REFERENCES

1. J. E. Acton. Assessment of Hazards Associated with Highway Engineering. Proc., Institute of Civil Engineers, Vol. 64, Part 1, Aug. 1978, pp. 381-391.
2. J. R. Benjamin. Applied Statistical Decision Theory for Dam Construction. Presented at Conference on Dam Safety, Stanford Univ., Palo Alto, CA, 1978.
3. S. O. Russell. Civil Engineering Risks and Hazards. British Columbia Professional Engineer, Jan. 1976.
4. H. Raiffa. Decision Analysis. Addison-Wesley, Reading, MA, 1970.
5. E. Hoek and J. W. Bray. Rock Slope Engineering, 2nd ed. Institute of Mining and Metallurgy, London, 1977.
6. S.-M. Chin. Cost as a Criterion for Evaluating Highway Level of Service. ITE Journal, Aug. 1978.
7. E. Kreyszig. Advanced Engineering Mathematics, 2nd ed. Wiley, New York, 1967, Section 18.10.
8. Pit Slope Manual: Chapter 5—Design. Canada Center for Mineral and Energy Technology, Department of Energy, Mines, and Resources, Ottawa, Canada, 1977.

Publication of this paper sponsored by Committee on Exploration and Classification of Earth Materials.

Abridgment

Advantages of Founding Bridge Abutments on Approach Fills

D. H. Shields, J. H. Deschenes, J. D. Scott, and G. E. Bauer

A set of controlled experiments has been carried out in which the ultimate bearing capacity at various locations within a granular approach fill for a spill-through bridge abutment was measured. It was shown that existing design procedures for spread-footing-supported abutments in approach fills are unduly conservative, and it is recommended that the experimentally determined bearing-capacity values be used as the basis for design.

Footing foundations would be competitive in cost with piled foundations for spill-through bridge abutments if the design bearing pressure for footings near slopes could be increased. That is, if the allowable bearing pressures could be located closer to the end slope of the approach fill, the resulting bridge length would be comparable to that of a bridge having a pile foundation.

Current bearing-capacity limits are based on theoretical considerations. This paper describes a set of controlled experiments in which the ultimate bearing capacity at various locations within a granular approach fill was measured. It was found that the theoretical approach seriously underestimates the capacity of footings close to the crest of a slope. Present indications are that piles can be omitted from existing spill-through abutment design, and the abutments can be placed directly on select, well-compacted gravel at lower cost. A concomitant benefit is that a footing-supported abutment and fill will settle as a unit; this will eliminate the maintenance cost often associated with bridge approaches that settle while the bridge itself does not.

In 1978, an actual underpass structure was built to a new design based on the tests reported here. The behavior of the structure is being monitored, and its performance will be compared with that of the corresponding model.

DEFINITION OF THE PROBLEM

Generally, one distinguishes two basic types of abutments—the retaining and the spill-through. In a retaining abutment, the approach fill is contained within the vertical abutment wall and the wing walls, whereas in the spill-through abutment, the approach fill is self-supporting and the bridge appears to rest on the fill near the top of the end slope. In fact, in the majority of cases, the bridge does not rest on the fill but is, instead, supported on piles that extend down through the fill to the natural soil or rock.

Why Use Piles?

Economics plays a large role in the design of bridges, in particular in the design of fairly routine highway and railway bridges of the overpass type. Based on present design practices, the economic advantage is nearly always in favor of founding spill-through abutments on piles rather than on spread footings. Generally, the bridge on spread footings is longer than the bridge on piles and the spread-footing alternative requires a fairly large zone of more-expensive, compacted select fill.

To design a spread footing for a spill-through abutment, the designer must resolve the dilemma of determining the probable ultimate capacity and settlement of the footing. At present, there are at least eight bearing-capacity theories that engineers can use, and all eight purport to take into account the effects of the proximity of the sloping face of the approach fill. The problem is that all eight give different answers.

Most of the theories are applicable only to a footing located right at the crest of the slope; only two—those of Meyerhof (1) and Giroud (2)—treat the general problem of the capacity anywhere within a slope and also use acceptable analytical techniques. Because it is unlikely that a designer would locate an abutment footing right at the crest at the end slope of the approach fill, Meyerhof's and Giroud's theories are the most widely used for design. Even then, the difference between the two theories can be considerable—particularly in dense material within the region close to the crest of the slope.

Figure 1. Basis for cost comparison.

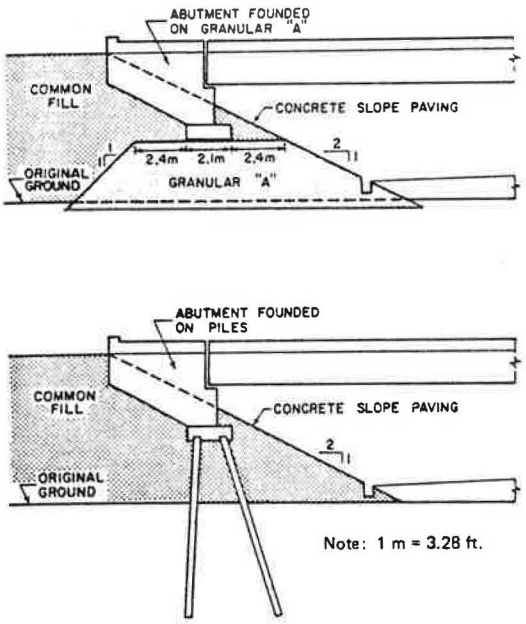
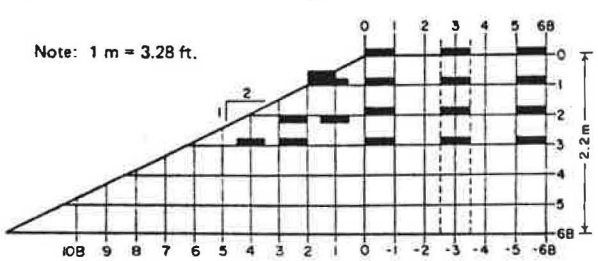


Table 1. Cost comparison for abutment footings: typical highway underpass structure.

Ontario Location	On Piles ($/bridge)	On Spread Footing ($) Per Cubic Meter of Fill	On Spread Footing ($) Per Bridge	Saving ($/bridge)
Southwest	17 900	10.80	16 650	1 250
East	16 400	5.25	8 800	7 600
Central	16 950	7.85	12 300	4 650
North	20 000	3.90	7 000	13 000

Notes: 1 m^3 = 1.31 yd^3.
Costs in 1976 Canadian dollars (1.00 1976 Canadian dollar = 1.01 1976 U.S. dollars).

Figure 2. Locations of test footings.



A New Look at the Footing Alternative

If experimental proof were available to show that existing design procedures for spread-footing-supported abutments in approach fills are unduly conservative, what economic benefit would there be in choosing spread footings over piles? This question can be approached as follows: Consider the possibility that a spread footing could be designed to carry an abutment located at the exact position of a pile-supported abutment; this would mean that the bridge length and thus cost would be the same. Consider also the possibility that the width of the footing would be equal to the width of the pile cap; this means that the abutments themselves would be the same, and they would cost the same. Thus, comparison of the costs of pile- and footing-supported abutments will be limited to comparing the cost of the piles and their installation with the cost of the select granular zone under the footing. The two systems compared are shown in Figure 1. The piled foundation is made up of thirty-eight 32.4-cm (12.8-in) outside-diameter, 9-m (30-ft) long, steel tube piles, each designed to carry 22.7 Mg (25 tons), and the spread-footing foundation is a 3.4-m (10-ft) thick zone containing approximately 1150 m^3 (1500 yd^3) of granular A. The cost-comparison results are shown in Table 1 (this comparison makes allowance for the fact that common fill would be required for the piled foundation so that the cost for granular A shown is the difference in cost between granular A and common fill).

The conclusions that can be drawn from this are that

1. In all cases, there is a saving in selecting the spread-footing foundation and
2. The magnitude of the saving depends on the cost of granular fill at the bridge location.

If longer piles were required to reach the bearing stratum or if cheaper gravel could be used (or both), the savings would be even greater.

Another advantage of using spread footings to carry bridge abutments in approach fills is that the bridge and the fill will settle together, and there will be no bump such as occurs when the abutment and the fill settle differentially. Of course, a settling abutment may not be desirable for a multispan continuous bridge structure.

PROVIDING THE PROOF

Given the worthwhile savings that could result from a change in spread-footing design practice, the Ontario Ministry of Transportation and Communications and the geotechnical group at the University of Ottawa conducted large-scale experiments to measure the bearing capacity of spread footings adjacent to a 2:1 slope of granular material—the standard design slope of the ministry. Two test series were envisioned—one series in compact material and another series in dense material. A grid of 12 footing locations was chosen for the tests (see Figure 2), and it was decided to make the footings long in comparison with their width to simulate an infinitely long, continuous strip footing in the tests.

Previous research (3) on footings on flat ground had indicated that a 300-mm (1-ft) wide footing is the minimum that can be used on granular soil to simulate a full-scale footing. This dimension controlled the size of the bin that was required in which to perform the tests. The actual bin (or sand box) is 15 m (50 ft) long, 2 m (6.6 ft) wide, and 2.2 m (7 ft) high, but it is divided into two equal-length bins 7.5 m (25 ft) long to facilitate material storage between tests. The width of the sand box was arbitrarily fixed at six times the width of the footing; because the footing stretched from one side of the bin to the other, the sides of the bin were made rigid so that the soil could move only in plane strain. The length of the test was ample to allow full development of the failure zone from under the footing out into the slope. [The test arrangement is described in detail by Shields and others (4).]

To overcome the potential error due to friction on the sides of the box, the footing was made in three equal parts or segments.

The next decision that had to be made was the choice of the granular material. Ideally, granular A should have been chosen but, because each test required moving 40-50 Mg of material, making obvious that mechanical handling would be required, and the equipment to pick up,

Figure 3. $N_{\gamma q}$ contours for dense sand.

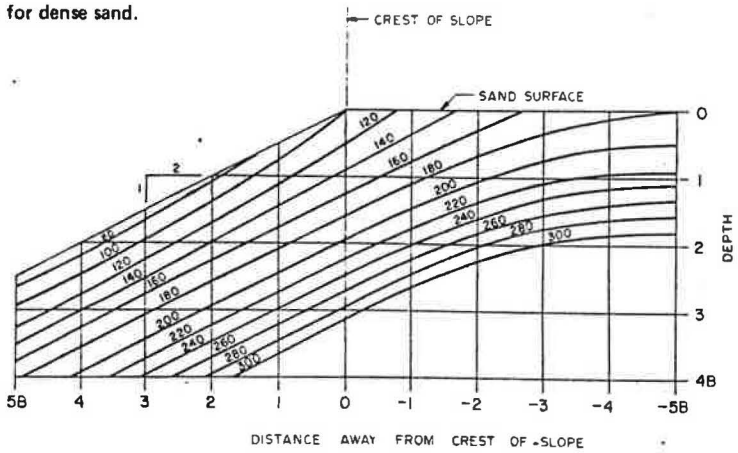
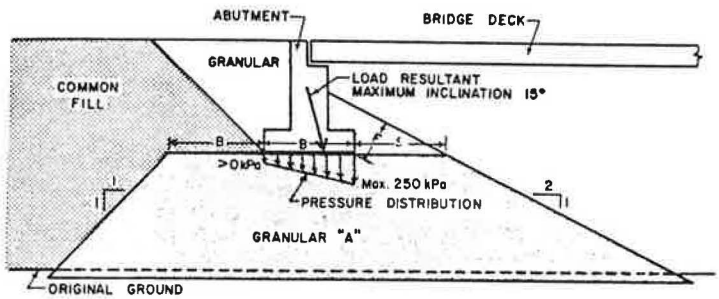


Figure 4. New (1978) bridge design.



s depends on minimum frost depth F.

Note: 1 kPa = 6.89 lbf/in^2.

transport, and deposit the material in a controlled fashion could handle only sand, a crushed quartz sand was chosen to simulate the granular A. Tests showed that the quartz sand had a lower strength (lower angle of internal friction) and was more compressible (lower Young's modulus) than either crushed stone granular A or crushed gravel granular A at the same relative density. This meant that the footing tests on the sand would give results that were conservative with respect to the performance on gravel.

Test Procedure

Increments of load were applied to the footing until failure was reached. Each of the three footing sections was loaded independently, but all sections were forced into the sand at the same rate. The load on the middle section was recorded, as was the amount of settlement of the footing under each load.

Measurements were also made of the movement of the surface of the sand.

Test Results

Because the test results would have to be scaled up from the 300-mm-wide footing used in the sand box to field size [typically 1.8-4.6 m (6-15 ft)], the test results are presented in terms of $N_{\gamma q}$ contours, where $N_{\gamma q}$ = experimental bearing-capacity factors. Figure 3 shows the experimental results for dense sand. These results do not agree with either Meyerhof's or Giroud's theoretical values.

DESIGN RECOMMENDATIONS

For the present, it is recommended that the experimental $N_{\gamma q}$ values be used empirically as the basis for the design of abutment footings in approach fills. It seems, therefore, that bridge lengths can be comparable for both pile- and footing-supported abutments and that the economic advantages of footings will in fact be realized.

The bridge structure built in 1978 to the new specifications is shown in Figure 4. The movement of the structure is monitored, as is the overall settlement behavior of the approach fill. The structure is instrumented to measure the distribution of bearing pressures on the foundation and the earth pressures on the vertical surfaces. One goal is to ascertain the actual direction and location of the resultant force on the foundation.

As a cautionary note, it must be noted that many spill-through abutments act partly as retaining walls; the resulting horizontal earth pressures lead to foundation loads that are inclined and eccentric. A series of tests have recently been completed in the sand box to determine the reduction in bearing capacity that is brought about by inclining the load 15° to the vertical, and size effects have been studied by carrying out tests in which a 600-mm (2-ft) wide footing was used.

ACKNOWLEDGMENT

The financial support of the National Research Council of Canada, which enabled the test facility to be built and maintained, is gratefully acknowledged. GKN Keller (Canada) Limited donated the sand spreader. The project was financed by the Ontario Ministry of Transportation and Communications. J. Keen of the ministry supplied the cost estimates for the comparison between footing and pile foundations.

REFERENCES

1. G.G. Meyerhof. The Ultimate Bearing Capacity of Foundations on Slopes. Proc., 4th International

Conference on Soil Mechanics and Foundation Engineering, London, England, Vol. 2, 1957, pp. 301-332.
2. J.P. Giroud and Tran-Vo-Nhiem. Force Portante d'une Fondation sur une Pente. Annals d'Institute Technique Bâtiment et des Travaux Publiques: Serie—Théorie et Methodes de Calcul, Paris, No. 142, 1971.
3. Y. Lebegue. Essais de Fondations Superficielles sur Talus. Proc., 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, U.S.S.R., Vol. 4.3, 1973, p. 313.
4. D.H. Shields, J.D. Scott, G.E. Bauer, and A. Barsvary. Bearing Capacity of Foundations near Slopes. Proc., 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, June 1977.

Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.

Pile Design and Installation Specification Based on Load-Factor Concept

G. G. Goble, Fred Moses, and Richard Snyder

The use of load-factor procedures for the design of bridge superstructures is expanding rapidly. However, substructure design is still based exclusively on allowable stress methods. This paper presents an approach to load-factor design for pile foundations. The load factors suggested follow the current American Association of State Highway and Transportation Officials recommendations, while the resistance factors recommended are based on the capacity-determination methods and the construction control procedures used. Actual values are selected to be consistent with currently used procedures where they are available. The proposed specification can provide a framework for the use of more-appropriate resistance factors as they become available from ongoing research.

About two decades ago, dramatic changes began to occur in structural design philosophy. Before that time, structural designers sought to develop structural systems that would resist the effects of expected load applications with no structural distress. This was achieved by requiring that the stresses calculated by an elastic analysis of the structure when subjected to the expected design or working load not exceed some accepted, allowable stress. These allowable stresses were usually defined either explicitly or implicitly as a fraction of the yield or ultimate strength of the material involved. The fact that the loads were statistically distributed with substantially different probabilities of occurrence of different types of loads was ignored. Design loads were developed, and their effects on the structure were analyzed deterministically.

There are clear advantages to this approach. The structure is subjected to an elastic analysis, and the limit on allowable stresses is placed well below the elastic region, so it can be expected that, even though the structural engineer is primarily concerned with the design of a structure having sufficient strength, many serviceability questions will be satisfied indirectly. For instance, in such an approach, one can expect that deflections will be tolerable and acceptable. The structure is subjected to elastic analysis and, therefore, indirectly deflections are controlled.

Another important but less understood advantage of an elastic-analysis-and-working-stress approach is that there is a clear and direct redesign process available to the structural engineer. Those portions of the structure that are found in the analysis to be overstressed can be increased in size while parts of the structure where stresses are less than the allowable can be decreased in size. This approach provides a simple redesign algorithm.

There are also important disadvantages to working-stress design. For instance, a statically indeterminate structure that has a high degree of redundancy will have a different factor of safety to collapse than will a statically determinate structure. When such structures are designed by working-stress procedures, the actual factor of safety (FOS) for a particular structure can vary considerably. Because the loads that must be carried by the design can come from a variety of sources, the accuracy and reliability of the determination of their magnitude can vary widely. Likewise, our ability to predict the behavior of various types of structural elements varies, as does the consequence of failure (the collapse of a column is usually more serious than is a beam failure). There are other considerations that motivate the change in practice. For instance, the behavior of reinforced concrete members does not satisfy working-stress analysis because of time-dependent and inelastic deformations.

On the other hand, if working-stress analysis is completely abandoned for an exclusively strength-design-based procedure, then difficulties can arise with other performance aspects of the structure. For example strength evaluation procedures completely neglect questions of deflection.

In summary, traditional working-stress design procedures have come under criticism because they do not recognize the statistical distribution of loads and the nondeterministic character of structural-element strength. These factors, together with considerations of the varying consequences of failure for different element types, all point to the need for design procedures that will produce factors of safety that include these consequences.

One solution is the procedure known as limit-state design or load-and-resistance-factor design. This procedure deals directly with the questions involved in structural design. The structure is designed to satisfy requirements of strength and serviceability, both directly and separately. By serviceability in structural design,