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Integral Abutment Bridges

Early Iron Bridges

Lightweight Concrete Roof Slabs

Engineering Design Using Spreadsheets

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Electron Inactivation of Pathogens
Concrete Formwork
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Early Iron Bridges

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Design Considerations

General Design Details for Integral Abutment Bridges

Extensive damage caused by faulty or leaking bridge deck expansion joints has resulted in progressively increasing maintenance and rehabilitation costs. To avoid these problems, more than half of the state highway departments have developed guidelines for the design and construction of integral abutment bridges.

Traditionally, a system of expansion joints, roller supports and other structural releases has been provided in highway bridges to permit thermal expansion and contraction. An example of a bridge with expansion joints is shown in Figure 1. The desirable characteristics of an expansion joint system are water-tightness, smooth rideability, low noise level, wear-resistance and resistance to damage caused by snowplow blades. However, the performance of many joint systems is disappointing. When subjected to traffic and bridge movement, they fail in one or more important aspects, most notably in water-tightness.

The flow of runoff water through open bridge deck joints or leaking sealed joints has been one of the major causes of extensive maintenance and costly rehabilitation work on bridges in general. The problem is especially aggravated in the snowbelt states, where runoff water contains highly corrosive chlorides. An estimated 12 million tons of sodium chloride and calcium chloride are used annually during wintertime de-icing operations in the United States. This practice exposes bridge decks and substructural elements located near expansion joints to a highly aggressive corrosive environment.

Critical substructural elements that are commonly damaged by water runoff through expansion joints include steel girders and stringers, bearings, rollers and anchor bolts. In addition, reinforced concrete members such as piers and pier caps are often subjected to scaling and spalling caused by deck runoff through joints, which subsequently leads to the cor-
Continuity in steel stringer and other types of bridges has been an accepted practice since the early 1950s. In addition to the economies inherent in the use of continuous beams, wherein negative moments over interior supports serve to reduce midspan positive moments, one line of bearing devices was automatically eliminated at each interior support. The predominant problem with these continuous bridges was at the abutments where some kind of expansion joint was still used.

In order to alleviate the high costs associated with bridge maintenance and the rehabilitation of joints, several states began eliminating them altogether from bridges by building the girders integrally with the abutments. Figure 2 shows an example of a bridge with integral abutments and Figure 3 shows details of typical integral abutments. Each bridge is supported by a single row of vertical piles extending into the abutments. In addition to being aesthetically pleasing, integral abutments offer the advantage of lower initial cost as well as lower maintenance costs. Expensive bearings, joint material, piles for horizontal earth loads and leakage of water through the joints are all eliminated.

Today, more than half of the state highway agencies have developed design criteria for integral abutment bridges. Development of these bridges began on an experimental basis on rela-
FIGURE 2. Cross-section of a bridge with integral abutments.

tively short bridges ranging from 50 to 100 feet in length. Because rational design guidelines were not available, any subsequent increase in allowable length was based on reports of the successful performance of prototype bridges in the field. As a result, each highway agency has developed its own unique length limits and other design criteria. In addition to the erratic limits for the allowable lengths of integral abutment bridges, design guidelines have also been lacking for the treatment of other essential components such as approach slabs, piles, pile caps, wingwalls, backfill and provisions for drainage.

Responses to previous surveys concerning the use of integral abutments have indicated

FIGURE 3. Typical integral abutment details.
that most state highway departments have their own limitations and criteria in designing integral abutments.\textsuperscript{2,3} The bases of these limitations and criteria are shown to be primarily empirical. The use of integral abutments in bridge design has so far been accepted by 28 state highway departments and the District Construction Office of Federal Highway Administration (FHWA) Region 15. Information on general policy concerning integral abutment design, provision for bridge movement, approach slabs, wing-wall configurations and details, and general design details and guidelines by highway departments in Tennessee, New York, and California, and the Federal Highway Administration is provided from survey responses. A summary on current practice by all the 28 states and the District Construction Office of FHWA Region 15 is given in Table 1.

**General Policy on Integral Abutment Design**

**Tennessee.** Structures must be designed to accommodate the movements and stresses caused by thermal expansion and contraction. Bridge designers should not accommodate these movements by using unnecessary bridge deck expansion joints and expansion bearings. This solution can create more problems than it solves. Structural deterioration due to leaking expansion joints and frozen expansion bearings constitute major bridge maintenance problems.

To eliminate the problems associated with leaking expansion joints and frozen expansion bearings, bridges must be designed and constructed with continuous superstructures, fixed or integral bearings at the piers and abutments, and no bridge deck expansion joints unless absolutely necessary. When expansion joints are necessary, they are provided only at abutments.\textsuperscript{4}

**New York.** The New York Department of Transportation currently has tentative integral abutment guidelines that list the design parameters that must be satisfied by designers if they elect to use an integral abutment type structure. Integral abutments are allowed on structures with span lengths up to 300 feet provided they satisfy the tentative guidelines. Span lengths between 300 and 400 feet are approved only on an individual basis.

The main concern regarding span length is the longitudinal movement and the large passive pressures that are generated as the structure expands against the compacted backfill. The general policy is to try to select a span arrangement and bearing types that result in approximately equal movements at each abutment. The 300-foot limitation results in movements that can safely be handled.\textsuperscript{5}

**California.** The end diaphragm is an integral part of the bridge superstructure. Frequently, this diaphragm is extended below the soffit of the superstructure to rest directly on piles or on a footing. This type of support is an *end diaphragm abutment.* An end diaphragm abutment cannot be used where the roadway on the structure is designed to carry storm water.\textsuperscript{6}

**Federal Highway Administration.** Bridges with their overall length less than the following values should be constructed continuous and, if unrestrained, have integral abutments:

- Steel: 300 feet
- CIP: 500 feet
- Pre- or post-tensioned concrete: 600 feet

Greater values may be used when experience indicates that such designs would be satisfactory.\textsuperscript{7}

** Provision for Bridge Movement**

**Tennessee.** The total superstructure movement should be based on the following design parameters:

- Concrete structures with 25 to 95°F temperature range, 0.0000060 coefficient of expansion and 0.505 inches/100 feet total movement.
- Steel structures with 0 to 120°F temperature range, 0.0000065 coefficient of expansion and 0.936 inches/100 feet total movement.

The total movement per hundred feet is applicable to the structure length measured from the theoretical fixed center of the structure.

When the total anticipated movement at an abutment is less than two inches and the abutment is not restrained against movement, no joint is required and the superstructure and
### Table 1
Summary of Current Practice

<table>
<thead>
<tr>
<th>State</th>
<th>Piling Stresses Due to Lateral Movement Are Calculated</th>
<th>Design Considerations</th>
<th>Criteria for Maximum</th>
<th>Design Assumptions &amp; Details</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alaska</td>
<td>Only for long bridges</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>On the basis of experience</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel: ≤ 300 ft.</td>
<td></td>
<td></td>
<td>Hinge</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete: ≤ 400 ft.</td>
<td></td>
<td></td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>Arizona</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>On the basis of experience</td>
<td></td>
<td></td>
<td>Hinge</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel: ≤ 253 ft.</td>
<td></td>
<td></td>
<td>No</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete: ≤ 303 ft.</td>
<td></td>
<td></td>
<td>Tied to abutment</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Prestressed: ≤ 400 ft.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>California</td>
<td>Piles are driven into pre-drilled holes &amp; success due to lateral movement are neglected</td>
<td></td>
<td></td>
<td>Partially restrained</td>
<td></td>
</tr>
<tr>
<td></td>
<td>On the basis of experience</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel: ≤ 240 ft.</td>
<td></td>
<td></td>
<td>Hinge</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete: ≤ 260 ft.</td>
<td></td>
<td></td>
<td>No</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Prestressed: ≤ 150 ft.</td>
<td></td>
<td></td>
<td>For bridge length ≥ 200 ft., use approach slab</td>
<td></td>
</tr>
<tr>
<td>Colorado</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>On the basis of experience</td>
<td></td>
<td></td>
<td>Fixed</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel: ≤ 200 ft.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete: ≤ 400 ft.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Connecticut</td>
<td>No</td>
<td></td>
<td></td>
<td>Free translation, free solution roller</td>
<td></td>
</tr>
<tr>
<td></td>
<td>On the basis of experience</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel: ≤ 400 ft.</td>
<td></td>
<td></td>
<td>Expansion joint between approach slab &amp; bridge slab</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete: ≤ 400 ft.</td>
<td></td>
<td></td>
<td>Expansion joint</td>
<td></td>
</tr>
<tr>
<td>Georgia</td>
<td>No</td>
<td></td>
<td></td>
<td>Free translation, free solution roller</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Based on total anticipated lateral movement of ≤ 1 ft.</td>
<td></td>
<td></td>
<td>Free drainage material</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel: ≤ 300 ft.</td>
<td></td>
<td></td>
<td>Free drainage material</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete: ≤ 400 ft.</td>
<td></td>
<td></td>
<td>Free drainage material</td>
<td></td>
</tr>
<tr>
<td>Idaho</td>
<td>Only for those that involve unique feature that would warrant such calcula-</td>
<td></td>
<td></td>
<td>Free drainage material</td>
<td></td>
</tr>
<tr>
<td></td>
<td>tion Based on FHWA guidelines &amp; the state's experience</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel: ≤ 200 ft.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete: ≤ 400 ft.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Prestressed: ≤ 400 ft.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Indiana</td>
<td>No</td>
<td></td>
<td></td>
<td>Free drainage material</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Steel: ≤ 150 ft.</td>
<td></td>
<td></td>
<td>Free drainage material</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concrete: ≤ 150 ft.</td>
<td></td>
<td></td>
<td>Free drainage material</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Prestressed: ≤ 150 ft.</td>
<td></td>
<td></td>
<td>Free drainage material</td>
<td></td>
</tr>
</tbody>
</table>

Bridges with integral abutments may be constructed with spread footings or piling. As longer bridges without expansion joints are found to be without problems, the length limit has increased to 400 ft. for concrete bridges.

As of May 1983, only one integral abutment has been designed & constructed. The design of this 245-ft. long, two-span continuous bridge was based on information received from the South Dakota Dept. of Trans.

Integral abutments have been used only at sites where steel H-piles are suitable. Steel H-piles are placed so that they bend about their weak axis.

Assume that passive earth pressure at abutments tends to restrain movement & reduce deflections from calculated values. Skewed 3-span steel girder bridge with integral abutment was built; rotational forces from lateral earth pressure on end walls caused failure in pier anchor bolts on exterior girder.

Only vertical piles are used with integral abutments. When bridge skew > 30°, length limit for concrete bridges is ≤ 100 ft. Integral abutments have been used for many years with no adverse experience. On longer bridges, the integral connection is eliminated, substituting a precast bearing pad or continued on next page.
<table>
<thead>
<tr>
<th>State</th>
<th>Design Consideration</th>
<th>Piling Stresses Due to Lateral Movement Assumed Calculated</th>
<th>Criteria for Maximum Length for Bridges With Integral Abutments</th>
<th>Design Assumptions &amp; Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indiana</td>
<td>Yes</td>
<td>Based on allowable bending stress of 55% of yield plus 30% overstress. Axial in pile found by a rigid frame analysis considering relative stiffness of the superstructure &amp; the piling. Assume piles to be 10.3 ft. &amp; neglect soil resistance. Analysis showed that allowable pile deflection was about 0.325 in. Steel: s; Concrete: s; Prestressed: s; 100 ft.</td>
<td>Fixed Neglect Neglect Roof-way fill</td>
<td>Comments: expansion device, use alternating vertical &amp; battered piles in the cap &amp; still neglect lateral forces on the piles.</td>
</tr>
<tr>
<td>Iowa</td>
<td>Yes</td>
<td>Based on allowable bending stress of 55% of yield plus 30% overstress. Axial in pile found by a rigid frame analysis considering relative stiffness of the superstructure &amp; the piling. Assume piles to be 10.3 ft. &amp; neglect soil resistance. Analysis showed that allowable pile deflection was about 0.325 in. Steel: s; Concrete: s; Prestressed: s; 100 ft.</td>
<td>Fixed Neglect Neglect Roof-way fill</td>
<td>Comments: expansion device, use alternating vertical &amp; battered piles in the cap &amp; still neglect lateral forces on the piles.</td>
</tr>
<tr>
<td>Kansas</td>
<td>No</td>
<td>Based on experience Steel: s $750-1000$ ft. Concrete: s $300$ ft. Prestressed: s; 100 ft.</td>
<td>Hinge File caps not used</td>
<td>Comments: Have used integral abutments for case-in-place bridge structures for many years &amp; have encountered no difficulties. Expect to increase length limits in the future.</td>
</tr>
<tr>
<td>Kentucky</td>
<td>No</td>
<td>Based on experience</td>
<td>Fixed or partially restrained</td>
<td>Comments: Have used integral abutments for case-in-place bridge structures for many years &amp; have encountered no difficulties. Expect to increase length limits in the future.</td>
</tr>
<tr>
<td>Missouri</td>
<td>No</td>
<td>Based on experience Steel: s; Concrete: s; Prestressed: s; 100 ft.</td>
<td>Hinge</td>
<td>Comments: Have used integral abutments for case-in-place bridge structures for many years &amp; have encountered no difficulties. Expect to increase length limits in the future.</td>
</tr>
<tr>
<td>Montana</td>
<td>No</td>
<td>Based on experience &amp; engineering judgement Steel: s; Concrete: s; Prestressed: s; 100 ft.</td>
<td>Hinge</td>
<td>Comments: Have used integral abutments for case-in-place bridge structures for many years &amp; have encountered no difficulties. Expect to increase length limits in the future.</td>
</tr>
<tr>
<td>North Dakota</td>
<td>No</td>
<td>Steel: s; Concrete: s; Prestressed: s; 100 ft.</td>
<td>Fixed</td>
<td>Comments: Have used integral abutments for case-in-place bridge structures for many years &amp; have encountered no difficulties. Expect to increase length limits in the future.</td>
</tr>
<tr>
<td>Nebraska</td>
<td>No</td>
<td>Based primarily on past experience &amp; recommendations from other agencies Steel: s; Concrete: s; Prestressed: s; 100 ft.</td>
<td>Hinge</td>
<td>Comments: Have used integral abutments for case-in-place bridge structures for many years &amp; have encountered no difficulties. Expect to increase length limits in the future.</td>
</tr>
<tr>
<td>State</td>
<td>Design Consideration</td>
<td>Piling Stresses Due to Lateral Movement Are Calculated</td>
<td>Criteria for Maximum Length for Bridges With Integral Abutments</td>
<td>Design Assumptions &amp; Details</td>
</tr>
<tr>
<td>---------------</td>
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<td>------------------------------------------------------</td>
<td>-------------------------------------------------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>Nebraska, cont.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Pile Head</td>
</tr>
<tr>
<td>New Mexico</td>
<td>No</td>
<td>-</td>
<td>-</td>
<td>Partially restrained or fixed</td>
</tr>
<tr>
<td>New York</td>
<td>No</td>
<td>Steel: ≤ 30 ft.; Concrete: ≤ 400 ft.</td>
<td>-</td>
<td>Hinge</td>
</tr>
<tr>
<td>Ohio</td>
<td>No</td>
<td>Based on experience &amp; engineering judgment Steel: ≤ 300 ft.; Concrete: ≤ 300 ft.; Prestressed: ≤ 300 ft.</td>
<td>-</td>
<td>Hinge</td>
</tr>
<tr>
<td>Oklahoma</td>
<td>Based on allowable lateral movement of 0.5 in. Steel: ≤ 300 ft.; Concrete: ≤ 300 ft.; Prestressed: ≤ 300 ft.</td>
<td>Partially restrained</td>
<td>-</td>
<td>Approach slab tied to pile cap</td>
</tr>
<tr>
<td>Oregon</td>
<td>Based on engineering judgment Length varies depending on location in state Steel: ≤ 300 ft.; Concrete: ≤ 300 ft.; Prestressed: ≤ 300 ft.</td>
<td>Hinge</td>
<td>Tie approach slab to abutment</td>
<td>Granular</td>
</tr>
<tr>
<td>South Dakota</td>
<td>No</td>
<td>Fixed</td>
<td>-</td>
<td>Tie approach slab to abutment</td>
</tr>
<tr>
<td>Tennessee</td>
<td>No</td>
<td>Based on experience Steel: ≤ 400 ft.; Concrete: ≤ 600 ft.; Prestressed: ≤ 600 ft.</td>
<td>Hinge</td>
<td>Tie approach slab to abutment</td>
</tr>
<tr>
<td>Utah</td>
<td>No</td>
<td>Steel: ≤ 300 ft.; Concrete: ≤ 300 ft.; Prestressed: ≤ 300 ft.</td>
<td>Hinge</td>
<td>Tie approach slab to abutment</td>
</tr>
<tr>
<td>Virginia</td>
<td>No</td>
<td>Steel: ≤ 245 ft.; Concrete: ≤ 454 ft.</td>
<td>Hinge or fixed</td>
<td>Uniform width &amp; parallel to bridge skew</td>
</tr>
<tr>
<td>Vermont</td>
<td>Steel: ≤ 150 ft.; Concrete: ≤ 644 ft.</td>
<td>Partially restrained</td>
<td>Flexible pile cap</td>
<td>Approach slab notched to abutment</td>
</tr>
</tbody>
</table>

continued on next page
Design Consideration: Design Assumptions and Details

<table>
<thead>
<tr>
<th>State</th>
<th>Piling Stresses Due to Lateral Movement Are Calculated</th>
<th>Criteria for Maximum Length for Bridges With Integral Abutments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Washington</td>
<td>No</td>
<td>Mainly based on past experience</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Steel: ≤ 200 ft.</td>
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<tr>
<td></td>
<td></td>
<td>Concrete: ≤ 300 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Prestressed: ≤ 300 ft.</td>
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<tr>
<td></td>
<td></td>
<td>Pile Head: Hinge</td>
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<td></td>
<td></td>
<td>Approach Slab: Approach slab attached to abutment with sufficient allowance for expansion</td>
</tr>
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<td></td>
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<td>Back Fill: Granular</td>
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<tr>
<td>Wisconsin</td>
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<td>Based on various studies, reports, etc.</td>
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<td></td>
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<tr>
<td></td>
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<td>Concrete: ≤ 300 ft.</td>
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<tr>
<td></td>
<td></td>
<td>Prestressed: ≤ 300 ft.</td>
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<tr>
<td></td>
<td></td>
<td>Pile Head: Fixed</td>
</tr>
<tr>
<td></td>
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<tr>
<td>Wyoming</td>
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<td></td>
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<td>Steel: ≤ 500 ft.</td>
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<td>Concrete: ≤ 500 ft.</td>
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<td></td>
<td>Prestressed: ≤ 500 ft.</td>
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<td></td>
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<td>Pile Head: Plastic hinge</td>
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<td></td>
<td>Concrete: ≤ 300 ft.</td>
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<tr>
<td></td>
<td></td>
<td>Pile Head: Hinge or partially retained</td>
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<tr>
<td></td>
<td></td>
<td>Approach Slab: Pile cast in pile cap 1 ft.</td>
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</table>

Comments:

- Length 150 feet or less: No provision for expansion is required.
- Length over 150 feet up to 300 feet: Provision should be made for expansion at the end of the approach slab. If at all possible, the span arrangement and interior bearing selection should be such that approximately equal movements will occur at each abutment.
- Length over 300 feet up to 400 feet: Lengths in this range are approved on an individual basis. Provision for expansion should be made at the end of each approach slab.
- Lengths over 400 feet: Not recommended at this time.

California. Thermal movements are easily absorbed by this type of abutment. Abutments of conventionally reinforced continuous concrete bridges of over 400 feet in length have shown no evidence of distress even though the end diaphragms were supported on piles. However, movement of the abutments from shrinkage and temperature changes result in an opening at the paving notch allowing for the intrusion of water. Prestressed structures exacerbate the intrusion problems because of the ad-

New York. Since the approach slabs are connected to the bridge slab, the distance from the end of approach slab to end of approach slab should be considered the length for an integral abutment structure. The criteria specified by the state are:

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- Length over 150 feet up to 300 feet: Provision should be made for expansion at the end of the approach slab. If at all possible, the span arrangement and interior bearing selection should be such that approximately equal movements will occur at each abutment.
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ditional movement resulting from plastic shortening.

Because of the problems arising from the movement at the abutments, the use of the end diaphragm abutment should be limited to the following values unless mitigating measures are used (based on movement required equal to 0.75 inch/100 feet):

- For a temperature range of 80°F: steel 240 ft., reinforced concrete 260 ft., PC concrete 240 ft. and CIP/PS 150 ft.
- For a temperature range of 100°F: steel 200 ft., reinforced concrete 210 ft., PC concrete 200 ft. and CIP/PS 130 ft.
- For a temperature range of 120°F: steel 160 ft., reinforced concrete 180 ft., PC concrete 170 ft. and CIP/PS 120 ft.

Federal Highway Administration. Thermal movements are predicted on the cold climate temperature ranges specified in the American Association of State Highway and Transportation Officials (AASHTO) bridge specifications, Article 1.2.15. State standards specifying other temperature ranges require adjustment of any of those values indicated.7

For structural steel supported bridges, Article 1.2.15 specifies a cold climate temperature range of 150°F with a thermal coefficient of 0.0000065, resulting in a total thermal movement of 1.25 inches (32 mm) of movement per 100 feet (30.5 m) of the structure.

For concrete superstructures, the AASHTO bridge specification specifies a cold climate temperature range of 80°F, a thermal coefficient of 0.0000060 and a shrinkage factor of 0.0002. However, this shrinkage effect can be reduced, provided the normal construction sequence allows the initial shrinkage to occur prior to the completion of the concrete operations. Based on an assumed shrinkage reduction of 50 percent, total allowance for thermal and shrinkage movement in a concrete structure would be approximately 0.75 inches (19 mm) per 100 feet (30.5 m).

For prestressed concrete structures, a somewhat smaller total movement occurs once the prestressing shortening has taken place. Movement of 0.625 inch (15.9 mm) per 100 feet (30.5 m) of structure is a reasonable value. This design criterion permits thermal movement and assumes that there would be no effect from shrinkage and long-term creep. This value has been substantiated in the field as a reasonable value for movement for normal highway overcrossing structures. In long pre- or post-tensioned concrete structures, long-term creep may occur, but this creep is normally insignificant insofar as provision for movement is concerned.

The flexibility of individual substructure units affects the distribution of the total movement between specified joints.

In cold climate conditions, the provisions for total bridge movement that are depicted in Figure 4 should be adopted. For those areas with moderate climate conditions, a 20 percent reduction of the AASHTO Article 1.2.15 values of 120°F for steel and 70°F for concrete may be used.

**Approach Slab**

**New York.** Approach slabs should be 20 feet long (maximum) and the end of the approach slab should be parallel to the skew (30° maximum skew angle). A tight joint should be placed directly over the backwall between the approach slab and bridge slab in order to provide a controlled crack location, thus preventing a random crack pattern from developing. Epoxy coated dowels should pass through the joint and be located near the bottom of the slab in order to keep the joint tight, but still allow the approach slab to settle without causing tension cracking in the top of the slab.

There has been considerable discussion and no agreement on whether the joint should be formed or saw cut. A formed joint can provide positive assurance that the joint would wind up exactly where it should be located and the approach slab would always be supported on the backwall. In many instances, the approach slab is not as wide as the bridge slab. In those instances, the joint is U-shaped and can be formed neatly and easily. The disadvantage to the formed joint is that it requires the approach slab to be poured separately from the bridge slab.

A saw cut joint allows the bridge slab and approach slab to be cast in a single operation. There is some concern as to how soon the saw cutting operation could be commenced and
FIGURE 4. The required provision for total bridge movement under cold climate conditions.

whether cracking would occur before the saw-cutting was started. Also, the saw cut is only to a partial depth and there is no guarantee that the crack that eventually develops between the bottom of the saw cut and the bottom of the slab will be a vertical line. If it cracks on a diagonal, the bottom of the crack may fall outside of the backwall, thus jeopardizing the approach slab support.

Reliable poured, or caulk applied, sealers could be used to seal the joint. If sealers are used, the joint should be formed rather than sawed. On structures over 150 feet, an expansion joint should be placed between the end of the approach slab and another short slab approximately 15 feet long. A short sleeper slab should be placed directly beneath the expansion joint. The 15-foot slab and sleeper slab are stationary, while the end of the approach slab is free to slide back and forth on top of the sleeper slab. The expansion joint is filled with some type of compression seal or perhaps asphalt concrete.

The purpose of the expansion joint is to
Approach Slab

- 14' Min.
- Anchoring Rebar

FIGURE 5. Approach slab details.

prevents a possible maintenance problem at the end of the approach slab. The joint at the end of the approach slab is a working joint. It opens and closes due to thermal expansion and contraction. The longer the span, the greater the opening and closing. Photos taken of joints at the ends of approach slabs for two different integral abutment structures that do not have any provision for expansion reveal that there is potential for future maintenance at these joints.

Federal Highway Administration. Approach slabs are needed to span the area immediately behind integral abutments in order to prevent traffic compaction of material where the fill is partially disturbed by abutment movement. The approach slab should be anchored with reinforcing steel to the superstructure and have a minimum span length equal to the depth of abutment (1 to 1 slope from the bottom of the rear face of the abutment), plus a 4-foot minimum soil bearing area. A practical minimum length of slab would be 14 feet. See Figure 5 for details.

The design of the approach slab should be based on AASHTO Specifications for Highway Bridges, Article 1.3.2(3) Case B, where design span "S" equals slab length minus 2 feet. Positive anchorage of integral abutments to the superstructure is strongly recommended. North Dakota provides a roadway expansion joint 50 feet from the end of bridge to accommodate any pavement growth or bridge movement.

Wingwall Configurations & Details

Tennessee. No. 4 bars should be used for wingwall lengths of 6 to 7 feet, No. 5 bars for wingwalls 7 to 10 feet and No. 6 bars for wingwalls 10 to 12 feet. These values may be adjusted by individual design. For wingwall lengths greater than 12 feet, the designer should use a comprehensive analysis for each case.

New York. Wingwalls should be in-line or flared. U-walls are not allowed and were eliminated because of design uncertainty, backfill compaction difficulty, and the additional design and details that have to be worked out for the joint between the wingwalls and approach slab.

Wingwall lengths in excess of 10 feet should be avoided. Generally, the controlling design parameter is the horizontal bending in the wingwall at the fascia stringer caused by the large passive pressure behind the wingwalls. When the wingwalls are longer than 10 feet, areas of steel greater than No. 11 bars at 6 inches may be required. The 10-foot dimension is a projected dimension and should be measured along a line perpendicular to the fascia stringer. Thus, flared wingwalls may be longer than 10 feet, providing the projected length does not exceed 10 feet.

Stem thickness should be 2 feet minimum. Wingwalls may be tapered to less than 2 feet in

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order to reduce vertical dead load. On structures that have been designed to date, the controlling design parameter has been horizontal bending in the wingwall at the fascia girder that is caused by the large passive pressure behind the abutment. Since it is not certain what the horizontal pressure will be, the state has elected to use the maximum pressures that were obtained in the testing conducted by South Dakota State University for the South Dakota Department of Highways in 1973. In their testing program, they jacked against the backfill in quarter-inch increments and measured the corresponding passive pressures in the backfill material. The anticipated structure movement is then calculated and a corresponding passive pressure from their test data is selected. This pressure is placed over the entire surface area of the abutment, then the horizontal bending moment in the wingwalls caused by the passive pressure trying to bend the wingwalls about the fascia stringer can be calculated.

**General Design**

**Details & Guidelines**

**New York.** Foundation Type: All integral abutments should be supported on piles. Steel H or CIP may be used for structure lengths of 150 feet or less. Only steel H-piles should be used for structure lengths over 150 feet. All piles should be in one single line and be oriented so that bending takes place about the weak axis of the pile. When steel H-piles are used, the web of the pile should be perpendicular to the centerline of the stringer regardless of the skew.

Construction: Steel or prestressed concrete superstructures may be used. Only straight stringers are allowed. Curved superstructures are allowed, providing the stringers are straight. Curved stringers were eliminated to guard against the possibility of flange buckling caused by the stringers trying to expand between the restraining abutments. Stringers should be parallel to each other. The abutments should also be parallel to each other. The maximum vertical curve gradient between abutments should be 5 percent. Stage construction is not allowed when integral abutments are used.

**California.** Restraining Forces: The values listed below for resistance offered by various end conditions are applied at the base of the end diaphragm to determine the proper reinforcement. These values do not take into account the special situations where very long piles or small limber piles offer little resistance to longitudinal movement. The values are for the design longitudinal force (service level) for each abutment type:

- End diaphragm on CIDH piles — 25 kips per pile
- End diaphragm on concrete-driven piles — 20 kips per pile
- End diaphragm on 45T steel piles — 15 kips per pile
- End diaphragm on neoprene strip or pads — 15 percent of dead load
- End diaphragm on rollers — 5 percent of dead load

The values for CIDH, concrete-driven and 45T steel piles are for the design of the end diaphragm only.

Earthquake Forces: Shear keys must be provided in order to resist transverse and longitudinal earthquake forces acting on the structure. These keys should be normally placed behind and at the ends of the abutment wall on narrow structures. On wide structures, additional keys may be located in the interior. One half-inch expansion joint filler should be specified at the sides of all keys to minimize the danger of binding.

Drainage: No previous material collector or weep holes are required for flat slab bridges. Continuous pervious backfill material collector and weep holes may be used for abutments in fills or well-drained cuts and at sites where a 5-foot level berm is specified. For an unprotected berm, the weep hole discharge is directly on the unprotected berm; for full slope paving, the weep hole discharge is on the spacer or groove in the paved surface. When there is continuous permeable material and a perforated steel pipe collector discharging into corrugated steel pipe, overside drains should be used for all other abutments. Corrugated steel pipe overside drains must be coordinated with the road plans. If there is no discharge system and no collector ditch, the outfall must be located away from the...
toe of slope to prevent the erosion of the end slope. Abutment drainage systems should be coordinated with the slope paving.

Backfill Placement: Unless there are special soil conditions or unusual structure geometrics, the method or timing of backfill placement need not be specified. Passive resistance of soil in front of the end diaphragm offers little restriction to structure movement due to stressing; nor will the active pressure of backfill behind the end diaphragm materially alter the stress pattern even if the fill is completed at one abutment before being started at the other.

Summary

Previous surveys concerning the use of integral abutments have indicated that most state highway departments have their own limitations and criteria in designing integral abutments. The bases of these limitations and criteria are shown to be primarily empirical. Twenty-eight states and the District Construction Office of FHWA Region 15 are known to use integral abutments. The current thinking and practice in integral abutment design by the state highway departments and the District Construction Office of FHWA Region 15 are summarized in Table 1. Information on general policy on integral abutment design, provision for bridge movement, approach slabs, wingwall configurations and details, and general design details and guidelines by selected state highway departments (Tennessee, New York and California) and the Federal Highway Administration has also been provided.

Iowa, South Dakota and FHWA Region 15 indicated that piling stresses due to lateral movement are calculated for integral abutment bridges. Alaska and Idaho indicated that such calculations are warranted only for integral abutment bridges that involve some unique feature. The remaining states neglected piling stresses due to lateral movement, although some states like California required some type of mitigating construction detail such as driving the piles into pre-drilled holes.

Construction details vary widely from state to state. Pile head fixity conditions may be of the hinge, fixed or partially restrained type. Pile caps may, or may not, be used. Approach slabs are, in some states, tied to the abutment with dowels and move back and forth with the superstructure, while other states claim that the expansion joint between the approach slab and bridge slab is needed to prevent possible maintenance problems.

While granular material is the most widely used backfill material, some states like New Mexico no longer use specified backfill. Wingwalls may be in-line or flared. Some states like New York do not allow U-walls. U-walls are disallowed because of design uncertainty, backfill compaction difficulty and the additional design and details that have to be worked out for the joint between the wingwalls and approach slab. New York recommends avoiding wingwall lengths in excess of ten feet.

Generally, the controlling design parameter is the horizontal bending in the wingwall at the fascia stringer caused by the large passive pressure behind the wingwalls. When the wingwalls are longer than ten feet, areas of steel greater than No. 11 bars at six inches may be required. The ten-foot dimension is a projected dimension and should be measured along a line perpendicular to the fascia stringer. Thus, flared wingwalls may be longer than ten feet, providing the projected length does not exceed ten feet. Tennessee recommends the use of No. 6 bars for wingwalls between ten and twelve feet, and requires the designer to use a comprehensive analysis if wingwall lengths greater than twelve feet are to be used.

The maximum allowable lengths for bridges with integral abutments used by the different states are summarized in Table 1. The length limitations were set for the most part on the basis of experience and engineering judgment. Many of the states have been progressively increasing the length limitations over the past thirty years primarily as a result of observing the satisfactory performance of actual installations. As of 1983, the length limitations for non-skewed integral abutment bridges had the following range:

- steel — 150 to 400 feet;
- concrete — 150 to 800 feet; and,
- prestressed concrete — 200 to 800 feet.

Most states use the same length limitations for skewed integral abutment bridges.
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LOWELL F. GREIMANN received his structural engineering Ph.D. degree from the University of Colorado, Boulder, in 1968. He is currently a Professor at Iowa State University and is involved in research in the finite element and maintenance areas.

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A Forerunner in Iron Bridge Construction: An Interview With Squire Whipple

Preserving the accumulated knowledge based primarily in practice is an important part of furthering the profession of civil engineering.

FRANCIS E. GRIGGS, JR.

KNOWN AS the “father of iron bridges,” Squire Whipple not only built hundreds of bridges himself in the middle of the nineteenth century, but also wrote a book in 1847 that for the first time anywhere described the methods by which a truss can be analyzed and designed. This book had a profound effect, changing the design and construction of truss bridges from an art to a science. He was the first man elected to honorary membership in the American Society of Civil Engineers (ASCE) after its rebirth following the Civil War. Whipple died in 1888 and was buried in the Albany Rural Cemetery in Menands, New York, in accordance with his request to be laid to rest in “as quiet a manner as the law allows.” His book on bridge building remained current until the 1890s.

While this interview is fictitious, its intent, however, is that the civil engineering profession needs to document the thoughts, ideas and experiences of the senior members of our profession. Oral history is perhaps the most effective way of recording this information. Even though this interview never actually occurred, the facts and thoughts expressed are the result of careful research on Whipple and his times.

Griggs: Thank you for talking with me. As I told you earlier, it is important that future generations of civil engineers know about and appreciate the work done by men like yourself. Oral history is an excellent means of obtaining insights into the personality, character, experiences and associates of, in our case, Squire Whipple. May I call you Squire?

Whipple: You most certainly may, as much as I am used to people asking me where my manor is. You must remember though, that Squire was
Squire Whipple

not so unusual a name around 1800.

Griggs: Okay then Squire, let’s begin. Could you tell me where you lived and the names of your wife and any children?

Whipple: I lived at 236 State Street, Albany, New York. My wife’s name was Anna Case Whipple. We were not blessed with any children, although we did raise from childhood one of my wife’s nieces.

Griggs: What was your childhood like? Who were your parents, your brothers and sisters? Where did you live, say up to the age of 17?

Whipple: My parents were James and Electra Whipple, and I was the ninth of ten children. I was born in 1804 in Hardwick, Massachusetts, just northwest of Worcester. My father ran a farm there until 1812, the year of the war with England, when he built a cotton mill in a small town called Greenwich on the Swift River in Massachusetts. He did well during the war. I worked with him in the factory when I wasn’t in school. When the war ended and English cotton goods came back to the United States, he couldn’t compete. In the end, he sold the mill and we moved to Otsego, New York, not too far from Cooperstown, where we took up farming again.

Griggs: That must have been a major trip for you. How did you travel and what routes did you take?

Whipple: It was a long trip. We travelled by horse and wagon along the Western Turnpike — it’s now called the Mohawk Trail.

Griggs: How did you cross the Hudson River?

Whipple: We went by way of Waterford, New York, and crossed the magnificent wooden arch bridge there. Much later in my life I found out that it was designed and built by Theodore Burr in 1803. It was the first major bridge that I had ever seen. The rest of the trip was along the south side of the Mohawk River, following what is now known as Route 20, until we arrived at Otsego.

Griggs: What happened after that?

Whipple: We farmed there for five years and I attended school during the winter months. In 1822, my father decided to move to Springfield Center, located on the northerly end of Otsego Lake, where he bought a new farm.

Griggs: So, at this time you were 17 years old with, I assume, many years of schooling behind you. You must have been ready to go on to college or to get a job, isn’t that right?

Whipple: Well, not exactly. As I told you, I was one of the youngest members of my family. I had shown a great interest in learning and a certain ability, too. Since my father didn’t need me in the fields, he encouraged me to pursue my studies, learn languages, play the violin and learn several trades such as carpentry and tinworking. I also had time to conduct my philosophical experiments.

Griggs: What were these philosophical experiments?

Whipple: Oh yes, you might not know what they are since you are one of the younger
generation. Well, we called what you call now physics, mechanics, electricity and such by the name natural philosophy.

**Griggs:** I see. What were some of your experiments?

**Whipple:** That was some time ago, but I do remember performing more than quite a few experiments on electricity and electrical phenomena.

**Griggs:** Did you attend college?

**Whipple:** Most certainly. I attended Hartwick Academy for several terms and studied under Dr. Hazelius. I also went to Fairfield Academy, located just outside of Herkimer, New York. I studied under Professor Avery there. It was at these academies that I first learned about Union College. At about 80 miles east of my home in Schenectady, New York, it was the best college in the area.

**Griggs:** So you went to Union?

**Whipple:** Yes. I first went to visit the campus and talked with Dr. Eliphalet Nott who was President of the College. He told me that many students came to Union to finish their college careers and that while I was a little older than most, 24 at the time, I could finish in one year if I did some self-study in Natural Philosophy, Law and other courses before enrolling in September of 1829.

**Griggs:** Did you look at other colleges, such as the Rensselaer School, which I understand was founded in 1824?

**Whipple:** Well, Rensselaer at that time was very small and Union was one of the larger schools in the country. President Nott had instituted a scientific option in 1828 which gave me the choice of pursuing something other than a classical course of studies.

**Griggs:** Tell me, what were your experiences at Union?

**Whipple:** As I said, I came into Union with advanced standing, so most of the courses I took were senior level courses. I took, for instance, differential and integral calculus; electricity, magnetism and optics; elements of criticism; astronomy; moral philosophy; chemistry; anatomy, etc.

As I recall, I excelled in every course. My strongest impression of the college was of its President, Eliphalet Nott. When you were in his presence, you just knew that he was something special. At this time he had already taken out several patents on wood stoves and was working on an anthracite burning boiler for the steamship Novelty. He was President of the college for 62 years, can you imagine that? Sixty-two years!

**Griggs:** When did you graduate?

**Whipple:** I graduated in 1830. The graduation ceremony always was held on the third Wednesday of July. Each student had to deliver a talk on a topic of his own selection. I was very nervous about giving mine, which was incidentally on natural philosophy. While I feel comfortable putting my thoughts into writing, I was reluctant to speak in front of large groups — especially in front of President Nott and the other faculty.

**Griggs:** What did you do after graduation?

**Whipple:** By this time I knew that I wanted to go into the field we now call civil engineering and that meant into canal and railroad building. I talked with John Jervis, who obtained his early experience and reputation in the building of the Erie Canal, about working with him on the design and construction of the Mohawk and Hudson Railroad. It had a groundbreaking ceremony only a short time after my graduation, and it was located only about a mile from the college.

**Griggs:** Can you tell me a little about the Mohawk and Hudson Railroad?

**Whipple:** Since Jervis already had a staff of assistant engineers that he had worked with on the canal, I didn’t get the job, so I can’t tell you first hand about it. I do know that it was built...
A map showing the Baltimore and Ohio Railroad, the New York and Erie Railroad and the Erie Canal — all of which were projects Whipple worked on in the middle part of the nineteenth century.

to speed Erie Canal packet boat passengers from Schenectady to Albany. The canal was very slow along that stretch, since you had to go through the Waterford flight of locks and drop more than 200 feet from the Mohawk River to the Hudson River. As I remember it, Jervis used an inclined plane to come up out of Schenectady and to drop down into Albany. The first steam powered locomotive was the Dewitt Clinton, pulling the passenger coaches along the plateau between the inclined planes.

Griggs: Could you tell me a little more about Jervis?

Whipple: He was, like myself, a small man with an innate ability to look at a problem and come up with a reasonable solution. He started work on the Erie Canal as an axeman and, I might say, as a protégé of Benjamin Wright. After that, he worked on the Delaware and Hudson canal, the Erie enlargement, many other canals and railroads, and finally the Croton Aqueduct Project, which fed water into New York City. I worked near him on many projects, but never for him.

Griggs: Where did you start your engineering career?

Whipple: I started as a rodman on the construction of the Baltimore and Ohio Railroad. They were just outside of Baltimore when I joined. They were building a staff of engineers to handle the detail layout and the design of the right of way. Col. Stephen Long and Jonathan Knight conducted the original surveys, and deter-
mined that it was feasible to build a railroad between the city of Baltimore and the Ohio River.

Griggs: It is very hard to read anything about the early days of engineering in the United States without constantly coming across those two men. Can you tell me a little about them?

Whipple: I knew Jonathan Knight on a personal level, since he was the engineering supervisor during the two years I worked on the railroad. What I could tell you about Col. Long is purely second hand. He left the railroad just before I arrived, so I never get to work with or for him.

Griggs: Why did Col. Long leave the railroad?

Whipple: The Government in 1824 assigned members of the Army with engineering or surveying experience to work with private companies who were building the canals and railroads to provide better access to the interior of the United States. The reason for this was that about the only school in America training men in this area was the military academy at West Point. Long was not a West Point man, I think he went to Dartmouth College, but his knowledge of mechanics and surveying got him a commission in the Army. He spent most of his early Army career as an explorer out west, especially around the Rocky Mountains and the Yellowstone River. When he was assigned to the Baltimore and Ohio Railroad at a level equal to Jonathan Knight, he got along very well in the early survey period. When they started construction, however, they had a major disagreement.

Griggs: What was the disagreement about?

Whipple: It seems that some members of the company went to England to look over what the English had done with their railroads and locomotives. They paid particular attention to George Stephenson and his work on the Stockton and Darlington Railroad. They noted that Stephenson had used masonry for all of his bridges. When they came back to the States, Knight and his assistant Caspar Weaver were convinced that all bridges on the Baltimore and Ohio should also be of masonry. Col. Long was equally convinced that the overpass structures should be made of wood, since they would be faster and cheaper to build. Since no one knew how the railroad would develop, they could be discarded without great financial loss.

Griggs: What happened then?

Whipple: Long was given authorization to build a bridge of wood and Weaver to build one of masonry. Long built a wooden truss bridge which he called the Jackson Bridge in honor of President Andrew Jackson, and Weaver built the Carrollton Viaduct in honor of Charles Carroll of Carrollton, the last remaining person alive who had signed the Declaration of Independence, and who incidentally was at the ground breaking ceremony on July 4, 1828. Both structures were magnificent, but I was amazed at Long’s bridge because it seemed like a spider web in the sky with very light truss members. It was very different from the Burr Bridge at Waterford or the Timothy Palmer Bridge I passed in Easton, Pennsylvania. Long said that he simply used the law of the parallelogram of forces to size the members. At the time I didn’t pursue this idea, but I always kept it in the back of my mind. The directors of the company decided to go with the Knight and Weaver masonry bridges, so Long decided to leave the project, with a great deal of chagrin, I might add.

Griggs: I’ve heard it said that the men of the Baltimore and Ohio really were attending the first school on railroading in the U.S. Is that true?

Whipple: We didn’t call it that at the time, but in a sense it’s true. When the excavation for the road was started, we didn’t know what kind of rail or sleepers we were going to use or whether the cars would be horse drawn or drawn by steam locomotives. We didn’t know what the maximum grade was that a horse or a locomotive could pull a carriage up, and we didn’t know if we would be using inclined planes or not. When we started laying track, we set up test sections. These sections consisted of various ways of attaching the rail to the road bed to see which systems worked best. Before very long we determined that a wooden tie crossing
under both rails was the cheapest and fastest method. As it turned out, that system was more than adequate for the job. I’ll never forget seeing Peter Cooper, a great man by the way, and his Tom Thumb steam locomotive racing a horse drawn carriage back in 1830.

Griggs: How about the other engineers who worked on the railroad?

Whipple: It was a remarkable group. In addition to Jonathan Knight, Benjamin Henry Latrobe, Jr., Wendell Bollman and Albert Fink (all names I am sure you are familiar with) worked on the railroad. At that time, Bollman was a carpenter and Latrobe’s brother also worked with us. This group had a very long career with the Baltimore and Ohio, with each making a contribution to iron bridge building. I didn’t like their trusses, because I thought mine were more efficient, but nonetheless many Bollman trusses and Fink trusses were built in the 1850s.

Griggs: How about Latrobe? We’ve heard a great deal about him.

Whipple: Latrobe’s father was one of the leading architect/engineers in the U.S. in the early nineteenth century. Since he had been educated in England, his talents were in great demand when he came here. Benjamin, Sr., sent his two sons off to college, Benjamin, Jr., to be a lawyer, and his brother to be an engineer. Much to his surprise, as Benjamin, Jr., told me after their graduation from college, they switched careers and both became important to the company. Benjamin was Chief Engineer and his brother was Company Attorney, Historian, Painter and more. I can’t recall a time in my career in which I worked with such an impressive group of men. They were all dedicated to their work, intuitive and imaginative. And, above all, they were all good friends.

Griggs: Why did you leave the Baltimore and Ohio?

Whipple: We were making good progress and were converging on Point of Rocks, Maryland, but so was the Chesapeake and Ohio Canal. The route on the south side of the Potomac River that we both wanted to go through was very narrow, and it was thought by some that both a canal and a railroad would not fit through it. It turned out that the Chesapeake and Ohio had more friends in the Congress than the Baltimore and Ohio did, so we were stopped from proceeding further. Actually, work stopped from sometime in 1832 to the middle of 1833. After we had installed our track as far as we could go, there was very little work to do so I resigned and headed back home to New York.

Griggs: Could you have stayed on?

Whipple: Yes, but I wanted to get on with my career and I heard that plans were being made to enlarge the Erie Canal. A fair number of engineers were needed on that project. Another factor was that my father had been killed when his team of horses had run away and overturned his wagon. I thought that my mother could use my support.

Griggs: Anything else about the Baltimore and
Ohio before we move on to the Erie Canal enlargement?

Whipple: Well, in addition to the experience I obtained and the men I worked with, I also had the opportunity to work with the first transit made in the United States. It was made by a man named Young from Philadelphia, I think. It was much lighter than the English theodolites we had worked with before. It also gave me the idea that I might be able to make a better transit.

Griggs: So then you went back to work on the Erie Canal?

Whipple: Yes, I moved to Utica, New York, and worked in the offices of Holmes Hutchinson. We surveyed the existing canal and prepared plans for enlarging the canal from the original 40' x 24' x 4' prism to 78' x 50' x 7'. The locks were also increased in size to handle larger barges. In addition to his being a good friend, Holmes was a great man to work for. His experience on the original Erie and other canals since then was unmatched.

I also did a great deal of surveying for building lots in and around Utica in my spare time, and oh, before I forget, I almost died of smallpox in 1833 during an epidemic. You probably noticed the pox scars on my face. I started wearing a beard after that to cover up some of the marks. Some people think my beard is a little shaggy. What do you think?

Griggs: I think it is distinctive. How long did you work on the enlargement?

Whipple: I was there, on and off, until 1836.

Griggs: What do you mean by “on and off”?

Whipple: In those days there was very little security in your job. When work was slow, you were let go.

Griggs: What did you do in those slow times?

Whipple: I tried my hand at making mathematical instruments and surveying instruments that were better than those currently available. So, in my spare time, I designed and sold all kinds of instruments to other engineers. I also built myself a house up on Steuben Street on a lot that George Hooper gave to me as a part of the fee for my survey and subdivision of his father’s estate.

Griggs: Was there anyone else you worked with on the enlargement project that made an impression on you other than Holmes Hutchinson?

Whipple: Oh, yes. There was John Jervis again, Julius Adams, John McAlpine and, maybe the most important, Edwin F. Johnson. He was a Norwich University graduate and he became one of the great railroad engineers of my generation. I had the good fortune to work with him on several railroad projects.

Griggs: By this time in your career you had worked on a major railroad project and a major canal project. Did you have any thoughts at that time as to which of the two would win out in the end?

Whipple: Time has answered that, of course. But in the 1830s no one was sure of the potential of the railroad and particularly of the steam locomotive. We also had a great deal of experience with canals as did the English. We knew what a canal could do and weren’t sure, then, what a railroad could do. So yes, I think the enlargement plans for the canal made a lot of sense.

Griggs: Do you want to take a break for a while?

Whipple: No, I find this to be very stimulating. Even though I came into this interview with some misgivings, I am really enjoying talking about my career and the people I’ve met along the way.

Griggs: Okay, let’s talk about your next position.

Whipple: That would be on the New York and Erie railroad. I went as Edwin Johnson’s assistant to lay out the railroad. It was planned to run from a point just north of New York City at
Whipple bowstring truss restored on the campus of Union College. It was originally built across the Erie Canal and later built across a stream in Johnstown, New York. It has been designated a National Historic Civil Engineering Landmark.

a town called Piermont along the southern border of New York state up to a point on Lake Erie. The preliminary surveys had been conducted by Benjamin Wright and by a man who was just starting to make his mark in the engineering profession, Charles Ellet. In 1835, they had made a report to the directors stating that a railroad could be built and gave a preliminary route and a working budget.

I never met Ellet at this time, but from his report and writings, I knew that he was a man to watch. We had the job of surveying the final route the road was to take between Piermont and Painted Post. After a fast start in 1836, money ran out in early 1837 and Johnson had to lay us all off. That was my experience with the New York and Erie until I was to build a few iron bridges for them in the late 1840s.

Griggs: What did you do then?

Whipple: I moved back to Utica, married Anna Case in 1837 and was employed on and off doing canal work, small railroad jobs, local surveying and instrument making. In 1839, I began work with Johnson again on a survey for a railroad running from Ogdensburg to Lake Champlain in New York. I had the easterly portion to survey and lay out, and H. Lee had the piece out of Ogdensburg.

The road was eventually built, but not along my route. I mentioned in my report that the gradient would be much better if the easterly terminus point were not on the west shore of Lake Champlain at Plattsburg but should be shifted north into Canada. That recommended route was eventually followed.
Another interesting point I could make, and few people know this fact, is that I inscribed two cross hairs on the lens of my level in such a way that I could measure distances from the level to the rod without the use of chainmen. Today they’re called stadia hairs, but I did it in 1839 and mentioned in my report to Johnson that it was an excellent technique to use in preliminary surveys.

Griggs: As I understand it, it was in 1840 that you started working on your own. Is that true?

Whipple: Yes it is. If I may express a little self-satisfaction, I think that this was to be the most creative decade of my life.

Griggs: Can you give me some examples?

Whipple: First of all, in August 1840, I had been thinking about the sad condition of the wooden bridges that crossed the Erie Canal and the speed at which the wood would rot. The lifetime of a wooden bridge was only 8 to 15 years. I also thought back to Long’s Jackson Bridge and his parallelogram of forces idea. Having read in the English journals what they were doing with cast and wrought iron, I started to work on a method to design an iron bridge. In almost no time I had a design method and an estimate of the cost for a bowstring truss that used cast iron for the compression members and wrought iron for the tension members, and had a wooden deck that could be replaced without affecting the strength of the bridge.

I went to visit Oliver Shipman, an ironmaker in Springfield Center who incidentally had married my sister Sophronia, to talk about making castings to my plan. He said that it would be an easy task. I then tried to convince the Erie Canal Commissioners to fund the construction of an iron bridge to my design over the canal. They would not take a chance on it so, after applying for a patent, I took $1,000 of my own money and built my first bowstring truss. I placed it on a lot adjacent to the drug store in Utica, so that the Canal Commissioners and the public could see what I had in mind. It was a beautiful sight and everyone who saw it gave me their support, except of course for the Canal Commissioners.

Griggs: And that is why you decided to write a
book to inform the world about your bridge-making method?

Whipple: I felt that if truss bridge building, and particularly iron bridge building, was ever going to be accepted, the bridge builders must be able to put away their "cut and try" methods and adopt a scientific method which I, and only I, knew. So in 1847 I published my *Treatise on Bridge Building*. It was a little book, but I am confident that it had a major impact on my chosen profession.

I was particularly pleased that my old friend Edwin Johnson reviewed the book in the *American Railroad Journal*, which was the journal that was most read by my fellow engineers. You may not know this, but I also wrote another book in 1847 entitled, *The Way to Happiness*, which in my opinion was a first rate little book, but one that did not have much impact at the time. But I was pleased for having done it. In it I described my philosophy on life and what it takes to make yourself and those around you happy. You ought to read it. Union College still has a copy, I think.

Griggs: Could you summarize the message of *The Way to Happiness*?

Whipple: That's a tall order, but I think that a quote from my concluding chapter sums up my philosophy fairly well. It reads: "If all would labor reasonably, spend economically, associate freely and extensively, all might enjoy plenty, all might, and would probably be enlightened, polite and agreeable, all might enjoy to the full, the sweets of social intercourse, and the benefits of private study and reflection, the pleasing alternations of labor and ease, and be in all respects as happy as is consistent with the
nature of things. What more can the heart of man wish or hope?"

**Griggs:** Those are some excellent thoughts. So that was it for the 1840s?

**Whipple:** Well, not entirely. I decided that it was time that iron bridges were given a try on the railroads. Fortunately, my old friend Julius Adams, whom I had met on the Erie Canal enlargement, was Resident Engineer on the Newburgh Branch of the New York and Erie Railroad that was then making rapid progress. Adams had faith in my ideas, so he gave me a contract to build four iron bridges on the Newburgh branch. I built a bowstring, a trapezoidal and two so-called Warren Girder Deck bridges in 1848 and 1849.

They were all designed for a load of 2,000 pounds per foot and worked fine up until 1850 when, as a result of an iron bridge failure on the New York and Harlem railroad, the company decided to replace all iron bridges on the line with wooden ones. This bothered me greatly since I had written letters to newspapers and the American Railroad Journal stating that a bridge I had inspected just like the one that collapsed, a Rider Bridge, was unsafe and very deficient in member size. So when the Directors of the New York and Erie ordered my bridges removed in 1850, even though they were performing perfectly, I felt I had to make my position clear in another letter, this time written directly to the Board of Directors with a copy to the New York City newspapers. Needless to say, I did not receive any more calls from them to build iron bridges.

**Griggs:** That brings us to the 1850s — the decade in which the Erie Canal enlargement was finally funded. Did you play a role in the enlargement?
Whipple: As I mentioned earlier, I did build several iron bridges over the canal in the 1840s and they all had performed very well. When the Canal Commission was considering a new method of funding the enlargement, I urged them to specify that all major bridges — that is, not farm bridges — should be made of iron. In 1852, they made the decision to go with iron bridges. As a part of a "big letting," they put out to bid at one time all of the work remaining on the enlargement. I, of course, placed a bid to build iron bridges using my bowstring plan. Other firms that had virtually no experience in iron bridge building also submitted proposals for various plans of bridges. To my dismay, a contract to build all of the iron bridges went to Erastus Corning's firm located in Albany. The decision of the Canal Commissioners was, as they stated, purely political. So even though my bid was lower, it was rejected in favor of Corning's. I might add that he was now my neighbor, since I had moved to Albany, New York, in early 1850.

Griggs: Did he build them all?

Whipple: No, fortunately for me and, I think, for the State, the State Supreme Court declared that the method of financing the enlargement was unconstitutional. All of the contracts were therefore voided. I then bid on the bridges, and with my nephew James M. Whipple we built a hundred or so across the canal. All of these bridges performed well. I also licensed several other firms to use my plan for a fee. As a result, my bridge was almost the standard iron bridge on the canal. I might add that the Canal Commissioners also accepted my design for wooden farm bridges that crossed the canal. So, in answer to your earlier question, I would say yes, I played a role, quite possibly a major one, in the enlargement of the Erie Canal.

Griggs: Did that occupy you for the entire decade?

Whipple: Oh, no. I was also building roadway and railway bridges across the state. One of my most visible bridges was a five-span bridge to Goat Island just above Niagara Falls that was built in 1851. It was during that project that I met John Roebling. He was starting the construction of his suspension bridge just below the falls.

Griggs: You knew Roebling?

Whipple: Oh, yes. I first became aware of him through his letters and articles in the *American Railroad Journal*. He, for instance, wrote to the journal commenting on the failure of the As-tabula Railroad Bridge as I had also done. So we knew each other through our writing, but I met him for the first time at Niagara Falls.

A funny thing was that in my 1847 book on bridge building I inserted a small section on suspension bridges in which I wrote that while I hadn't spent much time on this type of bridge, I had heard about plans to span the gorge with such a bridge that was to carry both rail and wagon traffic. Several engineers submitted plans and proposals for this bridge and all of them had one very wide deck with rail and wagon traffic side by side with pedestrians. Charles Ellet received the contract, but after building only a foot bridge, he was discharged and the contract was given to Roebling. Well, getting back to my point, I had suggested in my book that the bridge be made a double deck bridge with rail traffic above and pedestrian and wagon traffic below. It seemed to me that this would make for a much stiffer deck. Well, when Roebling got the contract, he recommended to the Board that he be allowed to build a double deck bridge, which of course he did, and it's still standing today.

Griggs: What kind of a man was Roebling?

Whipple: He was completely absorbed by his work and he was a man who did not suffer fools well. We got along very well, because in addition to our technical backgrounds, we both fancied ourselves to be philosophers of sorts and we both were musicians of sorts, too. Even though it was early in his career, and he had only built a few small bridges in Pittsburgh and on the Delaware and Hudson canal, I knew that Roebling had unlimited potential. His Brooklyn Bridge that opened after 14 years of construction, and 14 years after his death, is a fitting capstone to his career.
Griggs: Did you know Charles Ellet, one of Roebling's greatest competitors?

Whipple: I never met the man, but of course I know of his Fairmount Suspension Bridge as well as his Wheeling and Niagara Bridges. I also had read his articles in the railroad journal and his report on the location of the New York and Erie Railroad. It is a shame he had to die at such a young age in the Civil War.

Griggs: Did you do any more work on railroad bridges in the 1850s?

Whipple: Oh, yes. I almost forgot, but I did build two trapezoids of long span in 1852 and 1853. The first bridge was on the Utica and Black River line where it crossed the Mohawk River near Utica, and the other one was for the Troy and Schenectady Railroad near west Troy, New York, across a branch of the Hudson River. These bridges have both served very well up to this day even though the loads on them are much larger than I designed them to hold. I would suspect that they will have to be removed soon since the size of locomotives is very rapidly increasing.
Whipple: Maybe this is a small thing, but it may give you an appreciation of the state of engineering in the 1840s and 1850s. Many iron bridges were falling down during this time. The primary reason was that few people knew how to design them scientifically. It seemed that my book did not reach that many bridge engineers. I then decided to write a series of articles for the American Railroad Journal and Appleton's magazine in which I described my methods of bridge design. The articles were very well received, and I hope they made a contribution to the science of bridge building. I might add in passing that my friend Julius Adams was then serving as editor of Appleton's.

Griggs: That sounds like a very full decade and one in which you, by your efforts, advanced the science of iron bridge building from an experimental stage to an accepted technology. You must have felt very pleased with yourself.

Whipple: Thank you. I probably feel better about it now looking back than I did then.

Griggs: Why is that?

Whipple: I never liked the business end of bridge building. It seemed I was always trying to get clients to pay me, fighting patent infringement cases in court, or collecting fees from people who had agreed to pay me for the use of my patent. It seemed to me then that it was very burdensome and did not give me much satisfaction. So I decided in 1860 to retire from the practice of bridge building and turned the business over to my nephew J.M. Whipple.

Griggs: That is almost like retiring at the peak of your career. As a relatively young man of 56, what did you do with your life for, say, the following twenty years?

Whipple: During the Civil War and thereafter, I spent a lot of my time writing and printing pamphlets on various topics and preparing an appendix to my 1847 bridge building book which I published myself in 1869. I also spent a lot of time in the courts trying to receive payment for work I had done in the 1850s, but I became a little restless and decided to take on a few bridge building jobs. I built an iron swing bridge over the Portland Canal near Louisville, Kentucky, several trapezoidal bridges in southern New York and some lift and swing bridges over the Erie Canal in the early 1870s.

Griggs: Can you tell me about the lift bridges?

Whipple: Of course. I had seen that as our cities along the Erie Canal grew and the boats kept getting larger, that fixed bridges made it difficult to improve on the streets and stores along the street. I then conceived of designing an iron lift bridge with a movable level deck that would solve the problem nicely. I received a patent on this type of bridge and built the first one on Hotel Street in Utica in the early 1870s. Many people copied the design and used it on the canal, but I never received any patent fees. I was getting too old and tired to enter in a legal fight again. I was just happy that I had done it and that it had worked and is still working well today.

Griggs: What else did you do after that?

Whipple: I continued to write, worked on my philosophical experiments, read the technical journals and, on occasion, attended meetings of the American Society of Civil Engineers.

Griggs: Were you a member of ASCE for long?

Whipple: When the Society was first formed in 1852 I was so busy with my own work that, unfortunately, I did not have time to travel to distant meetings. That was the case with other civil engineers, so the Society disbanded just before the Civil War. After the war, however, I was not as busy, and I started attending meetings and, much to my surprise and pleasure, I was elected an honorary member in 1867, the first person to be so honored after the reconstitution of ASCE. John Jervis was elected an honorary member shortly after me.
to the transactions of ASCE?

Whipple: Yes. I felt then as I had in the 1850s that if successful practicing engineers shared their knowledge and skill with other members of the profession, we would all be the better engineers for it. Perhaps my best contribution was in a series of articles that James Eads and I did on the most efficient bridge structure to use—whether an arch like he had used in St. Louis, or a long span truss which I was proposing.

Griggs: I read those articles. I think that you proved your point well as you seemed to have done throughout your long and distinguished career. Before we close this interview would you give us a list of those men you were associated with that you felt made the greatest contributions to civil engineering practice in the nineteenth century?

Whipple: That is a big order, but the names that come to mind, not in any rank or order are: John Roebling, Charles Ellet, James Eads, Edwin Johnson, Holmes Hutchinson, Julius Adams, Benjamin Crofton, Canvas White and Benjamin Latrobe. There are many more that I will probably think of as soon as you leave, but those men stick out in my mind.

Griggs: Thank you, Squire, for taking the time to talk with me. I am sure that your fellow engineers will be appreciative of the thoughts and words you have expressed in this interview and the perspective you have given them on the period of time from 1830 until 1880. I only wish that I had the opportunity to talk with some of those engineers that you mentioned. The profession would be a better one if we took more time in tapping the wisdom of our senior members. Thanks once again.
Whipple: Thank you for listening to an old man talk about the way it was. I wish you luck on your future interviews.

NOTE — The Committee on the History and Heritage of American Civil Engineering (CHHACE) of ASCE urges local sections to institute programs that will lead to the building of a tape library of conversations, much like that with Squire Whipple, to be housed in the Engineering Societies Library in New York City. For information on the details of interviewing, recording and making your tapes, contact Herb Hands at the American Society of Civil Engineers, 345 E. 47th St., New York, NY 10017-2398, (212) 705-7496.

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Electron Inactivation of Pathogens in Sewage Sludge & Compost: A Comparative Analysis

Pathogen reduction and sludge stabilization in the treatment of sewage sludge prior to land application may be best achieved by using a process sequence of mesophilic composting or anaerobic digestion followed by electron treatment.

SAMUEL R. MALOOF

THE USE of raw municipal sludge on American farmlands dates back to as early as 1881 in Pullman, Illinois. Sewage farming, or "broad irrigation" as it was referred to then, was practiced mainly in the midwest and New England. By the turn of the century, this method of sewage disposal lost favor as increased urban development, the rising cost of suitable land close to cities and the growing potential threat of disease transmission to humans via the consumption of raw vegetables forced sanitary engineers to develop more technology-intensive systems for the treatment and disposal of sewage.1

In recent years, there has been renewed interest in the use of anaerobically digested sludge and composted sludge for soil amendment, crop production, land reclamation and horticultural applications.2,3 Both products are the result of the microbial decomposition of the organic matter in sewage sludge. However, whereas the process of anaerobic digestion is carried out in the absence of air, mesophilic composting (mesophilic bacteria grow in a temperature range of 10 to 40°C, with an optimum of 37°C) is carried out under controlled conditions of temperature, moisture and oxygen resulting in a more stable, odor-free humus-like substance.

Neither process produces an end product that is free of all disease-producing organisms (pathogens). Consequently, even composted sludge, which has a lower level of residual pathogens, should be further disinfected before the product can be safely used in the growth of...
root crops and leafy vegetables that are destined to be eaten raw.

Complete elimination of all pathogens in anaerobically digested sludge and composted sludge can be accomplished in a safe and economical manner by the technology of electron beam (EB) treatment. A dosage of 400 kilorads is adequate for anaerobically digested sludge, while a higher dose level (0.4 to 1 megard) is needed for primary raw sludge and composted sludge.

Present regulations of the U.S. Environmental Protection Agency (EPA) for land application of sewage sludge for beneficial reuse require that the sludge first be treated by a process to significantly reduce pathogens (PSRP) or a process which further reduces pathogens (PFRP). The PSRP processes listed in Section A of Appendix II of the EPA regulations include aerobic and anaerobic digestion (less than 55°C), mesophilic composting (40°C) and lime stabilization which, on their own, do not yield a pathogen-free product. Therefore, crops for direct human consumption must be planted 18 months following the land application, or incorporation, of sewage sludge treated by such processes. If crops for direct human consumption are grown prior to the 18-month waiting period, then the sludge must be treated prior to its application by one of the following PFRPs:

- high temperature composting (at least 55°C),
- heat drying (80°C),
- heat treatment (180°C), or
- thermophilic aerobic digestion (thermophilic bacteria have a range of approximately 45 to 75°C, with an optimum near 55°C).

More recently, the EPA has proposed a new set of regulations (unpublished: 40CFR, Part 503) in which the emphasis is placed on a process sequence to achieve specific pathogen reductions. For example, the reduction of pathogens to below detectable limits and the reduction in the attraction of disease vectors are accomplished by a process sequence to eliminate pathogens (PSEP) rather than by a single process as required of a PFRP in the present regulations. The process sequence must include a process that eliminates pathogens (PEP) to a level that is similar to that produced by the PFRPs in the present regulations with the exception that the PEP is not required to reduce the attraction of disease vectors. The EB process qualifies as a PEP, and if it follows a process such as mesophilic anaerobic digestion or composting, the combination can satisfy all the proposed specific requirements of a PSEP — namely, the reduction of pathogens below detectable limits, the reduction of the attraction of disease vectors and the reduction of the potential for the regrowth of bacteria.

Electron disinfection of anaerobically digested sludge, if followed by wide-area ocean dispersal, should be a viable disposal option for coastal cities like Boston. Carried out under controlled conditions, it would be environmentally safe and could represent a valuable source of nutrition for the planktonic life essential for fish production. Moreover, it appears to be a more practical and economical solution with fewer potential disadvantages than the technologies of composting or the incineration of sludge. Table 1 briefly compares the estimated general costs of these sludge disposal alternatives in 1985 dollars.

**Early Studies**

Research began in May 1974 with a grant from the National Science Foundation (NSF) for small scale studies on the ability of electrons (energized by acceleration in a machine accelerator and injected into a moving layer of the liquid material) to destroy pathogenic bacteria, viruses and parasites in raw and digested municipal sludges. Biological and chemical effects were measured on sewage materials treated over a wide dosage range with a three-million-volt Van de Graaff electron accelerator at the High Voltage Research Laboratory, Department of Electrical Engineering and Computer Science at the Massachusetts Institute of Technology (MIT).

These initial bacteriological studies on the effects of electron irradiation demonstrated that the inactivation of a variety of pathogenic bacteria and viruses in liquid sludges was effected at moderate electron dosages. Liquid digested sludges were effectively disinfected by an ionization dose of 400 kilorads. At this dosage,
the total bacterial count is reduced by about four logs (orders of magnitude) and total coliforms, salmonellae and shigellae are reduced to non-detectable levels. Poliovirus and coxsackie virus are reduced by one or two logs — adequate safe levels for anaerobically digested sludge, but marginal for raw sludge at the late summer peak of virus input. However, increasing the dose to 1 megarad can reduce the virus content by 3 to 5 logs, which is more than adequate to inactivate the virus content in raw sludge ($10^2$ to $10^3$ per ml). 8

The use of higher electron dosages to inactivate all pathogens in raw sludge is not recommended. In the absence of digestion, sludge stabilization will not occur and the potential for regrowth of contaminating bacteria will be increased due to a reduction in the number of the more resistant gram-positive sporulating bacteria that can serve as competitive organisms to minimize regrowth. Rather, the use of a process sequence such as anaerobic digestion followed by EB treatment at a dose of 400 kilorads is preferred to meet the proposed EPA requirements for a PSEP.

These initial studies also showed that the effective control of pathogenic microorganisms in wastewater residuals could be achieved at economic dosages with efficient in-line modular electron disinfection equipment. Together, these studies provided the basis for proceeding with the engineering and construction of a scaled-up in-line electron treatment facility.

### Sludge Dynamic Studies

With the cooperation of the Sewage Division of the greater Boston area Metropolitan District Commission (MDC), predecessor to the Massachusetts Water Resources Authority (MWRA), the Division of Water Pollution Control of the Massachusetts Department of Environmental Quality Engineering and a firm specializing in electron beam technology, a full scale modular electron research facility was assembled during the second year of support by the National Science Foundation at the MDC’s primary wastewater treatment plant at Deer Island, Boston. 9 This facility, which was rated at 50 kilowatts (kW) and designed to deliver a disinfecting dosage of 400,000 rads to 100,000 gallons of municipal sludge per day, was brought into operation in April 1976.

In 1980, the firm specializing in electron beam technology undertook a complete restructuring and expansion of the facility with the approval of the MDC. The new EB treatment system was rated at 1.5 megavolts (MeV), 50 milliamps (mA) and 75 kW. Its sludge treatment capacity was increased to 170,000 gallons per day (gpd) and it became operational in early 1981.

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### Table 1

<table>
<thead>
<tr>
<th>Sludge Disposal Alternative</th>
<th>Capital Cost (Million $)</th>
<th>Total Annual Cost, Millions ($)</th>
<th>Unit Cost $/Dry Ton**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Incineration</td>
<td>56</td>
<td>10.5</td>
<td>240</td>
</tr>
<tr>
<td>Pyrolysis</td>
<td>57</td>
<td>10.5</td>
<td>240</td>
</tr>
<tr>
<td>Landfill</td>
<td>25</td>
<td>9.6</td>
<td>218</td>
</tr>
<tr>
<td>Composting &amp; Land Application</td>
<td>38</td>
<td>8.6</td>
<td>197</td>
</tr>
<tr>
<td>Electron Disinfection &amp; Ocean Dispersal</td>
<td>22</td>
<td>4.2</td>
<td>96</td>
</tr>
</tbody>
</table>


** Based on 120 dry tons per day from 720,000 gallons per day of 4 percent liquid digested primary sludge.
Table 2
The Number of Bacteria in Deer Island Wastewater Residuals

<table>
<thead>
<tr>
<th>Bacterial Type</th>
<th>Raw Primary Sludge</th>
<th>Anaerobically Digested Sludge</th>
<th>400 Kilorad Electron Dose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Bacteria</td>
<td>$2 \times 10^8$/ml</td>
<td>$4 \times 10^6$/ml</td>
<td>$10^2$ to $10^3$/ml</td>
</tr>
<tr>
<td>Total Coliforms</td>
<td>$7 \times 10^7$/ml</td>
<td>$8 \times 10^5$/ml</td>
<td>Not detectable</td>
</tr>
<tr>
<td>Fecal Coliforms</td>
<td>$2 \times 10^9$/ml</td>
<td>$1 \times 10^7$/ml</td>
<td>Not detectable</td>
</tr>
<tr>
<td>Gram-Negative Bacteria</td>
<td>$1 \times 10^9$/ml</td>
<td>$1 \times 10^6$/ml</td>
<td>Not detectable</td>
</tr>
<tr>
<td>Salmonella</td>
<td>$1 \times 10^9$/ml</td>
<td>$4 \times 10^5$/ml</td>
<td>Not detectable</td>
</tr>
<tr>
<td>Fecal Streptococci</td>
<td>$5 \times 10^4$/ml</td>
<td>$5 \times 10^3$/ml</td>
<td>10</td>
</tr>
<tr>
<td>Gram-Positive Bacteria</td>
<td>$1 \times 10^9$/ml</td>
<td>$2 \times 10^6$/ml</td>
<td>$10^2$ to $10^3$/ml</td>
</tr>
</tbody>
</table>

At the plant, the liquid sludge was first ground to a slurry and then pumped at the rate of 120 gallons per minute into a weir, from which it flowed in free fall as a thin, smooth sheet 1.25 meters wide. A beam of electrons energized at full power was injected horizontally into the falling sludge sheet in the region below the weir where its velocity was about 2 meters per second. Disinfection took place in the fiftieth of a second that the sludge took to fall through the rapidly scanning electron beam. A modular sludge treatment unit with 100 kW of electron beam power could treat about 240,000 gpd at 4 percent solids (40 dry tons per day) to a dose of 400,000 rads. Four such modular units in parallel, on 0.5 acre of land, would be sufficient to disinfect the MWRA-projected 1995 sludge output of 720,000 gpd (120 dry tons per day) for the Boston area.

Studies performed at the Deer Island plant proved to be an excellent testing ground for the electron disinfection process. Despite wide fluctuations in the solids content of the primary digested sludge over the range of 1 to 8 percent, these studies demonstrated that such sludges could be reliably disinfected.

A similar facility was purchased and installed in 1983 by the Miami-Dade Water and Sewer Authority at Virginia Key in Miami, Florida, and became fully operational in 1984. This facility has proven successful for the complete disinfection of about 25 percent of the total daily output of liquid digested sludge at Virginia Key at the recommended dose of 400 kilorads. Nevertheless, it has been used only infrequently since 1985. However, the facility will be reactivated this year under a National Science Foundation grant to evaluate its effectiveness for the destruction of pathogens and toxins in domestic wastewaters. A second work phase will be devoted to evaluating the ability of high energy electrons to reduce natural aquatic organic compounds in groundwaters that are used as sources of drinking water.

Experimental Data

The bacterial counts determined in Deer Island wastewater residuals are presented in Table 2. They are characteristic of levels reported elsewhere from an extensive review of domestic and foreign data covering the period 1940 to 1980. Primary raw sludge usually contains over $10^8$ bacteria per ml. The total bacterial count is about evenly divided between the electron sensitive gram-negative bacteria such as salmonella and shigella and the more resistant gram-positive sporulating bacteria. After an electron dose of 400 kilorads, the bacterial counts for anaerobically digested sludge are essentially reduced to non-detectable levels.

Figure 1 shows the survival curves for total bacteria, total coliforms and salmonella in raw primary sludge as a function of the electron

dose. A plot of the logarithm of the percent survival \( \log(N/N_0 \times 100) \) versus the electron dose should be a straight line if a single homogeneous organism is present, where \( N_0 \) is the original number of bacteria or viruses and \( N \) is number surviving after electron treatment. For \( N \) equal to \( N_0 \), the logarithm of \( N/N_0 \times 100 \) equals 2 (origin). The survival curves for total salmonella and total coliforms are nearly linear; their initial counts are reduced by about 8 orders of magnitude (logs) at a dose of about 200 kilorads. However, the survival curve for total bacteria is linear only at small dosages. Once the electron sensitive gram-negative bacteria have
been destroyed, the more resistant gram-positive sporulating bacteria (mostly non-pathogenic) cause the survival curve to fall less rapidly with increasing dosage.

The $D_{10}$ values for various microorganisms in raw and anaerobically digested primary sludge are presented in Table 3. These values represent the electron dose required to reduce the initial population by a factor of 10 (90 percent), or by one log on a semi-logarithmic plot of the logarithm of the surviving fraction ($N/No$) versus electron dose. $D_{10}$ values for total coliforms, fecal coliforms and salmonella are among the lowest; whereas the values for viruses are higher by a factor of at least 10. Viruses are smaller in size and are tougher targets for electrons. Nevertheless, a higher dose of 1 megarad is adequate to reduce the levels of viruses commonly found in raw sewage to non-detectable levels.

The $D_{10}$ value for ova of the parasitic roundworm, *Ascaris lumbricoides*, is of particular interest. The value of 30 kilorads determined on semi-dried sludge using gamma rays at the Sandia Research Laboratories is of the same order of magnitude of those of the more electron sensitive bacteria. This low value indicates that complete inactivation of *Ascaris* eggs in sewage sludge should be feasible at low dosages using either gamma rays or energized electrons. If the sludge contained no parasites, the required waiting periods before crops for direct human consumption could be reduced.

Gamma rays have a greater ability to penetrate masses than electrons. Therefore, gamma rays are more practical for the irradiation of thick materials and are competitive economically with electrons for the treatment of materials where the power requirements are low (15 kilowatts or less). For the treatment of large volumes of digested sludge or composted sludge in thin layers where higher power is required for large throughputs, the use of electrons is more economical. Assuming the

---

**Table 3**

<table>
<thead>
<tr>
<th>Bacterial Group</th>
<th>$D_{10}$ Dose (kilorads)</th>
<th>Calculated Log$_{10}$ Reduction for 400 Kilorad Electron Dose***</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total bacteria in raw primary sludge</td>
<td>103</td>
<td>3.9</td>
</tr>
<tr>
<td>Total bacteria in anaerobically digested sludge</td>
<td>133</td>
<td>3.0</td>
</tr>
<tr>
<td>Total coliforms in raw primary sludge</td>
<td>25</td>
<td>16.0</td>
</tr>
<tr>
<td>Total coliforms in anaerobically digested sludge</td>
<td>28</td>
<td>14.0</td>
</tr>
<tr>
<td>Fecal coliforms in raw primary sludge</td>
<td>28</td>
<td>14.0</td>
</tr>
<tr>
<td>Fecal coliforms in anaerobically digested sludge</td>
<td>29</td>
<td>14.0</td>
</tr>
<tr>
<td>Fecal streptococci in raw primary sludge</td>
<td>157</td>
<td>2.5</td>
</tr>
<tr>
<td>Fecal streptococci in anaerobically digested sludge</td>
<td>110</td>
<td>3.6</td>
</tr>
<tr>
<td>Clostridia in raw primary sludge</td>
<td>600</td>
<td>0.6</td>
</tr>
<tr>
<td>Clostridia in anaerobically digested sludge</td>
<td>500</td>
<td>0.8</td>
</tr>
<tr>
<td>Salmonella in raw primary sludge</td>
<td>26</td>
<td>15.0</td>
</tr>
<tr>
<td>Poliovirus type 2 in anaerobically digested sludge</td>
<td>365</td>
<td>1.1</td>
</tr>
<tr>
<td>Coxsackievirus type B3 in anaerobically digested sludge</td>
<td>400</td>
<td>1.0</td>
</tr>
<tr>
<td>Echovirus type 7 in anaerobically digested sludge</td>
<td>335</td>
<td>1.2</td>
</tr>
<tr>
<td>Reovirus type 1 in anaerobically digested sludge</td>
<td>330</td>
<td>1.2</td>
</tr>
<tr>
<td>Adenovirus type 5 in anaerobically digested sludge</td>
<td>300</td>
<td>1.3</td>
</tr>
<tr>
<td>Ova of the roundworm <em>Ascaris lumbricoides</em> in air-dried sludge**</td>
<td>30</td>
<td>13.0</td>
</tr>
</tbody>
</table>


** Determined at Sandia Research Laboratories, Albuquerque, New Mexico, by J.R. Brandon and S.L. Langley.**

*** A 1 Log Reduction ($D_{10}$ dose) represents the reduction of one order of magnitude (10-fold reduction) or 90 percent in surviving organisms.
Table 4
Density Levels of Bacteria & Viruses in Deer Island Raw Primary Sludge & Anaerobic Digested Sludge

<table>
<thead>
<tr>
<th>Bacterial Strain</th>
<th>Density Level per ml</th>
<th>Calculated Log10 Reductions Following Treatment of Raw Sludge by:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Anaerobic Digestion</td>
<td>Electron Treatment at 200 Kilorads</td>
</tr>
<tr>
<td>Total Bacteria</td>
<td>$2 \times 10^8$</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>$2 \times 10^6$</td>
<td>1.9</td>
</tr>
<tr>
<td>Total Coliforms</td>
<td>$7 \times 10^7$</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td>$1 \times 10^5$</td>
<td>1.9</td>
</tr>
<tr>
<td>Fecal Coliforms</td>
<td>$2 \times 10^6$</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>$1 \times 10^5$</td>
<td>7.1</td>
</tr>
<tr>
<td>Fecal Streptococci</td>
<td>$5 \times 10^5$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>$5 \times 10^5$</td>
<td>1.3</td>
</tr>
<tr>
<td>Salmonella</td>
<td>$1 \times 10^6$</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td>$4 \times 10^5$</td>
<td>7.7</td>
</tr>
<tr>
<td>Enteric Viruses</td>
<td>$10^2$ to $10^3$</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>1 to 10</td>
<td>1 to 2</td>
</tr>
</tbody>
</table>

* Calculated log reductions following anaerobic digestion versus electron treatment at doses of 200 and 400 kilorads.

Comparison With Other Technologies
Table 4 shows the log reductions achievable for various bacterial organisms and enteric viruses in raw sludge by the processes of electron treatment and anaerobic digestion. The process of electron treatment shows remarkable results in its ability to reduce bacterial pathogens. Anaerobic digestion reduces the bacterial organisms shown by no more than 1 to 2 logs (1 to 2 orders of magnitude) with the exception of salmonella; whereas with electron treatment, even at the low dose of 200 kilorads, reductions of 1 to 8 logs were obtained.

The levels of human enteric viruses in raw sludge are fortunately far less abundant than bacteria. A reduction of 1 to 2 logs at the dose of 400 kilorads is adequate for anaerobically digested sludge, but somewhat marginal for the density levels of $10^2$ to $10^3$ per ml found in raw sludge. A log reduction of about 3 is therefore indicated for the viral inactivation of the raw sludge that can easily be achieved at a dose of 1 megard.

Although the study did not include any data for the electron inactivation of parasites, it reported that the $D_{10}$ dose for the inactivation of ova of Ascaris lumbricoides, the most resistant of parasitic worms, was approximately 30 kilorads. This value is comparable to that for the electron inactivation of total coliforms and fecal coliforms (see Table 2). Since the content of Ascaris ova in raw sludge is low when compared to bacterial content, complete inactivation at a dose of 400 kilorads was easily achieved. On the other hand, the process of anaerobic digestion was not particularly effective for inactivating parasite organisms.

An advantage of the electron treatment process is that most of the bacterial pathogens, parasites and viruses in anaerobically digested sludge and raw sludge can be reduced to non-detectable levels at moderate electron dosages. For anaerobically digested sludge, the minimum electron dose for disinfection is about 400 kilorads, and somewhat higher for raw sludge. Microbiological studies carried out on the regrowth of bacteria seeded into raw and digested sludge electron treated at 400 kilorads and 1 megard showed that the growth was far less in samples treated at the lower dosage. This reduction in regrowth was attributed to more of the gram-positive sporulating bacteria (non-pathogenic) remaining in samples treated at the lower dosage that can serve as competitive organisms to resist the regrowth of residual bacteria or contaminating pathogenic bacteria.

The data shown in Table 5 were derived from several sources. The process of electron treatment is more effective for the inac-
Table 5
Comparison of the Log Reductions Achieved With Various Disinfection Processes for the Inactivation of Bacteria, Parasites & Viruses in Raw Sludge

<table>
<thead>
<tr>
<th>Type of Organism</th>
<th>Indicator Bacteria</th>
<th>Pathogenic Bacteria</th>
<th>Parasitic Ova (A. lumbricoides)</th>
<th>Enteric Viruses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mesophilic Composting</td>
<td>2 to 4</td>
<td>1 to 3</td>
<td>Not effective</td>
<td>3</td>
</tr>
<tr>
<td>Anaerobic Digestion</td>
<td>1 to 2</td>
<td>1 to 2</td>
<td>Not effective</td>
<td>1 to 2</td>
</tr>
<tr>
<td>Lime Stabilization</td>
<td>1 to 7</td>
<td>1 to 2</td>
<td>Not effective</td>
<td>1 to 2</td>
</tr>
<tr>
<td>Electron Treatment at 400 Kilorads</td>
<td>2.5 to 16</td>
<td>13</td>
<td></td>
<td>13</td>
</tr>
</tbody>
</table>

* Of the three indicator bacteria, fecal streptococci are the most resistant to inactivation with the log reductions at the low end of the ranges given above for each process. Such bacteria regrow to near-original densities within 24 hours following lime stabilization with a drop in the pH below 11.0. Regrowth is also evident after mesophilic composting, suggesting that these processes are not effective in eliminating fecal streptococci. However, the level in raw sewage shown in Table 2 could be completely eliminated by electron treatment at a dose calculated as 733 kilorads.

** Temperature range of ambient to 40°C; for further details, see Reference 10.

Electron Treatment of Composted Sludge

Research carried out by Finstein et al. at Rutgers University has cast doubt on the EPA guidance for the composting of sludge with respect to the control of the growth and proliferation of pathogenic fungi during the composting operation and the regrowth of residual or contaminating bacteria. According to the researchers, the EPA's basic approach leads to an early peak temperature of 80°C in the bulk of the material that not only results in the early destruction of pathogens, but also in organisms that are needed in the decomposition (stabilization) process as well as those that can serve to combat the growth of contaminating bacteria. Instead, strict adherence to a temperature ceiling of 60°C is recommended, which they claim will address their objections to the EPA guidance policy on composting. However, whatever the composting temperature, the fact remains that the edge material is relatively cool and, therefore, will not be properly disinfected. In other words, it is practically impossible to achieve all the sanitation objectives desired in a single composting operation.

Complete pathogen destruction cannot be guaranteed. However, further disinfection of compost is recommended before the product is used in the production of root crops and leafy vegetables that are destined to be eaten raw. Evidence that composted sludge is not pathogen-free is shown in Table 5. Samples were taken from a compost pile at Durham, New Hampshire, after 31 days at a depth of about 15 inches. A portion of the sifted compost was taken to the University of New Hampshire for microbiological analysis, while a second portion was delivered to the firm specializing in EB technology for EB treatment. This second portion was subjected to a dose of 1 megaread and returned to the university for microbiological analysis.
Table 6
Microorganism Counts in Composted Municipal Sludge Before & After High Energy Electron Treatment

<table>
<thead>
<tr>
<th>Organism</th>
<th>Before Electron Treatment</th>
<th>Number per Gram of Compost After 1 Megarad Electron Dose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Coliforms</td>
<td>$7.0 \times 10^3$</td>
<td>2.3</td>
</tr>
<tr>
<td>Fecal Coliforms</td>
<td>$3.3 \times 10^3$</td>
<td>0.0</td>
</tr>
<tr>
<td>Total Streptococci</td>
<td>$4.8 \times 10^6$</td>
<td>6.0</td>
</tr>
<tr>
<td>Fecal Streptococci</td>
<td>$1.7 \times 10^3$</td>
<td>0.0</td>
</tr>
<tr>
<td>Salmonella</td>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>Aspergillus fumigatus</td>
<td>$8.0 \times 10^3$</td>
<td>0.0</td>
</tr>
<tr>
<td>Thermophilic actinomycetes</td>
<td>$4.0 \times 10^6$</td>
<td>0.0</td>
</tr>
</tbody>
</table>

As seen in Table 6, essentially all pathogens were destroyed at the dose of 1 megarad. Only a few coliforms and streptococci survived, but these were not of fecal origin. It is possible that a lower dosage would be adequate for the disinfection of composted sludge. Liquid sludges that were electron treated at 400 kilorads and 1 megarad showed little or no regrowth in samples treated at lower dosage. Since the regrowth of bacteria in composted sludge is a significant problem of some concern, the feasibility of using lower electron dosages for the disinfection of sludge composted at mesophilic temperatures should be further investigated.

Electron treatment of compost can be economical. Costs are estimated at $20 per dry ton or 1 cent per pound. Moreover, its use could help remove the uncertainty that presently exists regarding the choice of the optimum temperature and level of disinfection needed to achieve all the sanitation objectives desired in the composting process.

Summary
The use of electron treatment for the inactivation of pathogenic bacteria, parasites and viruses can be more effective for the inactivation of bacterial pathogens, helminth parasitic ova (A. lumbricoides) and the indicator organisms than the processes of anaerobic digestion, mesophilic composting and lime stabilization. An electron dose of 400 kilorads can be adequate for the reduction of the bacterial, parasitic and viral content of anaerobically digested sludge. A somewhat higher electron dose (0.4 to 1.0 megarad) is indicated for complete viral inactivation of raw sewage sludge and possibly for the destruction of residual pathogenic bacteria and fungi in composted sludge (compost).

Anaerobically digested sludge and composted sludge should be treated further by the process of electron treatment prior to use on agricultural land for the growth of crops for direct human consumption and for the use of compost in horticultural applications. Treated by such a process sequence, the end product can be stable and free of pathogens, resistant to regrowth from contaminating bacteria and can meet all federal requirements to obviate the need for any health-related constraints on land use.

An advantage of the electron treatment process is that the electron dose can be controlled in order to inactivate all bacterial pathogens, parasitic ova, viruses and fungi that may be present in raw sludge, anaerobically digested sludge or composted sludge. However, the dose should be kept to a minimum to retain more of the gram-positive (non-pathogenic) organisms that have a higher resistance to ionizing radiation and that can serve as competitive organisms to minimize the potential for the regrowth of pathogenic bacteria. Electron disinfection costs are estimated at $42 per dry ton ($7 per 1,000 gallons) for liquid sludge and $20 per
dry ton or 1 cent per pound for compost.

ACKNOWLEDGEMENTS — The author acknowledges, with thanks, the use of the microbiological data on raw primary sludge and anaerobically digested sludge collected by the late Dr. John G. Trump and the team of scientists at the Massachusetts Institute of Technology who were provided with financial support from the U.S. National Science Foundation and from the High Voltage Engineering Corporation (HVEC), Burlington, Massachusetts. Dr. Pascal Levesque, President and Chief Executive Officer of HVEC, kindly reviewed the manuscript.

SAMUEL R. MALOOF received his B.S. and M.S. degrees from MIT in 1945 and a Ph.D. in Material Science from the Pennsylvania State University in 1949. His early work in the private sector included studies on refractory materials and corrosion problems that resulted in several publications. In 1971, he entered public service at the state level and became involved in transportation and environmental issues. Since late 1977, he has served as consultant to HVEC on the application of electron beam technology for the disinfection of liquid sludges and composted sludge for beneficial use on land and in the ocean. He also serves as consultant to Escatex, Sao Paulo, Brazil.

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Engineering Design Using Microcomputer-Based Spreadsheets

The first spreadsheet designed for use on microcomputers was introduced in 1978. Since that time, business managers have made extensive use of these new electronic "tools," resulting in documented increases in productivity. Practicing engineers have been relatively slow in adopting this technology. Specifically, little use has been made of the powerful graphics capabilities these programs possess.

W. LEE SHOEMAKER & STEVE WILLIAMS

As the use of microcomputers, or personal computers (Pcs), has proliferated over the last ten years, civil engineers have put them to use in applications that require extensive and complicated calculations in fields such as structural engineering, hydrology, etc.

Engineers can use high-level programming languages (such as FORTRAN, Pascal, C, etc.) to write their own applications. The actual time spent writing, testing and debugging programs developed by engineers or software specialists, however, may not be cost-effective. Currently available commercial programming environments and program generators may reduce program development time, but it still may not be to the advantage of either the engineer or engineering firm to be in the "software development" business.

While off-the-shelf engineering software applications are available, they are in most cases too costly, or are either too broad or narrow in scope to meet the specific engineering requirements.

However, some engineers have begun to use commercial spreadsheet software that is readily available for use on personal computers for various engineering applications. Although the similarities between ledger calculations that this software was originally developed for and the solution of engineering equations might not be obvious, engineers are beginning to realize that spreadsheet concepts can be readily adapted to almost any manipulation of numeri-
Spreadsheets are also being utilized as effective teaching tools, primarily because these programs offer the unique opportunity for changing parameters in order to see their effects on the solution. Even though the quantity of spreadsheet applications in engineering design that are being reported has been increasing over the past few years, relatively little mention has been made of the type of design applications that can be readily handled by using the graphics capabilities built into most popular commercial PC spreadsheets.

**Spreadsheet Fundamentals**

An electronic spreadsheet is simply a matrix of
cells. Each cell may contain text, a number or a formula, and is most commonly identified by a column letter and a row numeral (e.g., A1 represents the uppermost left cell of the spreadsheet, C4 represents the cell at the intersection of the third column and fourth row). Usually, only a portion of the available spreadsheet is visible on the microcomputer's screen at one time—the size of the viewing area is regulated by the type of monitor and video display adapter that the personal computer is equipped with. However, the user can view other parts of the spreadsheet by using cursor control keys or a mouse. The maximum spreadsheet size varies for different software vendors, but it is typically of the order of 8,000 rows by 250 columns (2,000,000 cells).

To solve an engineering problem, the spreadsheet must be designed to accommodate input data, governing equations and the displayed results. One of the most useful features of a spreadsheet is that after setting up the governing equations, any or all of the input data may be changed and the new results will automatically be recalculated. This feature lets the user quickly iterate on design alternatives. Recalculation is also performed extremely fast; it is virtually instantaneous for normal-size spreadsheets that have less than a few hundred equations.

Example of Spreadsheet Setup. In order to illustrate the setup and use of a spreadsheet, values for the shear force, bending moment and deflection will be calculated for a common loading condition for an indeterminate beam that is fixed at one end and pinned at the other. The problem is illustrated in Figure 1 along with the governing equations. The required input data are:

- uniform load
- beam length
- modulus of elasticity
- moment of inertia

The desired result is to produce a graphical rep-
representation of the shear, bending moment and deflection for a beam of this type.

Spreadsheet Input Area. The area on the PC's screen for data input is created in a portion of the spreadsheet as shown in Figure 2. Text is entered by locating the cursor at a cell and typing in the desired characters. At times, some of the text will extend into the next column. If one of these overlapped cells was subsequently filled by an entry at that location, it would simply hide from view the extended portion of the text.

Setting up an input section in a spreadsheet is much like using any common PC word-processing program. Unlike setting up an input area using a conventional programming language, a spreadsheet lets the user see the formatting of the text, data input areas, equations and results display areas on the screen as it will appear when the spreadsheet is completed. Note that the cursor is positioned at the first data entry at cell F6. Input values are entered by locating the cursor at the appropriate cell (cells F6 to F9 in this case) and typing in the numerical value. Figure 3 shows the spreadsheet with a set of input data entered. The format of the data in a cell may be set to any desired number of decimal places. Symbols such as hyphens and equal signs can be entered across a row of cells to isolate and highlight portions of the spreadsheet as was done in this example.

Equation Entry. The next step is to enter the appropriate equations that serve to manipulate the input data in order to produce the desired results. The equations most often utilize standard arithmetic operations, but can take advantage of any functions from the spreadsheet's library of mathematical functions such as sine, square root, etc. These functions are similar to those provided by any common programming language. Descriptive text can also be entered in nearby cells to clearly identify the results and/or the procedure used. In the example here, the equations for the vertical reactions have been entered into cells J7 and J8.
The equations entered can refer to the cell addresses in manipulating the data they contain as shown at the top of Figure 4. Alternatively, a technique of interest to engineers accustomed to dealing with variable names may be used. This technique, called range naming, involves assigning a name to a cell or group of cells similar to a variable name in other programming languages. This name may then be used in place of the cell address in the equation. For example, by naming cells F6 and F7, \( w \) and \( L \), respectively, the equation in cell J7 for the left reaction takes on the more meaningful form as shown at the top of Figure 5. This technique is highly recommended since it eliminates the need to keep track of cell addresses and it is much easier to debug equations in this form.

The remainder of the equations are set up in a table as shown in Figure 6, with the shear, moment and deflection calculated at 1/10 span intervals. At first glance, it may seem like a great deal of work to enter the 33 equations to produce the results shown in the table, but most spreadsheet software provide sophisticated means of copying cells, either exactly or by incrementing certain cell addresses as needed. For example, in this case, all of the equations at the different intervals are identical except for the variation in the distance \( x \). The following steps represent the method used to copy the deflection equations:

1. Enter the "basic" equation as shown at the top of Figure 6 for the deflection in cell B19.
2. Specify that the cell address of variable \( x \) is the only one in the equation that is to be incremented as the equation is copied to the next cell.
3. Copy the basic equation to cells C19 to L19.

This copying sequence is carried out using only a few keystrokes. Note that range names have
been given to all of the input variables as specified in Figure 7.

Spreadsheet Graphics. A plot of the deflection values (see Figure 8) is easily obtained using the integrated graphics found in most spreadsheets. Depending on the spreadsheet used, the software’s command menu will essentially prompt the user for the information required after selecting the graph option (see Table 1). The plot is then displayed as shown in Figure 8. Embellishments such as axis names can be easily added by going through more graph command sequences in a similar procedure. Once the plotting parameters have been defined, it is not necessary to go through the graph creation steps again. In most cases, the user just pushes a key to view the deflected shape after making any changes to the input data. In a similar manner, the bending moment and shear diagrams can be “programmed” into the spreadsheet.

Once the spreadsheet is set up as in this example, it may be saved and reused on any similar problem. It is then an easy task to adjust

| Table 1 |
|-----------------|-----------------|-----------------|
| **Screen Prompt/Action** | **Response** | **Comments** |
| Select Graph Type | Line | Choice of line, bar, X-Y or pie graph |
| Set X-Axis Range | X_Values | Select previously defined range (Fig. 7) |
| Set First Data Range | Y_Values | Select previously defined range (Fig. 7) |
| View Graph | View | Deflection diagram appears |
BEAM FIXED AT ONE END, SUPPORTED AT OTHER -- UNIFORMLY LOADED

INPUT VALUES

- Uniform Load (W) = \( \frac{W}{L} \)
- Beam Length (L) = \( \frac{L}{L} \)
- Mod. of Elast. (E) = \( \frac{E}{E} \)
- Moment of Inertia (I) = \( \frac{I}{I} \)

END REACTIONS
- \( R_1 = 27 \) k
- \( R_2 = 45 \) k

SHEAR, MOMENT, AND DEFLECTION VALUES AT (1/10) SPAN INTERVALS

\[ X = \begin{array}{cccccccc} 0 & .1L & .2L & .3L & .4L & .5L & .6L & .7L \end{array} \]

\[ \begin{array}{cccccccc} \text{Y} \end{array} = \begin{array}{cccccccc} 0 & 342 & 740 & 1138 & 1536 & 1934 & 2332 \end{array} \]

<table>
<thead>
<tr>
<th>END REACTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>27k</td>
</tr>
<tr>
<td>45k</td>
</tr>
</tbody>
</table>

FIGURE 7. The range names used in the spreadsheet example (equivalent to variable names).

FIGURE 8. Deflected plot as displayed using spreadsheet graphics capabilities.
any input quantity and see the effect on the results, either numerically or graphically, almost instantaneously. This simple example illustrates some of the advantages that spreadsheets have over conventional programming languages. One of the most time-consuming programming tasks is dealing with input and output. Writing additional program source code to provide graphical display of results greatly increases the time needed to develop the software.

This example shows that spreadsheet “programming” is similar to working out a problem with a handheld calculator and a piece of paper. There is no need to learn a specific programming language — spreadsheet “programming” is menu- or command-driven. This makes spreadsheet solutions amenable to some unique problems that may not warrant the time to program conventionally. The more complex design example in the next section uses these same techniques to make the interactive graphics a key design aid as opposed to simply a “nice effect.”

Design Example

Problem Statement. The design of reinforced concrete beams or columns for combined axial loads and bending moments can present a formidable task with regard to the number of calculations and iterations required to converge on an economical solution. The assumptions and recommendations presented by the American Concrete Institute (ACI) reduce the analysis of a trial section to a pure statics problem. However, depending on the relative magnitude of the axial load and bending moment, the column may have a primary failure mode caused by compression in the concrete or it may fail due to a yielding of the reinforcing steel. Solutions to this type of problem are presented in virtually all reinforced concrete design textbooks (see, for example, Wang and Salmon).
The manual computations required to produce an interaction diagram for a given cross-section are very tedious. Non-dimensionalized diagrams are available as design aids, as well as tabular design aids. While these design aids do serve to reduce the work (as compared with a totally manual analysis), they still have some major limitations. Most require some preliminary calculations prior to using a non-dimensionalized chart, and few give the user an opportunity to view an interaction diagram for a specific section. Computer programs are available for the design of reinforced concrete columns, and for determining key points required to plot an interaction diagram. However, the use of a spreadsheet with built-in graphics capabilities can allow a designer to easily vary any parameter and instantly see the interaction diagram.

The interaction diagram is shown in Figure 9. Curve (a) represents the combinations of axial load, $P$, and bending moment, $M$, that would cause ultimate failure in the column. Curve (b) represents the design interaction curve that incorporates a strength reduction factor $\phi$, a specified minimum eccentricity that flattens out the curve from points 1 to 2, and a linear transition from a specified minimum axial load to pure bending that takes into account a transition in strength reduction factors from points 4 to 5. Curve (c) is a modified design curve taking into account the five points shown. This modified curve would be conservative, but yet not overly so. The modified design curve is the one produced by the spreadsheet application described.

**Spreadsheet Operations.** As noted in Figure 10, the "home" screen of the spreadsheet includes areas for primary and secondary input. Primary input includes five design parameters that vary frequently. The secondary input includes five other design parameters that may be changed, but usually remain constant and are thus set to default values. Also shown in this figure is an area for preliminary calculations. These calculations are performed automatical-
FIGURE 11. Interaction diagram for the data shown in the “home” screen in Figure 10.

ly by the spreadsheet and are based on the current input values. All input data are automatically checked to insure that the appropriate ACI Code provisions are satisfied. Messages regarding conformance or non-conformance to these code requirements will cause a “flag” to appear in the comments section of the screen as shown in Figure 10.

After the designer has entered the desired input, an interaction diagram for this particular column can be displayed. For example, Figure 11 shows the screen display of the interaction diagram produced from the input data of Figure 10.

In addition to the column material and cross-sectional data, the designer may wish to enter the factored axial load, \( P_u \), and bending moment, \( M_{u} \), that are acting on the column as a secondary input item. This applied loading combination is plotted on the interaction diagram as shown in Figure 11. If this point is within the boundary of the interaction diagram, the chosen design parameters represent an acceptable solution. Generally, the closer this point is to the boundary, the more efficient is the solution.

After viewing an interaction diagram, the designer returns to the original input screen. If any input changes are necessary, the cursor is moved to the appropriate cell and new data is entered. The spreadsheet instantaneously recalculates all values anytime a change is made in either the primary or secondary input. The subsequent interaction diagram may again be viewed. This process may be repeated many times within a short period of time in order to determine the most desirable solution.

A situation commonly encountered in reinforced concrete design is the adjustment of the area of steel within a given column size to provide a required capacity. A change in column size could require a reanalysis of the building frame since reinforced concrete structures are generally monolithic and statically indeterminate. It is also economically unsound to vary column size to suit the load on each floor.
level. Greater economies are achieved by maintaining the same column size for the entire building height and varying the reinforcement to correspond to lighter loads at higher levels.14 As an example of varying this design parameter, Figure 12 was generated by changing the size of the reinforcing bars from No. 10 (diameter equal to 1.27 in.) to No. 9 (diameter equal to 1.128 in.) on the original input screen shown in Figure 10.

The software used for this example generated the new interaction diagram in under 10 seconds. A designer using hand computations, even with the aid of design table or diagrams, will usually be satisfied when a trial section is within allowable limits and will be less likely to check for more economical alternatives. This example gives an indication of how this spreadsheet approach would enable a designer to refine a design very quickly and produce a more economical solution. The designer would also know exactly where the applied loading combination fell on the interaction diagram, providing more confidence in the design.

Conclusions

The spreadsheet application example discussed herein was not developed with the intention of competing with existing computer programs or design aids, but rather to illustrate the tremendous potential these programs have with regard to engineering applications.

Spreadsheets offer four distinct advantages over conventional programming languages, programming environments and specific off-the-shelf engineering applications software.

- They can be developed without requiring the knowledge of higher level programming languages and programming skills.
- They offer superior input/output capabilities.
- They can provide “instant” graphical solutions.
They are specifically designed for the "What if?" effects required for sensitivity analysis.

In addition to these primary benefits of using commercially available spreadsheet software for PCs, there are other adjunct advantages such as:

- the ability to import and export data to and from other database or word-processing applications software;
- built-in database functions that are available with some commercial spreadsheet software;
- options to protect data, input areas and formulas from accidental change or intentional tampering; and,
- the ability to create a "template" spreadsheet that can be used for different applications and by many users.

Engineers need to give serious consideration to the many features that electronic spreadsheets have to offer. Because of the potentially large increase in productivity, engineers who do make use of this new technology will have time to investigate more variations, thus creating better, more cost effective designs.

NOTE — The spreadsheet software used for this article was Lotus 1-2-3 Version 2.0 run on an IBM PC. Menu and command sequences, as well as the specific options and functions, provided by specific commercial spreadsheet software for microcomputers varies widely and cannot be generalized. Most commercial PC software do fulfill the functions/requirements as set forth in this article.

REFERENCES
7. Wenzel, T., "Use of Spreadsheet Programs in Teaching Reinforced Concrete Design," Computer Applications in Concrete Technology, ACI SP-98, American Concrete Institute, Detroit, MI, 1987.
9. American Concrete Institute, Building Code Requirements for Reinforced Concrete (ACI 318-83), ACI, Detroit, MI, 1983.
A Comparative Experimental Study of Reinforced Lightweight Concrete Roof Slabs

Reinforced lightweight concrete one-way slabs made using perlite, styropor and pumice lightweight aggregates and aerated concrete slabs can behave favorably in terms of load-carrying capacities, deflections, initial crack loads and crack widths.

MURAT GÜROL, MEHMET A. TASDEMIR & FERRUH KOCATASKIN

UMICE, VOLCANIC TUFF and volcanic cinders are the most abundant natural lightweight aggregates in Turkey, where pumice reserves in the regions of Sarikamis, Agri, Van, Bitlis, Kayseri and Nevsehir are estimated to be 15 billion cubic meters. Also, there are approximately eight billion tons of perlite reserves in Turkey. Generally, artificial lightweight aggregates are used for structural lightweight concrete, but artificial ones are not yet available in Turkey and other parts of the world. However, before an artificial lightweight aggregate industry is developed in these areas, these abundant natural lightweight aggregate reserves should be considered for possible utilization. Natural lightweight aggregates do have lower strengths and endurance compared with artificial ones. In spite of this fact, if necessary precautions against moisture effects are taken, it is possible to produce reinforced concrete structural elements such as roof slabs and wall panels of moderate strength and of low thermal conductivity using natural lightweight aggregates. The use of lightweight reinforced concrete in the earthquake-prone areas of Turkey, for example, can also be beneficial in reducing the lateral forces and moments caused by earthquakes.

Experimental Study
The main objective of the study was to find out how roof slabs produced using natural pumice aggregates compared to slabs of other types of lightweight concrete. Nine reinforced...
lightweight concrete one-way slabs fabricated using perlite, styropor and pumice lightweight aggregates and three aerated concrete slabs were tested in flexure under short-term four-point loading until failure. For comparative purposes, unit weights (800 kg/m$^3$), dimensions and reinforcing of all slabs were kept equal to the aerated concrete ones. In addition, three cylindrical samples were cast from each slab mix for uniaxial compression testing.

The midspan deflections and extreme fiber strains of the slabs were measured during the flexure tests. Compression and deformation tests were also performed on the cylinders.

**Materials**

The natural sand used in perlite and styropor slabs possessed a continuous grading 0 to 4 mm with a specific gravity of 2.66 and a bulk unit weight of 1.54 kg/dm$^3$. Moisture absorption of the sand was 2 percent.

Pumice lightweight aggregate brought from the Kayseri-Develi district had a size range of 2 to 16 mm. In order to obtain a homogeneous lightweight aggregate, only particles that floated on water were used in the casting of the concrete. Some of the properties of these floating fractions are given in Table 1. The water absorption of the pumice aggregate was 84.3 percent insoluble SiO$_2$ and 6 percent soluble SiO$_2$ with small amounts of Al$_2$O$_3$, FeO$_2$, CaO and MgO.

The perlite used in the experiments was in the size range of 0 to 3 mm with a specific gravity of 0.450 and a bulk density of 0.435 kg/dm$^3$. The water absorption of the perlite lightweight aggregate was 174 percent with respect to its dry weight after 24 hours.

The styropor was a mixture of three size fractions 1 to 2, 2 to 4 and 4 to 8 mm. Their specific gravities were 0.034, 0.027 and 0.016, respectively. The water absorption of styropor was zero.

The cement used was a Type I cement.

The reinforcing bars used in the slabs were cold-drawn plain bars of 4.25 mm diameter. Their yield strength, $F_y$, was 610 N/mm$^2$; ultimate strength was 670 N/mm$^2$; and total elongation was 8 percent.

**Mix Design**

The absolute volumes method was used in calculating the mix proportions. An air entraining agent (Tricosal LP) was used in the pumice concretes in order to reduce the unit weight of the concrete to approximately 800 kg/m$^3$.

Mixes were prepared in a small laboratory mixer with vertical rotation axis by forced mixing. A 30-minute period of water absorption was allowed before mixing the lightweight aggregate. This absorption period prevented difficulties in workability and placing from occurring. Duration of the mixing was 2 minutes. Vibration was used for short durations while the fresh concrete was poured into the molds. Together with every slab, three test cylinders were cast from the same mix. The molds were taken off after 2 to 3 days. The concrete was moist-cured for 7 days under polyethylene sheets and wet burlap. Afterwards, they were cured in air for 56 days in a room of 65 percent ($\pm$5 percent) relative humidity and 20°C ($\pm$3°C) temperature.

The mixes and production costs for a typical plant are given in Table 2 with 1987 prices. The factory production costs of aerated slabs are known to be $64/m$^3$.

![Table 1](image)

<table>
<thead>
<tr>
<th>Size Fractions (mm/mm)</th>
<th>Specific Gravity</th>
<th>Bulk Density (Oven Dry)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Oven Dry</td>
<td>Saturated</td>
</tr>
<tr>
<td>2/4</td>
<td>0.80</td>
<td>0.78</td>
</tr>
<tr>
<td>4/8</td>
<td>0.78</td>
<td>1.00</td>
</tr>
<tr>
<td>8/16</td>
<td>0.74</td>
<td>0.74</td>
</tr>
</tbody>
</table>
The Mix Design & Production Costs of the Concretes Produced

<table>
<thead>
<tr>
<th>Type of Concrete</th>
<th>Cement Content (kg)</th>
<th>Total Water (dm³)</th>
<th>Sand (kg)</th>
<th>Admixture</th>
<th>Total Weight of Aggregate (kg)</th>
<th>Total Weight of Water (kg)</th>
<th>Total Weight of Fresh Unit Weight (kg/m³)</th>
<th>Total Cost Water/Cement Ratio ($/m³)</th>
<th>Total Cost of Material (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perlite</td>
<td>346</td>
<td>400</td>
<td>66</td>
<td>—</td>
<td>107.0</td>
<td>1019</td>
<td>1.44</td>
<td>65</td>
<td>1.44</td>
</tr>
<tr>
<td>Styropor</td>
<td>402</td>
<td>182</td>
<td>324</td>
<td>—</td>
<td>17.7</td>
<td>926</td>
<td>0.45</td>
<td>104</td>
<td>0.45</td>
</tr>
<tr>
<td>Pumice</td>
<td>315</td>
<td>213</td>
<td>—</td>
<td>1.58</td>
<td>389.0</td>
<td>919</td>
<td>0.68</td>
<td>40</td>
<td>0.68</td>
</tr>
</tbody>
</table>

Reinforcement of the Slabs

The reinforcement used in the slabs consisted of 4.25 mm diameter plain, hard steel bars with welded cross-bars of the same steel as shown in Figure 1. The reinforcement of the slabs consisted of four 4.25 mm diameter bars at the bottom for tension, $A_t$, and two 4.25 mm diameter bars at the top for compression, $A_c$, or utilized for constrictive reinforcement.

Flexural Tests of the Slabs

Flexural tests were performed on the slabs after the 56-day aging period using quarter-point

![FIGURE 1. Reinforcing details of the doubly reinforced slab.](image-url)
loading as shown in Figure 2 according to the Turkish Standard No. 453. At each loading increment, crack lengths were marked on the slab and crack widths were read with a feeler gage that provided 0.05 mm accuracy. At the end of every loading period, deflections for the center and quarter points of the slabs were read with dial gages having 0.01 mm accuracy. For each loading, longitudinal strains were read at five locations along the height of the slabs using Demec points on the side surface of the concrete at the center of the span over a 200-mm gage length. Each division of the scale on the dial corresponded to a strain level of $8 \times 10^{-6}$.

Also, short-term compression tests were performed on $15 \times 30$ cm cylinder specimens taken from perlite, styropor and pumice concretes at the end of the 56-day aging period. The modulus of elasticity of the concrete was obtained from the slope of the stress-strain diagram by linear correlation up to the level of $1/3$ times the compressive strength. In order to obtain the modulus of elasticity and compressive strength of aerated concrete, prismatic specimens of $100 \times 100 \times 200$ mm were prepared. The loading duration for the flexural tests of the slabs was taken as about 35 minutes.

**Ultimate Strength Design for Flexure**

As shown in Figure 3, the design of the slab
with compression reinforcement consists of two parts: the total resisting moment, \( M \), is equal to \( M_1 + M_2 \). If \( T_1 = A_{S2}F_y \) and \( A_{S2} = A_S - \bar{A}_b \), then the nominal resisting moment, \( M_2 \), becomes \( (A_S - \bar{A}_b)F_y(d - \alpha/2) \) for part 1, where \( a = A_{S2}F_y/0.75F_{c'}/b = (A_S - \bar{A}_b)F_y/0.75F_{c'}/b \) in the case of lightweight concrete. If the area of the concrete stress-block is smaller than normal concrete, calculations similar to those used for normal concrete slabs can be applied. The moment for part 2 is \( A_{S2}F_y(d - d') = \bar{A}_b F_y(d - d') \), where \( \bar{A}_b = A_{S2} = A_S - \bar{A}_s \), and \( A_{S2}F_y = C_2 = T_2 \). Hence, the total resisting moment can be written as:

\[
M = (A_S - \bar{A}_b)F_y(d - \alpha/2) + \bar{A}_b F_y(d - d') \tag{1}
\]

Since Equation 1 is valid only as \( \bar{A}_b \) yields, the strain compatibility check was also performed. Theoretical ultimate moments and ultimate loads for the slabs can be predicted from the above equation taking \( E_g = 210 \, kN/mm^2, A_S = 56.8 \, mm^2, \bar{A}_b = 28.4 \, mm^2 \), yield strength of the steel, \( F_y = 610 \, N/mm^2 \), \( d - d' = 72 \, mm \), and \( F_{c'} \) values from Table 3. On the other hand, the ultimate strength design of aerated one-way structural roof slabs was performed according to the rules given in the West German Standards (DIN 4223). In these calculations, the selfweight of the slabs was considered also to behave as concentrated forces acting at the quarter points for convenience. The theoretical ultimate loads and the averages of actual test results for each type of slab are given in the last two columns of Table 4.

### Results & Failure Modes

The results of the cylinder tests are shown in Table 3. All test results are average values of three tests performed for each type of concrete. With regard to the results of the flexure tests of the slabs, the average characteristic loads and unit weights for each type of slab are shown in Table 4. Dimensions of the slabs were 100 x 470 x 1,985 mm. Maximum permissible crack width was taken as 0.41 mm. All slabs, except for the perlite ones, lost their carrying capacities through flexural failure. One of the main differences between

<table>
<thead>
<tr>
<th>Slab Type</th>
<th>Average Weight of the Slab (kg)</th>
<th>Average Unit Weight of the Slab (kg)</th>
<th>Initial Crack Load (kN)</th>
<th>Average Load for 0.41 mm Crack Width (kN)</th>
<th>Average Load of Failure (kN)</th>
<th>Predicted Theoretical Ultimate Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perlite</td>
<td>76.0</td>
<td>815</td>
<td>2.10</td>
<td>2.30</td>
<td>7.85</td>
<td>11.50</td>
</tr>
<tr>
<td>Styropor</td>
<td>82.1</td>
<td>880</td>
<td>2.70</td>
<td>5.50</td>
<td>11.00</td>
<td>11.70</td>
</tr>
<tr>
<td>Aerated</td>
<td>70.5</td>
<td>755</td>
<td>4.25</td>
<td>6.45</td>
<td>9.60</td>
<td>9.50</td>
</tr>
<tr>
<td>Pumice</td>
<td>79.3</td>
<td>850</td>
<td>2.50</td>
<td>5.45</td>
<td>14.00</td>
<td>12.50</td>
</tr>
</tbody>
</table>
Table 5
The Safety Factors Against Failure & First Crack Loads

<table>
<thead>
<tr>
<th>Slab Type</th>
<th>Average Safety Factor Against Failure</th>
<th>Average Safety Factor Against First Crack Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perlite</td>
<td>3.60</td>
<td>1.29</td>
</tr>
<tr>
<td>Styropor</td>
<td>4.81</td>
<td>1.50</td>
</tr>
<tr>
<td>Aerated</td>
<td>4.51</td>
<td>2.22</td>
</tr>
<tr>
<td>Pumice</td>
<td>6.03</td>
<td>1.36</td>
</tr>
</tbody>
</table>

the types of failure of the aerated concrete slabs and the others was that while the failure cross-section was near the mid-span for the aerated concrete slabs, in all others it took place under one of the applied loads at the quarter points. This failure mode was due to the higher rigidity of the aerated slabs as compared to the others. Generally, two major cracks developed under both loads; one of them grew excessively for higher loads and caused first tensile and then compressive failure, or shear failure (as for the perlite slabs).

All perlite slabs lost their carrying capacities by diagonal shear. Once shear cracking had occurred, the slab was a simple arch or a strut and tie system and the principal cause of failure was the destruction of the anchorage of the tie. Use of inclined stirrups in these critical sections can be expected to increase the load-carrying capacities for these slabs considerably. For the styropor slabs, flexural cracks were formed in the regions of the shear span but not far enough out to develop into serious shear cracks. The failures were flexural with yielding of the steel and subsequent crushing of the concrete in the compression zone.

For the aerated slabs, the situation was just the opposite. Since there was practically no bond between the steel and concrete because of the bituminous coating on the steel, the cracks were sudden and deep, indicating that they were almost purely flexural. If deformed bar reinforcement is used, the results may differ.

All the pumice slabs reached their ultimate strengths by the yielding of the steel followed by the crushing of the concrete in the compression zone. Their failures were also ductile, like those of the styropor slabs, making excessive deflections before one of the main steels broke off. After this, they still had a carrying capacity for slightly higher loads. Among the four types of slabs, they had the highest load carrying capacity.

Service Loads, Deflections & Crack Widths

For a standard roof slab, the permissible load is 120 kg/m² in the case of snow and wind loads. For the protective layer, the permissible load is 45 kg/m². According to manufacturers' recommendations, minimum permissible ratios of span, L, to deflection, Δ, is 300 under these loads.

For structural reinforced elements made with pumice lightweight concrete, the safety factor against failure given in Turkish Standards 2823 is: ²

\[
\eta = \frac{(P_F + R + W)}{(P_N + W)}
\]

where:
- \(P_F\) = failure load
- \(P_N\) = nominal load
- \(R\) = additional load caused by loading system, and
- \(W\) = self-weight of element

In Turkish standards, the safety factor against failure is 2.3. On the other hand, the safety factor against first crack load is 1.25, which was suggested by aerated concrete manufacturers. The safety factor against failure was calculated using Equation 2. In the calculation of the safety factor against the first crack load, the same equation was used (in this case \(P_F\) was taken as first crack load). The results obtained are given in Table 5. As shown in that table, all slabs are safe against failure load and the first crack load.
The safest one against failure load is the pumice slab; however, the safest one against first crack load is the aerated concrete slab.

If the value of $L/300$ is considered, the maximum deflection, $\Delta_{max}$, becomes $L/300 = 1.845/300 = 6.15$ mm. Loads corresponding to this value were taken from the load-deflection curves, the additional load, 245 N, caused by loading system was added to this load; however, the slab’s self-load was not added.

The average value of loads against the permissible deflection of 6 mm and the deflections under service loads are shown in Table 6. As shown in this table, all slabs have smaller deflections than the permissible deflections under service loads. Among these, the slab made with pumice lightweight aggregate deflected least.

If loads corresponding to the deflection of 6 mm are considered, the aerated concrete slab has the highest value of load. At lighter loads, aerated concrete slabs deflect less.

### Capillary & Water Absorption

Capillary and water absorption tests were carried out on all concrete specimens that were used in the slabs. The capillarity coefficient, $k$, was obtained by using the following expression:

$$\frac{Q}{A^2} = kt$$  \hspace{1cm} (3)

where:

- $Q$ = the amount of water absorbed by capillarity (cm$^3$)
- $A$ = the cross-section of specimen that was contacted by water (cm$^2$)
- $t$ = time (in seconds)
- $k$ = the capillarity coefficient of the specimen (cm$^2$/sec)

Water absorption experiments were performed on the $10 \times 10 \times 10$ cm cube specimens and the capillarity tests were performed on the $10 \times 10 \times 5$ cm prism specimens.

As seen in Table 7, the water absorption of perlite concrete was found to be 127 percent and of aerated concrete to be 122 percent higher, while that of styropor was 21 percent lower than the water absorption of pumice concrete. The capillarity coefficients of perlite, styropor and aerated concretes were 60, 2 and 10 times higher than those found for pumice concrete, respectively. Pumice and styropor concretes exhibit lower water absorption and lower capillarity. Pumice concrete has the

### Table 6

<table>
<thead>
<tr>
<th>Slab Type</th>
<th>Average Load for the Deflection of 6 mm (N)</th>
<th>Deflection Under Service Loads (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perlite</td>
<td>2796</td>
<td>2.50</td>
</tr>
<tr>
<td>Styropor</td>
<td>3532</td>
<td>2.13</td>
</tr>
<tr>
<td>Aerated</td>
<td>4169</td>
<td>1.75</td>
</tr>
<tr>
<td>Pumice</td>
<td>3335</td>
<td>1.33</td>
</tr>
</tbody>
</table>

### Table 7

<table>
<thead>
<tr>
<th>Type of Concrete</th>
<th>Water Absorption (%)</th>
<th>Capillarity Coefficient (cm$^2$/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Perlite</td>
<td>62.0</td>
<td>$60 \times 10^{-5}$</td>
</tr>
<tr>
<td>Styropor</td>
<td>21.6</td>
<td>$2 \times 10^{-5}$</td>
</tr>
<tr>
<td>Aerated</td>
<td>60.6</td>
<td>$10 \times 10^{-5}$</td>
</tr>
<tr>
<td>Pumice</td>
<td>27.3</td>
<td>$1 \times 10^{-5}$</td>
</tr>
</tbody>
</table>
lowest capillarity coefficient among the four types, probably due to the air entraining agent used in its production which forms microscopic air bubbles in the concrete.

Conclusions
The results obtained in this study can be summarized as follows:

1. All four types of slabs furnished acceptable results in terms of accepted load-carrying capacities, deflections, initial crack loads and crack widths in the standards.
2. The failure types for the styropor and pumice slabs were highly ductile, while the aerated and perlite slabs failed in a more brittle manner.
3. All slabs except perlite failed due to flexure. Use of inclined stirrups is also expected to increase the load carrying capacities for perlite slabs. Perlite failed through diagonal shear.
4. Of the four types of slabs, the ones with pumice exhibited the highest load-carrying capacities, together with the lowest cost of production and capillary absorption, therefore being the most advantageous. Utilization of pumice lightweight aggregates can be recommended for structural as well as non-structural lightweight concrete elements in Turkey and other earthquake-prone regions where these materials are in abundance.

The test results showed that all the slabs behaved favorably in terms of load-carrying capacities, deflections, initial crack loads and crack widths with respect to the accepted standards. The slabs made with pumice lightweight aggregate had the highest carrying capacity. Since the locally available natural pumice aggregate is abundant in Central and East Asia Minor, it is concluded that a lightweight, insulating, and economical roofing slab can be obtained by utilizing it.

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REFERENCES
The Place of Stability Calculations in Evaluating the Safety of Existing Embankment Dams

Thorough investigations of site conditions and construction records should have precedence over stability analyses for determining the safety of embankment dams.

RALPH B. PECK

THE PURPOSE of evaluating the safety of an existing embankment dam is to ensure that the catastrophic loss of the reservoir will not occur. Many reports in which the safety of existing dams is evaluated relate the safety to the results of stability analyses. Yet, seismic considerations aside, stability analyses are often irrelevant and may even be misleading.

To be sure, one of the great achievements of soil mechanics has been the development of limit-equilibrium methods of stability analyses. Every soils student learns about them. Sophisticated computer programs exist for carrying them out. They have an important place in embankment dam engineering, primarily in design, but they can be misleading in evaluating dam safety if too much dependence is placed on the numerical values of factors of safety derived from them.

Purposes of Stability Analysis in Design

Before the implications of stability analyses with respect to the safety of existing dams can be considered, the ways in which such analyses are useful in design should be reviewed. The discussion herein is limited to limit-equilibrium analyses. Other techniques are required for investigating seismic behavior and liquefaction.

In practice, the slopes for embankment dams are not chosen on the basis of stability calculations; they are chosen by precedent. The initial
selection is based on the designer's judgment that takes into account foundation conditions, economics, the availability of materials, logistics and a whole series of technical and non-technical considerations. Having tentatively selected both exterior and interior slopes, the designer carries out stability analyses to ensure that conventional factors of safety are achieved. This application of stability analyses is a legitimate step in design.

Furthermore, inasmuch as every embankment dam differs in some respect from any other, it is useful to have a basis for comparing the proposed dam with others whose performance is known. Factors of safety provided by stability analyses for the proposed design and for existing dams constitute such a basis.

Moreover, stability analyses are valuable in comparing the efficiencies of various arrangements of the zoning of the dam. The analyses can provide insights regarding the relative merits and economies of placing superior or inferior materials of different costs in different parts of the embankment.

Stability analyses assist greatly in avoiding shear failures during construction. Many dams have experienced upstream or downstream sliding failures during construction because of weak seams in the foundation. Such failures can often be predicted and avoided by careful investigation and appraisal of the foundation conditions and appropriate equilibrium analyses.

In addition, failures during construction have been known to occur as a consequence of pore pressures induced in relatively impermeable zones by the addition of fill. These failures can be predicted by suitable equilibrium analyses combined with investigations of pore-pressure coefficients in the relevant materials and studies of the rates of dissipation. These failures also can be avoided by implementing a monitoring program that ensures that sufficient dissipation of excessive pressures occurs by instituting appropriate waiting periods during filling.

Finally, stability analyses are used to provide assurance that a dam will not fail under operating conditions. A large enough factor of safety is specified to guard against downstream sliding under a full reservoir and, in addition, an appropriate factor of safety is specified against an upstream failure resulting from rapid drawdown.

All these uses of stability analyses are legitimate parts of design. They require knowledge of foundation conditions and of the pertinent properties of the various materials involved. The necessary information from the field and the analyses are usually developed in successive steps of increasing refinement.

Stability Analyses of Existing Dams

The application of stability analyses in the design phase makes use of idealized or generalized soil properties, assumed known geometries and idealized surfaces of sliding. In contrast, if the factor of safety of an existing embankment dam is to be determined correctly, facts rather than idealizations are needed. Obtaining these facts is no simple task.

First, both the external and internal geometry of the dam must be ascertained, which may be difficult if reliable as-built drawings of the dam are not available. Second, the properties of the materials need to be determined, either from good records as they were actually placed or by investigation. The geometry of the surface of sliding can be determined by measurements if possible, or it can be established realistically from knowledge of the subsurface conditions. The shear strengths have to be ascertained in terms of effective-stress parameters along the surface of sliding including that portion of the surface within the foundation. The pore pressures on the actual surface of sliding (or on the potential surface of sliding if the actual one is not known) must be determined as well.

All these data are at best expensive to evaluate realistically, and in many instances may be impractical to determine. Obtaining them may necessitate exploratory work within and beneath the dam, itself often an undertaking detrimental to the safety of the dam.

Indeed, it is fair to say that if good construction records are unavailable, it may be impractical or virtually impossible to get adequate data for calculating the factor of safety reliably.

However, this limitation is only one of several considerations leading to the conclusion that stability analyses may be irrelevant or lead-
ing with respect to predicting the safety of an existing dam.

Failure of Embankment Dams

Embankment dams can fail either catastrophically or non-catastrophically. A catastrophic failure of whatever nature is defined as one that results in the uncontrolled loss of the reservoir with consequent loss of life and damage to property. It is the avoidance of catastrophic failures that justifies the authority given to regulatory bodies to require assessments of dam safety and to mandate remedial measures where safety appears to be questionable or inadequate. It is the legitimate goal of government, through regulatory bodies, to avoid future St. Francis (California), Teton (Idaho), Johnstown (Pennsylvania) or Baldwin Hills (California) catastrophes. Non-catastrophic failures, which may be expensive, annoying or embarrassing, should also be avoided. Owners of dams may be well advised to evaluate the probability of such failures and to take steps to prevent them. Yet, since their consequences fall far short of the calamities associated with the flood following a catastrophic failure, they fall outside the domains of public safety and the regulatory powers of government, and they do not fall within the scope of this study.

Catastrophic failures have one of four causes: overtopping, piping by backward erosion, liquefaction, or downstream sliding at high reservoir (possibly associated with toe failure due to piping by heave). The first three of these types of failures — overtopping, backward erosion and liquefaction — cannot be predicted by stability analyses. Hence, the legitimate application of stability analyses to catastrophic failure is restricted to downstream sliding, with or without loss of toe support, when there is enough water in a reservoir to do catastrophic damage if released.

Non-catastrophic failures can occur by downstream or upstream sliding, including sliding originating in the foundation, when there is no pool or when the pool is so small that its release is inconsequential. In addition, failures can occur by rapid drawdown, but such failures are not in themselves catastrophic even if the reservoir contains a high pool.

Rapid drawdown has led to significant damage in a number of instances, but there appears to be no record of catastrophic loss of a reservoir resulting from this mode of failure. Therefore, it is not included as a cause of catastrophic failure. However, the potential for catastrophic failure exists if a rapid drawdown slide could block outlet works and if spillway capacity would be inadequate to prevent overtopping in the event of such blockage. Under these circumstances the potential for a rapid drawdown requires assessment.

Critical Periods for Sliding

If an embankment dam were to fail under conditions that could be appropriately defined by a limit-equilibrium analysis, it would do so at one of three critical periods. The first of these is during construction. As the embankment rises, the factor of safety against a slope failure, and particularly against foundation failure, decreases. Such a slide would not be catastrophic unless the pool had been allowed to rise against the embankment as it was being placed. Under these circumstances, whatever pool had been accumulated might escape and cause flooding.

The second critical period is the first filling of the reservoir. If the dam survives the initial filling and if there is no blowup at the toe, the dam can be considered safe (in effect, proof-tested) against failure by piping due to heave.

The third critical period is achievement of maximum pore pressure under a full reservoir. If the dam has not failed when this condition has been reached, its safety against downstream slope failure has been demonstrated. Under many circumstances, including the presence of relatively thin cores or ample well-drained downstream shells, pore-pressure maxima follow so rapidly after the first filling that the survival of the first filling can be considered to be a demonstration of the ultimate safety of the dam under full-reservoir conditions. However, if the impervious section of the dam is thick and impermeable enough to create a time lag between the rise of the reservoir and the rise of piezometric levels in the core or supporting downstream zones, pore-pressure equilibrium may not occur for several years after the reservoir is first filled and the critical period may be delayed. So-called
homogeneous dams, especially in regions of high intensity of rainfall, are potentially vulnerable, and at least one failure, a small dam near Ponce, Puerto Rico, has occurred under these conditions.

In principle, it remains generally correct to postulate that if a dam survives the first filling and the corresponding pore-pressure increases under the filled condition, its safety against failure of the downstream slope has been demonstrated. There is one possible exception to this statement.

If the factor of safety is so close to unity that cyclic loading by the pool causes strain softening and critical loss of strength, the factor of safety may decrease. Aside from this special case, surviving the first filling and corresponding pore-pressure increase ensures that the dam is safe against catastrophic failure by any mode to which an equilibrium analysis is applicable.

**Failure of Walter Bouldin Dam**

Notable for its absence from the listed causes of catastrophic failure is upstream sliding leading to overtopping. Yet, the official cause of failure of Walter Bouldin Dam in Alabama, as set forth in reports of the Federal Power Commission, is an upstream slide that occurred without preceding drawdown and that breached the crest of the dam. The failure resulted in loss of the reservoir within a few hours.

It is important to the profession and to the public that the failure of Walter Bouldin Dam be correctly explained, as it would otherwise be necessary to include upstream sliding as a cause of catastrophic failure. In this case, the failure was clearly the result of subsurface erosion. The official reports and other sources can be consulted for details about the dam and of the investigations after the failure. The investigations disclosed shortcomings in both design and construction. However, these shortcomings were not responsible for the failure. The investigators did not ignore the possibility of piping, but they concentrated their attention with respect to piping on zones where seepage had been noted and extensive observational and control measures had been
FIGURE 1. Simplified diagram of the power house and dam as seen from downstream.

FIGURE 2. Longitudinal section through the east end of the Walter Bouldin power house and the junction with east wing dam.

established. They discounted the likelihood of piping at a lower elevation where it probably occurred.

The essential features of the project included an earth dam about 165 feet high across the deepest part of a valley, flanked on the west and east by wing dams founded on Pleistocene terrace deposits at a higher level. A power house was embedded in the downstream slope of the highest part of the dam; the roof of the power house was at the same elevation as a berm on the downstream slope. The power house's foundation extended into Precambrian schist bedrock overlain by Cretaceous sediments consisting largely of slightly cohesive sands and silts with layers of stiff clay. Beneath the wing dams these sediments were overlain in turn by the terrace deposits. The relationships are shown diagrammatically in Figure 1. Figure 2 depicts a longitudinal section of the area through the east end of the power house and its junction with the east wing dam; it shows the excavation made through the Cretaceous soils into the schist to reach suitable foundation support for the power house raft.

There was one eyewitness to the events leading up to the failure: Mr. Sanford, the night guard. He was interrogated many times in the course of the ensuing investigation and recounted a remarkably consistent series of recollections. As a non-technical person, he had no hypotheses about the causes of failure and apparently had no reason to report other than what he experienced.

The chronology of Sanford's activities on the cloudy, moonless night of the failure can be traced with reference to Figure 1, a simplified sketch of the power house and dam as seen from downstream. He went on duty about 9:45 p.m., made his first routine inspection starting at his office in the reception room (A) at the northwest corner of the roof of the power house, and returned about midnight without having observed anything unusual. After reading the Sunday paper for some time in the reception room, he glanced out a west window...
and noticed that the paved gutter along the north edge of the roof was running nearly full of muddy water. He was unable to see the drain or embankment directly north of the reception area, because there were no windows on that side of the building. Recognizing that something was wrong, he called his supervisor. The time was 1:10 a.m. Before he completed his brief call, water was coming beneath the door into the room. At first, it was one or two inches deep, but it quickly became deeper.

By this time he had decided to leave. As he did so, he looked around from (2) and saw that muddy water was now flowing over the powerhouse roof, apparently gushing from the northeast corner near the back stairwell (B). He waded through water, now 4 to 6 inches deep, to his truck (T) and drove from (3) to the gate (4). At (4) he got out of the truck, opened the gate, and looked back for a few minutes. He observed that the light (C), which was located 26 feet east of the powerhouse on the crest of the dam, was illuminated, but he could not observe many other details of the dam because of the intervening switchyard and other objects. He thought he saw a cavity behind the stairwell and mist rising behind it, possibly from beyond the crest of the dam.

He then noticed that the water on the road and on the powerhouse roof was getting shallower, so he decided to walk back for a closer look. He stopped (5) in front of the warehouse and while standing there heard rocks begin to fall on the roof and on some of the equipment it supported. There was then an electric flash and the lights went out, including the one at (C). The clock in the control room was later found to have stopped at 1:33 a.m., about 23 minutes after Sanford’s phone call to his supervisor. Shortly thereafter, other plant personnel arrived; by that time they could see, with the aid of their automobile lights, that a gap existed in the crest of the dam and that water was pouring through.

Sanford’s account is fully compatible with the progressive collapsing of the roof of an erosion tunnel that had penetrated through the dam at the east end of the powerhouse, as suggested schematically in Figure 1. When the elevation of the tunnel reached that of the berm, water was able to flow over the powerhouse roof. As erosion progressed, one of the successive collapses partially blocked the tunnel and the flow decreased, prompting Sanford to return as far as the warehouse. The final collapse, accompanied by falling riprap, breached
the crest. The similarity of the events to the development of the erosion tunnel through Teton Dam is evident (see Figure 3).

The official reports concluded that the failure started as an upstream slide extensive enough to breach the crest to a level below the reservoir surface, whereupon the water flowed through the gap and initiated the erosion. This scenario is incompatible with Sanford’s account. Foremost among the facts that cannot be explained by the upstream slide hypothesis is that the light at the crest of the dam, located where the gut ultimately developed, remained illuminated for more than 20 minutes after muddy water began to flow over the roof of the power house. Had overtopping occurred through a slide-produced gap, the lamp post or its power supply cables would have been among the first casualties, and the events described by Sanford would have taken place in the dark. Furthermore, the temporary decrease in the flow that he observed when he reached the gate is characteristic of blockage caused by collapse of material overlying a tunnel, whereas the flow through an open channel would only have increased with time. It is also noteworthy that the horizontal thickness of the dam at the water line was some 64 feet. To allow the reservoir to escape, a slide would have had to extend upstream at least this distance. The extent of such a slide parallel to the axis of the dam would have had to be of comparable magnitude, several times greater than the 26 feet from the end of the power house to the lamp post at (C). Thus, the crest light would have disappeared with the first earth movement. Yet, it continued to function for at least twenty minutes after the dam was releasing water.

From the time that Sanford first detected trouble, his description fits the classic mechanism of the upward development of an erosion tunnel. The scenario would not be complete, however, unless a set of physical conditions existed that would permit the initial undetected development of such a tunnel. The official reports correctly noted that seepage was expected by the designers and had occurred extensively at the downstream toes of the east and west wing dams where they rested on permeable terrace deposits. Relief wells had been installed, along with other remedial and observation works, to control the seepage. Indeed, the record is replete with references to the attention given to the prevention of piping at and above the Tertiary-Cretaceous interface. The investigators, however, discounted the
FIGURE 5. Erosion tunnel at the contact of two layers of Cretaceous soils in excavation at the Bouldin damsite. Note the vertical joint in the cohesive gravelly material.

possibility of piping in the underlying Cretaceous materials, although these materials contained many slightly cohesive, highly erodible sands and silts (see Figures 4 and 5).

The foundation raft of the power house was cast inside formwork in an excavation extending a few feet into somewhat weathered schist underlying the Cretaceous beds. An approximate section through the edge of the raft and adjacent materials is shown in Figure 2. The geometry of the backfilled zone would have favored seepage along its boundaries. Seepage could also have developed readily through the joints and more pervious zones in the Cretaceous deposits. Indeed, the excavation for the power house and intake structure required dewatering by well points; three tiers were installed, the lower two of which were entirely in the Cretaceous beds. Thus, avenues for seepage were undoubtedly present. Backward erosion could have gone unobserved below tailwater level for a long time until finally an erosion tunnel reached the reservoir and permitted concentrated destructive flows to cause rapid enlargement and failure.

The course of events before the tunnel reached the level of the power house roof is speculative, but the existence of conditions favorable to the development of a tunnel by backward erosion is not. Neither are the events observed after the tunnel reached the level of the power house roof. The conclusion seems inescapable that failure actually occurred by piping, and that any supposed shortcomings in the construction of the embankment, even if they existed, were irrelevant.

The official reports postulate an upstream slide without the destabilizing influence of a drawdown. This explanation in itself is logically questionable. An earlier shallow drawdown slide in the steep upstream slope had occurred, however, and had been repaired by dumping crushed stone and rockfill in the affected area. The investigators reasoned that the strength of the clayey materials under the dumped fill had gradually deteriorated as their moisture content increased, and that at the time of failure the strength had reduced to the extent that the slide was reactivated. This hypothesis, like that of any upstream slide, is incompatible with the events recounted by Sanford and with the extent of a slide that would have been required to lower the top of the dam below reservoir level.

In short, the failure of Walter Bouldin Dam occurred because of piping by backward erosion. As no other example of catastrophic failure and loss of reservoir has been attributed to an upstream slide, it may be concluded that a dam that has successfully survived construction will not experience a catastrophic upstream slope failure. Any analysis that indicates otherwise must be erroneous. Further, a stability analysis to investigate the safety of the upstream slope under full reservoir is irrelevant. If an upstream slope does not fail during construction, its factor of safety must exceed unity under the more favorable condition of reservoir loading. Rapid drawdown may induce a failure, but such a failure is shallow and has never been known to cut back into
the embankment far enough to permit overtopping and cause the catastrophic loss of a reservoir.

There is a remote possibility that a dam consisting of fine-grained soils possessing shear strength due to capillarity, or containing stiff cohesive materials susceptible to swelling, may lose strength when submerged if the thickness of stable upstream shell material is inadequate. The upstream slopes of dams containing such materials are usually fairly flat and failure surfaces would tend to be shallow, with consequences similar to those of drawdown failures. Again, no catastrophic failure of this type is known.

If Walter Bouldin Dam is eliminated from the category of failure by upstream sliding, then it may be concluded that once a reservoir has been filled and the associated pore-pressure increases have been achieved, the factor of safety is at least equal to unity with respect to limit-equilibrium conditions, and that any calculation showing a factor of safety less than unity must be in error.

Downstream Slope Failures

The factor of safety of a dam that survives its first filling and the associated increases in pore pressure will increase with time, unless this factor of safety is so close to unity that cyclic loading produced by fluctuations in the level of the pool causes strain softening and a critical loss of strength. If, however, the factor of safety is indeed so close to unity, downstream slope failure will be preceded by progressive and increasing increments of movement at successive full pool levels. Any calculation showing a factor of safety appreciably different from unity under these conditions must be erroneous. Stability calculations are thus irrelevant in assessing the safety of such a structure; observations of movement must take their place. If successive periods of full reservoir are accompanied by decreasing increments of movement, the stability of the structure is increasing. If the contrary occurs, the stability may be decreasing. Stability calculations may be useful in judging the influence of various remedial measures, but the computed magnitudes of the factor of safety are meaningless. An outstanding example of the irrelevance of stability calculations under conditions of decreasing increments of movement is Gardiner Dam on the South Saskatchewan River in Canada. The case of this dam illustrates the limitations of equilibrium stability analyses.

Behavior of Gardiner Dam

Conception, design and construction of Gardiner Dam took place during the quarter century in which understanding of shear strength was undergoing its most radical revisions, and at each step in the evolution of the design the geotechnical studies reflected the new frontiers of knowledge. The following history is greatly abbreviated, perhaps beyond tolerable limits, but since the project has been exceptionally well documented, the interested reader can readily learn the details 51617

At the site the South Saskatchewan River flows in a valley cut into the Cretaceous Bearpaw formation, of which the main shale member at the site, the Snakebite, is of high plasticity and contains bentonite or bentonitic zones with liquid limits ranging up to about 300. The depth of the shale bedrock valley at the site is about 75 m, but the bottom 30 m are filled with aluvium. The valley is bordered by wide zones of slump or landslide topography giving testimony to the propensity for stability problems during excavation and fill placement (see Figure 6).

In 1943, the Prairie Farm Rehabilitation Administration of Canada (PFRA) began studies for an irrigation project involving a dam across the river. Total stress stability analyses were the rule at the time, and the initial design was based on two sets of undrained peak-strength parameters: $c = 15$ to 20 psi, $\phi = 10^{\circ}$; and $c = 20$ psi, $\phi = 0^{\circ}$. Circular surfaces of sliding were assumed, and a factor of safety, $FS = 2.7$, was adopted for the end-of-construction condition.

The early geotechnical studies were carried out by Robert Peterson with equipment representing the latest Harvard designs. Subsequently, Arthur Casagrande, engaged as a consultant, turned the emphasis to a study of the slumped slopes in the vicinity supplemented by laboratory tests on a few select samples, by a test drift in which surfaces of sliding in the shale could be observed, by instrumentation to detect movements and by installation of...
Backfiguring the undrained strengths of the shales from the observed natural slumped slopes, on the assumption of circular surfaces of sliding, led to the conclusion that the most appropriate values for design were $c = 3$ psi and $\phi = 3^\circ$. A factor of safety of 1.1 was considered adequate for the end of construction because the interpretation was believed to be conservative. Even so, the design slopes required flattening.

During the early stages of construction from 1959 to 1961, however, minor excavations reactivated slips at calculated factors of safety greater than unity, according to analyses based on the design parameters, and a re-evaluation was undertaken. By now, effective stress analyses had become the norm. When the reactivated slips were backfigured, with composite surfaces of sliding and with piezometric levels assumed to be near the upper surface of the shale, shear parameters $c' = 0$, $\phi' = 8.5^\circ$ to $11.5^\circ$ were found. A reanalysis of the slumped slopes as they existed before construction activities, carried out under the same premises, indicated $c' = 0$, $\phi' = 8^\circ$ to $7^\circ$. The design was then revised on the basis of $c' = 0$, $\phi' = 9^\circ$ in the foundation, and the slopes were flattened accordingly.

A minimum value $FS = 1.4$ was postulated for short surfaces of sliding; $FS = 1.2$ was considered sufficient for long surfaces emerging near the toe at locations where stabilizing fill could be added, and $FS = 1.3$ was required where such stabilization would not be practical because of physical constraints. Construction proceeded on this basis through 1964. Although up to 0.3 m of foundation displacement toward the river channel was noted during stage construction on the west side of the river, the deformations were considered acceptable because there was no visible indication of upstream cracks in the embankment or of overthrusting at the toe. As piezometric data accumulated, the calculations were refined by incorporation of observed values of the pore-pressure ratio, $n_u$.

However, when the river-section embankment was raised from a height of 26 m to 47 m in 1965, horizontal displacements of as much as 0.8 m took place in the downstream direction. In 1966, when the dam was raised only 11 m from this height to nearly its full height at the center line, further movements of about 0.3 m occurred. The slopes, which had already been flattened after each redesign, were flattened once more. The final slopes were extended to a distance of about 1,200 m downstream to
prevent an overthrust from developing from the ancient shear zone located some 50 m below the valley floor. Three of the successive stages in the flattening of the slopes are shown in Figure 7. Although the increment of fill near the crest had produced a disproportionately large increment of movement, the rate slowed rapidly, and the embankment was completed to its designed crest elevation. In all, a maximum displacement of over 2 m occurred in the region about 150 m downstream of the center line. The displacement was associated with an upstream-downstream compression of the downstream shell, as the displacement at the toe of the embankment 1,200 m downstream of the center line was less than 1 cm during the same time interval.

The reservoir was raised for the first time, although not to full pool, in 1967. The raising was marked by a substantial increment of downstream displacement. Each annual rise of the reservoir has been accompanied by an additional increment, although a trend for the magnitude of the increments to decrease is evident if the different peak annual reservoir levels are taken into account. However, in view of the large movements already experienced during construction, the reservoir-induced movements prompted further evaluation of the structure’s safety. Many factors were taken into account, including the numerous features of the design that were introduced to cope with the large movements that had been anticipated:

- a cross-section capable of accommodating substantial deformations without rupture of the core;
- the remoteness of the possibility of cracking and leakage through the core;
- the great depth of the preexisting shear surface below the river bed; and,
- the extensive installations of slope indicators, piezometers and other means of field observations provided to permit close surveillance.

Indeed, this and subsequent evaluations by the PFRA and its ongoing Boards of Consultants, which have enhanced confidence in the inherent safety of the dam, have placed principal dependence on the results of the observational program and the favorable trends that it has disclosed.

In his 1964 Rankine Lecture, Skempton introduced the concept of residual strength on surfaces along which large displacements had occurred. It had by then become apparent that the surface of sliding constituting the seat of principal movement was an ancient shear zone in the hard shale near the base of the Snakebite member. The zone extended from an area beneath the upstream shell of the dam to at least 1,200 m downstream of the crest. According to Skempton’s concepts, it should certainly have been at residual strength.

Reversing direct-shear tests on samples from the shear zone indicated \( c' = 0, \phi' = 2.7° \) to \( 3.3° \); rotational-shear tests indicated \( c' = 0, \phi' = 3.5° \) to \( 4.5° \). Back analyses using measured piezometric levels and composite surfaces of...
FIGURE 8. Results of stability calculation for the river section of the Gardiner Dam on the basis of the best information available in 1979.

sliding corresponded to $\phi' = 0$, $\phi'$ ranging from 2° to 7°. (In reality, three different embankments make up Gardiner Dam — all experiencing a similar history. Part of the range in the shear parameters may be ascribed to differences in the geometry and geology of the sections.) Although the shear-strength parameters from the tests and back calculations are in general agreement and leave little doubt that the residual strength is the pertinent parameter, the range in back-calculated values of $\phi$ demonstrates that the margin of uncertainty in such stability calculations is still appreciable.

One of the most recently published examples of such a calculation for the river section is illustrated in Figure 8. It represents a parametric study in which the factor of safety was calculated for various residual friction angles from 2° to 9°. The composite surface of sliding included the observed shear zone in the foundation, an active segment (taken at 3 different positions) along which peak strength was assumed, and a passive segment at the downstream end at a location corresponding to observation. Measured piezometric levels along the shear zone and in the embankment were used. The results show that for $FS = 1.0$ a residual friction angle of only 2° is needed, but even at this small value a maximum shearing displacement of over 2 m occurred during construction. The calculations illustrated are for full pool, corresponding to conditions at the end of winter, show a negligible influence of pool elevation on the factor of safety. Inasmuch as each application of the reservoir load has actually produced a distinct increment of displacement amounting to several millimeters, it is apparent that even these calculations are not yet at a stage where they are definitive predictors of behavior.

Considering the talent that was brought to bear on this project over a period of more than forty years; the degree of care and sophistication in soil testing; the data obtained from some 83 slope indicators, over 500 piezometers and an even greater number of surface reference points; and the care and continuity with which the observations have been carried out, there arises an inescapable conclusion. Forty years of research have reached the point where a limit-equilibrium stability analysis can indeed demonstrate that the calculated factor of safety of the structure is equal to the known value of approximately unity. Even so, the analysis is still deficient as it does not yet realistically account for the influence of the water level in the reservoir. The analysis is still deficient in spite of the outstanding resources of the PFRA and the unusual effort that has been expended as compared to the investigations possible in connection with the usual dam-safety assessments.

The unusual geometry and physical proper-
ties of the mass undergoing movement in Gar­
diner Dam impose inherent limitations on
limit-equilibrium analyses. The limitations
have been recognized, and finite-element
studies have been carried out with sophisti­
cated refinements. By assigning what ap­
peared to be reasonable values to the physical
properties of the various materials involved,
and by adjusting these values as required to
achieve agreement between predicted and ob­
served behavior, a model was developed that
could reproduce deformations similar to those
in the field, including the pattern of response to
cyclic reservoir operation. Prediction over
many load cycles, however, was not satisfac­
tory. Although the study provided valuable in­
sight into the behavior of the dam, it is clear,
nevertheless, that no such finite-element study,
without the calibration afforded by extensive
observational data, can yet be depended on to
indicate the degree of safety of this or probab­
ably any other existing dam.

Conclusions
The foregoing discussion leads to the con­
clusion that stability analyses are unreliable
bases for assessing the stability of an existing
dam with respect to catastrophic failure. The
conclusion strictly applies only to static condi­
tions; insight regarding the behavior in an
earthquake may be gained by analysis, al­
though not generally by limit-equilibrium
analyses.

If the pool has been filled and pore-pressure
equilibrium has been reached, the results of
stability analyses may be assessed as follows:

1. If the calculated factor of safety is less
   than unity, it must be erroneous.
2. If the calculated factor of safety is
   greater than unity, the results merely indi­
cate the obvious. The calculation is unneces­
sary to show that the dam is standing. Fur­
thermore, no definitive conclusion about the
degree of safety can be drawn from the
numerical value of a computed factor of
safety; hence, satisfying some prescribed
criterion for this value is not in itself a
suitable indicator of safety.
3. If progressive movements are occur­
rating, a calculation is irrelevant because the
factor of safety is obviously close to unity.
The actual safety can be assessed only on the
basis of monitoring the movements and as­
associated events. The procedure is ex­
emplified by the studies at Gardiner Dam,
where the crucial observations were those
indicating decreasing increments of move­
ment under successive comparable reservoir
fillings. Nevertheless, if the calculated factor
of safety is approximately unity, limit-equi­
librium calculations may be useful in judg­
ing the effectiveness of various alternatives
for increasing the safety. This use of equi­
librium analysis is justifiable, and its effec­
tiveness has been demonstrated not only
with respect to dams, but with respect to
many natural slopes. It should be clear,
however, that the absolute value of the fac­
tor of safety resulting from any of the cal­
culations is of no significance.

Stability analyses are tools for the guidance
of the investigator. They have their limitations
with respect to evaluating the stability of exist­
ing dams. It is not meant that they should never
be performed. However, the numerical values
for the factor of safety should carry little if any
weight in judging the actual safety of the struc­
ture with respect to catastrophic failure.

The great danger in placing too much em­
phasis on stability calculations is that they may
be regarded as a substitute for the much more
difficult and expensive field investigations and
historical research needed to establish the real
character of the structure in question. Some
dam owners may prefer the relatively small ex­
penditure for a perfunctory stability study in
contrast to costly and time-consuming field
studies. Of greater importance, because of the
greater potential danger, some regulatory
bodies may take more comfort in orderly
stability calculations based on unsupported or
unverifiable assumptions than in qualitative
judgments based on experience and careful in­
vestigation. Yet, the former may have little or
no relation to the real safety of the dam,
whereas the latter are essential in assessing the
likelihood or possibility of a catastrophic
failure.

This discussion quite possibly conveys a
negative impression about our ability to deter­
mine whether or not an existing embankment dam is demonstrably safe against a catastrophic failure. This impression is not the intention of this study, nor would it be a correct summarization. On the contrary, a sound engineering appraisal can almost always be reached, but such an appraisal will often require painstaking field investigations; searches for construction records and, if they are found, their assessment and interpretation; inspection of all visible features; location and interpretation of maintenance records; and in many instances installation and monitoring of instruments and devices capable of disclosing not only present behavior but also future trends. The latter requirements may sometimes preclude an immediate evaluation of the safety of the dam, and they may sometimes prove to be expensive. On the contrary, they may also sometimes establish the safety quickly and beyond reasonable doubt. However, factors of safety derived from stability analyses, even when based on what appear to be the most reasonable of assumptions, are dangerous substitutes for the thorough investigations that are needed to reach sound, even if qualitative, judgments.

ACKNOWLEDGEMENTS — A scenario concerning the failure of Walter Bouldin Dam, quite similar to that described herein, was developed independently and roughly concurrently by Thomas M. Leps. It is contained in Advanced Dam Engineering for Design, Construction, and Rehabilitation, R. B. Jansen, ed., Van Nostrand Reinhold, 1988. The Prairie Farm Rehabilitation Administration (PFRA), Agriculture Canada, provided data concerning Gardiner Dam and granted permission to publish the account contained in this article. This permission, as well as review and comments by the PFRA engineering staff and by James E. Watson, are gratefully acknowledged. Thanks are also extended to John Dunnicliff, who critically reviewed the entire manuscript. This article was originally prepared as the Third Casagrande Lecture presented to the Boston Society of Civil Engineers Section/ASCE on October 14, 1987.

RALPH B. PECK received the degrees of C.E. and D.C.E. from Rensselaer Polytechnic Institute in 1934 and 1937, respectively, and studied soil mechanics at Harvard from 1938 to 1939. He was a professor of civil engineering at the University of Illinois from 1943 to 1974. He now resides in Albuquerque, New Mexico, and is a consultant on dams, tunnels and landslides. An honorary member of ASCE, he has received numerous awards from that Society. He is a member of the National Academy of Engineering and, in 1974, received the National Medal of Science from President Gerald Ford.

REFERENCES
Concrete Formwork: Constructability & Difficulties

**Saving on formwork labor costs which are the leading costs in concrete construction should be a design objective.**

Ali Touran

The impact of formwork on the cost of a concrete structure cannot be overemphasized. Under current practice, formwork that is designed and built by the contractor is by far the most expensive item in a typical concrete building. The cost of formwork sometimes exceeds the cost of the concrete and reinforcement for the structure. The most important cost item in formwork is labor, which sometimes accounts for more than 30 percent of the total concrete cost, especially when the forms are custom-built. Formwork labor, therefore, has the highest potential for cost saving in a typical concrete job and is thus the prime target for value engineering analysis. Unfortunately, most value engineering endeavors focus on cutting the costs of concrete without giving formwork the attention that it deserves.

Concrete formwork is one of the more difficult items of a construction contract to estimate. Labor productivity (square feet of form contact area constructed per hour) is usually difficult to assess. Most of the formwork is constructed in open air, thus leaving the construction process susceptible to inclement weather. Project location, labor availability and management conditions all affect formwork construction productivity. Formwork configuration — i.e., the geometrical characteristics of the structure — is another major factor that complicates the estimator’s task in establishing productivity rates for various formwork components such as beam, column or slab.

The traditional method of construction contracting minimizes the interaction between designer and constructor before construction initiation. All too frequently, construction contracts are bid on in a great hurry, leaving the estimator little time to visualize and consider all the difficulties that may arise during the construction phase. A lack of communication between designer and builder frequently results in designs that are unnecessarily difficult and expensive to build. Unless the estimator is highly experienced and proficient, budget overrun will occur, especially in labor-intensive areas such as formwork construction. Any deviation from the most simple and repetitive pattern in forming — e.g., wall openings, windows, peculiar beam-slab intersections,
bulkheads, blockouts, etc. — will result in lower productivity rates and higher costs, since the estimator would have a difficult time considering all these factors and allowing for them in the estimate. If some quantitative factors can be calculated to allow for each type of irregularity (or difficulty), then the estimator increases the chances of estimating the formwork costs more accurately, thus precluding faulty and under-budget estimates.

There are means available to quantify such "difficulty factors" for formwork estimating. These factors can alert less experienced estimators to determine whether there will be formwork difficulties, to account for these difficulties and arrive at more accurate formwork estimates quickly. Because of the nature of construction practice, these factors depend on the individual company and its own method of managing the job. In other words, universal factors cannot be developed to be used by all contractors because of various elements affecting the productivity rates in different companies.

**Formwork Economy**

Formwork costs account for between 40 to 60 percent of the total concrete costs.\(^1\) Since formwork labor typically averages 2 to 3 times the formwork material cost, it accounts for about 30 percent of the total concrete cost and thus is the most important cost item in reinforced concrete projects.\(^2\) These data apply to forming systems such as flying forms or gang forms. If custom-built forms are used, this percentage can go higher. The high cost of formwork has spurred a great deal of innovation and technological development by formwork systems' manufacturers in the past two decades. Today's trend is toward the prefabrication of forms, assembly and erection by mechanical equipment and their reuse.\(^3\) Obviously, forming systems are justified only if enough reuse can be realized. Therefore, these systems are particularly efficient on large jobs with modular or repetitive designs.

Given the current cost breakdown for a concrete structure with formwork costs so high, trying to reduce the size of structural members in the hope of economizing the design is often a futile pursuit and usually results in more expensive structures. Generally, the more slender and delicate a member, the more time it takes to form — the cost of formwork becomes higher than the cost of concrete and reinforcement.

Computing productivity rates in the construction industry is much more complicated than in the manufacturing industries. So many variable factors affect productivity in construction that average productivity rates can be seriously misleading when applied to specific jobs. The more important factors that affect formwork productivity can be divided into two major groups.

The first group consists of factors that do not depend on the type and shape of the structure. Weather, project location, type of labor (union vs. open shop), management and contractor's experience are among the more important factors in this group. The effect of some of these factors is so profound that computing average productivity rates without categorizing the type of project would have little benefit to the estimator. Research on methods for quantifying the effects of these factors on construction productivity has been conducted.\(^6\)\(^7\)

The second group consists of factors that depend on the formwork requirements and the geometrical shape of the structural members. These factors are at least as important as the factors mentioned in the previous group. Most of the extra costs are a result of these factors and can be eliminated by carefully designing the structure, thus resulting in substantial cost savings. The designing of deep and narrow spandrel beams; changing column cross-section (e.g., from circle to square or rectangle); reducing the column dimensions every few floors; and the placement of wall openings and windows of different sizes, blockouts and pilasters and many other peculiarities in the structure all contribute to the already high cost of forming the structure.

The topic of constructability has received much publicity recently in discussions on ways to improve construction productivity and quality in the United States. O'Connor and Tatum present detailed treatments of this important subject.\(^8\)\(^9\) With regard to constructability as related to formwork and concrete structures, in order to make a concrete building easier to form and cheaper to build (yet meet-
ing all the quality standards, three design steps should be taken:

1. The design should consist of as many as possible similar modules. Using similar plans for the building floors and repeating the same layout in various building components will enable the contractor to use the same forms several times and allow the workers to become familiar with the work, thus improving the productivity rate because of learning curve effects.

2. Since manufacturers' prefabricated forms are readily available in standard sizes, designs should conform to these standard sizes, thus enabling the contractor the opportunity to employ them. Even in the case of custom-built forms, the member dimensions should be selected with the standard lumber sizes in mind, thus saving a lot of time in carpentry and reducing waste.

3. Structural members should be designed with the least amount of variation in dimensions. Using the same depth for beams and girders, the same height for floors, identical cross-sections for columns from floor to floor will all facilitate and reduce the cost of formwork construction.

This design for constructability philosophy is summarized in New Formwork Perspectives as follows:

"This approach does not ask the building designer to assume the role of a formwork planner, nor does it make the structural design a slave to formwork considerations. Its basic premise is merely that practical awareness of formwork costs may help the designer take advantage of less expensive structural solutions that are equally appropriate in terms of aesthetics, quality, and function of the building. To use this pragmatic approach the designer need only visualize the forms, visualize the field labor required to form various structural members and be aware of the direct proportion between complexity and cost."

Difficulty Factors
Difficulty factors or complicating factors are defined as factors that quantify the effect of irregularities in formwork productivity. For example, Richardson suggests that a factor of 4.5 be multiplied by average formwork productivity (manhours/square foot) for walls to arrive at the productivity rate (manhours/linear foot) for forming the intersection shown in Figure 1. Or, according to Walker's, the cost of forming the face and sides of pilasters runs from 25 to 50 percent more than plain wall forms both for material and labor (p. 8.25).

Good data on difficulty factors are hard to find. Not only is it difficult to accumulate this type of data, but the competitive nature of the business discourages its dissemination. The following sources of information on this type of data are available:

I. Construction estimating reference books. Well-known estimating reference books usually provide limited information regarding formwork difficulty factors. Richardson covers some of the situations in an independent section. Walker's and Means treat the subject while covering various formwork costs and productivity rates. For example, the wall formwork difficulty factors mentioned in Walker's cover wall pilasters, radial wall forms, forming openings in walls, and forming setbacks and haunches on walls (pp. 8.25-8.27). Since these reference books provide average data, they should be used with care.
Many contractors do not use these references because they believe that their work and conditions are very different from the average values that are presented for cost and productivity in these references.

2. The estimator’s own knowledge and experience. This type of knowledge is the most common source used during the estimating phase. Experienced estimators who have been working with a certain contractor for a number of years have a thorough knowledge of the organization’s efficiency and capability and can determine the effect of formwork difficulty factors rather quickly. The most successful estimators are those who pay a great deal of attention to detail. Estimators should routinely count the number of corners, openings, blockouts, etc., in estimating the formwork and allow for the cost increase in their estimates.

Unfortunately, the short period of time that is usually available for estimating and preparing a bid on a proposal limits this approach to only the most experienced and competent estimators. Such individuals are vital to the contractor’s success. Their knowledge is largely intuitive and not documented, or easily documentable, for use by others.

3. The contractor’s historical data. Few smaller construction companies maintain an extensive database of construction costs and productivity rates. The larger construction companies, however, with in-house computer capabilities, maintain detailed data from previous and current jobs. With the widespread use of personal computers, their simplicity and declining prices, it is quite conceivable that data collection and reduction will become more affordable for even the smallest construction firms. The level of detail in data collection and reduction varies from firm to firm. One of the more detailed systems used for data collection consists of weekly reports of progress on the different items of formwork for each floor (e.g., formwork for spandrel beams, slabs, columns, etc.) and the comparison of these data with the pre-estimated measures of productivity to compute budget overrun/underrun. The general procedure is that the project engineer assigns values for the quantities of work performed under the various categories of beam, column, etc. Data pertaining to spandrel beams that had been collected in this process were found in many instances to be assigned quantities of progress in different categories without a high degree of accuracy. In these cases, the resulting data can be hard to categorize. For example, forming the beam-column intersection, especially in complicated configurations (e.g., round columns) takes considerable time. It is not clear that this time should be assigned to beam formwork or column formwork. In order to clarify these ambiguities, footnotes and extra information should be included in the progress reports. For this reason, useful reporting on even moderate sized jobs requires a greater commitment than most contractors are prepared to make. Still, the importance of accurate and detailed productivity and cost reporting cannot be overemphasized for the serious contractor. These data, if carefully collected, can be invaluable in developing accurate estimates, winning contracts and operating at a profit in today’s extremely competitive building market. This competitive edge is extremely important. For example, a few projects involving building correctional facilities on the west coast last year were bid upon and the low bidder was not more than 0.1 percent apart from the second lowest bidder.

If a rather detailed cost and progress reporting system exists at a construction company that is based on the collected data from specific projects and the estimators’ knowledge, experience and estimating references, the values of difficulty factors can be determined for various formwork categories. These difficulty factors can then be used for estimating future projects with speed and accuracy.

Methodology

An example can be used in demonstrating the general methodology that can be applied to estimating. Assume that the concrete frame of a multi-story building has been recently completed and the contractor wishes to estimate the
difficulty factors in formwork so that they can be used in future biddings. In this case, only spandrel beams are considered. The productivity data (or total manhours) per floor have been compiled by the contractor. There are 10 bays of 20-foot and 6 bays of 25-foot spandrel beams in each floor (see Figure 2). Four types of beam cross-sections are used as shown in Figure 3. Depending on the number of floor and architectural considerations, different beam types are used on different floors. Therefore, each floor consists of a combination of different beam types with different lengths, some with blockout and some without.

Some components of these beams are more difficult to form. For example, forming the bulkhead for construction joints is much more time consuming than forming the beam sides. The bulkhead should allow for all the longitudinal rebars to pass through it, causing a lot of problems. Forming beam soffit can take much longer than beam sides because of the time involved in constructing the support system beneath the beam. Beam-slab intersection can also be relatively slow. One suggested method is to key a form into the beam face and punch it to permit the slab rebars to pass through. Building beam blockouts present another challenge that usually takes more time than routine forming. The objective is to find out the productivity rates for forming beam sides, beam soffit, bulkheads, blockouts and the beam-slab intersection. The following general equation can be developed for each floor:

$$\sum_{i=1}^{n} A_{ij} x_i = y_j$$

where:

- $n =$ total number of unknown productivity rates, and
- $j =$ floor number.

![FIGURE 2. Plan view of the building.](image)

![FIGURE 3. Beam cross-sections.](image)
For example, for Floor 1 there is:

\[ A_{11}X_1 + A_{21}X_2 + A_{31}X_3 + A_{41}X_4 + A_{51}X_5 = Y_1 \]  

(2)

where:

- \( A_{11} \) = area of beam side (ft.\(^2\)) Floor 1
- \( A_{21} \) = area of beam soffit (ft.\(^2\)) Floor 1
- \( A_{31} \) = area of beam bulkhead (ft.\(^2\)) Floor 1
- \( A_{41} \) = length of beam-slab intersection (linear feet) Floor 1
- \( A_{51} \) = area of beam blockout (ft.\(^2\)) Floor 1
- \( X_1 \) = productivity rate for beam side (manhours/ft.\(^2\))
- \( X_2 \) = productivity rate for beam soffit (manhours/ft.\(^2\))
- \( X_3 \) = productivity rate for beam bulkhead (manhours/ft.\(^2\))
- \( X_4 \) = productivity rate for beam-slab intersection (manhours/ft.)
- \( X_5 \) = productivity rate for beam blockout (manhours/ft.\(^2\))
- \( Y_1 \) = total manhours spent in Floor 1 on spandrel beams

Similar equations can be constructed for each floor. The values for \( Y_j \) can be taken from the final progress reports. The values of the \( A_{ij} \)'s can be computed from the building drawings. The unknown components are the productivity rates, \( X/\)'s, that need to be quantified. The less the number of variations in each floor, the more similar these equations should be from one floor to another. In the case of similar floors, the effect of repetition will cause the total manhours spent on each floor, \( Y_j \), to decline on the higher floors. This effect can be analyzed by the theory of learning curves. If the floors are not identical, the effect of learning curves are less profound and probably can be disregarded in the first trial. If it could be assumed that productivity rates were consistent from floor to floor, then it would be possible to use any five sets of equations to solve for five \( X/\)'s. The computed values of \( X/\)'s then should fit into the rest of the equations as well. However, for several reasons as cited above, it is difficult to imagine a situation where productivity rates remain "absolutely" constant for every floor. The

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**Table 1**

Sample Spreadsheet for Regression Analysis of Hypothetical Problem in Figure 2

<table>
<thead>
<tr>
<th>Floor No.</th>
<th>Sides (ft.(^2))</th>
<th>Soffit (ft.(^2))</th>
<th>Bulkhead (ft.(^2))</th>
<th>Beam-Slab (linear ft.)</th>
<th>Blockout (ft.(^2))</th>
<th>Total Actual (manhours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A(_{11})</td>
<td>A(_{21})</td>
<td>A(_{31})</td>
<td>A(_{41})</td>
<td>A(_{51})</td>
<td>Y(_1)</td>
</tr>
<tr>
<td>2</td>
<td>A(_{12})</td>
<td>A(_{22})</td>
<td>A(_{32})</td>
<td>A(_{42})</td>
<td>A(_{52})</td>
<td>Y(_2)</td>
</tr>
<tr>
<td>3</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
<tr>
<td>n</td>
<td>A(_{1n})</td>
<td>A(_{2n})</td>
<td>A(_{3n})</td>
<td>A(_{4n})</td>
<td>A(_{5n})</td>
<td>Y(_n)</td>
</tr>
</tbody>
</table>

---

**Table 2**

Regression Matrix for the Hypothetical Problem of Figure 2

\[
\begin{pmatrix}
A_{11} & A_{12} & A_{13} & \ldots & A_{1n} \\
A_{21} & A_{22} & A_{23} & \ldots & A_{2n} \\
A_{31} & A_{32} & A_{33} & \ldots & A_{3n} \\
\vdots & \vdots & \vdots & \ddots & \vdots \\
A_{n1} & A_{n2} & A_{n3} & \ldots & A_{nn}
\end{pmatrix}
\begin{pmatrix}
X_1 \\
X_2 \\
X_3 \\
\vdots \\
X_n
\end{pmatrix}
= 
\begin{pmatrix}
Y_1 \\
Y_2 \\
Y_3 \\
\vdots \\
Y_n
\end{pmatrix}
\]
analyst should remain content with a moderate amount of variation in the values of $X_i$. Therefore, the general problem is reduced to finding values for $X_i$ that fit into all the developed equations (one per floor) as closely as possible. It is suggested that a multiple regression analysis be performed on the data. The most simple way is, of course, to start with linear regression. Performing regression analyses on five variables requires a rather large number of equations. In the case where there are less than ten equations, for example, it can be very difficult to find meaningful values for the variables. It is possible that negative values are found for some of the $X_i$. In such a case, one or two variables should be estimated using estimating reference books or the company's historical data. After reducing the number of variables, the regression should be performed on the rest of the variables. One efficient way to do this is to use the regression routine available on some of the available commercial microcomputer-based spreadsheets. Several analyses can be done efficiently and in a short period of time. The regression matrix should appear similar to the matrices presented in Tables 1 and 2.

After the values of $X_i$ are computed, the results should be compared to the results of analyses performed on other similar projects. If the results are consistently similar across a few projects, then those productivity rates can be used in estimating future jobs. Also, the values of computed $X_i$ for a completed job can be compared with those assumed productivity rates in estimating the job when it was bid so that reasons for any significant differences can be determined.

Case Study
This proposed method for costs analyses was applied to estimating the formwork costs for a 20-story building. In the design of this 600,000 square-foot correctional facility, 59 different types of spandrel beams were used. The cross-section of beams varied from floor to floor and within each floor. The contractor had a difficult time forming all these different configurations. When the job was finished, it was decided to investigate the effect of these radical variations in beam configuration on the cost of concrete formwork. Major reasons for variations, apart from various beam dimensions, were:

- Slab tied into the beam at different levels of beam at various floors.
- Beam breakout configuration changed.
- Column pilaster changed (in some floors the column cross-section changed from square to circle)
- Difficult bulkheads due to the amount of longitudinal rebars

Based on the above observations, a number of difficulty factors were considered. Regression analysis was not successful on the first nine floors because there were so many variations on these floors that 15 difficulty factors had to be accounted for. The analysis was performed on Floors 9 to 18 by considering four difficulty factors. Beam configurations in these floors were much more consistent. The results of regression analysis, augmented by the estimator’s viewpoints, quantified some of the more important difficulty factors. For example, it took twice as long to form the beam soffits as compared to beam sides. A major cause for delay and expense was the beam-slab intersections. A form was keyed into the beam face at the intersection location and drilled to accommodate protruding slab dowel bars. On average, forming this intersection required ten times as much labor as forming the beam sides. Interested readers in the project and methodology should refer to Qabbani.

Conclusions
Formwork labor is the most expensive item in a typical concrete job. Trying to save on formwork labor, therefore, should be the designer’s objective rather than saving on concrete material. Repetition, consistency and standardization are three key issues in coming up with economical formwork construction.

The concept of formwork “difficulty factors” was discussed and a methodology proposed for quantifying these “difficulty factors.” The nature of competitive bidding in construction allows very little time to the estimator to consider all details. However, if the estimator has access to a set of appropriate difficulty factors, then the estimate can be more accurate. Due to the large number of factors affecting
productivity rates in construction, every contractor should develop their own set of difficulty factors.

The brief case study presented above demonstrates the application of a suggested method for applying difficulty factors to a specific construction project. It revealed that forming beam-slab intersections takes about ten times as long as forming beam sides. This huge difference points out the importance of considering the effect of formwork irregularities and difficulties when bidding on complex jobs.

Ali Touran is an Assistant Professor of Civil Engineering at Northeastern University. He holds Masters and Ph.D. degrees in civil engineering with an emphasis on construction management and engineering from Stanford University. His areas of expertise are construction cost and productivity analysis, simulation modeling, and heavy construction equipment and methods. He has performed research in concrete productivity, simulation and tunneling, material handling and bridge deck construction. He has lectured on simulation of construction processes, computerized data collection and data analysis.

REFERENCES
Awards

BSCES Honorary Members

The Society honors four members for their professional excellence, achievements and contributions.

At the 1988 Annual Meeting of the Boston Society of Civil Engineers Section/ASCE (BSCES), the Society paid special recognition to four of its members — John T. Christian, William J. LeMessurier, Maurice A. Reidy, Jr., and Kentaro Tsutsumi — by making them Honorary Members. Honorary Membership is the Society’s highest level of acknowledgement of its outstanding and distinguished members based on their contributions to BSCES, the engineering profession and society. Honorary Member status for these individuals was attained through a rigorous process of nomination by a BSCES committee, petition by its membership and election by the Board of Government. They join six other living Honorary Members — Harl P. Aldrich, Jr., John B. Babcock, Paul S. Crandall, Albert G.H. Dietz, Donald R.F. Harleman and John A. Volpe — to form a select group that comprises less than one-half of one percent of the Society’s membership.

John T. Christian
A pioneer and leading authority in the use of computer methods in geotechnical engineering, John T. Christian has had extensive experience in foundation engineering, earth dam analysis and design, engineering mechanics and earthquake engineering. Dr. Christian’s activities within the field of geotechnical engineering have spanned a wide spectrum of activities from preliminary design stage to the construction stage. He is a registered professional engineer in Massachusetts and Maine.

Dr. Christian received his B.S., M.S. and Ph.D. degrees in civil engineering from the Massachusetts Institute of Technology (MIT) in 1958, 1959 and 1966, respectively. After receiving his doctorate, he taught at MIT and also served as a private geotechnical consultant. His research and teaching were focused on such topics as the application of finite element methods to problems in geotechnical engineering, including consolidation, behavior of braced excavations, stability of slopes, inelastic soil deformations, earthquake problems and flow through soils. Other research topics included the field behavior of levees, development of computer-aided slope stability analysis and earthquake engineering.

After attaining the position of Associate Professor, Dr. Christian left teaching full-time at MIT in 1973 to become a consultant in the geotechnical division at Stone & Webster Engineering Co. Three years later he became Consulting Engineer and, finally, Senior Consulting Engineer in 1980. He is now the Manager of the Consulting Group at Stone & Webster. Dr.
John T. Christian

Christian has performed geotechnical and seismological work on nuclear power plants in a variety of locations. This work included probabilistic assessments of seismic hazard. He has also worked on offshore caissons, offshore mooring facilities, earth dams for storing fuel oil and water, offshore pipelines, underground rock openings, slope stability studies, highway embankments, oil field subsidence and foundation investigations for conventional buildings. Dr. Christian has performed seismological evaluations and analytical studies of earthquake effects including soil amplification, liquefaction and soil structure interaction.

In the computer field, Dr. Christian has developed finite element analysis programs for a number of applications. He has conducted studies of the flow of water through porous media, dams and other geological structures and has developed computer programs for performing the analyses and for presenting analysis results graphically. At Stone & Webster he has developed procedures for documenting computer programs and has implemented procedures for controlling the quality and accuracy of computerized calculations.

Well-versed in the interpretation of field data from soil instrumentation, he is co-editor of the book, *Numerical Methods in Geotechnical Engineering*. He authored more than 100 reports, papers and articles. A fellow of the American Society of Civil Engineers (ASCE), Dr. Christian is a member of the Technical Advisory Panel for the Seismic Research Group at the Electric Power Research Institute and of the Seismic Advisory Committee for the State Board of Building Regulations and Standards. He was also past Chairman of the Geotechnical Engineering Division of ASCE and past Chairman of the Computer Group of BSCES. He is a member of numerous professional organizations and committees. His contributions to engineering practice have been recognized by such awards as the Desmond Fitzgerald Medal from BSCES and Outstanding Correspondent Award from ASCE.

William J. LeMessurier

Trained first as an architect and then as an engineer, William J. LeMessurier has been devoted to generating professional understanding between engineers and architects throughout his career in teaching and in practice. He graduated in 1947 from Harvard with an A.B. in architecture and received his M.S. in civil engineering from MIT in 1953. A registered professional engineer in Massachusetts, the District of Columbia, New York, Tennessee and Colorado, William LeMessurier is founder and chairman of the board of LeMessurier Consultants. He has practiced structural engineering design in projects throughout the world. Major works of his were completed in Abu Dhabi, Bahrain, Egypt, France, Iran, Saudi Arabia and Singapore.

Throughout his career, William LeMessurier has been dedicated to extending engineering practice through structural innovation, constantly advancing the state of the art while paying special attention to the aesthetic aspects of structures. He is credited with several innovations in engineering. He developed the Mah-LeMessurier System, a pre-cast concrete high-rise housing system. While at MIT, Mr. LeMessurier conceived and developed the now widely used Staggered Truss System for high-rise steel structures. He also conceived, developed and applied the Tuned Mass Damper System to reduce tall building motion in the Citicorp Center, New York, and the John Hancock Building in Boston.
In recent years, he has centered on the problems posed by tall and very slender structures, tackling the effects of bending, shear and vibration. His solution to these problems is a system that possesses vertical continuity in a continuous partition at the furthest points from the horizontal center. To him, the chimney form represents the perfect super-tall structure, thinking of a skyscraper as a beam cantilevered from the earth. For LeMessurier, the chimney form is opened by transforming the wall into columns stabilized by a lattice of cross-braces or rigidly joined frames. In his design philosophy, a design is finished when there is nothing to change via adding elements or subtracting them.

William LeMessurier has been involved in the structural engineering design of such projects as the new Boston City Hall, Shawmut Bank of Boston, First National Bank of Boston, Federal Reserve Bank of Boston, Citicorp Center in New York, the National Air and Space Museum in Washington and the Ralston Purina Headquarters in St. Louis. He has taught at MIT and Harvard University Graduate School of Design where he continues as adjunct professor. He has also served as visiting lecturer at a number of institutions including Yale University, the University of Michigan, Cornell, Northeastern and the University of California at Berkeley. LeMessurier has co-authored sections of the Structural Engineering Handbook. A member of many professional societies including ASCE and AISC, he has received many awards throughout his career. Among them are Engineering News-Record awards for professional service, many Awards of Excellence and a Special Award from AISC, and the American Institute of Architects' Allied Professions Medal. He was elected to the National Academy of Engineering in 1978, and was made honorary member of the Boston Society of Architects in 1985 and honorary member of the American Institute of Architects in 1988.

Maurice A. Reidy, Jr.
Over the course of his 47-year professional career, Maurice A. Reidy, Jr., has applied soil and structural engineering principles to the field of structural and foundation design and, in later years, to problems of structural damage and failures. After earning a B.S. and M.S. in civil engineering from Harvard in 1940 and 1941, respectively, where he studied under Drs. Arthur Casagrande and Karl Terzaghi, he began his career as a soils and foundation engineer with Frederick R. Harris, Inc., in New York, working on the design and construction of Navy drydocks. From 1943 to 1946 he was an aerodynamicist for the Republic Aircraft Corporation, becoming responsible for design air loads on all components of aircraft.

In 1946, he joined the structural and foundation consulting firm of Maurice A. Reidy Engineers, founded by his father in 1924. He was involved in design of a number of well-known buildings in Boston including the Jordan Marsh Main Store, major additions to Filene's, St. Anthony Shrine, the Harvard Graduate Commons and Aldrich Hall at the Harvard Business School. An unpublished report written by Reidy on the snowdrift loads on the Jordan Marsh Squantum Facility roof in the 1978 blizzard is cited frequently in the literature on drift loads.

In the early 1950s, he received attention for demonstrating that design error was responsible for the collapse of the Sullivan Square overpass. As an expert witness, Reidy has earned a reputation for fairness and resourcefulness in a wide range of litigations relating to
Maurice A. Reidy, Jr.

structural failures. In the most noted of these cases, Reidy represented Trinity Church in its successful suit against the John Hancock Insurance Co. for damage to the church caused by construction of the neighboring Hancock Tower. For this case he developed, in 1984, the “takedown theory,” a new method for assessing the value of partial damage caused by some disturbance to a masonry building, where reconstruction is not warranted or not feasible. The theory compares the curvature or angular distortion existing in the building before and after the disturbance, each expressed as a percentage of some larger, hypothetical degree of angular distortion, the “takedown” condition, at which the building is so pervasively damaged that, by consensus, it must be taken down. The dollar value of the damage increment is the increase in that percentage times the cost of takedown and reconstruction. This theory withstood appeal to the Massachusetts State Supreme Judicial Court.

Reidy has been involved in structural rehabilitation of several important historical buildings. Among these are the Old Corner Bookstore, the Old South Meeting House and the Old North Church in Boston.

He has been in charge of Maurice A. Reidy Engineers since 1961. A registered professional engineer in Massachusetts, New York, Connecticut, the District of Columbia, New Hampshire, New Jersey and Oregon, he is a Fellow in ASCE and has served as chairman of the BCSE/ASCE Sections Structural Group. He has also served on the BSCES Seismic Advisory Committee, Massachusetts State Board of Building Regulations and Standards, and as president of the Boston Association of Structural Engineers.

Kentaro Tsutsumi

A professional engineer, teacher and inventor for more than 50 years, Kentaro Tsutsumi is a nationally recognized authority on isolating and stabilizing vibration. With his teaching deeply rooted in professional experience, Prof. Tsutsumi has taught students from all over the world and even the children of his students over his long career, instilling in them his clear sense of design and precision about the art of engineering.

Born in Hawaii, Prof. Tsutsumi received his B.S. in civil engineering from the University of Hawaii in 1936 and an M.S. in civil engineering from MIT in 1938. His contributions to engineering education started in 1939 when he taught structural analysis and design courses for the Massachusetts Dept. of Education, Division of University Extension. Prof. Tsutsumi also had teaching appointments at Northeastern University and for the Tufts University Engineering Science Management War Training Course during World War II. He returned to Tufts in 1963 as Associate Professor of Civil Engineering and was promoted to Professor in 1966.

Demonstrating the breadth of Prof. Tsutsumi’s vision as a teacher, he taught an analog computer course for engineers and scientists at Tufts in 1965. This course was the first of its kind in New England. Later on, in 1981, he introduced personal computers in the undergraduate engineering laboratory. Prof. Tsutsumi also incorporates the writings of the 12th century Persian poet and scientist, Omar Khayyam, into his classes. Another topic he has introduced is the analysis of a Picasso painting in structural engineering terms.

His professional career ranges from the design of buildings and bridges, foundation design and hydraulic design to the design of a
stable test platform for the calibration of inertial guidance systems. Holder of two patents, Prof. Tsutsumi worked on the design of several Boston Edison buildings in the 1940s and along with LeRoy Hersum, a consultant he knew from MIT, he helped rebuild Massachusetts bridges, and other structures, damaged or destroyed by the hurricane of 1938.

His longstanding consultant career includes work for MIT’s Draper Laboratory, Jackson & Moreland, NASA, Raytheon and Itek. His expertise in vibration isolation and instrumentation has been evidenced in his work for Draper Labs. He has designed such test equipment as gyro test foundations, optical equipment test foundations, elastic limit testers, radial force testers, axial force testers, centrifuges, gimbal systems and vibrator for centrifuges, and shock and vibration isolation analyses for components. His “Type T” (named after him) isolation pier that utilizes ordinary building materials isolates movements to a millionth of an inch. In 1964, he invented an instrument testing platform that is not affected by the random tilting of the earth. And in 1944 he designed a 634-foot wind tunnel that generated winds up to 600 mph and which he recently refined to generate supersonic wind speeds.

Prof. Tsutsumi retired from Tufts in 1986. He is currently the chairman of the Massachusetts State Board of Building Regulations and Standards and serves on the Mayor’s Advisory Board of Public Buildings for the City of Newton. He is involved with a number of professional organizations including ASCE, the American Institute of Steel Construction (AISC), the Society for Experimental Stress Analysis, the American Institute of Aeronautics and Astronautics, American Geophysical Union and the Seismological Institute of America. Author of many papers and articles, Prof. Tsutsumi has been awarded the Tufts Service Citation and AISC’s Special Citation Award for Exceptional Professional Achievement.
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