the uncertain resistances, and the inevitable inaccuracies of engineering analyses. A statistical probabilistic approach to possible failure when applied to a bridge or dam in a highly populated area is probably unacceptable. Sowers proposes a component approach that would establish values to be required on the basis of the reasonableness of appraisal of the forces involved (loads, pore pressure, etc.) and on the individual contribution to the entire problem.

This RECORD will be of immense practical value toward enhancing the "philosophy" of competent soils engineering.

Contents

Non-Engineering Elements in Factors of Safety in Soil and Foundation Engineering

PHILIP KEENE, Engineer of Soils and Foundations,
Connecticut State Highway Department

As in other fields, various elements are considered in choosing a safety factor in soils and foundation engineering. The engineering elements usually considered are the forces causing stresses and strains in materials and the resisting forces of those materials. A safety factor is chosen commensurate with the accuracy and reliability of analyzing these forces. During or following these analyses, non-engineering elements modifying this safety factor are or should be considered. These may include economic, sociological, psychological, and political elements. They may be mixed together in certain cases or clearly separate. Usually some are absent. Most cannot be quantitatively predicted. Numerous cases are described to illustrate these non-engineering elements.

•IN EARTHWORK and foundation engineering, the technical elements that enter into producing an approximate factor of safety against failure are usually recognized, although not always understood. These elements are the stresses and strains produced by the applied forces involved and the resistances of the materials to withstand them, all of which, for various reasons, may be difficult to determine accurately. Other elements should also be considered when choosing a factor of safety. These are economic, sociological, psychological, and political. It is the purpose here to discuss how these non-engineering elements should be included, when necessary, in determining the approximate final factor of safety.

In past eras it was sometimes felt, especially by laymen, that the engineer simply analyzed the engineering elements, usually by rules based chiefly on his own or others' experience, and applied a factor of safety commensurate with his confidence in his analyses. It was also felt that the construction cost should enter into his final design; that is, extravagant design or construction should be avoided. To this extent some of the economic elements were considered.

Today the picture is changing. In soils and foundations on many projects fairly reliable numerical values can be placed on stresses, resistances, and the accompanying strains, thanks chiefly to modern soil mechanics and sophisticated instruments and methods used in field and laboratory tests and observations. Recent examples are basic research on pile driving and soil-pile interaction in Michigan, Texas, and elsewhere. But the picture is also changing in other respects due to our modern complex, integrated society, the tremendous growth of knowledge and the speedy means of transportation and communication. Many fields of activity have become interwoven. The civil engineer is coordinating his work with city planners, economists, sociologists, conservationists, insurance men, and others. The soils and foundations specialist is sharing in this endeavor. His importance can be judged from the fact that "soils" or "foundations" is mentioned in one-third of the business "cards" appearing in the professional directories of civil engineering magazines. It is proper, therefore, that he

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be conscious, when choosing a factor of safety, of the economic, sociological, psychological, and political elements inherent in the situation. In the words of Cicero, "Probabilities direct the conduct of the wise man."

ECONOMIC ELEMENTS

The cost of construction for increasing or decreasing the factor of safety by varying a design can be estimated fairly easily. It is or should be common practice to estimate the additional construction cost to increase the safety factor and, conversely, the saving in cost to decrease it. "Safety and economy" are the skilled engineer's usual goal. His estimates here include not only the initial construction, but also the necessary land, the future maintenance costs, and others. But the economic cost of a failure should also be considered, even though the monetary value of this can be calculated less accurately. The cost of repairing or replacing a bridge, building, dam, or highway pavement can be estimated. To this should be added the monetary costs of injury and death to persons, damage to property, and loss of "use and occupancy", such as found in insurance policies.

The economic or cost element is often interwoven with the sociological, psychological, and political. Some cases, chiefly economic, are given here:

Case 1. A test section of a 40-ft high roadway embankment on a deep deposit of soft varved clay in a rural area was built to determine the shearing strength of the clay. Unfortunately the location was chosen at an abutment of a major bridge and the filling was stopped 2 ft short of completion for fear of a failure. As a consequence, the design was changed to provide for an additional span and wide flanking berms at an additional cost of $100,000, even though the shearing strength of the clay had not been properly determined by the field test. The motives for the changes apparently were economic (fear of large extra costs of a slide at the abutment site) and psychological (fear of being blamed).

Case 2. A large 25 by 50-ft twin-box concrete conduit was built in a deep cut in a deposit of soft varved clay 100 ft deep. Where it crossed under a busy three-track railroad the cut was 40 ft deep. Here the factor of safety against sliding was extremely low, and before construction had progressed the cut slopes and overhead temporary railroad trestle were revised to give an adequate safety factor. The elements here were the economic cost resulting from a possible slide and the possible plunge of a train into the cut, the sociological (humanitarian), the psychological (fear of blame for a catastrophe), and perhaps the political.

Case 3. Many years ago, a major river bridge collapse occurred during construction, resulting in the death of 16 men and injury to many more (1). The girders for the first river span, where the collapse occurred, were destroyed, as well as the erection equipment. The tragedy was due to the failure of the temporary river pier made of two clusters of timber piles having timber bracing above the water only. Reconstruction some months later (2) included the use of two temporary piers, instead of one. The new piers were made of very heavy steel H-piles, pointed and reinforced at their tips and driven to refusal in bedrock with a very heavy pile hammer. A third improvement was to provide a huge steel frame, lowered to river bottom, at each temporary pier; the frames had a well for each pile through which the pile was driven and then braced soildly to the well with steel wedges. It is sad to think that any one of these three improvements would probably have averted the tragedy. The inadequate safety factor was due to skimping on the bridge erection costs. The economic cost of the failure far exceeded the anticipated savings. The sociological aspects are obvious. The psychological included an ultra-safe reconstruction to persuade construction workers that it was safe to return, and the political produced some serious undercurrents that fortunately were cleared up.

Case 4. With the use of flatter grades and wider roadways, rock cuts for highways are becoming much deeper. Consequently, the problem of rock falling onto the roadway has become more serious. The use of wide ditches at roadway grade, presplitting the rock before production blasting, steepening rock slopes to reduce bounce of falling rock, and barricades are some methods to reduce the problem. These usually increase the initial construction cost but may save much in roadway maintenance and in injury to motorists and their vehicles, in addition to removing apprehension for traveling on such roadways.

Case 5. Another situation involving the economic element is the problem of scour due to flood waters. It can cause extensive damage to highway and railroad embankments but more spectacular and usually more costly damage to bridges, buildings, and other structures. Significant research has been made on scour but choosing a factor of safety against movement or failure due to scour has uncertainties. The disastrous floods of August and October 1955 in Connecticut damaged or destroyed 300 bridges (3), washed out several miles of highways, cost over 100 lives, and destroyed $220 million worth of property. No state highway bridges, old or new, that had pile foundations or were founded at satisfactory depths were damaged by scour of the foundations. Of the bridges built without piles by the state since 1940, when soils and foundations engineering was started by the state, only one was injured by scour. This consisted of failure by scour under the west pier, which dropped 3 ft. The initial saving by avoiding piles was $11,000 at the piers and $15,000 at the abutments. The cost of reconstruction was about twice the initial saving. The expense of a temporary Bailey Bridge was considerable. Another substantial cost was to the traveling public, who were deprived of a river crossing here for a few months.

Case 6. The unusual and extremely difficult project of building a railroad embankment across the Great Salt Lake in Utah is described by Casagrande in his Terzaghi Lecture on the calculated risk (4). In this paper Casagrande illustrates his discussion by describing some unusual projects in which he was vitally involved. On the Great Salt Lake project he tells how the skill of the board of consultants and the design and railroad engineers was continuously pitted against the thick, soft, sensitive clay below the lake bottom and the project cost ceiling of $50,000,000. Because of the unusual conditions, most of the earthwork construction had to proceed on a semi-empirical basis, with strong reliance on continuous field measurements and on shear strength values derived from analyses of failure and near-failures during construction. The factors of safety during construction were close to 1.0, as the economic element was of paramount importance. The project was completed within the cost limitation and one year ahead of schedule.

SOCIOLOGICAL AND PSYCHOLOGICAL ELEMENTS

Sociology is the study of society in groups, while psychology deals with human behavior and is more associated with the individual. These elements tend to overlap and hence are discussed together.

An obvious sociological aspect is the humanitarian. This includes injury and death, and also hardship or inconvenience to people from disrupting their customary living conditions. Other elements, chiefly psychological, that may be harmful often result from an unprofessional attitude by engineers or others connected with a project, careless or dishonest work on construction, division of responsibility between design and construction, fear of being blamed, and poor publicity practices.

The humanitarian is the most spectacular element and is usually well provided for. "Safety" is becoming a national watchword. Government at various levels, insurance companies, modern technology, and other forces are tending to improve the humanitarian aspects, especially in construction work. An instance at hand is the trench cave-in accident study recently begun by the Engineering Research Institute at Iowa State University (5). In a different type of problem, the design of underground reinforced concrete pipe, Spangler (6) recommends a factor of safety of 1.0 if the design is based on the 0.01-in. crack strength of the pipe. This recommendation is based partly on the assumption that "failure of this type of structure does not involve the safety of human life".

A less obvious element is connected with over-design. The owner pays for the project and there is sometimes a tendency by the designer or the contractor to build unnecessarily large, which may increase the fee or the profit. Also, over-designing is faster and saves design payroll costs. Fortunately the profession recognizes this and various methods of compensation have been devised to improve this situation. Akin to this, but more difficult to remedy, is the psychological element of careless or dishonest work in design or construction. Sometimes this may arise because of lack of coordination between the designers and the constructors, which will be discussed below.

A more excusable cause for over-design is the fear of being blamed if something should go wrong, generally during or after construction. This fear is a basic human trait. It is present in the soils and foundations engineer, the designer, the construction man, and others. The blame may be wholly, partly, or not at all justified. The penalty may vary with the project and the positions of those involved. On the other hand, in some cases this fear will spur the engineer to do a more thorough and complete job, if he has the competence.

A final component in this category is in poor publicity practices. All too often the expensive project is aggrandized in newspapers and magazines with phrases such as "the bold solution to bad foundation conditions" or "elaborate drainage system installed in bad soil", when actually a better insight into the situation or more reliance on the soils engineer's recommendations could have led to a simpler, more economical solution. Conversely, expert modern techniques in soils and foundations work, bringing large savings to the owner when compared with conventional methods, are seldom publicized. Mr. Big is more appealing to the reading public.

In his paper (4), Casagrande deals chiefly with monumental projects having risks involving engineering judgment and experience, where past knowledge and conventional methods of analysis were insufficient. These, of course, had large economic elements of risk by their very nature. His discussion of human risks also stresses the perils in the division of responsibility between designers and supervisors of construction. These can be especially serious where there is poor communication between the groups or the attitude of the construction men is one of indifference or ignorance. Casagrande also discusses the engineering and psychological difficulties arising from division of responsibilities in design. He says, "Even a brilliant man can be very sensitive when his carefully prepared design. . . is attacked by someone who on the basis of a brief review believes that he has good reasons to criticize the design." Casagrande believes the solution on large projects is to have one board of consultants appointed jointly by all parties concerned with the design.

Casagrande also points out the difficulties that may arise from failure to "follow through" in design, especially in large organizations, and to furnish adequate subsurface information to contractors. These are often due to ignoring responsibility. He also deals briefly with the purely non-engineering element of corruption, "an age-old problem".

Similarly, Terzaghi (7) discusses the difficulties and perils resulting from a rigid division of responsibilities between designers and construction forces and from a lack of communication between them. This can be serious when underground conditions are revealed or develop during construction that were not anticipated during design because of insufficient or inaccurate borings, inadequate analyses, or other reasons. If the construction forces do not have personnel competent to diagnose the changes and prescribe proper remedies and if they do not communicate with those responsible for the design, then an improper remedy will be tried. In such situations, if the soils and foundations engineering is supplied by an outside consultant, the division can be especially serious and the consultant may be made a scapegoat when trouble develops. Sometimes the contractor may be of no assistance and may even wish to compound the difficulties, as his chief incentive is to increase his profits or reduce his losses. In his conclusion, Terzaghi urges that the soil mechanics department that supervised the soils work during design should inspect subsurface conditions during construction and compare them with those assumed during design; if necessary, they should request modifications in design in accordance with the findings.

In an interesting and informative book on foundation failures (8) illustrated by approximately 100 cases, Széchy points out that in many of these much of the trouble was due to the construction personnel failing to carry out the plans and specifications or to consult the designers when subsurface conditions differed from the expected. This was especially true in handling of groundwater problems. Similarly, Spangler (6) cites the case of a large city's engineering department using a high factor of safety in sewer design because the construction department often made changes during installation that might influence strength of the pipeline.

As noted in the descriptions of the six cases given earlier, nearly all contained sociological or psychological elements or both. Other examples are given below:

Case 7. A reputable consulting engineering firm strongly desired to use piles under three pairs of highway bridges it was designing. The bridges were to serve as grade separation structures; all were located over some 50 ft of soft varved clay. The state vetoed the use of piles and requested the firm to provide a modest embankment overload and waiting period at each bridge. This procedure caused most of the settlements to occur before building the bridges. The end results were structure settlements of less than 1 in. and a net saving of $250,000. The consulting firm's over-caution was psychological: it had been unfairly caught in a "squeeze" on two widely different projects, one involving an incinerator that allegedly caused air pollution (later proved false) and the other a low-cost housing project where faulty work brought trouble.

Case 8. The factor of safety, so-called, is sometimes used in connection with permissible strains, rather than stresses. These strains are vertical or horizontal movements, or both, and are usually concerned with bridges, buildings, and other structures that may be sensitive to movements. Total movements and differential movements between adjacent elements of the structure are estimated by the soils and foundations engineer to the best of his ability. He then gives these estimates to the structural engineer who decides whether they are tolerable for his structure. At this stage of the proceedings, the structural engineer may become over-cautious for various reasons, such as lack of confidence in the soils and foundations predictions, lack of confidence in his knowledge of how the structure will behave with the predicted movements, or an indifferent attitude toward structural analysis of such problems. Sometimes a designer seems alarmed when told his bridge will settle 1 in., but when settlement readings taken during and after construction show settlements of more than this, with no apparent damage, he then exhibits only passing interest.

POLITICAL ELEMENTS

A final type of non-engineering element to be discussed is the political. It is confined to projects in which government is involved in all or part of the cost and the responsibility. It is more prevalent in the smaller types of government—state, county, and municipal. It exists from the fear that persons or members of an opposition political party—the "outs"—might try to discredit the administration by pointing the finger of blame on an alleged wrongdoing on a government project. This would then involve the engineers connected with the project. Consequently they may choose to raise their factors of safety when they anticipate a restless political scene.

Some examples serve to illustrate the occasional effect of this element. It was mentioned in Cases 3 and 7 above. Several other examples come to mind:

Case 9. Some 33 years ago, a parkway in a metropolitan area was closed at a conspicuous spot only a few days after it had been opened to traffic with the usual ribbon-cutting ceremony. The closing was due to a failure of the soft mud foundation under the embankment. This was later investigated by Taylor and reported in his paper (9). Unfortunately the mishap occurred only a few weeks before election day and the "out" political party took great pains to try to discredit the administration.

Case 10. The failure of a dike and boulevard (10) was used in a political action that attempted unsuccessfully to unseat a high state official. The failure, which was predicted by the only soils engineers connected with the project, occurred partly because of division of responsibility between four government organizations and partly because soils engineering was not regarded then as highly as it is today.

Case 11. A miscalculation in the design of a bulkhead was the cause of a small political stir many years ago. The 600-ft bulkhead retains a fill of sand in a harbor having a thick deposit of marine mud. Although the error was rectified, without damage but with some expense, and the installation completed, members of the "out" party attempted to discredit the administration.

SUMMARY

In an increasingly complex and populous world, the engineer is becoming more involved with the non-engineering aspects of our society. The soils and foundations engineer, having attained a position of importance in civil engineering, should consider the non-engineering elements that enter into his work. These elements are economic, sociological, psychological, and political; their importance varies with different projects. The seasoned and conscientious soils and foundations engineer is aware of these elements and modifies his factors of safety upward or downward according to their importance. He agrees with the poet Gibson, who says through the lips of a North Sea fisherman's wife, ". . . and life itself's a chancy thing".

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Discussion

LYLE K. MOULTON and JAMES H. SCHAUB, Respectively, Assistant Professor of Civil Engineering and Associate Dean, College of Engineering, West Virginia University—Mr. Keene is to be congratulated for his fine paper dealing with the consideration of economic, sociological, psychological, and political elements in establishing factors of safety in soils and foundation engineering. He summarized the situation rather succinctly when he stated: "The seasoned and conscientious soils and foundations engineer is aware of these elements and modifies his factors of safety upward or downward according to their importance." Unfortunately, the seasoning of a soils and foundations engineer is not like the seasoning of lumber, and it often involves more than simple aging in an appropriate environment. Many soils and foundations engineers obtain their seasoning the hard way, through many years of enlightening and, sometimes, sad experience. While it is often said that there is no substitute for experience, papers such as Mr. Keene's can provide the younger and less experienced engineer with a measure of vicarious experience that can help to accelerate the seasoning process.

There are several situations involving one or more of the non-engineering elements presented by the author that, in the opinion of the writers, deserve special recognition and emphasis. These include the case of the soils and foundations engineer being called upon to solve a problem after failure has taken place; the lack of proper supervision during construction; the need for unusually rapid analysis and design brought about by an accelerated building program; and the sometimes tenuous relationship that may exist between factor of safety and economy.

Often, when the soils and foundations engineer is engaged after a failure has occurred, he finds that it is necessary to employ different soil parameters and a different factor of safety than he would normally use. Both the owner and the engineer may be sensitive to the economic impact of a second failure. The engineer may not wish to put his reputation in jeopardy and, because of the peculiar material and psychological conditions that often exist following a failure, may be tempted to adopt an unusually high factor of safety. On the other hand, there is the tendency on the part of some owners to want to cut corners in order to keep the unanticipated additional costs resulting from the failure as low as possible. If the engineer honestly attempts to satisfy the owner by holding down the cost of corrective measures while maintaining an adequate factor of safety, he often has to depend on good construction practices and a well-controlled sequence of operations to do so. Under these circumstances, the lack of proper supervision of construction can lead to very undesirable results.

These considerations are well illustrated in a recent project that involved the design and construction of several heavy structures and conveyor systems. The site was selected by the owner and it was decided that the structures would be founded on a bench cut into an existing slope. The site work and grading were designed by the owner's engineers, none of whom was a soils and foundations engineer, without the benefit of adequate soils explorations. After the excavation was well under way, the owner engaged soils and foundations engineers to supervise the foundation exploration and the foundation design for the structures and conveyor supports. It was agreed that the soils and foundations engineers would arrange for and supervise the subsurface explorations, perform any required tests and analyses, and provide foundation recommendations. It was stipulated that this work should be started as soon as the bench excavation was complete. However, when the level of the bench excavation was approximately 5 ft above finished grade, a slide involving between 150,000 and 200,000 cubic yards took place, and the construction was halted. Since the owner, by this time, felt that he was unalterably committed to the use of the site and the proposed structure grades, the soils and foundations engineers were required to provide recommendations for stabilizing the slope. The subsurface explorations were made, necessary testing and analysis performed, and a report containing recommendations for stabilizing the slope was submitted to the owner. The soil in the slope consisted of colluvium with numerous slickensides. Recommended limits of excavation were clearly defined in the report and the necessity for strict adherence to the recommended sequence of operations was emphasized. This sequence involved the unloading of the head of the slide and installation of surface and subsurface drainage before proceeding with any further excavation at the bench level. Unfortunately, the factors involved in the mechanics of slope failures of this type were not understood by the owner's engineers or the contractor. As a result, the supervision of construction provided by the owner was, at best, haphazard. When the bench excavation was almost to planned grade, the slope failed again. Cross sections taken after the slide showed that the as-built toe of slope was located approximately 30 ft farther into the hillside than had been recommended, the recommended unloading of the head of the slide had not been completed, and neither surface nor subsurface drainage had been installed, ostensibly because of a delay in the delivery of perforated pipe. Obviously the combination of these effects had reduced the factor of safety to less than one. Corrective measures were undertaken a second time, and, at the insistence of the soils engineers, the proper supervision of construction was established and the recommended sequence of operations was followed. Although the slope stabilization was successfully completed, the cost to the owner was quite high. It is likely that many of the problems and much of the expense could have been avoided if the soils and foundations engineers had been consulted during the site planning stage, or an early assessment of the influence of the pertinent non-engineering elements on factor of safety had been obtained.

Another situation that can exert considerable influence on factor of safety is brought about by the expanded commercial and industrial building program in many areas. Often, the foundation exploration, analysis, and design are scheduled to be conducted simultaneously with the structural design. At the same time major planning decisions are being made by the owner's executive personnel. All too frequently the structural designer may be pushing the soils engineer for allowable bearing capacities or other foundation recommendations when the design loads, size of footings, and even the location of the structural units have not been finalized. The discussants have found themselves involved in several such projects. Although in each case the possible alternatives have been known, the tendency was to use a higher factor of safety than might ordinarily be used, in order to take into consideration the effects of a change in location or configuration of the structure that could involve interpolation or extrapolation of boring data and the attendant uncertainties.

Although maximum safety and economy, as noted by the author, are generally accepted as being the basis for engineering decisions during design, the soils and foundations engineer is often faced with the question of how much money to spend to achieve an adequate factor of safety. More important, however, may be the determination of just what factor of safety is adequate, especially where human life is involved. In a recent project involving the expansion of an industrial plant, a soils and foundations engineer was charged with investigating the adequacy of an existing foundation for a gantry crane column. The load on the column was to be increased as a result of the proposed construction. Subsequent investigations led the soils engineer to the conclusion that the factor of safety with respect to bearing capacity under the new load would be quite low. However, the cost of providing a new foundation for this column greatly outweighed the cost of any damage resulting from settlement or tilting of the column. Therefore, it was recommended that the column be instrumented to provide warning of impending difficulty with settlement or tilting, and the foundation was left unchanged. In this instance, the relationship between economy and safety was clear-cut, and the potential danger to human life was negligible. This relationship is not always quite so clear. In the design of dams, use is made of the statistical probability of flooding. Experience has shown how dams designed for different storn frequencies have performed. Thus, a meaningful relationship between safety and economy has been developed. No such detailed statistical data are generally available with respect to the design and performance of many soils and foundations projects. Additional research and correlation of existing data are necessary to provide adequate insight into this problem.

Finally, it is the discussants' opinion that those involved in educating soils and foundations engineers can provide a measure of "instant seasoning" by frankly relating their experiences to their students and emphasizing the important non-engineering elements that can sometimes override strict engineering considerations.

PHILIP KEENE, Closure—The discussion by Professors Moulton and Schaub adds considerably to the paper, both in their emphasis on the possible consequences of poor communication and coordination in design and construction, and in their three illustrative cases. In many papers, such as this one, actual case histories serve to illustrate the points or principles expounded in the paper, with lasting value to the readers. The student who hears of these cases in classroom lectures not only learns more engineering but also is made aware of pitfalls to be avoided in his future career.

Factors of Safety in the Design of Buried Pipelines

M. G. SPANGLER, Research Professor of Civil Engineering, Iowa State University of Science and Technology, Ames

This paper examines the facts and circumstances that should be considered in determining the factor of safety to be used in the design of several types of buried pipelines. Factors of safety based on yield strength or on ultimate strength of the pipe are defined and considered for reinforced concrete pipe, nonreinforced rigid pipes, and flexible metal pipes. Recommended factors of safety vary from 1.0 to 1.5, depending on the circumstances of construction, i. e., the type of pipe, bedding, knowledge of the character of the soil involved, permissible deflection, and the type of strength test used on the pipe.

•THE STRUCTURAL design of a buried pipeline follows the same sequence of operations as does the design of more conventional sturctures, such as bridges and buildings, and consists of two principal phases. First, it is necessary to determine the loads and pressures to which the pipeline will be subjected during its functional life. The second phase is to select the materials, proportion the pipe, and design the pipe installation, all so related that the structure will adequately support the maximum expected load system with a reasonable factor of safety.

Factor of safety may be broadly defined as the ratio of the maximum load that a structure is capable of supporting to the load that it is designed to carry. This ratio may be determined in a variety of ways. For example, in the case of a steel structure it may be expressed as the ratio of the yield strength of the material to the moment design stress. Thus the factor of safety based on yield is

$$FS = \frac{\text{yield strength}}{\text{design stress}} = \frac{33000}{20000} = 1.65$$

In this instance it is assumed that the load-supporting strength of the structure is proportional to the yield strength of the steel. Some designers may wish to consider the ultimate strength of the steel as the appropriate criterion of supporting strength, in which case

$$FS = \frac{\text{ultimate strength}}{\text{design stress}} = \frac{64000}{20000} = 3.2$$

Obviously it is necessary to specify the basis upon which the factor of safety is applicable in order to keep the meaning clear.

Factors of safety may also be applied to other phenomena in place of or in addition to moment design stress, such as shear, torsion, or deflection, depending on the importance of these criteria in relation to the maximum load the structure can carry. In some situations a factor of safety may be applied in the design of a structural element

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to protect an adjunct material, as in the case of a beam that supports a plastered ceiling. In this case the choice of a factor of safety depends on the deflection characteristics of the plaster material, and not on the strength or deflection capabilities of the beam itself.

The purpose of a factor of safety is to guard the completed structure against damage or failure caused by applied loads or circumstances that may be greater or more damaging than those that a skillful designer can logically predict. Or it may be needed to protect against possible shortcomings and inadequacies in the most modern and widely accepted design methods, or possibly against normal and unpredictable variations in strength of materials.

A factor of safety should not be used to mask or cover up sloppy design work, careless construction, inadequate or incompetent inspection, or any other element that can be controlled by competent and alert engineering practice. Several years ago the author sat with a committee attempting to formulate a recommendation for factor of safety in sewer design. Quite naturally there were differences of opinion. One sewer design engineer on the staff of a large city argued for a high factor of safety because, he said, when a design left his department it was turned over to the construction department and they often made changes in installation that might influence strength of the pipeline. Therefore a high factor of safety was needed to guard against such a possibility. What that city really needed was a reorganization of its engineering staff to insure effective cooperation and understanding between designers and constructors. The structural performance of a buried pipeline depends nearly as much on the environment in which it is installed and the manner of its installation as it does on the inherent strength of the pipe itself.

These brief references to the employment of factors of safety in the processes of structural design illustrate the fact that there is no single basis for the application of this important factor and no single value is applicable to all elements of a structure or to all structures in a given category.

The choice of a suitable factor of safety cannot be made by the application of principles of engineering mechanics. It is not possible to establish basic criteria and then proceed by mathematical analysis to an estimate of an appropriate value as is normally done in the classical structural design process. The selection can only be based on sound engineering judgment founded on experience and observation of the performance of similar structures in similar environments and having similar functions. In the exercise of this judgment the basic conflict between reasonable prudence and reasonable economy must be kept constantly in mind. If the selected value of safety factor is too high, the design is uneconomical and the structure costs more than it should. Conversely, if the safety factor is too low, the risk of failure may be too great and the design is decidedly imprudent or marginal. Perhaps nowhere in the field of engineering is the need to exercise good, sound judgment more necessary than in the selection of a prudent, yet economical, factor of safety.

In the field of buried pipeline design, it is necessary to estimate the earth load to which the structure will be subjected. The most convenient and the most widely used tool for this purpose is the Marston theory of loads on underground conduits (1, Ch. 24 and 25). This theory was first announced in 1913 and has since gained worldwide acceptance and use. It is theoretically sound and both experimental evidence and long years of experience indicate that it yields results that are dependable and on the conservative side. It is applicable to all heights of fill and to all types of conduits, regardless of shape and material of composition.

Pipelines under relatively shallow cover may be acted upon by surface traffic loads (1, Ch. 16) such as truck wheels and airplane and railway traffic, including impact. Loads from these sources are combined with the earth load to obtain the total design load on the structure. Such loads may be estimated by means of the Boussinesq theory of stress distribution in elastic solids of semi-infinite extent or half space. Although the Boussinesq theory was developed with reference to an idealized elastic, isotropic, homogeneous material, and although soil definitely does not comply with these specifications, experimental evidence indicates that the theory is a valuable guide for estimating safe design surface load effects on buried conduits. After extensive measurements of loads transmitted to a culvert under various depths of cover, the writer and colleagues (2) offer the following conclusion:

> The theoretical formula (Boussinesq) seems to give a locus showing the maximum possible percent of load transmitted through any thickness of fill. In the experimental work, however, this maximum load generally was not reached, but when conditions were most favorable,... the experimental results came very close to the theoretical.

The load-supporting strength of a buried pipeline is intimately dependent on the shape of the pipe and its component materials. In this connection, two general classes of pipes are recognized. These are rigid pipes, such as those manufactured of plain or reinforced concrete, burned clay, asbestos cement, or cast iron, and flexible pipes, such as those fabricated of corrugated steel, corrugated aluminum, smooth steel, ductile iron, or reinforced plastic mortar.

Speaking broadly, rigid pipes are those that deform very little under load and fail by rupture of the pipe wall. Before cracks develop in the wall, rigid pipes deform a negligible amount under load, and lateral pressures that may act against the sides are considered to be active lateral pressures. Flexible pipes, in contrast, are those that deform relatively large amounts and normally fail by excessive deflection. The sides of a flexible pipe as it deflects under vertical load move outward against the enveloping soil enough to mobilize passive resistance pressures, and these provide a major portion of the pipe's ability to carry the vertical load. Reinforced concrete pipes, which are normally rigid when loaded beyond their initial cracking stage, may gradually become essentially semiflexible in character as cracking progresses. As such, they may deform enough to mobilize the passive resistance property of the enveloping soil as the sides of the pipe move outward. Under these circumstances a substantial portion of supporting strength of the originally rigid structure gradually shifts from its inherent strength characteristics to dependence on the passive resistance of the sidefill soil.

REINFORCED CONCRETE PIPE

Reinforced concrete pipes, widely used in sewer and culvert construction, obviously fall in the rigid pipe category. Their supporting strength in a field installation depends on three major factors: the inherent strength of the pipe, the quality of the pipe bedding as it affects the lateral distribution of the vertical reaction on the bottom of the pipe, and the magnitude and distribution of active lateral pressures acting against the sides of the pipe.

Inherent strength is the strength built into a pipe by the manufacturer. It depends on the wall thickness; the quality of the concrete; the kind, quality, and amount of steel reinforcement; and the placement of the steel. This matter of steel location is extremely important because pipe walls are relatively shallow elements and if the reinforcement is even slightly out of place the lever arm between the compression area and the tensile steel may be seriously modified. Some intangible factors that seem to influence inherent strength are the character of raw materials, the skill of the manufacturer, and quality control practices, including curing of the concrete, that are in effect at a production plant. Experience shows that pipes of exactly the same physical dimensions, made in different plants, may consistently vary in strength.

The inherent strength of rigid pipes is determined by the three-edge bearing laboratory test specified by the American Society for Testing and Materials (ASTM C 497). For reinforced concrete pipes, two separate and distinct criteria for measuring strength are specified. These are the load to produce a crack 0.01 in. wide and the ultimate or maximum load that the pipe can withstand. The three-edge bearing test is quite severe, consisting of a load and reaction concentrated along longitudinal elements at the top and bottom of the pipe, without the application of any lateral forces.

The load-carrying capacity of a pipe installed in the ground is almost always greater than its strength in the testing machine. This is because the loads and reactions are distributed over greater widths and because of the possibility of lateral pressures acting effectively against the sides of the pipe. The more favorable load distributions reduce the bending moment in a pipe wall in exactly the same way that a distributed load on a simple beam reduces the bending moment as compared with that caused by a concentrated load of the same magnitude. As a generalization, the earth load on top of a pipe in the ground is essentially uniformly distributed over the full width of the pipe, i. e., the outside horizontal diameter. The width over which the bottom vertical reaction is distributed depends on the quality and character of the bedding in which the pipe is installed and may vary from the highly detrimental situation represented by a circular pipe resting on a flat bed of strain-resistant material (Class D bedding) to the very high quality concrete cradle (Class A bedding).

Under favorable circumstances a pipe may be acted on by active lateral earth pressures. This is particularly true of projecting conduits or conduits under embankments. Lateral pressures tend to produce bending moments in the pipe wall that are opposite in direction to those produced by vertical loads. Therefore, every pound of lateral pressure that can reliably be brought to bear against the sides of a pipe increases its capacity to carry vertical load about one for one.

The ratio of the strength of a pipe under any stated load system to its strength in the three-edge bearing test is called the load factor. It furnishes a medium by which the strength of a pipe as installed in the ground can be evaluated in terms of its three-edge bearing test strength. Experimental and analytical procedures have been used to evaluate load factors for a number of commonly specified types of bedding, both with and without lateral pressures on the sides of the pipes (3).

Load factors usually vary between 1.1 for Class D or impermissible bedding to well over 3.0 for Class A concrete cradle bedding. A special case worthy of note is that of bell and spigot pipes. The seat of strength of this type of pipe lies in the barrel, and it should be installed so that all of the bottom reaction impinges on the barrel with none acting on the bell. Some recent experiments by a private research agency have indicated that the load factor for pipes that rest heavily on the bells may be as low as 0.5 to 0.75. That is to say, pipes bedded in this manner may fail under loads that are less than the three-edge bearing strength of the pipes. This indicates very strongly that bell holes should be provided when bell and spigot pipes are installed. These should be deep enough and wide enough to insure that all of the bottom reaction acts only on the pipe barrel.

An example of the design of a reinforced concrete pipe culvert is given to illustrate the application or determination of the safety factor. Consider a 60-in. pipe under 43 ft of fill with the pipe installed on a Class B bedding and by the imperfect ditch method of construction (1). Assumptions: $H = 43$ ft, $B_c = 6$ ft, $w = 120$ pcf, $p' = 1.0$, $r_{sd} = -0.5$, $K\mu = 0.13$.

Load calculation:

$$\frac{H}{B_c} = \frac{43}{6} = 7.17$$

$$C_n = 4.3$$

$$W_c = 4.3 \times 120 \times 6^2 = 18600 \text{ plf}$$

Strength calculation: Assume $m = 0.7$, $K = 0.33$, $N = 0.707$, $x = 0.595$;

$$q = \frac{0.7 \times 0.33}{4.3}(7.17 + 0.35) = 0.404$$

$$L_f = \frac{1.41}{0.707 - (0.594 \times 0.404)} = 3.02$$

Required 3-edge bearing strength:

$$\frac{18600}{3.02} = 6160 \text{ plf}$$

Required D-load strength:

$$\frac{6160}{5} = 1230 \text{ D}$$

Using ASTM C 76 Class III pipe, minimum D-load strength at 0.01-in. crack = 1350 D. Therefore, the factor of safety based on the minimum 0.01-in. crack strength is

$$\frac{1350}{1230} = 1.1$$

These calculations indicate that the factor of safety of this pipe installation will be 1.1 based on the minimum 0.01-in. crack strength of the pipe.

It is this writer's judgment that a factor of safety of 1.0 is a prudent, economical minimum value for use in the design of a reinforced concrete pipe installation when the design is based on the specified minimum 0.01-in. crack strength of the pipe. Reasons for this belief are as follows:

1. The failure of this type of structure does not involve the safety of human life;
2. The specified strengths of pipes at 0.01-in. crack are minimum values and the great bulk of pipes in a given class will have strengths greater than the value specified;
3. Reinforced concrete pipes have a large reservoir of load-carrying capacity beyond the 0.01-in. crack stage due to inherent strength and the strength imparted by passive soil pressures as the pipe deforms; and
4. A pipe in the ground does not fail suddenly or collapse completely, so there is adequate time for making repairs in case of accidental overloading.

The application of a factor of safety of unity, based on the minimum 0.01-in. crack test load, suggests the possibility that if all the factors influencing load and supporting strength operate at their assumed or calculated values, a small number of individual pipe sections in a pipeline will develop longitudinal cracks that are 0.01 in. or less in width. Such cracks are not considered detrimental to the structural integrity of the pipe and certainly should not be regarded as a failure situation. Ordinary reinforced concrete (not prestressed concrete) is expected to crack, and all standard equations for calculating stresses in reinforced concrete beams assume that the concrete in the tensile zone is cracked. Unless a crack in the protective cover of concrete is sufficiently wide to permit corrosion of the steel, it is harmless. It indicates nothing more than that the steel is being stressed as expected, and because the modulus of elasticity of concrete is very much less than that of steel, the concrete does not stretch with the reinforcement but develops cracks instead. Also, a crack 0.01 in. wide at the surface of the pipe wall may be only about two-thirds as wide, or roughly 0.007 in., at the reinforcement because of the requirement for a minimum protective covering over the steel.

Some engineers prefer to apply a factor of safety to the ultimate test strength of the pipe rather than the 0.01-in. crack strength. The ASCE Manual of Practice No. 37, "Design and Construction of Sanitary and Storm Sewers," (WPCF Manual No. 9), recommends using a factor of safety of 1.5 based on the ultimate test strength of reinforced concrete pipe. It is noted that this value gives exactly the same result as the value of 1.0 based on the 0.01 in. crack strength in the case of ASTM C 76 Classes I, II, III, and IV pipe, since the required ultimate strength for these classes is 1.5 times the crack strength. For Class V pipe, the required test strengths are 3750 D at ultimate and 3000 D at 0.01-in. crack. Therefore, a factor of safety of 1.5 based on ultimate strength is the equivalent of 1.2 based on the 0.01-in. crack strength. However, since the ultimate test strength of a reinforced concrete pipe has no equivalent or comparable counterpart when the pipe is installed in the ground, because of the development of lateral passive resistance pressures by the enveloping soil as a cracked pipe deflects, factors of safety based on ultimate test strength have no numerical meaning, in this writer's opinion (4). In the foregoing example, the minimum required ultimate three-edge bearing strength of Class III pipe is 2000 D. Therefore the factor of safety based on ultimate strength is

$$FS = \frac{2000}{1230} = 1.63$$

compared with the value 1.1 based on the 0.01-in. crack.

NONREINFORCED RIGID PIPES

The inherent strength of nonreinforced rigid pipes is determined by the three-edge bearing test, the same as for reinforced concrete pipe, except that there is only one test load criterion—the ultimate strength. When a nonreinforced pipe cracks under test load it is finished, and the first crack strength and the ultimate strength are essentially the same. In the testing machine, pipes of this kind normally break into quadrants of approximately equal size and then collapse. When excessively loaded in the ground they also break into quadrants, but may or may not collapse immediately because of lateral support provided by soil at the sides. Broken pipes thus supported may continue to serve as conduits, sometimes for a fairly long period of time. However, eventually they may collapse as the supporting soil is eroded away by leakage, such as from a sewer operating under a head in time of excessive run-off, or other causes. Such broken pipes also contribute heavily to undesirable groundwater infiltration in sewer lines, which adds greatly to water pollution control and treatment costs. Certainly every effort should be made by a designer of this type of structure to guard against cracking of nonreinforced pipes.

Since there is no residual strength in a nonreinforced pipe, except what is ephemerally provided by soil at the sides, the factor of safety must be applied to the minimum ultimate test strength of the pipes to be used. A factor of safety of 1.5 is recommended, unless very favorable conditions relative to knowledge of local conditions that influence loads and strengths of pipe can be relied upon. The conditions referred to are precise knowledge of the character of the soil to be encountered, its unit weight and friction characteristics, an appreciation and understanding of the influence of high-quality bedding on the strength of pipes, a conscientious and knowledgeable contractor, and an organizational setup that insures competent and adequate inspection of the construction procedures. Under such favorable conditions the safety factor can prudently be reduced to 1.4 or even to 1.3 if conditions are exceptionally favorable.

FLEXIBLE METAL PIPES

Flexible metal pipes, fabricated of corrugated steel or aluminum, are widely used in drainage, irrigation, sewerage, and allied fields of construction. As indicated earlier, they tend to fail by excessive deflection. As they deflect under vertical load, the outward movement of the sides of the pipe is sufficient to mobilize the passive resistance pressure of the enveloping soil. This lateral pressure becomes an important source of supporting strength for this type of structure. A logical basis for design of flexible pipes is to estimate the probable deflection of the pipe and compare this with some established criterion for maximum allowable deflection. In addition, it is necessary to investigate the stress situation in longitudinal joints or seams of the pipe.

A widely accepted criterion is to permit a flexible pipe to deflect, i. e., the vertical diameter to shorten and the horizontal diameter to lengthen, an amount equal to 5 percent of its initial diameter. This percentage is based largely on observations made by the late George E. Shafer, formerly Chief Engineer of Armco Drainage and Metal Products, Inc. He measured diameter changes on a large number of corrugated steel culverts under fills of various heights and from these measurements concluded that such pipes can deflect up to approximately 20 percent of their initial diameter before failure by collapse is imminent. Therefore, he applied a safety factor of 4 and established an allowable deflection criterion of 5 percent. Shafer's data are unpublished and an independent appraisal of the validity of this criterion is not possible. However, the writer has observed a number of corrugated pipes in service and has seen nothing to negate his recommendation. Therefore, it is accepted, at least until more definitive research indicates a need for modification.

Measurements of radial pressures (5) around the periphery of a flexible culvert pipe clearly indicate that such pressures essentially are uniformly distributed and that they increase in magnitude as the height of fill increases. Some observers have interpreted these facts to mean that the only stress in a pipe wall is a circumferential or ring compressive stress, much the same as prevails if the pipe is acted upon by externally applied fluid pressure. However, each increment of fill load causes a corresponding increment of deflection, and this deflection brings about the equalization of external pressure, which makes it appear to be hydrostatic in character. Also, when a circular pipe deflects, there is bending moment in the pipe wall. Therefore the true stress situation in a flexible pipe wall is a combination of direct thrust and bending moment, not thrust alone.

These facts are important in connection with the design of longitudinal bolted seams in field-erected pipes, especially those of larger diameter. Pipe manufacturers have frequently designed bolted seams on the basis of single shear in the bolts or bearing of the plates on bolts, with a factor of safety said to be 3.5 to 4.0. Such designs may be inadequate because they do not take into account the bending moment at the location of the seam (6). This bending moment creates a prying action at the lapped seam, which causes direct tension in the bolts in addition to the direct shear stress.

The function of a longitudinal seam is to transmit both bending moment and shear (tangential thrust) from one ring plate to another. Unless the bolted seam is designed to transmit this composite stress situation, trouble may ensue, and the rather generous-sounding factor of safety based on direct shear alone may be misleading. There is need for extensive and detailed research to develop a more rational procedure for the design of longitudinal bolted seams.

SUMMARY

In summary, the writer recommends that a factor of safety of unity is both adequate and economical for reinforced concrete pipe installations designed on the basis of the minimum 0.01-in. crack three-edge bearing test strength of the pipe. If it is preferred to design on the basis of the ultimate test strength, a factor of safety of 1.5 should be used. For nonreinforced rigid pipes, a factor of safety of 1.5 based on the minimum ultimate test strength is recommended for general design application, with possible reduction to 1.4 or 1.3 in unusually favorable circumstances. In the case of flexible metal pipe installations, a limiting deflection of 5 percent of initial diameter is recommended. This is approximately one-fourth (factor of safety of 4) of a critical deflection of 20 percent. The design of longitudinal bolted seams in flexible metal pipes should be based on the ability of the seam to carry a composite of shear and bending moment stresses and not on shear strength alone.

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Safety and Factors of Safety in Trench Construction

JACK L. MICKLE, Associate Professor, Department of Civil Engineering, Engineering Research Institute, Iowa State University, Ames

Soil trench cave-ins have been and will continue to be a major construction hazard. Present state and federal legislation is shown to be inadequate by example calculations of safety factors and reference to known soil behavior. Specifically, the term "angle of repose" referred to in most codes is shown to be meaningless and dangerously misleading. Methods used in current research on this problem in Iowa are described.

•SOIL TRENCH cave-ins have always been a construction hazard. With present-day concentration on water pollution control, increased demand for housing, and more federal aid for projects of this nature, the possibility of this hazard is even more pronounced. There were five deaths resulting from trench cave-ins in Iowa in the fall of 1967. All occurred in two accidents in a small town receiving federal aid to install a municipal sewer system. Nationally there were at least 125 similar deaths during the two-year period ending June 1967, as reported by Land (1). As a result of the Iowa tragedies, a soil trench cave-in accident study has been sponsored by the Engineering Research Institute at Iowa State University. At the time of four of the Iowa deaths, a new "safety rule for excavation" (2) had had the effect of law for 39 days. The rule was ineffective in preventing the cave-ins, and because of its inherently weak terms and penalty provisions probably would have been so regardless of its age. Pressure is being exerted on the state legislature to stiffen the requirements and penalty provisions of the law. Four cave-in deaths in Nebraska (3) in the spring of 1968 will probably have the same effect on the Nebraska state law governing excavation. Both the Iowa and Nebraska laws appear to suffer from a lack of teeth. Unfortunately, most laws and codes not only lack adequate means of enforcement but are inherently defective.

A soil trench cave-in failure is imminent when the fully mobilized shear strength on the weakest plane in an embankment is just equal to the shear stress on the same plane. It is assumed that the strains in the embankment are large enough to fully mobilize cohesion and frictional resistance before failure occurs. If the strains are not large enough, or if tension exists anywhere on the failure surface, then progressive failure is probable. In a progressive failure situation, localized failures occur at overstressed sections along the main rupture surface in a sequential fashion as opposed to one single failure. Sowers (4) indicates that a progressive failure situation exists in most instances that exhibit three signs. The first sign is subsidence of the adjacent ground surface. The second sign is the formation of tension cracks parallel to the trench. The final indication is the spalling of small pieces of soil from the cut face.

Analysis of progressive failure is extremely difficult, and current practice calls for an adjustment in the factor of safety to provide for this eventuality. The factor of safety as used in slope stability is normally arrived at by applying a factor to one or both components of the soil shear strength. If the factor of safety is with respect to shearing strength, then the same factor is applied to both the cohesion and tangent of the internal friction angle. If the factor of safety is with respect to cohesion, it is defined as the

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ratio between the actual cohesion and the cohesion required for stability with full friction mobilized. The latter is also called the factor of safety with respect to height, and is most frequently used for slope stability analysis.

Terzaghi (5) gives several equations for analyzing vertical slopes with inclined plane rupture surfaces. The critical height is given as

$$H_c = 4 \frac{c}{\gamma} \tan\left(45 + \frac{\phi}{2}\right) \tag{1}$$

where c is the cohesion, γ is the unit weight of the soil, and ϕ is the internal friction angle. The inclined plane slopes up from the ditch bottom at an angle of 45 + (ϕ/2) degrees with the horizontal. When a trench is cut in the soil a state of tension exists in the surface soil adjacent to the ditch. In time, vertical tension cracks develop parallel to the edge of the ditch. The depth of the cracks and the distance back from the edge of the ditch is about half the depth of the ditch. The length of time necessary for the cracks to occur varies but is probably a matter of hours. The cracks may be considered a step in a progressive failure. The critical height after the tension cracks have developed is

$$H'_c = 2.67 \frac{c}{\gamma} \tan\left(45 + \frac{\phi}{2}\right) \tag{2}$$

If surface water accumulates in the cracks, a hydrostatic pressure is exerted on the crack wall and the critical height is further reduced to

$$H'_c \cong 2 \frac{c}{\gamma} \tan\left(45 + \frac{\phi}{2}\right) \tag{3}$$

The reduction of the critical height caused by water in Eq. 3 results solely from the pressure of the water within the crack. If the soil becomes saturated then a further loss in stability occurs.

Consider as an example a particularly dangerous Iowa soil (6) with cohesion of 1.3 psi, a unit weight of 90 pcf, and a friction angle of 25 deg. Assume a ditch is to be dug 6.0 ft deep. What is the factor of safety?

Many factors of safety can be expressed, depending on the method chosen for defining the factor and the circumstances surrounding the excavation. Using Eqs. 1, 2, and 3 and the data given, the calculated critical heights are 13.1, 8.7, and 6.5 ft, respectively. The corresponding factors of safety with respect to height are 2.2, 1.5, and 1.1. If the soil is saturated by surface or ground waters a further reduction in the critical height can result from a loss of cohesion or from seepage pressures. The soil in the above example has a cohesion of 0.3 psi when saturated. Using the same equations and conditions as previously cited except for the reduced cohesion, the critical heights are 3.0, 2.0, and 1.5 ft respectively. The factors of safety with respect to height are correspondingly 0.5, 0.33, and 0.25. If seepage pressures are taken into account, a further reduction in stability results. A reduction of the cohesion from 1.3 to 0.3 psi is a reduction of 77 percent. Equal reductions in the critical heights and the factors of safety with respect to height are observed; i.e., 13.1 ft to 3.0 ft and 2.2 to 0.5 are 77 percent reductions. The friction angle remains the same and the frictional resistance is fully mobilized in all cases. As noted earlier and as shown above, the factor of safety with respect to cohesion is the same as the factor of safety with respect to height.

The factors of safety above could have been given with respect to shearing strength where the cohesion and tangent of the friction angle are reduced equally. For example, in the first calculation the factor of safety with respect to cohesion or height was 2.2; had it been with respect to shearing strength it would have been 1.8. Factors of safety with respect to height are used in slope stability analysis because of simplicity and ease of calculation.

The Iowa law states, "The sides of all trenches which are six (6) feet or more in depth, and where the earth is not sloped to the angle of repose, shall be securely held by shoring." Nearly all slopes of trench walls are being considered, if not accepted, as the angle of repose. Unfortunately, since the angle of repose does not exist in most cases, this section effectively negates whatever usefulness the law might have had. Although Iowa law indicates 6 ft, most laws of this nature require shoring for depths over 4 ft. In any case, specifying constant depth for shoring without regard for soil or circumstance is improper.

The preceding paragraph closely resembles the American Standards Association statement (7): "The sides of all trenches which are four (4) feet or more in depth, and where the earth is not sloped to the angle of repose, shall be securely held by timber bracing. The bracing shall be carried along with the excavation and must in no case be omitted unless the trench is cut in solid rock or hard shale." The National Safety Council (8), although having published a superior document with respect to cave-in problems, also speaks of the angle of repose. None of the codes or recommended specifications contained a definition of the angle of repose.

Terzaghi (9) states in a letter, "There is no such thing as an angle of repose of cohesive earth." Later in the same letter Dr. Terzaghi says, "For perfectly clean and dry sand or gravel the angle of repose is fairly independent of the heights of the heap and the method of dumping, and it is approximately equal to the angle of internal friction of the sand in the loosest state. The angle of repose of moist sand and of cohesive soils depends essentially on the height of the heap and on the method of dumping. Hence, in connection with such soils, the angle of repose has no meaning."

Although the angle of repose does not exist for cohesive soils, sloping trench walls as an alternative to shoring is sound engineering. The actual slope must be determined by acceptable theories tempered with appropriate factors of safety. Several design methods for determining safe slopes are available and the choice depends on the general steepness and height of the slope, complications resulting from adjacent structures and boundary conditions, and general soil conditions. For steep banks such as found in trench construction, the Culmann solution (10), which is the more general solution of Eq. 1, is probably acceptable. The safe height is

$$H = 4 \frac{c_d}{\gamma} \frac{\sin i \cos \phi}{\left[1 - \cos\left(i - \phi_d\right)\right]} \tag{4}$$

where c_d is developed cohesion, ϕ_d is the developed friction angle, and i is the slope of the trench wall with respect to the horizontal. The parameters c_d and ϕ_d were determined by dividing the actual cohesion and tangent of the friction angle by a factor of safety with respect to shearing strength. The Culmann method is justifiably criticized because of the assumed plane rupture surface, and even though the soil and water conditions are known and accounted for, a factor of safety with respect to shearing strength of at least 2.0 should probably be used (11). In any case, it becomes readily apparent that any statement calling for an angle of repose in a cohesive soil is meaningless.

Another section of the Iowa law states, "Excavated material and superimposed loads shall not be placed nearer than eighteen (18) inches from the sides of the trench, unless bracing has been installed of sufficient strength to withstand the load." This statement is the same as the American Standards Association statement except the latter ends as ". . . installed and designed to withstand the load." A similar statement from the National Safety Council is as follows: "The amount of soil to be removed as well as the nature of the soil structure will determine how far back from the edge of the trench the soil must be piled. Excavated material and other superimposed loads should never be placed nearer than 18 in. from the sides of the trench. It is, however, good practice to allow at least 24 in. to prevent rollbacks. When superimposed loads or equipment are within the limiting plane of rupture, timbering must be increased to withstand the resultant additional pressures."

Once again the National Safety Council seems to have the superior document. The Iowa law appears to be ambiguous and the intent of the statement is questioned; either it was meant to prevent rollbacks or to provide for surcharges. In either case the implication is that there is no danger resulting from surcharges, regardless of the depth of the ditch, if the loads are kept back 18 in. This is not the case. For example, in the first sample calculation where the ditch was 6.0 ft deep and Eq. 1 was used, the factor of safety with respect to shearing strength was 1.8. If a 400-lb per lineal ft surcharge is placed anywhere within 4.5 ft from the edge of the ditch, the factor of safety with respect to shearing strength drops to 1.5. Designing for loads "within the limiting plane of rupture" is much more meaningful than attempting to specify some specific distance for all cases.

The First National Conference of States on Building Codes and Standards (12) was held at the National Bureau of Standards in May 1968. Former Illinois Senator Paul E. Douglas in his keynote address stressed the urgency for uniform building codes. Douglas said, "Local building code regulations are a major obstacle to true low-cost housing." Gene A. Rowland, chief of the codes and standards section of the National Bureau of Standards, indicated that existing codes are "not bad," but rather "too much of a good thing." Douglas also said, "If the states do not find a way to get around unduly restrictive building codes, we'll have a national code. If you don't clean house, the federal government will."

Building codes are drafted to satisfy minimum standards of performance and safety. However, the right to choose the system that will satisfy the intent of the code must not be infringed upon. For economic reasons, if not other, the innovator must be free to create new systems for accomplishing the given task as well as to improve current procedures.

Engineers have not been active in the past in developing the majority of these codes. However, if the engineer does not become active and make decisions in this field, then someone else, perhaps not so well versed in the problem, will do so. The end result will be engineering by a political group with safety in mind and with little thought given to engineering economics, alternatives, or innovation.

There are diverse opinions as to what role the engineer should play in the soil trenching field. The prevailing opinion appears to place all of the responsibility with the contractor. Under this system, when a job is advertised the contractor takes his own borings, determines what difficulties he is to encounter, and submits his bid based on this information tempered by his past experience. At a recent ASCE section meeting, one contractor on a panel discussing trench excavation indicated that the engineer should furnish only enough information for the contractor to find the job.

While the preceding has probably been at least partly true in the past, Greer and Moorhouse (13) indicate that the standards of the profession are changing. It is time the engineer recognized and accepted his responsibility in subsurface construction. If the engineer makes a complete and proper study, he need not attempt to protect himself with a disclaimer, but will be in a position to supply prospective bidders with sound information. This information, because it removes doubt and duplication of effort, will eventually, if not immediately, lead to more economical and safer construction projects. This is far better than an uneconomical iron-clad code that is safe under all circumstances, in all soil types, at all times, or, because of its ambiguity, places the contractor at the mercy of the individual interpreting the law.

Early results from the Iowa State University study indicate that Terzaghi's solution for the stability of a vertical bank weakened by tension cracks is probably satisfactory. Only a limited number of observations are available, but Eq. 2 has proved satisfactory in evaluating cave-in failures. In Iowa, a field trip is made to each significant failure site in the state as it is reported. Close cooperation with the Iowa Bureau of Labor, whose field inspectors report the failures, makes this arrangement possible. Field strengths are evaluated with the newly developed bore-hole shear device (14).

Other objectives of the Iowa State study are to investigate the effects of time on the stability of trench walls, the effects of surcharge loads, vibrations, moisture content variation, ditch geometry, and construction procedures. All of these effects must be recognized either directly or indirectly through the factor of safety. Eventually, a manual of recommended practice for trench construction in Iowa soils will be published.

Stronger and more uniform national and state building codes are being demanded. If the engineer is to have a hand in drafting codes and specifications and is to furnish direction to his clients and to contractors, then he must have reliable information. Knowledge of this type can be gained only through thoughtful study and research. The engineer must act now, however, before national and state codes are written and rewritten by those less able to do the job.

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Discussion

TERENCE J. HIRST, Assistant Professor of Civil Engineering, Geotechnical Engineering Division, Lehigh University—The author is to be congratulated for drawing the attention of engineers to problems surrounding establishment and enforcement of safe and economical building codes. Once again we are faced with evidence of society's inability (or refusal) to act on behalf of individual safety until after tragedy has occurred.

The various safety factors that the author computed for a vertical cohesive embankment dramatically illustrate the meaningless nature of such factors unless each is accompanied by information concerning the method of analysis, the soil properties, and the assumed boundary conditions. For example, the factor of safety with respect to cohesion is not the same as the factor of safety with respect to shearing strength. Because the conditions necessary to the development of cohesion and friction are not clearly understood, it is difficult for the discussant to understand the rationale behind determination of a safety factor with respect to cohesion rather than shearing strength, particularly since the latter is numerically lower than the former.

Most of the currently acceptable methods of stability analysis employ a failure mechanism such as a plane or circular surface of sliding in conjunction with limit equilibrium. In an effort to provide additional insight into the validity of existing limit equilibrium analyses, research at the Fritz Engineering Laboratory at Lehigh University has been directed toward establishing alternative solutions to stability problems by assuming that the embankment soil behaves as a perfectly plastic material. Preliminary results suggest that, for the special case of vertical cohesive embankments, the use of limit equilibrium in conjunction with a plane surface of sliding yields critical heights similar to those obtained from an analysis that assumes plastic behavior and a logarithmic spiral surface of failure. However, such agreement is not evident for embankments whose slopes are not vertical.

Other factors, additional to those noted by the author, must be considered when investigating the stability of cohesive embankments. For example, the properties of cohesive soils are known to be time-dependent. Although it might be argued that most trenching operations are of short duration, a significant number of trenches do remain open for long periods of time, thus necessitating consideration of the influence of time on the soil properties. Indeed, the current state of the art does not provide the engineer with an economical means to measure appropriate soil properties or to perform comprehensive stability analyses for each soil type encountered in every trench.

In summary, although supporting the author's dislike of uneconomical iron-clad building codes, the discussant suggests that replacing existing arbitrary codes with oversimplified methods of analysis may lead to equally meaningless requirements similar to those currently established. Until there is a better understanding of all of the variables affecting stability analyses, an uneconomical, arbitrary—but safe—code is perhaps more attractive than a code based on a method of analysis that yields a safety factor of dubious reliability.

The Safety Factor in Excavations and Foundations

GEORGE F. SOWERS, Senior Vice President, Law Engineering Testing Company, Atlanta, and Regents Professor of Civil Engineering, Georgia Institute of Technology

•A SECONDARY road in a midwestern state was constructed on an embankment across a small ravine. The design of the embankment followed the usual standards of the highway department in providing a reasonable margin of safety against shear failure. The pavement also was constructed in accordance with the usual standards with a substantial margin of safety against failure under the traffic loads. A culvert was installed beneath the embankment to dispose of the runoff that accumulated in the ravine upstream from the embankment. Its design, too, incorporated a reasonable factor of safety.

After several years of satisfactory performance with no indications of distress, the embankment suddenly failed (Fig. 1). The failure was the result of a chain of circumstances. Debris, from cutting undergrowth nearby, was eroded from the steep slopes of the ravine, and it accumulated at the culvert entrance. The embankment with its clogged culvert became a dam with water ponded behind it. The soil in the embankment became saturated, a condition that was never anticipated in the original design. The continuing saturation and seepage through the embankment weakened the soil enough so that a shear failure developed on the downhill side. More than half of the road was taken by this failure.

This illustrates the complex nature of the problem of the margin of safety with respect to failure. The original design included adequate safety factors for the conditions anticipated in these highway embankments. It is obvious that there was not a sufficient margin of safety with respect to possible but unusual conditions. A peculiar chain of circumstances, unforeseen by the design, produced a rather ordinary type of shear failure.

The problem of safety in engineering and construction is becoming increasingly important. The traditional views of safety as well as the design provisions for providing safety are undergoing change for a number of reasons:

1. Designs are becoming increasingly daring, departing from the ordinary or from past experience. Excavations are deeper and structures are heavier. Moreover, these cause much more drastic changes in the environment.
2. The structures are becoming more critical—less tolerant to movement. Modern frame structures develop severe secondary stresses as a result of differential movements of the foundations; the complex structures erected today are much more sensitive to tilting and misalignment than the simple structures designed and constructed 50 years ago.
3. Sites for construction are becoming progressively poorer. In populated areas the best construction sites are already occupied by structures. Only the marginal land usually passed over because of unfavorable site conditions is available. Geometric design dictated by safety and speed dominates highway location, and considerations of foundation and material quality have become secondary.

In addition to the engineering problems involving safety, there are important legal and economic considerations that dictate a new look at safety requirements. First, the

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Figure 1. Slide of a highway embankment weakened by saturation caused by a clogged culvert.

economic squeeze imposed by increased design and construction costs has made it necessary to shave the safety factors to the barest minimum. At the same time, the lack of planning money has sometimes made it impossible to undertake the thorough investigation of site conditions that must accompany a reduction in the margin for safety.

Similarly, the squeeze of time has added its effect. The public demands better highways and better structures immediately; the urgency generated by public opinion frequently does not permit enough time for the necessary thorough evaluation of site conditions before construction begins.

Unfortunately, the demand for cheaper structures built in less time has been accompanied by increasing intolerance of risk by society. The public has become exceedingly claim-conscious, probably lured on by the exorbitant damage awards sometimes made by ill-advised juries in accident cases. In a recent magazine article a prominent attorney, a leader in the Association of American Trial Lawyers (the association of attorneys whose fees are generally a third to a half of the damages awarded in cases of failure), has stated that our society demands that engineers be morally and economically responsible for failures. He implies that this is a shift in the position that has governed American society since the building of this nation. The original American philosophy was that progress inherently requires risks and the damages resulting from this risk are the price that society as a whole must pay. With this traditional position the author agrees.

However, the attorney maintains that now the need for such progress at the cost of risk has ended. Society, elevated to a plateau of affluence, no longer requires a rate of progress that demands risk. It is the author's opinion that this attorney is seriously misinformed regarding the expanding needs of the world. Risk is inherent in life but particularly inherent in progress. This growing tendency on the part of certain members of the legal profession to demand payment by somebody for the risks of existence and progress (and legal fees in proportion) is a threat to the very progress that has transformed the life of this nation during the last half-century. The growing application of this concept can only result in a lack of progress and a stagnation of engineering initiative. However, the growing unwillingness of society to accept risks in obtaining progress now makes it imperative that the engineering profession reevaluate the philosophy of safety in design and construction.

SAFETY FACTOR

The safety factor is difficult to define accurately. In its fullest sense, it is the margin of resistance of the structure to failure. In a more restricted sense, it is the ratio of the resistance to failure to the unbalanced force that might cause failure. For a small, simple component of a structure like a beam, the safety factor can be defined accurately with ease. However, the evaluation of that safety factor is seldom precise because neither the resistance to failure nor the unbalanced forces causing failure can be determined accurately in advance.

The overall safety factor of the structure is more difficult to define because it depends on the interaction of all of the components of the structure. The individual components all may be adequately safe. When these components are joined together, however, certain secondary stresses that were considered in the evaluation of the individual components may govern the overall safety.

The complex nature of the safety factor can only be understood by considering all the technical components that are involved. Essentially, it is a technical measure of the unknown or, in less elegant terms, of the ignorance of the designer. The most important of the components that influence the safety factor are listed in Table 1. The site conditions are among the most difficult components to evaluate quantitatively because of the extremely complex nature of the soils, rock, and groundwater conditions. Moreover, these site conditions are dynamic, changing with the seasons, and even changing as a result of the new construction. The saturation of the embankment in Figure 1 is an example of an environmental change that led to failure.

The material properties of the site and the engineering structure are equally difficult to evaluate. Moreover, the site properties are dynamic, as was previously mentioned. The science of evaluating these properties has not progressed to the point that all of the future behavior of soil and rock can be predicted, even when the environmental changes are known. To a lesser degree, the dynamic changes apply to the materials of the engineering structure. Their properties are altered by temperature and with repeated loading. The recent collapse of a bridge over the Ohio River, years after it was placed in service, apparently was a result of fatigue, a dynamic change in the structural behavior of the material.

The magnitude of the load to which the structure will be subjected is difficult to evaluate. The dead load can be predicted accurately. The designer selects design live loads based on codes, laws, and experience. The capacity to resist these loads becomes an inherent property of the structure. However, the ignorance of the owner or the bowing of a legislative body to lobbying or political pressure can upset the engineer's design by arbitrarily permitting live loads greater than those that were anticipated. For example, state legislatures have increased the loads permitted on highway vehicles as blithely as if the change in the load law could cause a change in the strength of all of the components of the highway. (While we must agree that the politicians have many occult powers, it has not yet been demonstrated that they have the magic wand for increasing soil and rock strengths.)

Sometimes even engineers are misled into permitting load increases because there has been no sign of distress with the original design load. Such progressive failures as the fatigue of the Ohio River bridge are not likely to develop at low levels of stress, but beyond a threshold level of stress, creep, and fatigue, failures become increasingly important. Therefore, stresses cannot be increased just because a long life has indicated that the original design might have been conservative.

TABLE 1

THE ELUSORY OR ILLUSORY FACTOR OF SAFETY AND ITS COMPONENTS

I. Sources of ignorance involved in design
   - A. Site conditions
   - B. Material properties
   - C. Loads on structure
   - D. Inaccuracies in analysis
   - E. Changes produced by construction
   - F. Changes produced by structure
   - G. Changes of environment

II. Consequences of failure
   - A. Direct costs
     1. Cost to owner
     2. Cost to neighbors
     3. Cost to users
   - B. Liability of owners, designers, constructors

III. Failures not related to the safety factor
   - A. Complementary structure: natural and man-made
   - B. Excessive deflection producing failure

IV. Safe because of failure
   - A. Shear mobilization
   - B. Safety valve
   - C. Warning

A major component of the margin for safety, therefore, lies in the margin of uncertainty of the future loads: their magnitude, duration, and frequency. The total view of the problem is given in Table 1.

THE SOIL-STRUCTURE SYSTEM AND FAILURE

The resistance against failure is provided by an assemblage or system of structural components. Each individual component must be safe against failure. In addition, the assemblage of components acting together must also be safe against failure. The system includes anything that is interconnected with the structure under consideration. From the soil point of view, the total system includes not only the soil beneath the structure but the soil adjacent to the structure.

The major problem arises from the fact that engineering design is generally component-oriented. The structural designer determines the size and shape of each individual component in the system. While the design codes also establish certain requirements for the assemblage of components, the interaction of those assemblages is frequently so complex that their real behavior cannot always be evaluated by the usual methods of structural analysis. Furthermore, the designer is likely to focus his attention on only those elements of the structural assembly that are his direct concern. For example, the structural engineer concerns himself directly with the structure. The mechanical engineer concerns himself directly with the mechanical system. Only rarely does the mechanical engineer consider the effect of the vibration response of the structure on the performance of the air conditioning compressor. Similarly, the structural engineer seldom considers the dynamic loading of the air conditioning equipment in his structural design. The problem of the appropriate total system becomes more complex in the foundation because the soil system that is related to a particular structure may extend beyond the limits of the building site.

Many of the engineering difficulties involving foundations result from a failure to consider certain minor, innocent-appearing components in the system or the interaction of various components of the system. For example, a small landslide occurred in a deep railroad excavation in North Carolina in spite of a design analysis based on laboratory tests of the soil that required a minimum safety factor of 1.5 against soil sliding. Moreover, the failure occurred in one end of the cut where the total cut height was substantially less than at the center of the height of the cut. The upper scarp of the slide intersected the slope well below the top. The cause of the slide was a small slickensided surface present in the residual soil mass. This minor component of the total soil structural system had not been considered in design. Although it was known that such slickensides were present, it was not feasible to determine their location nor orientation in advance. Of the hundreds of slickensides present, only one was geometrically oriented in such a way that it precipitated a slide. Thus, the failure took place although the general design safety factor was 1.5. It is likely that the failure would have occurred had the slopes been so flat that the general safety factor would be even 2.

In a second example, failure occurred because the interaction of components was not considered. A bracing system for a deep excavation in a large city was designed to support not only the loads imposed by the surrounding alleyways, but also stresses transmitted to the bracing system by an adjacent 7-story building. The design was prepared with ample safety factors, and the installation of the bracing system generally followed the best construction practice. However, at the center of the site the contractor installing the bracing encountered an underground transformer vault that had been inadvertently left off the site plans. Some of the lateral load in the adjoining building was transmitted directly through the transformer vault to the bracing system, an interaction that the design had not contemplated. As a result, a portion of the bracing system failed, causing damage to the adjacent structure.

A study of case histories thus indicates that it is necessary to consider all of the components in the total structural system, including the soil, from the standpoint of both their inter-reaction and their individual safety. From the engineering point of view it is convenient to divide the total system into two parts: the immediate system, which includes the soil and the structure directly involved, and the complementary sysstem, which includes the adjoining mass of soil as well as any structures that contribute significant loads to it.

The Immediate System

As stated, the immediate system consists of the soil and the structure directly involved in the project. Failure can occur independently in the soil, the structure, or in both. In a deep excavation for an office building, the bracing structure that supported the soil was properly designed and did not suffer damage, although the soil failed and nearly disrupted the entire system (Fig. 2). The bracing consisted of vertical H-piles acting as soldier beams, with horizontal steel beam wales supported by diagonal steel beam rakers. The design of the bracing system and the sequence of installation were shown on the contract drawings. They required driving all the H-piles and then excavating to the level of the first wale. Wood lagging was required to be installed between the soldier piles as the excavation progressed downward. At that level, the wale was to be installed followed by the diagonal rakers. Following the placing of the wale and rakers the next level of excavation would proceed. The contractor reasoned that he could change the bracing procedure and save money. He excavated a narrow slot at the location of each soldier pile and drove the soldier pile in the slot, thereby reducing the skin friction and easing the driving. He then excavated to the level of the first wale and installed the rakers. He omitted the wood lagging in this stage because it would be inconvenient and time-consuming to install it level by level. He then proceeded to excavate between adjacent soldier piles from the first wale to the bottom of the excavation. Because workmen installing the lagging would interfere with machine excavation, he planned to install the lagging after the excavation had been completed to the bottom between the adjacent soldier piles. The weather was reasonably dry, and the soil appeared to stand without the need of the lagging, which confirmed his optimism. However, after several such excavations had been made and no lagging was yet installed, a severe storm occurred. The soil adjacent to the excavation became saturated and weakened. Large chunks of earth fell between the soldier piles where the lagging should have been. Unfortunately, an 18-inch water main was supported by the soil only a few feet from the bracing system. The fallout of the soil left the water main unsupported, and it ruptured. The flow of water washed a large hole beneath the adjoining pavement of a main thoroughfare, making it necessary to close two traffic lanes.

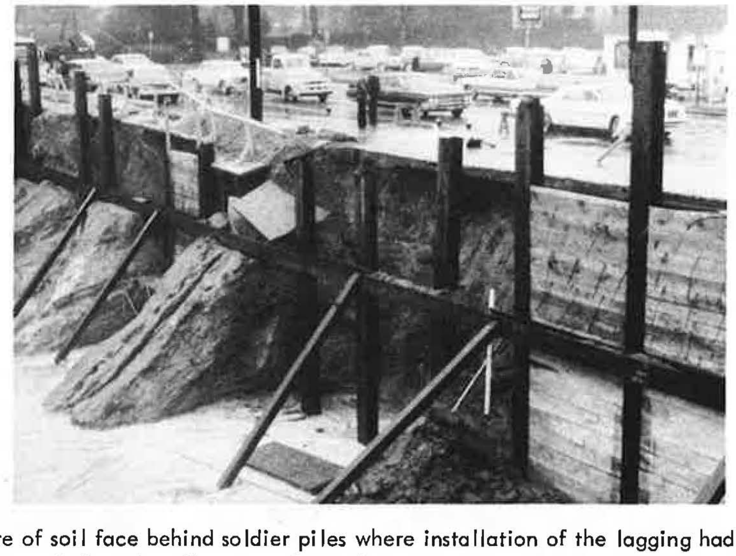

Figure 2. Failure of soil face behind soldier piles where installation of the lagging had been delayed. The dropout destroyed the sidewalk and an 18-inch water main and undermined a 6-lane pavement.

At the same time, the flow flooded the building site to a depth of 20 feet. Nearly a month was lost in pumping the water from the site, removing the slime that had accumulated on the bracing system, and cleaning the power shovels, air compressors, and other mechanical equipment in the job. The direct cost to the contractor was nearly $100,000. Moreover, more than a month's building occupancy was lost. Even more serious was the exposure of the public to potential loss of life from the undermined street and the potential loss of property due to interruption of fire protection in the adjacent area of the city. (Fortunately, the failure took place in the early morning hours before morning rush-hour traffic had commenced, and there was no loss of life or property except from the direct loss of the pavement support.) In this case, the safety factors of the bracing system design were adequate. Because of the omission of a portion of the system, the soil rather than the bracing system failed. The contractor had not considered the safety of the soil in his streamlined operation, and as a result nearly precipitated a disaster.

Just the opposite sometimes occurs. In one project, the excavation bracing system was left to the ingenuity of an incompetent superintendent. The bracing system was an assemblage of old steel beams and wood lagging obtained from a junkyard and from the demolition of an old building (Fig. 3). It consisted of a soldier pile wall driven along the perimeter of the site and supported by diagonal rakers. No two soldier piles were the same structural shape. The diagonal rakers were old wood beams that were badly bent. There was no wale to tie the structural system together. The bracing system supported a steep bank, beyond which was a 3-story brick building. Although the bracing system was condemned by the designing engineer, the contractor delayed in replacing it with something better. Two days after its condemnation, the bracing system failed. A deep but narrow I-beam soldier pile failed by twisting. Although it was propped by a wood raker, there was no provision for lateral stability; under the pressure exerted by the soil, the beam rotated, breaking its connection with the raker. Fortunately, only a nominal wedge of soil fell out and this was partially restrained by a piece of construction equipment at the base of the excavation. The soil, therefore, generally did not fail although the bracing system did. If the soil had not been stronger than the contractor anticipated, the 3-story brick building would have dropped to the bottom of the excavation with a substantial financial loss and possibly loss of life.

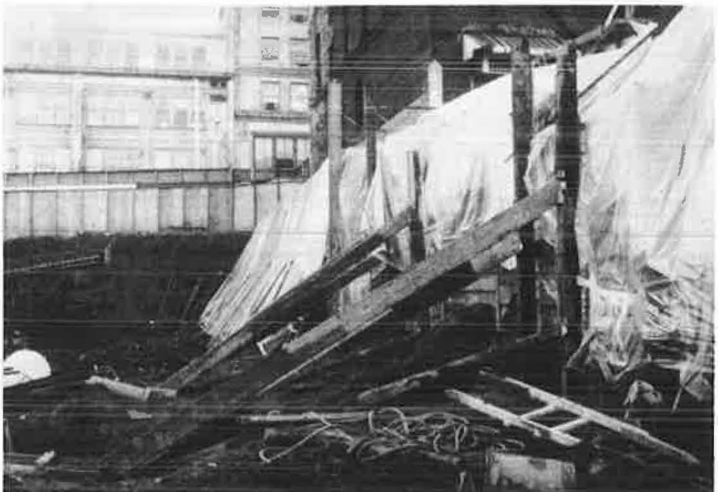

Figure 3. Poorly designed bracing system a few hours before failure.

Occasionally, the safety factor of a structure is affected by construction problems that were neither anticipated in the design of the foundation nor involved in the safety factor used in design. Thus, an unsafe structure is constructed in spite of an adequate design safety factor. An example of this type of problem in the immediate system was the construction of open pier foundations for a large office building on a site underlain by residual clay overlying limestone. The clay was approximately 20 feet thick below the bottom of the 30-foot basement excavation. Generally, the limestone was sound and the clay above it relatively stiff. However, in an elongated, narrow zone stretching diagonally across the site, the depth to rock was considerably greater than the average, and the rock contained numerous narrow slots filled with soft clay. The foundations were installed by drilling holes to the rock surface, and then installing temporary casing to support the clay. This was followed by excavation of a short socket into the sound limestone. After the hole was cleaned and inspected, it was filled at the same time the casing was pulled. In the slotted zone it was necessary to excavate far below the upper surface of the rock in order to reach sound, continuous limestone. Excavation in the slotted rock below the level of the casing was accompanied by a squeezing and flow of wet, soft soil into the socket. Large quantities of pasty soil were removed that were several times the theoretical volume of the hole. The ground surface in the vicinity of the slotted zone subsided and moved laterally toward the slot.

This movement had two serious effects. First, it generated movement of the excavation bracing system because the rakers of that system were supported by spread footings resting on the residual clay, well above the rock surface. Second, the movement of the residual clay produced lateral pressures on the pier foundations for which they were not designed. Some of the tops of the piers were moved out of their original position, and possibly some were damaged. This difficulty would not have been prevented by an analysis of the bearing capacity of the pier foundations. In fact, it is doubtful that even a ridiculously high safety factor could have prevented the movements and the foundation difficulties that occurred. It might be argued that the failures, as evidenced by the soil movement, were beneficial because they brought to everyone's attention a construction difficulty that had not been recognized by the resident engineer even though he had noted the excessive amounts of soil being removed from the pier excavations. It was only after the movements had occurred that action was taken to change the construction procedure so as to eliminate the squeezing of the soil through the slots of limestone.

The Complementary System

The complementary system includes the soil and the structures beyond the immediate limits of the project—the soil far below the level of support of the foundations, and the soil and adjacent structures beyond the construction limits. Problems in the complementary system may be reflected in damage or failure in the immediate system. In such instances the failure is not directly related to any safety factor that might have been used in the design of the structure. Instead, failure is related to external circumstances that must be evaluated in the design but are not reflected in the strength of the components of the immediate system.

The failure of a foundation caused by geologic processes unrelated to the structure or its load illustrates such a complementary failure. A highway bridge in West Florida was supported by pile foundation driven to refusal on the underlying limestone. The foundation was designed in accordance with the customary practice, and the piles were driven to provide the required support with ample safety. For years the bridge supported heavy traffic with no evidence of distress. Suddenly, two bents of the substructure dropped out of sight. Sections of the deck draped into the water, and one span was lost entirely. This occurred so quickly that automobiles traveling on the bridge ran off the open end into the water. Seven automobiles were lost and several people were drowned. This failure was apparently caused by the collapse of the underlying limestone, which supported the pile tips. Possibly, there was a thin rock arch overlying a cavity. There was enough continuing solution to enlarge the cavity or to weaken the rock until the mass collapsed. Although the bridge weight probably contributed to the extent of the collapse, there is nothing to indicate that the bridge was responsible for the geologic processes that brought about failure.

Foundation failures from erosion into underground cavities and sewers, mine subsidences, and similar phenomena involve the complementary system and are not influenced by the design safety factor of the foundations.

Occasionally, a poorly designed adjacent structure or faulty construction operation causes a failure that influences the primary structure under consideration. For example, the sudden collapse of foundations overlying cavernous limestone has been triggered by pumping water from those cavities remote from the point of failure. The lowering of the water table at one site may be responsible for serious settlement and even failure at an adjoining site. The shock, vibration, and changes in stress caused by building on one site may be reflected in changes in the soil conditions and the behavior of structures at some distance.

Failure of the structure produced by activity on adjoining sites is not always not related to the design safety factor of that structure. In some cases, ill-advised provisions to enhance the safety of the primary structure can even lead to its failure. An example of this occurred at a large country club. The club building and the adjoining swimming pool were on the top of a hill. An excavation had been made at the toe of the hill to provide a large level area for tennis courts. As a result, the slope of the hill was increased. Small landslides occurred in the slope following heavy rains. These endangered the swimming pool, although it had been designed structurally to resist some loss of support from movement in the hillside. (In this case, the increased safety of the pool saved it in spite of the continuing slides.) The club was determined to eliminate the increasing danger to the pool and directed an engineer to prepare plans for a retaining wall to support the slope. In preparing the plans, the engineer disregarded soil data showing that the hill consisted of alternate strata of sand and clay, with water under slight pressure confined within the sand seams. The design required a crib retaining wall backfilled with the "best clay-gravel" that could be obtained. The new wall was a dam that prevented the exit of water seeping through sand seams in the hillside. The pressure built up in the sand until eventually a major landslide occurred that seriously endangered the swimming pool and destroyed the very wall that was designed to protect the pool.

The complementary system, therefore, must be considered vital in the ultimate safety of any structure, even though the effects of the complementary system cannot be expressed in terms of a simple safety factor of the structure.

NONFAILURE

Failure of a structure may be caused by movement within some portion of the system that is not related to a failure of that portion of the system. The elastic deflection of the soil in the face of a braced excavation that causes settlement of the adjoining structures is a good example of such a nonfailure producing a failure. In one such case, the excavation for a new office building was within 8 feet of the outside wall of an old brick wall bearing structure. The contractor for the office building concluded that the old brick structure was too weak to withstand underpinning. Therefore, he undertook to protect the old building, its foundation, and the soil supporting its foundation by a strong bracing system. This was a reasonable solution because the foundation level of the old structure was not far above the ultimate excavation line of the new building. The bracing system consisted of interlocking concrete cylinders, installed by augering holes and filling them with concrete with appropriate reinforcement. The upper ends of these cylinders were supported by diagonal rakers, resting on foundations within the new building site. Shortly after the excavation was complete, movement was noted in the foundations of the old building. The foundation settled slightly and moved laterally toward the excavation about ½ inch. An analysis of the bracing system showed that the safety factor was adequate. However, it was necessary for the bracing system to deflect in order for it to mobilize any resistance to lateral movement. The bracing system (including the soil, the vertical interlocking cylinders, the steel rakers, and their foundations) absorbed considerable movement before their resistance was mobilized. Although the movement was sufficient to cause damage to the adjoining structure, the elastic deflection was unrelated to the safety factor of either the new building, the bracing system, or the old foundations. Failure could have been prevented by prestressing the bracing system to minimize deflection.

Consolidation of the soil is another factor in damage caused by nonfailure. Ordinarily, the abutment of a bridge that acts as a retaining wall to support an approach fill is designed to resist active earth pressure. The earth pressure causes the abutment to tilt outward away from the fill, a movement necessary to produce the active pressure. However, such a fill is frequently placed above a weak, compressible soil. In one instance, the consolidation of the compressible soil under the weight of the bridge caused the abutment to tilt away from the bridge far enough that the approaches had to be reconstructed. Equally serious is the effect of the movement on the earth pressure. The active earth pressure used on design presumes a small outward tilt of the abutment. The inward or reverse tilt could raise the pressure and create loads for which the abutment was not designed.

A subsidence produced by environmental changes is another form of nonfailure that is not related to the foundation safety factor. Yet a subsidence can cause a structural failure of major magnitude. Changes in the groundwater level produced by long-term changes in climate or by drainage that accompanies construction in large cities cause increased effective soil stresses and consolidation of soil strata. Although the most susceptible layers are clays, such consolidation settlements do occur in sand and silts.

Rapid fluctuations in the water table in a coastal city produced severe settlement cracks in a church that had no signs of distress for the first 50 years of its life. A long-term dry spell accompanied by drainage of a deep excavation nearby, followed by several periods of very wet weather, caused severe changes in the water table and subsidence of the heavy load-bearing walls of the building and accompanying cracks.

Drainage-induced consolidation frequently accompanies deep excavations that require well-pointing or other forms of accelerated construction drainage. The drawdowns associated with shallow wells sometimes produce settlement in adjoining structures. Occasionally, the rapid drawdown accompanying high rates of well pumping induces such severe gradients that seepage erosion occurs. The settlements in such cases are likely to be sudden and disastrous. The erosion-induced subsidence in this way differs from the progressive settlement produced by the effective stress increases resulting from drainage.

Poorly compacted backfill adjacent to bridge abutments and around drainage structures is a frequent cause of delayed settlement in highways. The loose soil when dry may be relatively strong and incompressible. When it becomes inundated because of changes in environment the hard lumps soften and rapid settlement takes place. Settlements of several inches to several feet are not uncommon in poorly compacted dry backfills. Dropouts in pavements above uncompacted utility trenches are common in nearly all cities. Such settlements occur rapidly, immediately after the change in environment. The damage resulting from such settlement is not directly related to the safety factor of a foundation or pavement supported on the compacted fill. Instead, the subsidence and resulting damage take place regardless of the safety factor. It is frequently difficult to differentiate between such subsidences and the rapid downward movement produced by a bearing-capacity failure. Where a foundation is directly supported by a dry, poorly compacted soil, the weakening of the soil upon inundation also could produce a bearing-capacity failure. The safety factor required to insure against such a failure due to soil softening upon inundation would be excessive, however.

BENEFICIAL FAILURE

Although failure is generally considered to be bad, some "failures" are beneficial. For example, the driving of pile requires successive bearing-capacity failures in order to advance the tip of the pile through the soil strata. Although successful driving of the pile requires failure of soil, it must not produce failure of the pile shaft. During driving the safety factor of the soil will be 1; the safety factor of the pile shaft at the same time must be sufficiently greater than 1 so that the pile shaft does not fracture under the hammer impact. The completed pile, therefore, must represent an unbalanced design with the pile shaft having a greater safety factor than the soil surrounding the pile. This has been ignored by some engineers, who have attempted to make a balanced design with all the safety factors equal. The result has been damage to the pile shaft during driving.

Certain engineering analyses require soil failure as a part of the ultimate design. For example, the design of a retaining wall for active earth pressure requires that the soil behind the wall fail in shear sufficiently to mobilize the strength of the soil. The wall, on the other hand, must have a safety factor large enough that it will not fail or move inordinately under the reduced pressure developed through soil shear. Paradoxically, it also requires that the wall be able to deflect enough under the loads produced by the failing soil that sufficient shear is developed to establish the active state. When this deflection of the wall is ignored, enough cracking and local failure will develop in the wall structure so that the required movement can occur. In this case, the wall may not necessarily be really failing; instead, it is moving as required by the design assumptions. The movement is frequently accompanied by cracks in the soil mass behind the wall, causing alarm to all concerned. If a structure is placed on the wall before the backfill is complete or if a structure's foundation is placed in a lower part of the backfill before the backfill is complete, the structure will move with the shearing soil and will suffer damage. In such a case, the circumstances include a beneficial failure in the soil accompanied by required deflection in the retaining structure but producing a damaging failure in an adjoining structure. In this case, an additional safety factor in the design of the foundation placed in the backfill or a structure supported on the wall could not prevent its movement and damage.

Occasionally, a failure is beneficial in that it gives warning of serious trouble that is developing or provides a safety valve that prevents further failure. A highway fill placed on hillsides underlain by water-bearing strata of sand can act as a dam and cause the water pressure to build up in the blocked strata. If the water pressures are great enough, the soil strength will be reduced until it is less than the stresses imposed and failure follows. If the failure involves the movement of the embankment, it may uncover the blocked stratum and allow drainage. In one such case, a large highway embankment slid down a mountainside after a clay fill prevented drainage from thin seams of jointed sandstones sandwiched between impervious layers of shale. The failure continued for several years as additional clay fill was placed over the pervious seam in order to keep the roadway at the proper elevation. Finally, someone hit on the idea of maintaining the roadway elevation by a pervious fill. The continuing movement of the embankment down the hill combined with refilling eventually brought the new pervious fill to the level of the water-bearing stratum. Thereafter, the rate of movement was much slower.

THE OVERALL VIEW

The Illusion of Safety

The foregoing discussion illustrates the problems associated with establishing safety factors for design. Too often, the safety factor is an illusion—an imaginary crutch that helps the designer over the difficult point of evaluating the unknown forces, the uncertain resistances, and the inevitable inaccuracies of engineering analyses. Unfortunately, the continuing use of safety factors without their accurate verification by detailed studies of failures can lead the engineer to the illusion that the numerical value of the safety factor is a real measure of the margin of safety of the structure. The illusion becomes a deception when the engineer is pressured into reducing the safety factor because of economic considerations or because other design disciplines (which confirm their real safety factors by pilot tests of full-scale models) can get by with lower safety factors as well as an occasional failure. The aircraft industry can afford to use a small safety factor because it checks the overall safety factors by test flights. The occasional inadequacy of the original safety factors is demonstrated by the fact that test pilots are very well paid and by the fact that sometimes aircraft must be recalled for modification after a rash of failures shows some deficiency.

The engineering profession must recognize that a computed safety factor greater than 1 does not insure safety. Moreover, a computed safety factor somewhat less than 1 does not necessarily mean that failure is inevitable.

The Elusive Nature of Safety

The safety factor is elusive because many of the factors that contribute to the safety of a foundation cannot presently be evaluated with accuracy. This elusive aspect of safety has led to a probability approach wherein the safety factor is related statistically to the reliability of the test data on the strength of the material, the reliability of the loading, and the probable errors in the computations. Such a probability approach is reasonable in manufactured products where the possibility of failure can be tolerated and where the cost of failure versus the value of safety can be evaluated statistically. However, the statistical possibility of failure of a major engineering structure, such as a dam, cannot be evaluated. In the first place, a statistical analysis becomes unreliable at the extremely low probabilities that must be considered in such a design. Furthermore, there is every reason to believe that there are upper limits for the forces that might be involved in engineering problems, whereas statistical analyses consider that even an unreasonable magnitude of force is statistically possible. The statistical approach, therefore, is appropriate only to those loads that occur frequently enough that a valid statistical analysis is possible. Until enough failures can be analyzed that a valid statistical analysis is possible, a statistical evaluation of safety will be impossible.

The admission of possible failure implied by statistical analysis raises a question of public response. The public apparently is reasonably content to deal with the statistical possibilities of individual accidents. The statistical possibility of the failure of a bridge or dam in a populated area, however, is probably inadmissible.

THE COMPONENT APPROACH

An interesting approach to the safety factor was suggested by Brinch Hansen in 1961 (1). He proposed varying safety factors to be applied to the loads acting on the structure as well as to the various soil properties used in analysis. (He did not consider the use of a safety factor to compensate for inaccuracies in analysis, however.) For the dead load on a structure he proposed a safety factor of 1.0 because the dead load should be capable of precise evaluation. For design, live load should be increased 50 percent to allow for unknown variations. For groundwater loads, the increase should be 20 percent. The author cannot agree with the latter recommendation because water loads can involve a greater degree of unknown than other live loads unless there are physical limits to the level to which the water can rise.

Safety factors are also applied to the components of soil strength individually. The apparent cohesion of a clay soil is divided by a number ranging from 1.5 to 2, depending on the accuracy of the soil tests and the sensitivity of the material. The tangent of the angle of internal friction (or factors derived from the angle of internal friction) is reduced by an amount equivalent to dividing the tangent of the angle of internal friction by 1.2. This is considered reasonable, because the range of the angle of internal friction in most soils is rather limited. It is the author's opinion that this approach is sound but that additional components of safety factor are necessary because of the uncertainties in the accuracy of the engineering analyses that are used.

THE EMPIRICAL APPROACH

Because of the illusory and elusory nature of the safety factor, the shortcomings of the statistical approach, and the uncertainties in the accuracy of engineering computations, the author prefers the traditional empirical approach of an overall safety factor developed from experience. Most practicing engineers utilize a safety factor derived from their own experience. After years of experience it is possible for the engineer to determine if that safety factor is inordinately low by the incidence of failure resulting from his designs. If he has no failures, he may congratulate himself that his safety factors have been adequate; on the other hand, he may merely have been unduly conservative.

A proper use of the empirical approach requires that there be a full study of all failures that occur so that the source of the error, if any, can be pinpointed and the uncertainties involved in loading, evaluation of resistances, and engineering analyses can be established. While such postmortems are embarrassing to those directly involved, such failures are the chief source of full-scale tests. Occasionally, the engineer is given the luxury of making a full-scale test of a structure, loaded so as to produce failure. When such an occasion arises, the engineer is professionally obligated to study that failure extensively and to make the results of the failure known to the profession. More often, the study of controlled failures is limited to models. Frequently, the models are so small that extrapolation of their results to full-scale structures is questionable, if not hazardous. Properly conducted and thoroughly evaluated large-scale tests, however, offer much promise in determining what the safety factors ought to be for engineering design. Unfortunately, in new situations the engineer is still confronted with a lack of empirical data and adequate analyses. In such cases he must rely on his intuition and good fortune.

Failure is a risk inherent in all endeavor, whether it be the design and building of an engineering structure or stepping across a crowded city street. The risk of failure can be eliminated only be eliminating endeavor itself. Unfortunately, the growing number of lawsuits filed against engineers in cases of failures or near-failures will only lead to more conservative and expensive designs and a stifling of initiative in new engineering developments. This will be a disaster for the engineer as well as for society.

The alternative is for society as well as the engineer to face the fact that engineers are not infallible and that failures occasionally occur in spite of the best designs. The risk of failure must be pointed out to those who request such designs and ultimately the owner and society as a whole must accept the responsibility for the risk rather than the designer, because ultimately the owner (and society) must bear the responsibility for the risk of failure, since they reap the benefit for the greater chance of success.

The engineer is obligated to minimize the risk by using his best talents with the most advanced engineering knowledge suitable to the task. He then must acquaint the owner with the possibilities of failure and the possible consequences. The owner is obligated to weigh these against the value of the completed structure. Only when the risk of failure is faced honestly by all concerned can proper designs be evolved.

Absolute safety is a myth. A quantitative statistical evaluation of safety is not technically feasible when enough full-scale failures to establish a valid statistical analysis cannot be tolerated. Instead, the profession must rely on its intuition and experience and make use of the knowledge gained from a full investigation of every failure that presents itself. Such a program can lead to more reliable safety factors as well as lower safety factors and cheaper structures, but rapid changes are not likely.

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