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TESTS UPON THIN DOMES BURIED IN SAND

By Robert V. Whitman,* Zvi Getzler,** and Kaare Höeg***

(Presented at a meeting of the Structural Section, B.S.C.E., held on December 12, 1962.)

Introduction

The nation's protective construction effort during the past decade has kindled anew the civil engineer's interest concerning the behavior of structures buried in earth. Whereas above-ground structures are readily destroyed by the air blast from a large explosion, it is possible to construct economical underground facilities which are quite resistant to blast loadings. Moreover, earth cover provides an effective shield against the radiation and heat generated by a nuclear explosion.

The transient nature of the loading from a blast wave has, of course, given rise to new design problems. In the broad view, however, the design of underground protective construction facilities is a direct extension of quite conventional problems with which the civil engineer has long wrestled: the design of tunnels, of culverts, of retaining walls, and even of grain storage bins. One conclusion is reached quickly by engineers in the protective construction field: the dynamic structural design problems would be relatively easy if only the corresponding static problems were understood clearly.

It has been somewhat surprising to discover the limitations of present knowledge regarding the response of soil-structure systems to

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static loading conditions. It is true that useful procedures have been developed for specific design problems, such as proportioning certain types of flexible culverts. It is also true that some important basic principles have been established, at least in a qualitative sense. However, the store of quantitative data regarding fundamental phenomena is inadequate for analyzing new types of design situations. Moreover, even the culvert problem gives difficulties if the parameters of the problem are changed beyond their usual limits.[1]

Present knowledge is especially scant when it comes to predicting the collapse loading for a soil-structure system involving a flexible tube, arch or dome. Not only can the earth "arch" around the structure, but the earth inhibits the usual patterns of structural deformation. While the general principles have been outlined by Marston and his associates (Spangler, 1948, 1956), there are still such important questions as: if the structure yields suddenly, can an "arch" immediately form in the surrounding earth, or what strength and stiffness are necessary in the surrounding soil to ensure that a thin arch will fail by compressive yielding rather than by buckling or bending? Such problems are difficult enough when the soil mass and structure are both cylindrical and the loading is a uniform radial pressure (Whitman and Luscher, 1962); they become very difficult when the loading is other than uniform and the structure becomes an arch or a dome.

As a step toward closing some of these gaps in the knowledge concerning soil-structure interaction, the M.I.T. Department of Civil Engineering has initiated a program of small-scale tests using static loadings. Although horizontal tubes and arches are simple forms in the structural sense, the problem of end effects always arise when testing such structures. Hence, it was decided that the program should involve structures having a vertical axis of symmetry, and a thin-walled dome was selected for the initial tests. So as to simplify the placement of soil around the structure in these initial tests, a coarse dry sand has been employed.

This paper discusses the development of testing systems and procedures, and presents the first of the experimental results.

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[1] Newmark and his associates at the University of Illinois have recently put together an excellent review of knowledge concerning soil-structure interaction. This review is distributed in very limited quantities by the Defense Atomic Support Agency as Part V of "Nuclear Geoplosics: A Sourcebook of Underground Phenomena and Effects of Nuclear Explosions." Spangler (1948, 1956) has provided excellent surveys of knowledge concerning the conduit and culvert problem. Tunnel lining behavior is discussed in the symposium by Lane and Burke (1960).

Test Bin and Loading Frame

The testing system, shown in Figure 1, involves a soil container 5 feet in diameter, a rubber bag through which a uniform static pressure is applied to the top surface of the sand mass, and a self-contained loading frame.

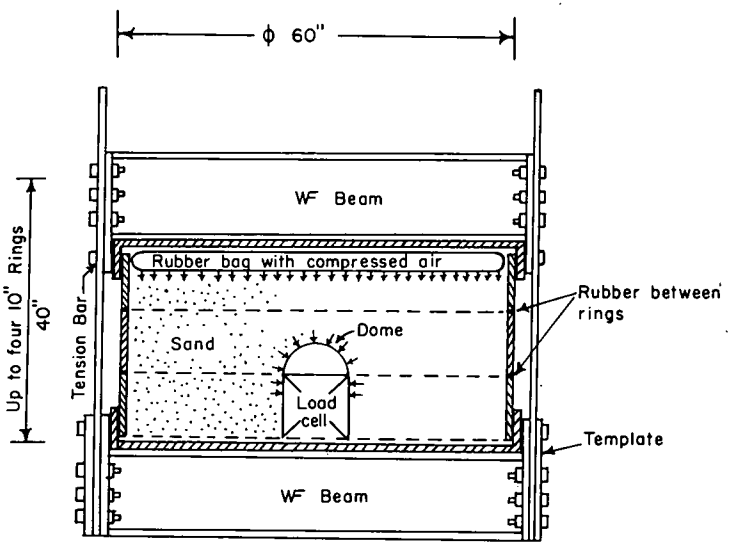

FIGURE I LOADING FRAME AND SOIL CONTAINER

The soil container consists of a set of rings and upper and lower cover plates, with all parts 3/8 inch in thickness. Each ring is 10 inches high (the prime limitation being that two men must be able to handle the ring). Adjacent rings are separated by a thin piece of rubber so that the container wall is flexible in the vertical direction, and thus the side-friction problem is minimized. A maximum of four rings can be fitted into the loading frame, and the maximum possible depth of sand is about 42 inches.

The rubber bag was fabricated from 1/16 inch black neoprene,

by vulcanizing together two circular disks and a long strip. The resulting bag was 4 inches thick, and fitted snugly within the upper cover plate. It was necessary to provide ½ inch thick rubber padding around the edge of the bag, to prevent the bag from blowing out through the inadvertent cracks between the cover plate and the topmost ring of the bin. Pressure was supplied to the bag from nitrogen bottles.

The loading frame is made up of four complete rectangular bents in parallel planes with templates at the lower corners to tie the bents together. Each bent involves two 14 WF 68 beams and two tension plates of 1 inch thickness connected by 8 high strength bolts at each corner. The maximum working pressure within the rubber bag is 300 lb/in^2, giving a maximum total force of 850,000 pounds. This huge force is carried internally by the frame, and only the dead weight (about 7.5 tons) of the testing system and sand reaches the floor of the building.

Obviously, many compromises had to be made when selecting the dimensions for the testing system. On the one hand, a large system means that the details of full-scale situations can be more fully reproduced in the small-scale tests, and the results of small-scale tests can in turn be applied to full-scale problems with but a minimum of interpretation and extrapolation. On the other hand, a large system requires the expenditure of considerable money, effort and time in order to accomplish a single test.

Since the experiments were to be somewhat exploratory in nature, and aimed at basic research rather than the modelling of full-scale situations, it was appropriate to select the minimum possible dimensions. The minimum size for the structure itself was set by the requirement of observing adequately the deformation of the structure: on this basis, a 12 inch diameter at the base of the dome was chosen. Then the clearance between structure and bin required to minimize boundary effects established the bin dimensions. By analogy to the footing problem, and by inspection of some photoelasticity results (for example, see Armour Research Foundation, 1962), it was decided that the bin diameter should be at least 5 times the span length of the small-scale structure and the bin depth should be at least 3 times this span length. The bin dimensions thus obtained were already at or close to those set by space limitations.

Placement of Sand

A sand-blasting sand, readily available and inexpensive, was used for these tests. As shown in Figure 2, this sand was of relatively uniform grain size and free from dust. Individual grains of sands were quite angular. There are unfortunately no standard tests for

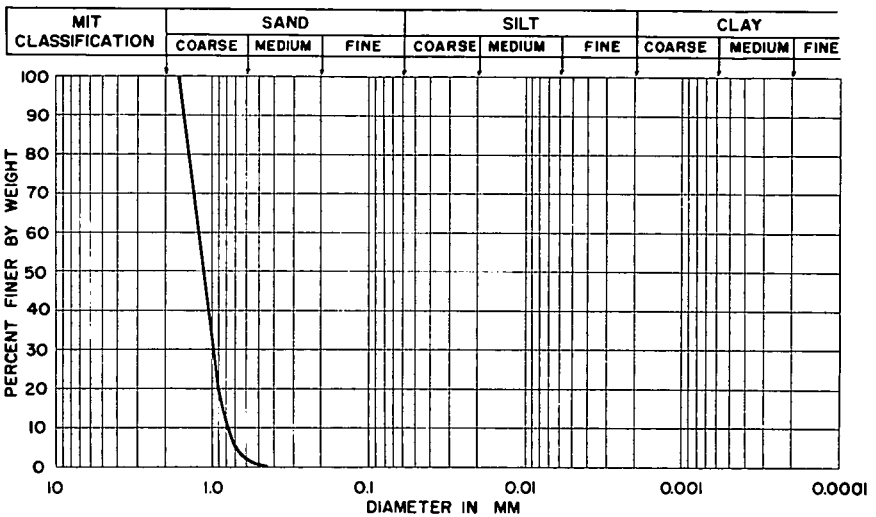

FIGURE 2 GRAIN SIZE DISTRIBUTION

determining the maximum and minimum unit weight for a sand, and hence a number of the commonly recommended tests were used. The minimum unit weight, for the sand in the air-dry state, proved to be about 90 lb/ft^3. The maximum unit weight was more elusive, but a representative value of 108 lb/ft^3 (achieved by vibration under surcharge) was finally selected. The friction angle of the sand ranges between 30° and 38°, depending upon the relative density.

After some experimentation, and much to our surprise, it was found that a very dense sand mass could be obtained by "showering" the sand, using a flexible hose muzzled with ¼ inch mesh wire screen: see Figure 3. When showering into small containers (about 5 inch diameter) at a rate of about 0.02 ft^3/min., a unit weight of 107.5 lb/ft^3 (97% relative density) was obtained consistently. As indicated by earlier studies (Kolbuszewski and Jones: 1961), the unit weight so obtained was a function of the height of free-fall, and

especially of the rate-of-flow of the sand. Apparently, when sand particles arrive more-or-less individually at an existing sand surface, it is possible for these particles to nestle down into the "holes" between existing sand particles in such a way as to give nearly optimum packing and maximum unit weight. As the rate of delivery of the sand is increased, there is more-and-more likelihood that two or more

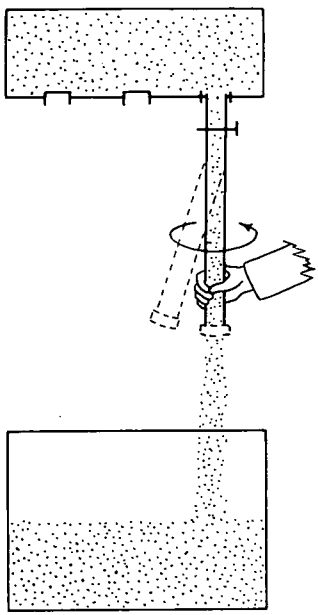

FIGURE 3 PLACING SAND BY SHOWERING

particles will try simultaneously to fall into the same "hole," thus leading to arch-like arrangements and a looser sand mass. The earlier studies would not have suggested that essentially 100% relative density would be obtained by showering, and it seems likely that such a result can be obtained only for coarse, uniform sands.

When placing "dense" sand in the test bin, a quantity of the sand was supported overhead in a box, and showered through a 2 inch diameter radiator hose. It was essential to keep moving the end of the hose back-and-forth over the sand surface, so as to keep this

surface free from hills and valleys. A rate of placement of 0.16 ft^3/min., well above the optimum rate of placement, was used to speed up the testing schedule.[2]

A simple penetrometer, using a flat disk 1.25 inches in diameter as a tip, was first calibrated vs. unit weight by tests in small containers, and then used to check the in-place unit weight of the large sand mass. In this way, it was found that the average unit weight of the sand involved in the buried dome tests was 102 lb/ft^3 (67% relative density). Thus the increased rate of showering caused quite a marked decrease in the resultant unit weight.

The in-place unit weight of the "dense" sand, as measured by the penetrometer, ranged from 101.5 to 104.0 lb/ft^3 (relative densities of 59% to 78%). A number of plate bearing tests (12 inch diameter plate) were carried out to check upon the uniformity of the dense sand mass, and it was found that the standard deviation from the mean was only 4%. It was concluded that the as-placed sand masses were indeed quite uniform.

To create a loose sand mass, sand was poured rapidly into the center of the bin from the box supported overhead. The sand of course formed a cone, and finally the sand in the top portion of the cone was pushed carefully to the sides of the bin so as to give a level surface. Tests with the penetrometer revealed that the average unit weight was reasonably uniform throughout the mass, at the minimum unit weight of 90 lb/ft^3.

Load Capacity of Unburied Domes

A cross-section through the dome used in these experiments is shown in Figure 4. This dome is a segment of a sphere with a radius of 9 inches, and the central angle for the dome is 85°. The domes were formed by pushing and spinning a flat sheet, and were then heat-treated to relieve worked-in stresses. Aluminum 6061 0, with a yield point (based on 0.2% strain) of about 8000 lb/in^2, was used.

A fixed-edge support condition was provided, as shown in Figure 4, by gluing the dome to a base plate with an epoxy adhesive. Calculations indicated that bending stresses arising from this support condition would persist only through an angle of approximately 10° from

[2] Approximately 7 hours were usually needed to fill the bin to 30 inch depth, of which about 5 hours were spent in actually showering the sand.

the support. Thus a membrane stress condition existed over the major portion of the dome.

According to the classical linear theory for the buckling of spheres (see Timoshenko and Gere, 1961), such a dome should be able to withstand a uniform radial pressure of 90 lb/in^2 before buckling.[3] However, recent research (for an up-to-date review, see Gjelsvik

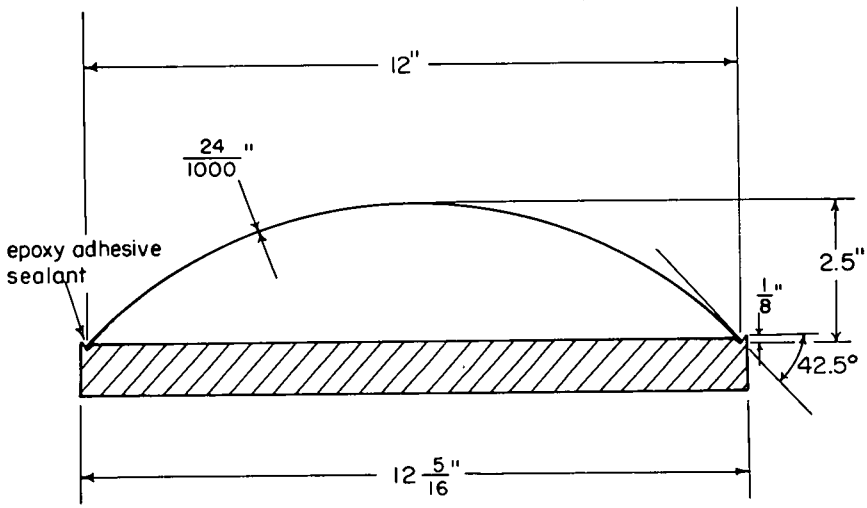

FIGURE 4 DOME AND BASE PLATE

and Bodner, 1962) has indicated that a "snap-through" will occur at a much lower pressure, owing to nonlinear effects. The buckling pressure for the domes of this test program was determined by subjecting the dome, already mounted on its base plate, to a water pressure. Buckling occurred at a pressure of 25 $\pm$2 lb/in^2, a result which is in satisfactory agreement with those obtained by Klöppel and Jungbluth (1953). Figure 5 shows a picture of a buckled dome.

Instrumentation for Tests With Buried Domes

The extent of the instrumentation used in the test program is shown in Figure 6. A load cell was placed under the base plate of

[3] Computed for the hypothetical case that the modulus of elasticity for aluminum remains constant.

the dome, so as to measure the total vertical force reaching the dome; strain gages were placed on the inside surface of the dome; deflection was recorded at various elevations of the dome-load cell system; vertical movements within the sand mass were measured; and finally the movement of the bottom of the test bin was monitored.

The load cell employed SR-4 strain gages affixed to a thin cylinder which carried the total load in axial compression. The strain gages and bridge circuit were arranged so as to provide compensation

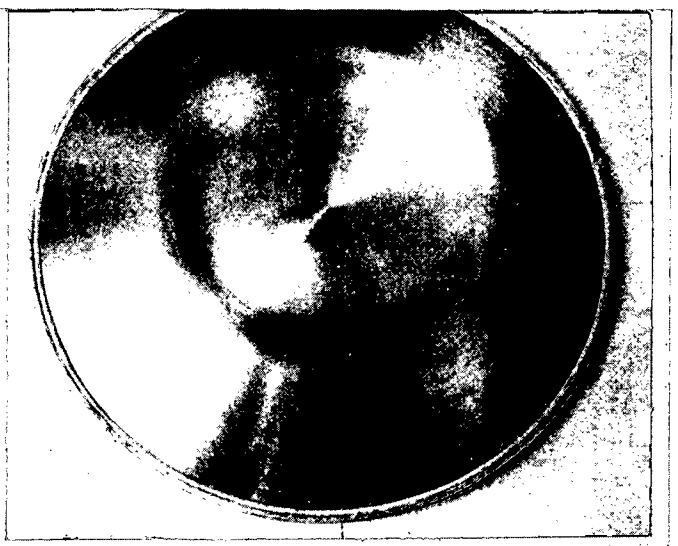

FIGURE 5 DOME BUCKLED BY HYDROSTATIC PRESSURE

for bending strains and temperature effects. As indicated in Figure 6, this cylinder was provided with bearing plates at both ends. A pipe kept sand away from the load cell, and a piece of tape covered the gap between the pipe and the base plate of the dome.

A typical arrangement of strain gages mounted on the dome is shown in Figure 7. While it would have been desirable to have gages on the outside surface as well, it was feared that such gages might affect the soil-structure interaction patterns.

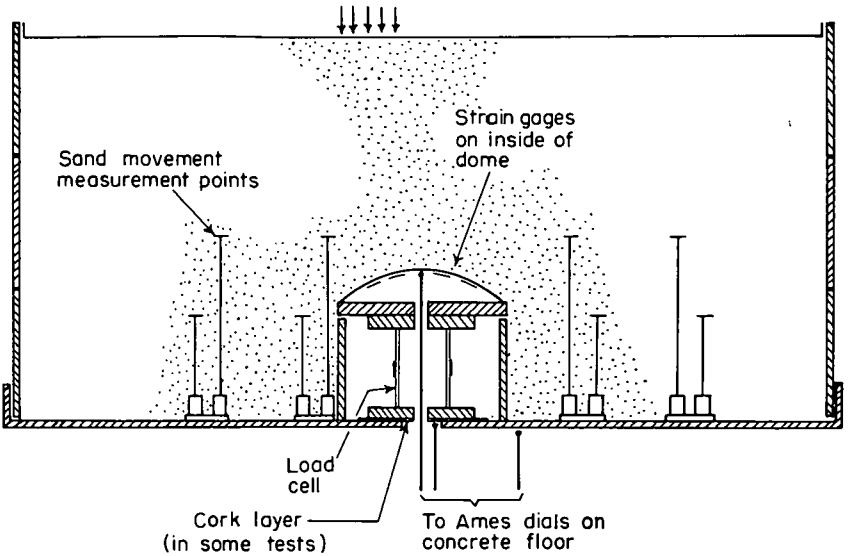

FIGURE 6 DOME SUPPORT AND INSTRUMENTATION

Vertical movement of the crown of the dome and of the lower flange of the load cell were measured using Ames dials. Rods from the Ames dials extended up through a hole in the bottom of the bin, through the load cell, and through another hole in the base plate of

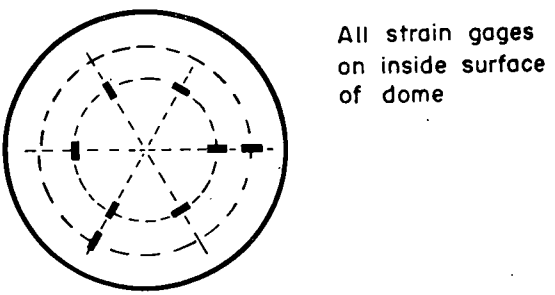

FIGURE 7 TYPICAL PATTERN OF STRAIN GAGES

the dome. Because of seating imperfections, the dome-load cell system proved to be slightly flexible. In some tests, a layer of cork was introduced below the load cell so as to give even more flexibility.

Figure 8 shows the system used to measure vertical movements within the sand. The anchor disks (1 inch in diameter and 1/16 inch

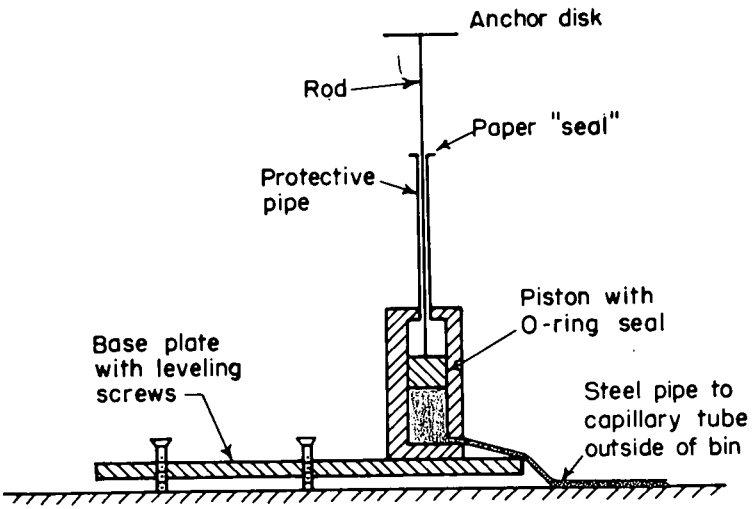

FIGURE 8 DEVICE FOR MEASURING SAND MOVEMENT

thick) moved with the sand, and as a result squeezed colored water from the hydraulic cylinders resting on the bottom of the bin. The volume of water squeezed from the cylinders was measured in capillary tubes located outside of the bin.

Because the lower beams of the loading frame seated imperfectly against the floor, and because of bending under load, the bottom of the bin moved during a test. Since the dome movements were measured with respect to the floor, while the sand movements were referenced to the bottom of the bin, it was necessary to measure the bin deflections and to use this data to put all movement measurements on a common basis.

Results and Analyses

The test program is summarized in Table 1. There were three preliminary tests that served to perfect the testing techniques. The

TABLE 1
SUMMARY OF TEST PROGRAM

Test No.	Sand	Burial	Support	Dome
4	Dense	17.5 in.	Rigid	Standard
5	Dense	5.0 in.	Rigid	Standard
6	Dense	5.0 in.	Flexible	Standard
7	Dense	17.5 in.	Flexible	Standard
8	Loose	17.5 in.	Flexible	Standard
9	Loose	17.5 in.	Rigid	Standard
10	Dense	5.0 in.	Flexible	Standard
11	Dense	5.0 in.	Rigid	Rigid

Notes: (a) Dense sand mass averaged 102 lb/ft^3 (67% relative density); loose sand mass averaged 90 lb/ft^3 (0% relative density).
(b) Depth of burial measured to crown of dome.
(c) "Rigid" support provided by load cell; "flexible" support achieved by introducing cork layer below load cell.
(d) Rigid dome is standard dome filled with gypsum.

depth of cover, relative density of the sand, and flexibility of dome support were varied from test to test as indicated. A test involving a rigid dome (a standard dome filled with gypsum) was included. Tests 6 and 10 were duplicate tests. In Test 7, the gas bag ruptured before failure developed in the dome. The applied surface pressure was generally increased in increments of 10 lb/in^2, although smaller increments were used as the point of structural yielding was approached. All instrumentation systems were read after each increment.

An initial failure developed along the spring line of the dome when the average vertical stress against the dome reached about 70 lb/in^2. At this point, the crown deflection suddenly increased, the average soil pressure against the dome suddenly decreased, and the strains within the sand alongside the dome suddenly increased. The surface pressure required to cause 70 lb/in^2 against the dome varied with the density of the sand and the flexibility of support for the dome. Following development of the yield condition, it was generally possible to increase the surface load further, without causing the structure to collapse.

The details of the test results are presented in the following subsections.

Compressibility of sand: From the measured movement of disks embedded within the sand mass, it was possible to determine the vertical strains caused within the sand by the applied surface pressure.

Assuming that the vertical stresses within the sand were equal to the applied pressures, the virgin stress-strain curves shown in Figure 9 have been developed. The data used to construct these curves came from a zone which lay from 8 inches to 14 inches from the bottom

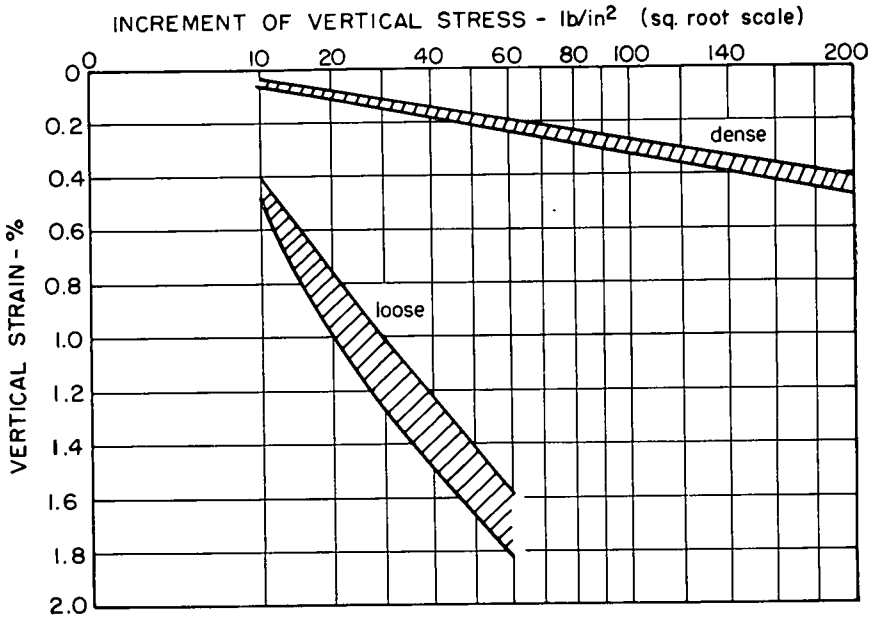

FIGURE 9 STRESS-STRAIN CURVES FOR SAND

of the bin (average depth below surface from 6.5 inches to 19 inches, depending upon test conditions), and from 6 to 14 inches to the side of the edge of the dome. It is believed that the actual stresses within this zone were affected only slightly by arching around the dome.

As suggested by Chaplin (1961), the strains within the sand were found to increase as the square root of the applied pressure; i.e., the compressibility of the sand decreased with increasing stress. The tangent modulus at a stress level of 30 lb/in^2 was 27,000 lb/in^2 for the "dense" sand and 4,000 lb/in^2 for the loose sand. From the former of these values, it is clear that the "dense" sand was indeed in a very compact state. The sand masses were essentially in a state

of one-dimensional strain, and the stress-strain ratio represents a constrained modulus.

"Arching" before yield: The average vertical pressures reaching the domes are shown by the curves in Figure 10. The ratio of the

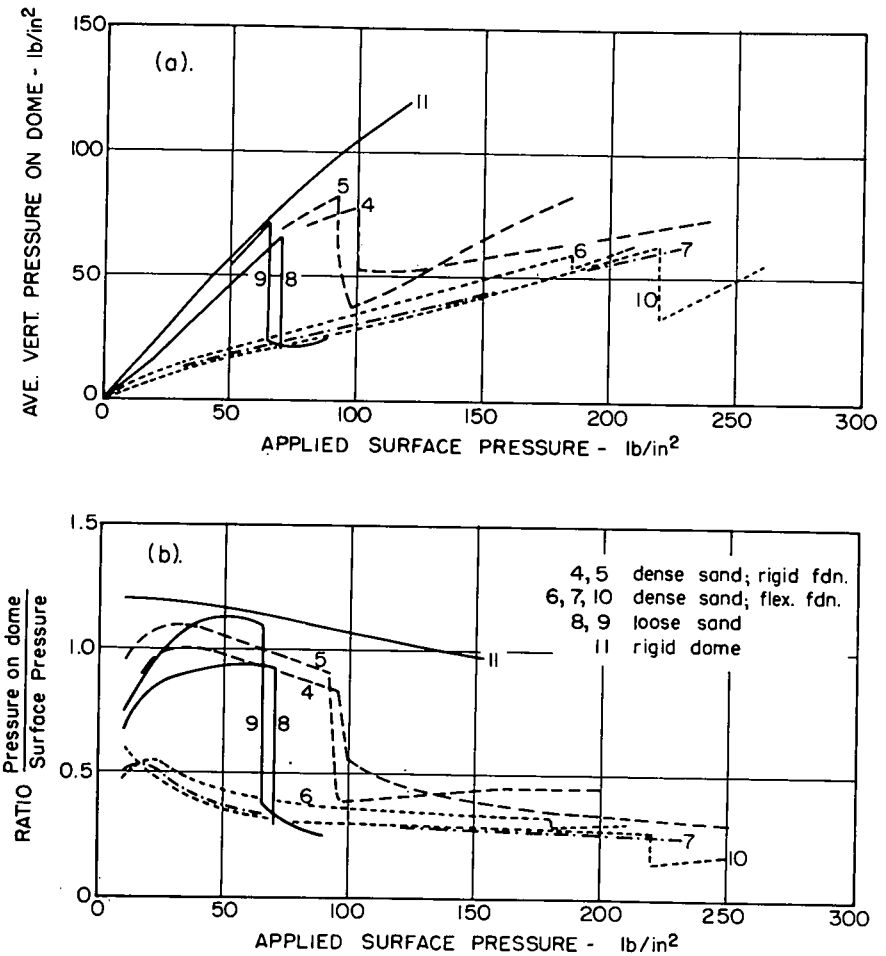

FIGURE 10 PRESSURE ON DOME vs. APPLIED SURFACE PRESSURE

pressure reaching the dome to the applied surface pressure is indicative of the "arching" or stress redistribution around the dome.

The curves of pressure ratio generally have a characteristic shape. (1) First, these curves move upward to a peak at a relatively low value of surface pressure. No satisfactory explanation has been found for this rather surprising behavior. It was suspected that the support for the dome might initially be "soft" as the result of poor seating between the various portions of the dome-load cell-bin system. While the data showed that seating errors were indeed present, there was, however, no evidence that the system was unusually "soft" during the first application of pressure. (2) Following the peak, the pressure ratio decreased. The decreasing compressibility of the sand presumably accounted for this trend. In addition, the compressibility of the cork also decreased with increasing stress, and this fact accounts for the flattening of the pressure ratio curves from Tests 6, 7 and 10. (3) The sudden drop in all curves, except those for Tests 7 (failure not reached) and 11 (rigid dome), signified yielding within the dome.

In all tests with a rigid foundation, and in the one test with both loose sand and a flexible support, the maximum value of the pressure ratio ranged from 0.95 to 1.20. In all other tests with a flexible support, the peak value of this ratio was in the range from 0.5 to 0.6. Qualitatively, these results are as would be expected on the basis of arching theory. However, when the actual magnitudes of the pressure ratio were studied in the light of the relative movements between dome and sand, the results were rather surprising.

Figure 11(a) shows these movements for the case of dense sand and rigid support. Although the peak pressure ratios in these tests were 1.0 and 1.1, the crown moved down more than the sand located some 2 to 14 inches off to the side of the dome. Figure 11(b) shows the relative movements for the case of dense sand and flexible foundation. Here the pattern of relative movement and pressure ratio is as would be expected. Finally, results for the tests involving loose sand are shown in Figure 11(c). Although the sand has moved much more than the crown of the dome, the pressure ratio values were not far from unity; i.e., there apparently was little negative arching.

In preparing these curves, it was necessary to correct the measured crown movements for the relative movement between the bin bottom and the floor of the laboratory room; while the exact magnitude of the correction was in doubt, it is believed that the relative

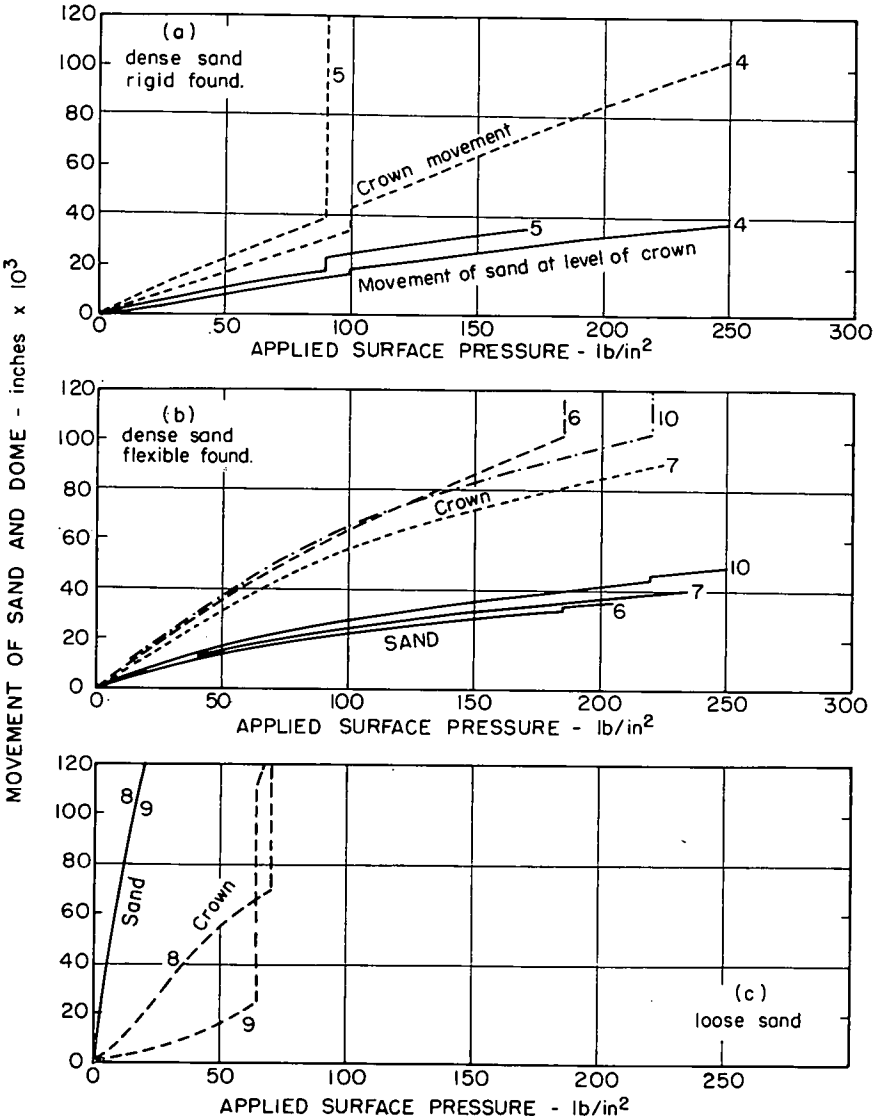

FIGURE II RELATIVE MOVEMENT OF SAND AND DOME

position of these curves is correct. Even with the so-called rigid foundation, most of the crown movement prior to yield was the result of compressions within the load cell and seating errors. The dome itself was relatively incompressible; i.e., the elastic deflection of the crown relative to its base plate was computed to be 0.004 inch at a pressure against the dome of 30 lb/in^2.

It would of course be dangerous to draw quantitative conclusions from these tests concerning the arching phenomena around buried domes. The preceding analysis has to some extent accounted for the presence of the load cell under the dome; however, there must be other effects of this unusual structural arrangement. The results would, in general, appear to support the conclusion that it is relatively easy to induce positive arching (pressure ratio less than unity) but that negative arching (pressure ratio greater than unity) may be relatively insignificant even when the soil is extremely compressible. Such a conclusion is in line with the suggestions outlined by Taylor (1947).

Stress required to cause yield: It is, of course, quite significant that the buried domes sustained an average vertical pressure much larger than the pressure which would cause snap buckling of this dome if unburied. Moreover, the buried domes yielded by development of excessively bending strains at the support, and in all tests save one, the only deformation in the dome was a bend along the spring line. The compressive strains measured near the crown reached values at least three times those existing just before snap buckling in the unburied dome. From all of these facts, it is clear that the sand surrounding the dome was extremely effective in inhibiting the development of elastic snap buckling, even when the sand was in a loose condition.

By analysis of the bending stresses expected near the support, it can be predicted that a pair of plastic hinges, as shown in Figure 12, should develop at the support when the average vertical pressure on the dome reached about 65 lb/in^2. Once these hinges develop, local plastic buckling should follow almost immediately. This prediction is in good agreement with the observed results, both as to magnitude and as to the nature of the observed yield: see Figure 13(a).

Starting from the recorded strain data, an attempt was made to determine the distribution of the pressure acting against the dome. Despite the difficulties inherent in such a procedure, it appeared that

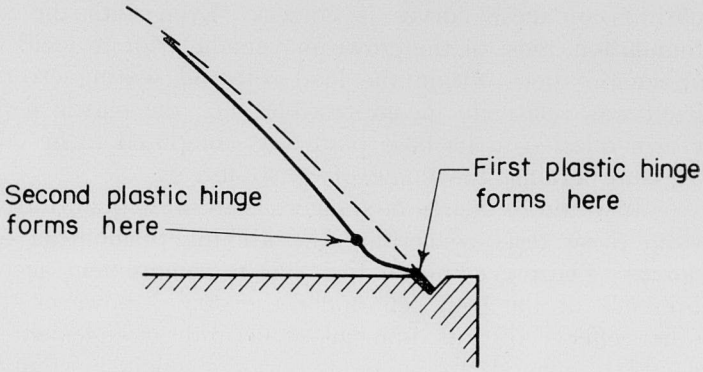

FIGURE 12 DEVELOPMENT OF FAILURE AT SUPPORT

the loading in Tests 6, 7 and 10 resembled a snow loading (i.e., a uniform stress per unit horizontal area) while in the remaining tests there was a greater concentration of pressure near the support of the dome. Thus, comparing the strains near the crown of the dome when failure developed near the support, these strains were much greater in Tests 6 and 10 than in Tests 4, 5, 8 and 9.

Finally, it should be noted that the pressure ratio at yield was unaffected by the change in the depth of burial, for the case of dense sand and a rigid support. Even though the dome in Test 7 did not yield, the trend of the curves in Figure 10 (b) suggests that this same conclusion applies to the case of dense sand and a flexible support.

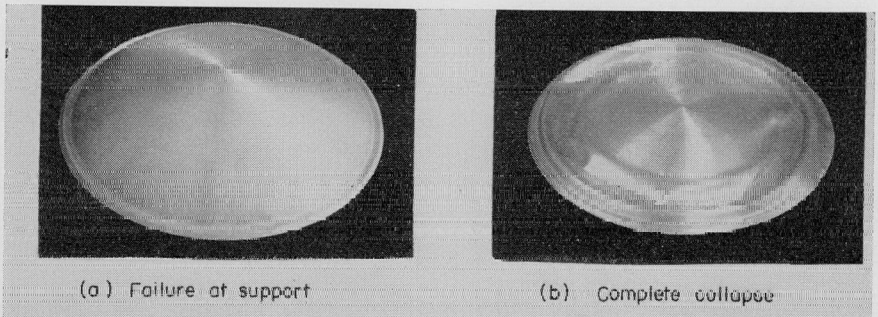

FIGURE 13 YIELDING AND COLLAPSE OF BURIED DOMES

Behavior following yield: When failure by local plastic buckling developed near the support (Figure 12), the crown of the dome suddenly moved downward, generally while the surface load remained constant. The magnitude of this sudden deflection varied widely from test to test, as indicated in Table 2. The dome in Test 6 collapsed completely, as shown by the photograph in Figure 13(b).

TABLE 2
CROWN DEFLECTION AT YIELD

Test No.	Deflection (% of height of dome)
4	1.5
5	3.5
6	Total collapse
8	5.0
9	4.0
10	9.0

The ratio between average pressure on dome and surface pressure dropped suddenly at this point of yield, and there was a transfer of stress to the surrounding sand. The amount of stress transfer was least in the cases where the pressure ratio prior to collapse was least. When the surface pressure was increased further following yield, the dome picked up some portion of the additional surface load. The ratio of additional pressure on dome to additional surface pressure seemed to be related to the magnitude of the dome's deformation during failure near the support.

These various observations might be explained by the following hypotheses. In each test, a soil "arch" (more strictly a "dome") had formed to a greater or lesser degree prior to yield. (Even in tests where the pressure ratio was near unity, there was a concentration of stress near the support line of the dome.) When the structure failed near the support, the "arch" had to adjust quickly to the new conditions. Where the "arch" was relatively unimportant before yield, and where the depth of burial was large, the "arch" remained intact and effectively accomplished the redistribution of stress both at and subsequent to yield. But where the depth of burial was shallow, or where the "arch" was well developed before yield, the "arch" was not able to adjust immediately and collapsed partially. In these latter cases, the deformations at yield were large, and there was less arching subsequent to yield.

In Test 4, the burial depth was large compared to the span of the dome, and the combination of dense sand and rigid foundation meant that there was little arching prior to yield. Thus all conditions were favorable for development of "arching" as the dome yielded, and the dome in this test deformed relatively little at and subsequent to failure at the support. In fact, the crown movements in Tests 4 and 7 (same as 4 except flexible foundation) were essentially identical at a surface pressure of 220 lb/in^2, even though the dome in Test 4 had long since yielded and the dome in Test 7 had not yet yielded.

On the other hand, the worst combination of circumstances was present in Test 6: the burial was relatively shallow and there was considerable arching prior to yield. The dome in this test collapsed immediately upon failure near the support, and a depression formed in the surface of the sand. These test conditions were duplicated in Test 10. This time there was no collapse, but the deflection was quite large when failure occurred near the support.

While the foregoing hypotheses are in general supported by the results, the behavior of the dome at and following yield undoubtedly was affected by a complex interaction between a number of factors, including the pattern and magnitude of stresses within the dome just prior to yield and possible slight differences between the properties, dimensions and support conditions for the individual domes. Moreover, the amount by which the dome deflects at yield must in turn influence the ability of the sand to form and/or sustain an "arch." While the data suggest, for example, that burial depth will by itself have a significant influence upon the deformations at and following yield (compare Tests 4 and 5), it would be dangerous to draw general conclusions concerning this complex problem on the basis of such limited data.

Conclusions

It appears certain that the type of small-scale test described in this paper can be of great use in studying the fundamental aspects of the soil-structure interaction problem. The tests discussed herein, even though preliminary in nature and rather crudely instrumented, have revealed the following patterns of behavior:

(a) While comparison of structure flexibility with the compressibility of the surrounding sand did not indicate any simple

relationship between these parameters and the amount of surface pressure transferred to the dome, it was notable that there was little tendency toward significant negative arching even when the sand was very loose.

(b) The domes, when buried to a depth greater than their radius, could withstand pressures several times the elastic snap-buckling pressure for the unburied dome. When the domes did yield, it was as the result of bending failure near the support, and not the result of elastic buckling.

(c) The overpressure at the sand surface, required to cause failure in the domes, was essentially the same for burial depths of 0.4 and 1.5 times the span of the dome.

(d) A sand "arch" will generally form when the structure yields. This "arch" will assume some of the pressure which had been acting upon the dome, thus preventing total collapse. "Arching" action will continue if the surface pressure is increased still further. However, unless the burial depth is great, there may be large structural deflections at and subsequent to yield.

(e) Beneficial "arching" action in the sand can be developed prior to yield by introducing foundation flexibility, even for a burial depth of approximately one-half the span. In such a case, however, the dome may deform excessively when failure occurs at the support.

These results will serve as the starting point for more detailed research concerning interaction between soil and buried structure.

Acknowledgments

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Many of the authors' colleagues have made valuable suggestions concerning the conduct of this research and the interpretation of the

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THE USE OF A SUBSAMPLE IN PARAMETER ESTIMATION

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ABSTRACT

If bivariate sample data demonstrate a tendency to be grouped into distinct clusters, each of which exhibits little or no correlation within itself, the overall sample correlation coefficient may nonetheless be quite high. If additional values of the independent variable are available for a particular group or cluster and, moreover, if that group is such that a significant correlation coefficient does obtain within it, regression analyses may be utilized to augment the sample and thereby improve the estimate of the population mean of the dependent variable for that single group in question. Three alternative techniques are available for augmenting sample data:

(i) use the regression equation defined by the entire sample,
(ii) use the regression equation defined by that subsample which is taken to include only those data in the single cluster in question, and
(iii) use neither (i) nor (ii), but rather rely on the original (i.e., observed) data alone as providing the most precise parameter estimate.

The criteria for choosing between these three alternatives in any given situation are presented.

THE USE OF A SUBSAMPLE IN PARAMETER ESTIMATION

LET there be n_1 concurrent measurements of two variables, X and Y, and let the scatter diagram of sample points in the X-Y plane be such that each individual cluster is comprised of n'_1 values. Suppose, for example, that two hydrologic phenomena are measured each month for a duration of several years, and that each group of monthly measurements except one, say period m, forms a distinct cluster with little or no correlation within the group. All the data taken together define a bivariate regression line of Y on X, which is characterized by a coefficient of correlation whose magnitude is due largely to cyclic (i.e., seasonal) fluctuations in the magnitude of the phenomena. Let n_2 additional values of the independent variable, X, be available for period m and let it be required to estimate the population mean of the dependent variable, μ, corresponding to the period m.

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Figure 1a represents a scatter diagram of monthly data which may be helpful in visualizing the problem. For the example represented thereon, $n_1 = 60$ so that $n'_1 = n_1/12$, giving 5 points per cluster (month). The points for period *m* are represented as circles whereas the remaining points are crosses. Four additional readings of the X_i are available during period *m*, and the estimated values of the Y_i are shown for each of the two regression lines, one defined by the entire sample of n_1 data and the second by the subsample of n'_1 data. It is required to determine which of these two lines, if either, should be used to estimate additional values of Y_i for the period *m*.

If the relationship between the X_i and Y_i be linear, the two regression lines may be represented by:

$$\hat{Y}_{i_a} = \overline{Y}_a + b_a (X_i - \overline{X}_x) \tag{1}$$

and

$$\hat{Y}_{i_m} = \overline{Y}_m + b_m (X_i - \overline{X}_m) \tag{2}$$

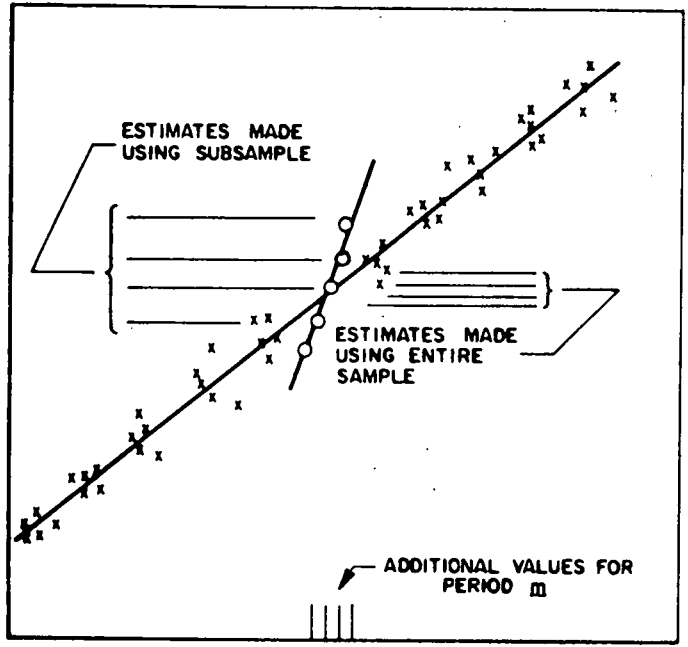
where Equation (1) defines the regression based on all the n_1 points and Equation (2) that based on the n'_1 points in period *m*, and for which:

$\hat{Y}_{i_a}, \hat{Y}_{i_m}$	are estimates of the value of Y_i using Equations (1) and (2), respectively,
$\overline{Y}_a, \overline{Y}_m$	are the sample means of the dependent variable for the entire sample and the subsample, respectively,
$\overline{X}_a, \overline{X}_m$	are the sample means of the independent variable for the entire sample and the subsample, respectively, and
b_a, b_m	are the sample regression coefficients computed from the entire sample and the subsample, respectively.

With reference to the three alternative procedures outlined above and repeated here for convenience, the estimate of the population mean for period *m*, denoted by $\hat{\mu}$, may be calculated as follows:

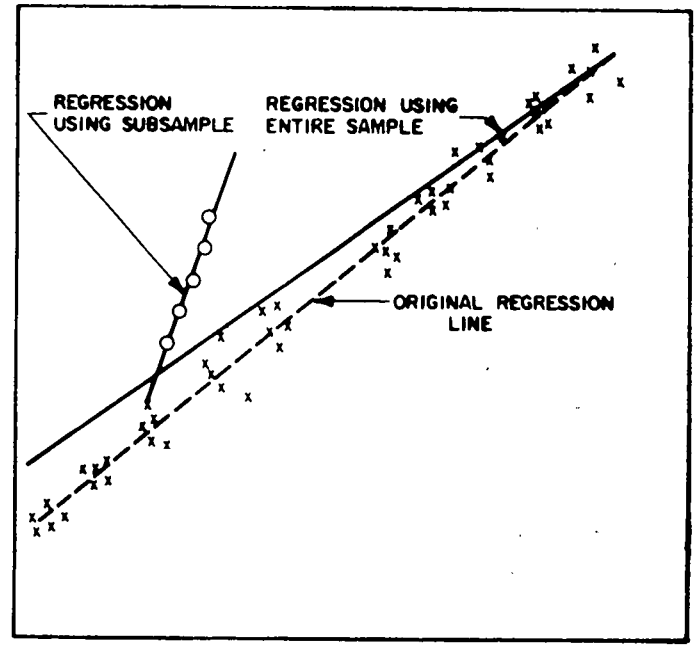
(i) use the regression equation defined by the entire sample, Equation (1), and compute the mean for period *m* as if these estimates were observed data,

(ii) use the regression equation defined by the subsample for



REGRESSION LINES BASED ON
SAMPLE AND SUBSAMPLE

FIGURE 1a



REGRESSION LINES BASED ON NEW
LOCATION OF POINTS FOR PERIOD m

FIGURE 1b

period *m*, Equation (2), and compute the mean as in (i) above,

(iii) use the observed data in period *m* alone, without augmenting the sample by virtue of (i) or (ii) above.

The criterion for selection of an alternative shall be the relative magnitude of the variance of the estimate of the mean associated with that alternative. That estimation technique which yields the smallest variance shall be utilized in preference to the other two. We consider the details of each alternative in turn, noting or deriving the estimating equation and the associated variance for each.

(i) *Use regression Equation (1), defined by the entire sample.*

We introduce here the following notation:

μ_{ya}, μ_{ym}:	population means of the dependent variable for the entire sample and the subsample, respectively,
μ_{xa}, μ_{xm}:	population means of the independent variable for the entire sample and the subsample, respectively,
$\sigma_{ya}^2, \sigma_{ym}^2$:	population variances of the dependent variable for the entire sample and the subsample, respectively,
$\sigma_{xa}^2, \sigma_{xm}^2$:	population variances of the independent variable for the entire sample and the subsample, respectively,
$\overline{X}_{n_2}$:	sample mean for the n_2 additional observations of the independent variable,
ρ_a, ρ_m:	population correlation coefficients for the entire sample and the subsample, respectively,
$K_1 = \mu_{ya}/\mu_{ym}$	
$K_2 = \mu_{xa}/\mu_{xm}$	
$K_3 = \sigma_{ya}/\sigma_{ym}$	
$K_4 = \sigma_{xa}/\sigma_{xm}$	
C_y	coefficient of variation of the dependent variable, σ_{ya}/μ_{ya}
C_x	coefficient of variation of the independent variable, σ_{xa}/μ_{xa}

Using Equation (1) we obtain the biased estimator:

$$\hat{\mu} = \frac{n_1'\overline{Y}_m}{(n_1' + n_2)} + \frac{n_2}{(n_1' + n_2)} \left[\overline{Y}_a + b_a (\overline{X}_{n_2} - \overline{X}_a) \right] \qquad (3)$$

If the X_i and Y_i are assumed to be normally distributed, estimates of the type given in Equation (3) are normally distributed about their population value, μ. Thus either the variance or mean-square-error of $\hat{\mu}$ is a measure of the precision associated with this estimator, but since Equation (3) is a biased estimator of μ it is necessary to compute its mean-square-error.

Assuming that $\overline{Y}_m$ and $\overline{Y}_a$ are uncorrelated, the result of the derivation may be summarized, after a good deal of simplification by:

$$\text{MSE}(\hat{\mu}) = \frac{\sigma_{ym}^2}{(n_1' + n_2)^2}\left[\frac{n_2^2(K_1 - 1)^2K_3^2}{C_y^2K_1^2} + n_1 + \right.$$

$$+ \frac{n_2^2K_3^2}{n_1}\left[1 + \left(\rho_a^2 + \frac{1 - \rho_a^2}{n_1 - 3}\right)\frac{1}{K_4^2}\left[\frac{n_1(K_2 - 1)^2K_4^2}{C_x^2K_2^2} + \right.\right.$$

$$\left.\left. + \frac{n_1 + n_2K_4^2}{n_2}\right]\right] + \frac{2n_2^2\rho_a(1 - K_2)K_3^2}{K_1K_2C_xC_y}\left[K_1 - 1 - \frac{\rho_aK_1K_2C_xC_y}{n_1(1 - K_2)}\right]\Bigg] \tag{4}$$

It is to be expected that the constants K_1, K_2, K_3, K_4, C_x and C_y would appear in the solution since they define the location of the data for period *m* relative to the remaining points. Thus it is anticipated that the data in Figure 1a would require a different decision than those in Figure 1b, in which all the points are identical except for those in period *m*, which are merely translated without altering their variance and orientation.

(ii) *Use regression Equation (2), defined by the subsample.*

Langbein and Hardison (1), Cochran (2) and Thomas (3) have obtained solutions for this case. The first of these is an approximate solution which has been used in water resources development programs to specify optimal stream-gaging schedules. The last two, exact solutions, are identical and were derived independently. The author (4) has obtained a solution for a multivariate model and has tabulated results for his and the Thomas-Cochran solution. Other earlier solutions were based on large samples and are not applicable here.

The Thomas-Cochran equations are given here, with their notation changed so as to conform to this paper:

$$\hat{\mu} = Y_m + \frac{n_2 b_m}{(n_2 + n_1')} (\overline{X}_{n_2} - \overline{X}_m) \tag{5}$$

$$\text{Var}(\hat{\mu}) = \frac{\sigma_{ym}^2}{n_1'} \left[1 - \frac{n_2}{n_2 + n_1'} \left(\rho_m^2 - \frac{(1 - \rho_m^2)}{(n_1' - 3)} \right) \right] \tag{6}$$

(iii) *Use neither (i) nor (ii) above.*

The variance of the mean computed from the n_1' values is written:

$$\hat{\mu} = Y_m \tag{7}$$

$$\text{Var}(\hat{\mu}) = \frac{\sigma_{ym}^2}{n_1'} \tag{8}$$

It remains only to choose from among the three estimates contained, respectively, in Equations (3), (5) and (7). The criterion for this choice is the magnitude of the variance of each estimate, as given in Equations (4), (6) and (8). Since numerical evaluation of Equation (8) is trivial, we concentrate here on examining Equations (4) and (6), choosing between them, and finally comparing that one possessing the smaller magnitude with Equation (8). It is a simple matter to specify the best estimate of μ as that estimate having the smallest associated sampling variance.

Define the relative information, I, as the ratio of the two sampling variances which are obtained from alternate estimating techniques. We write I_1 as the ratio of Equation (4) to Equation (6). When $I_1 > 1.0$, the variance of the estimate of the mean derived from the entire sample exceeds that of the estimate derived from the subsample, and therefore the subsample relation, Equation (5), should be used in preference to that for the entire sample, Equation (3). The converse is true when $I_1 < 1.0$. Similarly, we define I_2 as the ratio of Equation (8) to Equation (4) or (6), whichever ratio is larger. If $I_2 < 1.0$, Equation (7) should be used to estimate the mean. If $I_2 > 1.0$, use Equation (3) or (5), depending on whether I_1 is less than or greater than unity.

To summarize the strategy.

$$I_1 = \frac{\text{Eq (4)}}{\text{Eq (6)}} \tag{9}$$

$$I_2 = \frac{\text{Eq (8)}}{\text{Eq (4)}} \text{ or } \frac{\text{Eq (8)}}{\text{Eq (6)}}, \text{ whichever is larger} \tag{10}$$

If $I_2 < 1.0$, the techniques for augmenting sample data proposed here introduce so much additional variance into the new combined sample that n_1' data alone, those actually observed, should be used to estimate μ. If $I_2 > 1.0$, two choices are possible:

$I_1 > 1.0$: use Equation (5)

$I_1 < 1.0$: use Equation (3)

If I_1 or $I_2 = 1.0$, the solution indicates indifference to the alternatives in question.

The general behavior of I_1 was investigated using a program written by the author for the IBM 709 Data Processing System at the Western Data Processing Center (WDPC) University of California, Los Angeles Campus. I_1 was evaluated for many combinations of its arguments. Curves representing I_1 for a wide range of these combinations are given in Figures 2–7. In each case, the existence of monthly data was assumed for the subsample so that $n_1 = 12.n_1'$, but the program may easily be changed to accommodate other combinations of subsamples. Running time for the 17,496 evaluations of I_1, after perfecting the program, was 7.08 minutes. Care should be taken in extrapolating beyond or interpolating between the results given in Figures 2–7 because the value of I_1 is extremely sensitive to small changes in its arguments.

For typical values of the arguments which might be obtained in hydrologic studies of streamflows, e.g., $n_1 = 60$, $n_2 = 5$, $\rho_a = 0.9$, $\rho_m = 0.6$, $C_x = C_y = 0.25$, and $K_1 = K_2 = K_3 = K_4 = 2$, the relative information, I_1, is 0.6. I_2 is found to be 1.67.

Thus, for the given data, one would use Equation (3) in preference to Equation (5) (since $I_1 < 1.0$) and to Equation (7) (since $I_2 > 1.0$).

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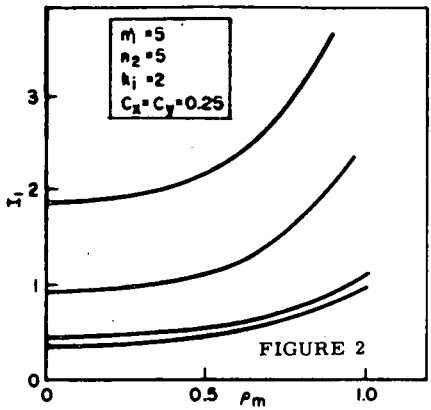

FIGURE 2

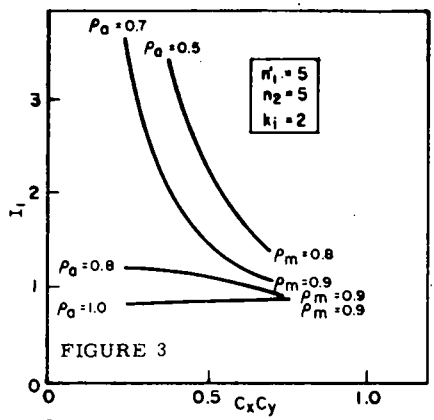

FIGURE 3

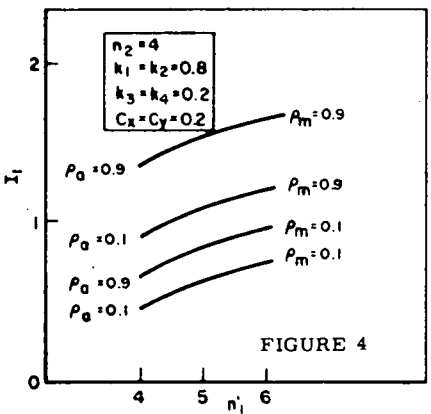

FIGURE 4

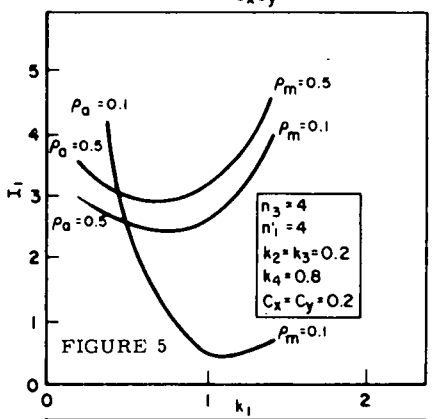

FIGURE 5

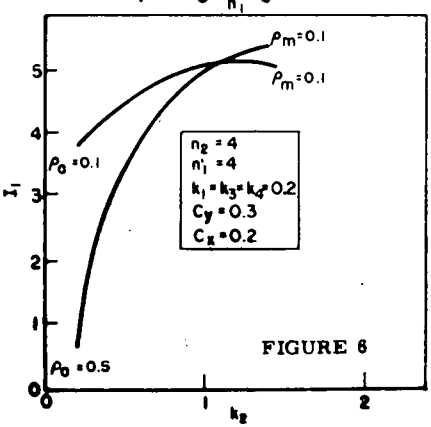

FIGURE 6

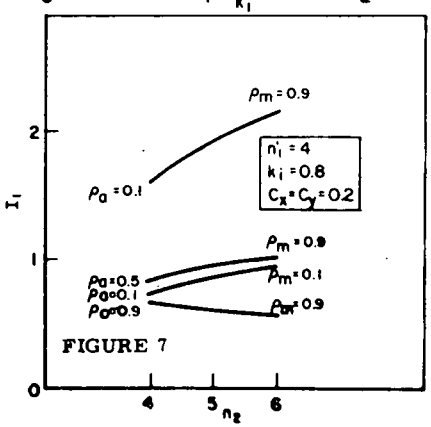

FIGURE 7

Data Processing Center at the University of California for cooperation in the use of the fine computing facility at the Center.

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SPECIFICATIONS—THEIR USE AND ABUSE

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Part I—Bridge Specifications

SIMON KIRSHEN*

INTRODUCTION

THE discussion of the uses of bridge specifications must inevitably include their abuses, either by inference or by direct citation.

Consider an entire specification; that is, the contents of a specification written for a private owner as distinguished from the public authority. This device may prove useful in that it will point out matters that should be reviewed by the specification writer working for the public authority that customarily uses published Standard Specifications.

IMPORTANCE OF CORRECT SPECIFICATIONS

A bidder can make a good bid only on good information. Good information is good in two ways. It not only permits a more equitable bid, it also avoids later difficulties in administration of the contract.

The specification for a bridge is essentially a form of purchase order. Ordinarily, if an article does not meet the specification, it can be returned to the seller. Unfortunately for our purpose, once delivered and placed on the bridge, a unit cannot readily be returned to the seller (in this case the contractor) and the remedy can be made at only great cost.

The specification must be so clear and convincing evidence of what was intended that there can be no dispute. The phrase "as directed by the Engineer" is useless as a buffer or an afterthought to supplement the specification. The final arbiters in contractual disputes are the courts, and in case after case the courts have decided that the meaning of a contract is determined by the intent of the

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parties to the contract. This intent is to be gleaned from the contract as a whole, and from the entire language used in the contract; not from particular words or phrases. Extrinsic evidence to sway the court can be introduced only when the language used in the ordinary sense is vague or has two meanings. In doubtful cases, where the contract has been prepared by one party and accepted by the other, the courts will generally resolve the doubts against the party that prepared the contract.

The foregoing is paraphrased from court decisions. It is sufficiently salutary to indicate the necessity for careful use of words in specification writing.

Information to be Included

Some items should be included in every contract. It was indicated previously that the bridge specifications would be treated as if they were written for a private owner, rather than for a public authority. In the latter case, the Standard Specifications issued by the authority include Definitions and General Covenants, as well as standards for construction.

Owner, Engineer, and Contractor, should be clearly defined. Abbreviations to be used in the text should be fully explained. The words: "plans," "work," "or equal," "Standard Specifications," and "Special Provisions" should be described. Extra Work should be distinguished from Alteration. It should be stated that the contract documents include all notices, specifications, plans, working drawings, and all correspondence regarding any change that affects the location, quality, or quantity of the work. Contract documents legally include every pertinent item about a project from request for bids to final acceptance.

If the contractor is required to furnish bonds and insurances, the amount should be stated, and, of extreme importance, it must be clear who is to pay and on whose behalf these documents are to be obtained. If the bridge spans a railroad right-of-way, the insurances on behalf of the railroad company are generally negotiated and paid for by the contractor.

There must be some subletting restrictions or the industry will be flooded with entrepreneurs who may know little about the work. The Federal Bureau of Public Roads, for example, requires that the prime contractor shall do at least 70% of the work with his own

forces, excepting work which is of a special nature and not usually done by a prime contractor. Such specialized work may be driving long piles or special piles. Other specialized work is erecting structural steel and reinforcing steel, erecting fences, and some minor items. When the major part of the bridge cost is structural steel (as, for example, where the contract is for only the superstructure on a bridge), the concrete deck and surfacing may be sublet as special items.

The General Covenants should include provisions setting forth the responsibility of the bridge contractor in his relations with contractors doing work in contiguous areas. Merely stating that work is being done near the bridge is not sufficient. Nor is it adequate to say that the Engineer will be the referee regarding who shall have priority. The bridge contractor is entitled to know what procedures he can expect on the part of the adjacent contractor, and the timing of such procedures. This is extremely important when relocation of railroad tracks or relocation of utilities is involved.

Not unusual is the case where, for a large bridge, the substructure and superstructure are constructed under two separate contractors. Separate contracts have also been used for small bridges when steel has been in a tight situation. In the latter case, steel has been ordered sometimes even before the substructure was completely designed, with the idea in mind that the steel can be delivered about the time that the substructure will be completed.

For the condition of separate contracts, the specification writer must visualize successive operations. Which contractor is to place anchor bolts? When? Does the superstructure contractor furnish the bolts, to be placed by the substructure contractor, or does the superstructure contractor enter on the unfinished work and place the bolts? Is the substructure to be entirely completed, with the abutments backfilled before the superstructure contractor comes to the site? These and similar questions must be resolved for each individual case by the specification writer.

If there is no statement for guarantee of the work, or any part of it, then it may be inferred that no guarantee is required.

Merely stating that a material will be tested or shall be tested means nothing. The United States Army uses will indiscriminately for will or shall. Army regulations and forms notwithstanding, common usage implies that will means that the owner intends to make

the tests and shall means that the contractor shall make the tests. But this is not sufficient. It should be stated clearly who is to make the tests and who is to pay for all costs of making the tests. An equitable way is to state that if the material is satisfactory, the owner pays the cost; if it is not satisfactory, the contractor pays the cost of testing and the cost of any remedial procedure.

Whatever the method of testing, the specification writer should know the approximate cost. The prescribed test should be one that can be made by familiar methods, using proven instruments.

Disposal of excavated materials seems a simple item. It isn't. In rivers under control of Army Engineers, excavated materials not required in the finished structure, or unsuitable for use, may be disposed of only in areas of the river designated by the Army Engineers. The specification writer should obtain in writing from the Engineers the specified locations, and these locations should be written into the contract. It isn't simple either when the crossing is over a highway or a railroad. Some towns have prohibited removal of excavated material beyond the town limits, especially those materials of good quality for construction. Further, some towns have prohibited trucking on certain streets. Worse still, the prohibition of removal of materials and trucking may be invoked after the contract has been awarded and even during the construction period. The remedy is to have the owner of the project ask the municipal authorities to declare in writing the prohibitions that will exist during the construction period, and write such prohibitions into the contract or note the absence of any prohibitions.

Sometimes the construction of a bridge, especially replacement of an existing bridge, requires use of a temporary bridge. The specification should be clear on materials permitted, minimum capacity required, minimum width of roadway, provision for sidewalks, temporary location of utilities, lighting, and signs for direction of the traveling public. Will the contractor be required to maintain the bridge? If he is permitted to suspend work during the winter, who is to do snow removal on the temporary bridge?

Public utilities must be given separate and special consideration. Whether a utility is in a location merely by permit from proper authorities and must therefore pay all costs for relocation, or whether the utility owns its right-of-way and must therefore be reimbursed for all costs of relocation, the service must be reasonably continu-

ous or alternative service must be furnished. Temporary or permanent easements must be obtained. Permanent locations on the bridge must be indicated. Such information should be obtained before the bridge design has gone too far because utility locations on the bridge may affect the design of cross bracing of the floor system.

Quite often a location for a temporary bridge is not convenient. Travel must, however, be maintained on the road below. Will the contractor be required to build a temporary roof over the highway? Or, in case of a double-barrelled highway, can traffic be made 2-way temporarily on either barrel? Where a railroad is crossed, it must be considered whether railroad operation is to be continuous. If so, some work may have to be done on Sundays, holidays, or between the hours of midnight and early morning. For crossing either a highway or a railroad in use, clearances must be considered for driving piles and sheeting and means must be provided for maintaining satisfactory drainage.

All of the foregoing items involve planning, work, and expense to both owner and contractor. All such items are either definite or the cost can be approximated sufficiently so that neither owner nor contractor is hurt. Eventually the owner must pay, but who shall pay in the first instance and how is the cost to be measured? Each item must be spelled out so that there is no ambiguity.

Payment for the tangible items incorporated in the structure is much simpler. The quantities should be measured in the field, or in some cases computed, insofar as possible using the method used in the particular industry. Consider structural steel as an example. The fabricator uses the nominal weights in the American Institute of Steel Construction Handbook and the actual length of rolled member or plate to obtain the weight. There is also an allowable over-run in certain size plates. The fabricator does not deduct for copes or holes. The same basis should be used, which will provide a means of checking, but copes and holes should be deducted if they are significant; the fabricator will forego the difference. The payment clauses should be definite in regard to over-run in plates. Castings are generally weighed. If an inspector's certificate is not available, the weight can be computed with an allowance for fillets. The specification should stipulate the method for computation of bolts, rivets, and welds. Some specifications stipulate that the weight of welds will not be included in the weight to be paid for. This stipulation has

some merit. The comparative insignificance of weight, and the arguments on type and size of weld, tend to make the cost of computation and checking more than is warranted by the money involved.

Fences, reinforcing steel, and some other items cause little friction between contractor and owner. Fences can be measured from the shop drawings or at the site. The weight of reinforcing steel can be taken from the shop drawings. These and other items can be measured from the drawings or in the completed structure. Concrete cannot be so readily measured, because part of it is buried under ground and the bottom of the footing is variable, plus or minus a few inches from that shown on the plans. Not only this factor, but other factors as well suggest that there should be an easier and more direct method of measuring the payment for a bridge.

Whether or not supported on piles, a bridge has definitely measurable components from the bottom of the footings to the top of surfacing, with definite lengths of curbs and fences. There is no valid reason why the payment should be broken down into components at unit prices. The time spent in argument between contractor and resident on really minor differences in quantities, then between contractor and finals department, and then between contractor and higher echelons, is unconscionable. Payment for the entire structure can and should be on a lump sum basis.

Of course, where a footing is on ledge and the base cannot readily be predetermined, the lump sum should apply only to the superstructure. Unit prices should then apply to the substructure. The subject of lump sum payment is introduced here because it should be the province of the specification writer to decide the method of measurement for payment.

Other major items of ambiguity in the specifications and consequent conflict are cofferdams and piles. If the bridge construction requires a cofferdam of substantial dimensions and cost, it should be a separate item. It should not be lumped, for example, with underwater concrete. If more concrete is used than was originally contemplated, the price to the owner may become exorbitant. If less concrete is used, the contractor can go broke; so there will be a claim and the contractor will win, justifiably. Obviously, including the cofferdam with the underwater concrete permits only the owner to suffer.

There has been and still is considerable controversy on the in-

terpretation of test piles as compared to pile loading test. A pile loading test seems definite enough, but does it include the pile? Most specifications state that the pile used for a loading test remains in the structure and is paid for as a pile, and payment is therefore only for the test. What happens if the pile fails? Some specifications allow for payment of the pile at the bid price per foot, even if it fails. Very few tell what to do or what substitution to make if the pile fails. The nature of the structure should indicate to the specification writer where the pile loading test should be made in order that he may then prescribe the remedy if the pile should fail.

Test piles present a different problem. They are presumably to be used to determine the length of piles to be ordered. The contractor is required to bring to the site his regular pile driving equipment to drive one pile; then leave the equipment idle or take it elsewhere while piles are being ordered. The careful engineer who really wants an answer will have the pile driven where the answer will be most practical: right in the middle of the area of the proposed footing. This is also the most economical place, from the engineer's view. Most specifications state that test piles used in the structure will be paid for only as piles. This is an unwarranted imposition on the contractor. If the need for a test pile is indicated, it should be paid for in a straightforward manner as a separate item, regardless of where it is driven.

There are some other items that force the bridge specification writer to do some research and considerable thinking. Suppose the services of a diver will be required. If the methods of construction require this service, it should be included as a bid item. If the service is required to check the construction, then the owner should pay for it directly. In either case, what is involved? A diver requires a tender, pump, and other equipment. The services may or may not require a report by the diver. Materials may be required for underwater repair of the structure. Here, too, the specification writer must visualize each successive step. He will not get the specification exact the first time, unless his library has a complete file on a previous and similar job, but it will be a better specification the next time. The trouble here is that the need for divers' services are so infrequent for bridge construction that the average specification writer never gets around to writing the second and better specification.

The General Covenants should state the basis for partial pay-

ments. In general, the contractor is paid periodically for parts of the structure that have been completed. There are occasions, however, when the materials for part of the structure are at the site but have not been incorporated in the structure. These may be piles, structural steel, reinforcing steel, fences, and similar materials that do not deteriorate while lying at the site. There must be some equitable payment provided for partial payment for such materials. It is implicit in the contract that, unless expressly written in, the owner is not required to finance the contractor. On the other hand, the contractor is often required to pay for some of these before they can be built into place. The contractor should therefore receive partial payment. About 80% of the cost to the contractor appears fair; he should be able to finance himself for the remainder.

The method of payment for Extra Work and for Alterations should be stated.

Contracts written by public authorities generally require that part of the partial payment due the contractor shall be retained as security for the fulfillment of the contract. It may not be immediately clear why any percentage should be retained in view of the condition that a performance bond in the full amount is required from the contractor by the owner. However, by the experience of many years, it has been shown to be a useful device to keep the contractor on the job. Further, in the absence of provision for a retained percentage the premium required by the bonding company would undoubtedly be sufficiently larger to offset the additional hazard of having the contractor abandon the job.

The feature that is not satisfactory is the one that states that this retained amount may be reduced at the option of the owner. The specification writer will be politically unwise to write something definite into the public authority contract so that all contractors would be treated alike. In preparing a contract for a private owner, as distinguished from a public authority, a limit to the amount to be retained should be stated. It is unfair for the contractor to lose the interest on his earned money. An equitable solution might be to permit the contractor to replace the retained money with negotiable bonds as security, whereby the contractor may receive his interest. Also, the maximum amount that will be withheld should be limited; say 5% of the total contract cost, with specific reductions as the contract nears completion.

Some public authorities do not state when the responsibility of the contractor ends. This is not quite fair. The contract should state that the responsibility ends when the bridge is accepted by the owner. It is also desirable that the acceptance by the owner be made in a reasonable time after the bridge is open to traffic.

The General Covenants should note the conditions under which the contract may be annulled. This is contract law and had better be prepared by a lawyer. It is general and can be written without knowledge of bridge construction.

Standard Specifications will include the items mentioned above and generally include much more. The Standards are designed for the general case. Having read them several times, the bridge specification writer will remember the gist.

Special Provisions are the real work of the specification writer. The definition itself indicates that they are to apply to the particular project for which they are written. The specification writer's knowledge of the work and his experience are thoroughly tested to such an extent that he may need some help.

Sources of Information for Writing Specifications

The office library should be invaluable for information similar to that required for the bridge being considered. This library should contain not only all the specifications prepared by the company, it should contain all specifications the company can obtain and all references from current literature. One should not be afraid of the charge of plagiarism by following the outline from another's work. The other man stood on the shoulders of those who preceded him and it is only fair that others stand on his shoulders. No one starts from the very beginning in all his work.

The use of "canned" specifications, just drawn bodily from the library and copied verbatim into the current specification, should be avoided. The case is cited where on a bridge job in Hawaii a "canned" specification was used and the contractor was directed to dispose of excavated material in a prescribed location in New York Harbor.

Another invaluable source is current literature issued by scientific groups, and in periodicals like Construction Methods and Engineering News.

Listening to salesmen is time consuming and sometimes irritating, but often enough rewarding in the information obtained. The

good salesman knows more about his product than the specification writer. One does not have to buy, but can listen.

Materials displays is a good place to meet salesmen, ask questions, and get literature on current materials and processes.

Attendance at special meetings by the Bridge Committee of the American Association of State Highway Officials is important. Matters discussed there do not appear in the literature for some time. Most important, the reasons for using or for not using a particular material or process are thoroughly discussed by men who have to deal daily with materials and procedures for bridge construction.

Bridge specifications should be discussed with the men in charge of construction. Some problems in construction or administration can be avoided, if the specification writer is informed of the unsatisfactory conditions the first time the problem arises. The bridge specification writer should sit in on every conference considering an extra or a claim by the contractor.

The perfect "devil's advocate" is the contractor. The specification writer should be present, if permitted, at the general conferences of contractors or of contractors joined with public authorities to discuss common problems. The contractor seeks, and is entitled to seek, short cuts to save time and money. That is the province and purpose of management. Information on these short cuts is valuable to the specification writer, both to permit them when they produce better quality for the money and to block such short cuts when they produce inferior results.

There is nothing so informative to the specification writer as visiting a job site. The scratches on the drawing paper and the verbosity of the specification book are too often the results of archaic office procedure and holding to the dead hand of the past. In one of Lord Chesterfield's letters to his son he observes: "People say things, at first, because other people have said them, and then they persist in them, because they have said them themselves." The visit to the site gives the specification writer the opportunity to see whether actual conditions can meet his oft-repeated precepts.

Pertinent court decisions on construction are important. From them one can learn mostly how to avoid pitfalls in certain types or conditions of construction. Abstracts of such decisions appear in several periodicals.

Value of Part of Specifications on Plans Instead of All in Separate Book

Consider to whom the specifications are addressed. It is not so much to the contractor in person as it is to his working foremen. It is also to the foreman in the fabricating shop, in the paint shop, and to the erecting crews for structural steel and for fences; to the estimators; and to the resident engineer, among others. Not all men engaged in the successive operations have seen or read the book, but all those responsible for the production have seen the plans. It may also be noted that the specifications are addressed to the Bureau of Public Roads and to the courts in the event of dispute. All have different interests.

Some writers prefer to confine specifications entirely to the book. Others prefer placing on the plans as much as can conveniently be placed, leaving to the book the general clauses and those specifications too lengthy for the plans. At least one state sets aside a full sheet of the bridge plans for writing only the specifications.

Someone has aptly said, "The technique of pictures with explanatory notes makes it possible to communicate, in a given time, more knowledge than is possible when communications are confined to the symbolisms of language." An effective method is to include on the sheets showing details of structural steel the notes for class of steel, painting, welding, use of high strength bolts, and other applicable items. Applicable notes on the appropriate sheets should be shown for concrete finish, pattern for form work, or sequence of placing concrete. Comparable notes should be used for fences, for waterproofing, and for piles. The value of such notes on the plans is that everyone connected with the bridge construction is made aware of the requirements. There is no recourse to the favorite: "I didn't know it was in the book."

The book is nevertheless important. The book can expand to the extent desirable on those notes which were necessarily brief on the plans. It will do no harm to repeat.

Methods of Writing

All methods of writing specifications aim at end results. The question arises whether the owner should specify the end result desired and leave the procedure to the contractor, or should specify

the procedure. It is not practicable to dictate both procedure and end result. If the specification writer specifies procedure, then the result, whether satisfactory or unsatisfactory, is not the concern of the contractor if he follows directions. Despite this handicap, specifying procedure is often desirable, even necessary.

Procedure must be specified when calling for a new product; or when the specification writer knows more, or thinks he knows more, about an obscure procedure; or when there are alternative procedures producing different degrees of quality of product and the writer wants the one he thinks is best suited for the conditions.

For examples, procedures should be specified for cofferdams and for backfill at rigid frames. The contractor knows how to backfill properly, but it costs less to do it otherwise. Procedure specifications was the usual method in specifications in earlier years of bridge construction. It is still used by many states and more often by railroads.

When end result alone is specified, the contractor has freedom to select his own suppliers, his equipment, and his work methods. The owner reserves only the right for design and for final acceptance. End result is perfectly satisfactory for the average case of structural steel, for concrete of a definite strength, for fences, for pile material, and so forth.

Whichever method is used the specifications should be made strict and equitable. A good contractor will do a good job under any specification. The marginal contractor and the tricky contractor will try to short-change the owner under any specification.

Qualifications of the Specification Writer

In the days of routine construction programs—they seem so long ago—speed in processing contract documents was of little moment. Also, in retrospect, methods of construction were comparatively standard. The specification writer was the top man or a man near the top in the organization.

New materials and new procedures in recent years have necessitated rather rapid changes in specifications. The top man has become the "contact man," and is no longer writing specifications or even reviewing them. It may be odious to ask who is writing the specifications now. It is preferable to suggest who should write them.

The specification writer should be one who can do bridge design, has construction experience, and has a good command of Eng-

lish. Further, he must like his job. This paragon of parts is somewhat difficult to find. Why do we need him?

These are days of specialization. The designer may not have had any experience on construction, and the construction man may not have had any experience on design. Some place in the organization there should be someone who understands both design and construction.

Undoubtedly the greatest abuse of specifications is to have them written by one who is lacking in knowledge of both design and construction and who, in addition, doesn't like the job. It is not necessary that the man must be the best designer and the best construction man in the organization. He must know enough of both to at least be inquisitive when he examines the bridge plans.

The bridge plans should be carefully reviewed by the specification writer. Every detail should be examined to see whether it is adequate in stage construction. Every note on the plans should be read to see whether it is clear in meaning. Abbreviations and phrases newly invented by the designer or draftsman must be avoided; only commonly accepted phrases and abbreviations should be used.

It was observed before that the writer should direct his specifications to the people who are to read them, and a partial list of readers was indicated. The writer must be able to transfer an idea clearly and logically from his mind to paper and then to the reader's mind. The wording must be so clear that a reasonable mind grasps the idea.

The writer must avoid all ambiguity in sentence structure, in punctuation, in vocabulary. If a word or phrase has no specific and unique meaning in the art of bridge construction it will be accepted in its dictionary meaning, which may not be the meaning intended by the writer.

The responsibility of the specification writer is considerable. He has the last free look at the bridge plans and specifications before bids are requested. An oversight by him can be very expensive to the owner.

Review of Specifications

The specifications should be reviewed by the construction department to make certain that procedures specified are not outmoded by procedures that have been developed even while the specifications under consideration were being written. They should also be re-

viewed by the proposed resident engineer, if it is known who will be resident.

The specifications should be reviewed by the division responsible for final payment. Here, too, recent developments may indicate a desirable change in method of measurement.

The owner or the group responsible for maintenance of the bridge should review both plans and specifications to see that the proposed construction and procedures will not introduce problems in maintenance.

SPECIFICATIONS—THEIR USE AND ABUSE

Part II—Highway Specifications

EDWARD C. KEANE,* MEMBER

GENERAL

IN these notes on highway specifications I shall assume that the specification writer is working within the framework of a set of standard specifications published by a state highway department. Such standards customarily cover the "general conditions" as well as detailed specifications for the items usually entering into highway and bridge construction. With the standard specifications as a basis, the special provisions have to include only the supplemental specifications or exceptions which may be applicable for a particular project, plus a general description of the work and a section on prosecution and progress. The special provisions must also include the sections required in Federal-aid work or by state laws or local ordinances, such as those containing minimum wage rates.

If the contract documents are being prepared for a private client, it may be advisable to use the general conditions forms prepared jointly by the American Society of Civil Engineers and the Associated General Contractors of America.

Whatever the source of the general conditions, it is important for the specification writer to become very familiar with them, not only so that he can determine where they should be supplemented or amended, but also to avoid duplicating them in the special provisions. I remember one case where the specification writer, starting his first highway job in a new state, spent several hours in drafting and perfecting the contractor's insurance requirements. He was chagrined to find out later that the subject was already well covered in the standards, and all he had to do was write in the limits of coverage.

* Fay, Spofford & Thorndike, Inc., Boston, Mass.

Suggested Procedure

Many a time and oft it has happened that the specification writer has been called in by the design engineer and informed that the contract drawings for a project will be ready for final printing in 48 hours or less, and that the special provisions must then be ready for final typing. The hapless specification man protests that he has had no previous warning, and has not had a chance to become familiar with the job. Then he asks for a set of prints of the plans, profiles, and typical sections. It turns out that this cannot be done just now, because the drawings are getting their final check and correction and cannot be spared for printing, but nevertheless the specifications must be written at once. The probable result is a sloppy job with many loose ends, and questions raised during the bidding period which should never have been necessary. There may be other questions which might well have been raised during the bidding period, but which are saved until later, during construction. At that time, the contractor may benefit considerably from an ambiguity in the contract, because, as Mr. Kirshen noted, the courts will generally resolve a doubt against the party that prepared the contract.

I may have overstated the case above; however, on important and complex work, it is very necessary to avoid the last-minute rush if a good job is to be obtained. I suggest the procedure outlined below, assuming the specification writer is not the same person as the designer or design supervisor.

(a) Keep in touch with required completion dates of current projects of the department for several weeks ahead in order to avoid being taken by surprise when a date is found to be close at hand.

(b) As early as possible, obtain a list of all the items that may enter into the project. In the case of large lump sum items, as for a bridge superstructure, list the sub-items which are gathered together in the lump sum item. A comprehensive item list serves as an excellent table of contents for the special provisions.

(c) Beside each listed item make a notation as to which of the following applies: (1) Covered by regular standard specifications, in which case no reference to the item is necessary in the special provisions; (2) Covered by standard

specification as modified in another recent special provision, which can be used or adapted to the new project; (3) Special specification needed for the particular project. The notations should be checked by the design engineer.

(d) Prepare drafts for classes (2) and (3) and place in a loose-leaf binder; keep up-to-date as the design progresses.

(e) Keep in touch with design all through the job. The designers will undoubtedly find it necessary to add or delete items from the original list, and possibly to create new classes of items to care for unusual conditions. The specification man must make a nuisance of himself, if necessary, to avoid missing something new. Whenever anything unfamiliar appears on the plans he should ask, "What item applies here? Do we need a new item? Can the operation called for be made incidental to an existing item?" If the latter is so, suitable statements are necessary in the special provisions or on the plans.

(f) Take the attitude throughout that the engineer of design must be finally responsible for the technical correctness of the specifications. No other person is in as good a position to know all details of the job and see that all operations are covered. He should actually write the sections on prosecution and progress, including information on traffic handling; and he should be responsible for approving the entire draft for final typing.

Helpful Hints

In the following paragraphs are hints about a number of matters which should not be overlooked. Some of these will be attended to by the designer, others by the specification writer, but there should be an understanding between the two as to who does what.

Utilities: As early as possible in the design period, inform utility owners affected by the work and obtain data on alterations. Alterations to be made by utility owners should be shown out of function on the plans, with a note that the work is to be done by others. Contractors must be told in advance about interferences that may affect the cost of their work. For complicated utility situations, call a conference of utilities affected and make sure they are coordinated

as to temporary and final positions, and, if necessary, as to sequence of work.

References to National Standards: Frequently, a standard of the American Association of State Highway Officials, or of some other national organization, will cover several types, classes, or strengths of material. If the state standard specifications do not state the particular properties desired, it is necessary to do so in the special provisions.

Protection of Public on Detours, etc.: When detours are specifically required under the contract, check to see that the contract items cover all necessary features, including special barricades, warning signs, or signals, and temporary lighting. Make clear to what extent removal of the detour construction will be paid for. State that temporary detours or other facilities installed by Contractor for his own convenience will be at his expense.

Site Availability: If parts of the site will not be available to the contractor at the start of the job, list dates when they are expected to be available. State conditions under which additional time will be allowed for completion, if properties do not become available when expected. State whether contractor will be allowed extra compensation on account of such delays, or extra time for completion.

Disposal Areas: Standard specifications usually refer to surplus excavated material with phrases such as "shall be wasted as directed" or "disposed of outside of the location" or "used for flattening slopes as directed" or something similar. In these days of improved profiles with consequent heavy grading, the handling of surplus material sometimes has an important effect on the contractor's costs, and the special provisions should be specific on this matter. If waste areas are to be designated it should be done in advance, and statements made as to approximate quantity desired in each location, whether hauling will be paid for, whether layering and compaction will be required, whether smoothing, seeding, etc. will be required, and if so, how paid for.

Highway Lighting: If lighting is to be included, ascertain what part of the system will be included in the general highway contract. Make sure the necessary agreements are made with the power company. If the municipality is to be called upon later to pay the annual bills for lighting, obtain concurrence of the city or town officials.

Railroads: If a crossing of a railroad is involved, either at grade

or with a grade separation, start early to make the necessary arrangements; these will have to be described in the special provisions. It is necessary to come to agreements with railroad officials on the design; on methods of construction (if disturbance of the track or operations is involved); on the days and hours when certain operations can be performed; on insurance coverage; on protection of railroad facilities including method of payment for flagmen, etc. It is often necessary for the state public utilities commission to hold a hearing, and time must be allowed for this.

Treatment of Borrow Pits: It has been generally realized in recent years that state highway commissions ought to be responsible, to some degree, for seeing that their contractors do not leave ugly scars in the countryside, resulting from borrow or waste operations. All areas used temporarily for such operations should be smoothed by bulldozers, graded to drain, and mulched and seeded. The special provisions should be specific as to whether this work will be paid for separately or considered incidental to other items.

Pay Limits: Whenever a project requires special detail plans not previously used, check to see that pay limits are definite. If plans do not cover the limits completely, this should be done in special provisions. Do not merely say "complete in place"—these words by themselves do not mean much. For example, in case of a pipe line item "complete in place" it is still necessary to say whether this item includes trenching, remedying poor soil conditions, special backfill, temporary pavement, permanent pavement, etc.

Embankment Delays Due to Poor Soil: With poor soil conditions it is sometimes necessary to stop in the middle of embankment formation, and allow underlying soils to gain strength under the partial load, so that a slide will not occur when the embankment is raised higher. In other cases, overloads may have to be placed and left for a few months while the underlying soil is consolidated, thus minimizing later settlement. If the designers feel that waiting periods may be required in the earthwork operations, this should be brought out clearly in the special provisions in order that there may be no claims later on account of the delays. (Incidentally, it might be well to inform the "front office" when such things are likely, so that publicity releases can be prepared, to inform the public as to why the delays are taking place.)

Certificates of Compliance: Standard specifications generally say

that certificates of compliance may be accepted, at the option of the engineer, in lieu of tests of materials by the owner or his agent. However, the contract documents do not often list the items for which such certificates should be provided. The obtaining of the certificates is often difficult if they are not requested in advance. It would be advisable to list in the special conditions the materials for which certificates should be provided, accompanied by a statement that the owner reserves the right to require samples and make tests on any material at any time.

Conclusion

At the risk of being redundant, I want to re-state my belief that the responsibility for technical sufficiency of the specifications ought to be on the person responsible for the design. The specification writer, if properly experienced in the office and in the field, can play a major part and can be entirely responsible for the form and editing, but he should not be held to account for the engineering aspects unless he is, in fact, in charge of the design also.

The importance of specifications is often discounted, unless or until disputes arise during construction as to the intent of the contract; then the specifications may become tremendously important. It should never be forgotten that clauses written specifically for the particular contract prevail over all other specifications, and also prevail over the contract drawings. If this fact is kept in mind by all concerned, the preparation of the specifications will receive the attention it deserves.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING
Boston Society of Civil Engineers

OCTOBER 17, 1962:—A Joint Meeting of the Boston Society of Civil Engineers with the Massachusetts Section of the American Society of Civil Engineers was held this evening at Worcester Polytechnic Institute, Worcester, Mass. The Student Chapters of the New England Colleges were especially urged to attend.

From 12:00 Noon to 4:00 P.M., a guided tour of the Alden Hydraulic Laboratory was made. At 6:30 P.M., dinner was served in Morgan Hall and delegates from Northeastern University, Mass. Institute of Technology, Tufts University, University of Rhode Island, University of Massachusetts, Norwich University and Worcester Polytechnic Institute were present.

President Bogren extended a cordial welcome to the students and expressed appreciation of the cooperation of the officers of the student organizations and the faculty for making this event so successful.

President Bogren introduced President H. P. Storke of Worcester Polytechnic who welcomed the students, and then introduced Dean Price, Chairman of the Board of Registration, who made a few remarks.

The Secretary announced the names of applicants for membership in the BSCE and that the following had been elected to membership October 15, 1962:—

Grade of Member—William D. French, Louis J. Goodman, Francis S. Harvey, Rev. Daniel Linehan, Thomas R. Parello, Ref. James W. Skehan

Grade of Junior—Thomas E. Morello, Jr., Khalid M. Minhas

Grade of Student—Dean K. White

President Bogren introduced William H. Mitchell, President of the Massachusetts Section, ASCE, and asked him to conduct any necessary business of ASCE at this time.

President Bogren then introduced the speaker of the evening, Mr. John H. Fullerton, Project Manager, Jackson & Moreland, Inc., who gave a most interesting illustrated talk on "Lunar Landing Research Facility."

One hundred and sixty-one members and guests attended the dinner and one hundred and eighty-one attended the meeting.

The meeting adjourned at 9:05 P.M.

CHARLES O. BAIRD, JR., *Secretary*

NOVEMBER 28, 1962:—A Joint Meeting of the Boston Society of Civil Engineers with the Transportation Section was held this evening at the United Community Services Building, 14 Somerset Street, Boston, Mass., and was called

to order by President George G. Bogren, at 7:00 P.M.

President Bogren stated that the Minutes of the previous meeting October 17, 1962 would be published in a forthcoming issue of the Journal and that the reading of those Minutes would be waived unless there was objection.

President Bogren announced the death of the following members:—

Frank H. Dillaby, who was elected a member May 18, 1910 and who died October 8, 1962.

John A. McAuliffe, who was elected a member September 23, 1942, and who died October 16, 1962.

James B. Flaws, who was elected a member April 30, 1910 and who died October 18, 1962.

The Secretary announced the names of applicants for membership in the Society and that the following had been elected to membership on November 28, 1962:—

Grade of Member—Irving M. Finberg, William D. Jordan, Warren J. Scott

President Bogren announced this was a Joint Meeting with the Transportation Section and called upon James W. Haley, Chairman of that Section, to conduct any necessary business at this time.

President Bogren introduced speaker of the evening Mr. Cranston R. Rogers, Senior Engineer, Chas. A. Maguire & Associates, who gave a most interesting illustrated talk on "Highway Transportation Problems Encountered During the Inner Belt Study."

A discussion period followed the talk.

Forty-nine members and guests attended the dinner preceding the meeting and sixty members and guests attended the meeting.

The meeting adjourned at 9:15 P.M.

CHARLES O. BAIRD, JR., *Secretary*

DECEMBER 5, 1962:—A Joint Meeting of the Boston Society of Civil Engineers with the Sanitary Section was held this evening at the United Community Services Building, 14 Somerset Street, Boston, Mass., and was called to order by President George G. Bogren, at 7:00 P.M.

President Bogren stated that the Minutes of the previous meeting held November 28, 1962 would be published in a forthcoming issue of the Journal and that the reading of those Minutes would be waived unless there was objection.

President Bogren announced the death of the following Member:—

Adnan N. Adsiz, who was elected a member May 14, 1951 and who died during the month of November, 1962.

President Bogren announced that this was a Joint Meeting with the Sanitary Section and called upon George W. Hankinson, Chairman of that Section, to conduct any necessary business.

President Bogren introduced the speakers of the evening, Dr. Harold A. Thomas, Jr., and Dr. Robert P. Burden, who gave an extremely interesting illustrated talk on "Rebirth of an Asian Breadbasket."

A discussion period followed the talk.

Thirty-nine members and guests attended the dinner preceding the meeting and fifty members and guests attended the meeting.

The meeting adjourned at 8:45 P.M.

CHARLES O. BAIRD, JR., *Secretary*

HYDRAULICS SECTION

NOVEMBER 7, 1962:—A meeting of the Hydraulics Section of the Boston Soci-

ety of Civil Engineers was held in the Society Rooms, 47 Winter Street, Boston, Massachusetts, and was called to order by the Chairman of the Section, Mr. Lawrence C. Neale, at 7:05 P.M.

Chairman Neale stated that the reading of the Minutes of the previous meeting, held on May 2, 1962, were published in the last issue of the Journal and that the reading of those minutes would be waived unless there was objection.

Chairman Neale requested the members to submit the desired lists of books for the B.S.C.E. Library relating to hydraulics to the officers of the Section or to Mrs. Budea, Secretary at the Society Rooms.

The Chairman then announced that, in accordance with past practice, the Executive Committee of the Section recommended that the nominating committee for officers of the section for next year would consist of the past three chairmen—John B. McAleer, Donald F. Harleman and Lawrence C. Neale. This motion was seconded and approved by the members in attendance.

Mr. Neale introduced the speaker of the evening, Mr. K. Peter Devenis, Project Manager, Charles A. Maguire & Associates, who gave a talk on "Proposed Dam, Navigation Locks, and Flood Control Pumping Station for Mystic River Basin."

A brief history and description of the Mystic River Project was followed by some of the design problems. These included the necessity and methods for controlling salt water pollution into the proposed Basin; hydrology and methods for water level control including pumping; and lock design including emphasis on recreational boat traffic. The talk was followed by a movie illustrating the contrast between the present unsightly condition of the Mystic River Estuary and the highly recreational uses of the Charles River Basin, as well as navigational, flood control, and other aspects of the project.

A question period and discussion followed the talk.

The meeting adjourned at 8:45 P.M. with 63 persons in attendance.

K. PETER DEVENIS, *Clerk*

STRUCTURAL SECTION

MAY 9, 1962:—A regular meeting of the Structural Section was held this evening in the Society Rooms and was called to order by Chairman Percival S. Rice at 7:00 P.M.

The Chairman introduced the speaker of the evening, Mr. Charles I. Orr, District Engineer, American Bridge Division, U.S. Steel Corp., who spoke on "Structural Aspects of Unisphere."

The speaker reviewed the pertinent design and construction details of the unisphere which will be the theme symbol of the New York World's Fair.

The unisphere will be 120 feet in diameter, inclined at an angle of 23½ degrees, supported on a tripodic steel base in a 300-foot diameter pool, and made up of stainless steel tubing from 8 to 10 inches in depth, weighing a total of 390,000 pounds.

The structure was designed to withstand 110 MPH winds and the analysis involved 670 simultaneous equations which were solved on an electronic computer.

Slides were shown to illustrate the general design and construction features of the project.

After a brief question and answer period, the meeting was adjourned at 8:15 P.M.

The meeting was attended by 28 members and guests.

MAX D. SOROTA, *Clerk*

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