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16. Abstract

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STUDY OF THE LATERAL PRESSURE OF FRESH CONCRETE AS RELATED TO THE DESIGN OF DRILLED SHAFTS

by

Juan B. Bernal
Lymon C. Reese

Research Report 308-1F

The Influence of the Characteristics of Concrete on the Load-Carrying Capacity of Drilled Shafts

Research Study 3-5-82-308

conducted for

Texas
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by the

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November 1983

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PREFACE

This report presents the results of an experimental investigation of the lateral pressure of concrete as related to the design of drilled shafts.

The authors wish to thank the Texas State Department of Highways and Public Transportation for their sponsorship of the work and express appreciation to their contact member, Mr. George Odom, for his cooperation during the project. Financial support from Dow Chemical Company and the Association of Drilled Shaft Contractors is also acknowledged.

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Juan B. Bernal
Lymon C. Reese

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ABSTRACT

A series of tests were conducted to determine the effect of the consistency of concrete, as measured by the slump test, on the lateral pressure of concrete. Testing conditions simulated the construction of drilled shafts as practiced by the Texas State Department of Highways and Public Transportation. The tests showed that increasing the slump of the concrete increased the maximum pressure and the length of shaft which was under hydrostatic conditions. Pulling the tremie increased the lateral pressure by variable amounts in an unpredictable manner.

The increased pressure against the sides of an excavation is desirable because the axial capacity of a drilled shaft is increased.

KEY WORDS: drilled shaft, skin friction, lateral pressure, concrete, superplasticizer.

SUMMARY

A series of tests were conducted to determine the influence of the consistency of concrete, as measured by the slump test, on the lateral pressure of concrete. Other variables known to affect the pressure of concrete were either kept constant or appropriate corrections were made to the results. The experimental set-up consisted of a column, 31 ft high, constructed using 42 in. diameter, circular steel formwork and instrumented on the sides to measure the pressure distribution as a function of depth.

The results of the tests showed that, keeping other variables constant, an increase in the slump of concrete produces higher lateral pressures and increases the length of column which is under hydrostatic conditions.

Previous investigations of the behavior of drilled shafts have not taken into consideration any concrete-related effects except for moisture migration from concrete into the soil. The results of the tests performed show that very different distributions of lateral pressure can be exerted by concrete. Increased lateral pressure will improve the ability of drilled shafts to carry load in skin friction.

The research also showed that the use of concrete with a high slump, made by use of a superplasticizer, facilitated construction operation. The concrete was placed rapidly and the tremie was easily withdrawn.

IMPLEMENTATION STATEMENT

The findings reported herein can be implemented following a study of the State Department of Highways and Public Transportation of the construction specifications for drilled shafts. Indications are that the implementation of the findings will result in improved construction procedures and improved performance of drilled shafts.

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CHAPTER 1. INTRODUCTION

A drilled shaft is a foundation element constructed by drilling a hole in the ground, placing reinforcement if required, and finally filling the hole with fresh concrete. Various methods of construction exist to cope with the different soil conditions that might be encountered (Reese, 1978).

Previous research into the behavior of this type of foundation has provided empirical correlations between load transfer and different types of soil (Whitaker and Cooke, 1966; Holtz and Baker, 1972; O'Neill and Reese, 1972). In addition, research has also established the influence of the construction method on the load transfer (Chadeisson, 1961; Farmer et al, 1970; Geffen and Amir, 1971; Reese et al, 1981). Still, there is a third aspect in the construction of a drilled shaft which has not been adequately investigated, namely, the influence of the concrete characteristics and concreting operation on the behavior of the foundation. Some of the research has tried to determine the effects of wet concrete on the adjacent cohesive soil. The results have shown that within about 3 in. from the soil-concrete interface there is an increase in moisture content of about 4 to 6% (Meyerhof and Murdock, 1953; Mohan and Chandra, 1961; Chuang and Reese, 1969).

More recently, Sheikh et al (1983) have examined the potential use of expansive cements in drilled shafts. The authors present the hypothesis that expansive cement concrete produces a stronger bond between the shaft concrete and the surrounding soil, which results in an increased shaft capacity and a lower settlement at working loads when compared with similar shafts constructed using normal cement. To substantiate the hypothesis, three shafts, 12 in. in diameter and 11 ft long, were constructed on the campus of the University of Houston and load tested. Two of the shafts were formed using expansive cement. The results of the tests appear to confirm the proposed hypothesis.

There are, however, other areas related to concrete which deserve attention. One of these is related to the ability of concrete to flow. Traditionally, the most important property of concrete has been its compressive strength. Therefore, low water contents (and consequently low slumps) have been used to attain desirable strengths in typical structural concrete. Furthermore, because structural concrete is typically vibrated there is no need for the concrete to have good flowing characteristics. In the construction of a drilled shaft, on the other hand, different requirements are placed on the concrete. Because no vibration is used, a concrete that flows easily through rebars and which compacts under its own weight is required. Some defective drilled shafts have been constructed because concrete either failed to flow through the reinforcement or failed to eject cuttings or other material from the hole. This kind of problem has been known to occur but little research, if any, has been carried out.

A second area related to concrete and the behavior of drilled shafts involves the influence on the skin friction of the lateral pressure developed by fresh concrete against the sides of the excavation. When a hole is opened in the ground, there is a stress relief, the total stress at the face of the borehole being reduced to zero. As a result of concreting, new lateral stresses are set up which may be higher or lower than the original in situ stresses. Recent articles (Reese et al, 1981; van Weele, 1982) stress the importance of the concreting operation on the final stress between the shaft and the soil and, furthermore, suggest the possibility of calculating the skin friction of drilled shafts using the lateral pressure of concrete. Based on these concepts, an experimental study of the lateral pressure of concrete was undertaken. This report presents the results of the investigation.

CHAPTER 2. INFLUENCE OF LATERAL PRESSURE OF CONCRETE ON BEHAVIOR OF DRILLED SHAFTS

COMPUTATION OF AXIAL CAPACITY

The computation of the ultimate axial capacity of a drilled shaft, Q_t, is typically done by calculating independently, and then adding the ultimate side capacity, Q_s, and the ultimate base resistance, Q_b. The procedures used in sands and clays are different as shown in the following paragraphs.

In a homogeneous clay profile, the ultimate side and base capacities are calculated as:

$$Q_s = \alpha\ s_u A_s \tag{2.1}$$

$$Q_b = c\ N_c A_b \tag{2.2}$$

where

α = reduction factor;

s_u = average undrained shear strength of soil along the sides;

A_s = peripheral area of pier shaft;

c = undrained shear strength of soil below the base;

A_b = area of the base; and

N_c = bearing capacity factors.

Values of N_c and α have been presented elsewhere (Skempton, 1959; Whitaker and Cooke, 1966; Reese et al, 1976).

In cohesionless soils the side capacity of a drilled shaft sometimes has been neglected (Winterkorn and Fang, 1975; Peck et al, 1974). Research in the past 10 or 15 years has provided data regarding the skin friction of piers in sands. Reese et al (1976) have presented the following recommendation for the calculation of the ultimate side capacity:

$$Q_s = \alpha_{avg}\ C \int_0^H p_z \tan \bar{\phi}_z \, dz \tag{2.3}$$

where

α_{avg} = factor that allows correlation with experimental results;
C = circumference of pier;
H = total depth of embedment;
p_z = effective overburden pressure at depth z;
ϕ_z = effective angle of internal friction; and
dz = differential element of length.

The authors recommend the use of 0.7 for shafts with a penetration in sand not exceeding 25 feet. They further suggest an α_{avg} of 0.6 for piers penetrating in sand between 25 and 40 ft, and 0.50 for shafts of greater penetration.

Regarding the ultimate base capacity of drilled shafts in sands, the approach has been to use the traditional bearing capacity equation (Terzaghi, 1943) modified as follows:

$$Q_b = \bar{\sigma} N_q A_b \tag{2.4}$$

where

$\bar{\sigma}$ = effective vertical stress at the base level;
N_q = a bearing capacity factor; and
A_b = area of the base.

Another approach has been suggested by Reese et al (1976) but will not be presented here.

From the above presentation it is obvious that the nature of the soil and the method employed to construct a drilled shaft affect its ultimate axial capacity. The influence of the concrete characteristics and concreting operation on the behavior of this type of foundation is yet to be established.

RELEVANCE OF LATERAL STRESS TO BEHAVIOR IN SKIN FRICTION

Some theories for determining the skin friction along a drilled shaft assume that the initial state of stress is reestablished around the pile after the end of construction. The stress relief due to opening of the hole and the creation of new stresses due to concreting are not taken into account.

There exists evidence which suggests that the skin friction of drilled shafts is strongly influenced by the pressure developed by fresh concrete against the natural soil. In this regard, some cases which lend support to this idea will be presented. One case is concerned with the behavior of grouted anchors or tiebacks. In the construction of a tieback, a hole is drilled in the ground and a high-tension cable or rod is installed. Part of the hole is then filled with a cement grout under pressure. Predicted capacities for these anchors, using skin friction values based on the overburden pressure, are much lower than the measured frictional resistance (Shields et al, 1978). Van Weele (1982) quotes frictional resistances of 5 to 8 T/sq ft in medium dense sands at depths of 16 to 26 feet. If the relation $\tau = \sigma \tan \phi$ is used, a unit friction of less than 15 T/sq ft is calculated at a depth of 25 feet. Thus, the argument can be made that the grout pressure has an influence on the load transfer of the tieback.

Another case was presented by Reese et al (1981). Two shafts constructed in Galveston in about the same soil conditions developed quite different load-transfer values. The soil in the top 40 ft in both cases was sand. One test showed no frictional resistance in the sand while the other developed load transfer values up to a maximum of 0.87 T/sq foot. The authors attribute this difference in behavior to the fact that the first shaft was 24 in. in diameter and the concrete delivered at the site was of questionable quality. It is postulated that the concrete took an early set so that little or no lateral pressures were developed at the interface of the concrete and the sand with the result that there was no load transfer. In the second test, the shaft was 48 in. in diameter and the concrete had excellent flow characteristics.

Whitaker and Cooke (1966) tested five 37-in.-diameter bored piles in tension to determine the frictional resistance developed in London clay in order to design the reaction system for a series of compression tests. Four of the piles were constructed using a Calweld machine and in one of the piles the concrete was vibrated. The concrete slump used in all the tests was 4 inches. The results showed a frictional resistance 20% higher for the vibrated pile as compared to the other piles.

Reynaud and Riviere (1981) present the results of an investigation conducted during the construction of a diaphragm wall. The authors measured the pressure developed by fresh concrete in order to verify the hypothesis they set forth. According to Reynaud and Riviere, in previous measurements of the total pressures at the soil-wall interface, the experimental value of K_0 (the ratio of horizontal to vertical effective stress in the soil) was found to be higher than that calculated using Jacky's formula ($K_0 = 1 - \sin \phi$). The soil profile at the site consisted essentially of sands for which a K_0 value of 0.26 was calculated while an experimental value of 0.80 was measured. Based on these observations, the authors proposed the hypothesis that the state of stress in the soil was modified and the soil went from an initial at-rest condition to a state of partial compression produced by the pressure of the concrete.

Thus, the cases just presented seem to indicate that the skin friction of drilled shafts is influenced by the lateral pressure exerted by the fresh concrete.

RELEVANCE OF LATERAL STRESS TO BEHAVIOR IN END BEARING

The construction of a drilled shaft produces a relief in vertical stress at the bottom of the excavation. The magnitude of the stress relief can produce significant changes in the properties of the soil. This effect seems to be more critical in granular soils and therefore the following discussion refers principally to sand profiles.

The magnitude of the stress relief, inflow of water into the excavation, and poor techniques of construction can combine to produce some loosening of the soil at the base. After the hole is constructed and cleaned, concrete is poured producing a new value of the total vertical stress. At most, this pressure could reach a value equal to the length of the shaft times the unit weight of the concrete. Whether or not this maximum value is obtained is dependent on the concrete characteristics and the concreting operation. The pressure could be less, equal, or greater than the original in situ vertical stress depending on the stress history of the soil. It is desirable to apply the maximum pressure possible at the base of the shaft to produce densification of the loosened material, and densification of other soil which might collect at the bottom of the hole. This compaction by the pressure produced by the column of concrete will improve the behavior at the base.

CHAPTER 3. PORTLAND-CEMENT CONCRETE

INTRODUCTION

The construction of drilled shafts involves the use of Portland-cement concrete, probably the most commonly used material of construction. Concrete as a material is widely available, is versatile, is adaptable to various jobs, and frequently offers economic advantages. There is a significant and continuous amount of research devoted to concrete. The research has been responsible for new technological advances which have extended the range of conditions under which concrete can be used (Neville and Chatterton, 1979). Thus, concrete can be vibrated, pumped horizontally or vertically considerable distances, or placed under water. As long as proper construction procedures are observed, a concrete of high quality is obtained. Nevertheless, concrete is at the same time a composite, complex material whose properties and behavior are not yet fully understood although great advances have been made with the application of new technologies (Ramachandran et al, 1981).

Before addressing the main topic of this investigation, the lateral pressure of concrete, it seems appropriate to review some basic information related to concrete. This chapter, therefore, will be devoted to a brief presentation of three aspects of concrete. The first will be concerned with the production, composition and properties of Portland cement, while the second part will deal with the behavior of a fresh mass of concrete, placing emphasis on the concepts of workability and setting. The third section will briefly address the subject of admixtures with emphasis on chemical admixtures. For a thorough coverage of these topics the reader is referred to texts on concrete technology (Popovics, 1982; Powers, 1968). In this report the word cement will refer to Portland cement unless otherwise specified, and likewise concrete will refer to Portland-cement concrete.

PORTLAND CEMENT

This type of cement consists mainly of silicates and aluminates of lime. It has the property of setting and hardening under water and, thus, is called a hydraulic cement. Its origins can be traced back to the first half of the 19th century when the first cements of this type were produced. The name Portland cement was derived due to the similarity of the hardened cement paste to a naturally occurring limestone (Portland stone) quarried in England at that time.

Production

Materials containing lime, silica, alumina, and iron oxide are required to manufacture Portland cement. Most commonly limestone, chalk or other calcareous material provides the lime while shales and clays are typically the sources of the other required compounds. These raw materials are first ground, mixed in certain proportions and then partially fused in a large rotary kiln at a temperature of 1400° to 1500°C. The material coming out of the kiln (called clinker) consists of dark grey, porous balls which, after cooling, are ground to a fine powder with a small amount of gypsum (up to 5%) to form the final product.

Composition

Portland cement is a mixture of several compounds. For all practical purposes there are only a few major constituents of the cement. These are shown in Table 3.1 along with some typical proportions. Note that the percentages in the table do not add to 100% due to the presence of other minor compounds and impurities in the cement. The calcium silicates (C_3S and C_2S), comprising roughly about 75% by weight, are by far the most important compounds in Portland cement and are responsible for its cementing qualities.

In addition to the major compounds listed in Table 3.1, there are some minor ones which account for only a small percentage of the weight of cement. Of these, some can be of considerable importance with regard to concrete performance. For example, the presence of excess free lime (C_aO) and magnesia (M_gO) can lead to excessive expansion and eventual cracking of the concrete, a problem known as unsoundness. Expansive deterioration of concrete might also be the result of a reaction between some types of siliceous aggregates and the alkalis (Na_2O and K_2O) in the cement. To avoid these problems the percentages of these and other potentially troublesome constituents in cement are limited.

Hydration

When cement is mixed with water, the individual compounds in cement undergo a series of chemical reactions which are collectively called hydration. These reactions liberate heat and eventually lead to the hardening of the con-

TABLE 3.1. TYPICAL PROPORTIONS OF MAIN COMPOUNDS IN PORTLAND CEMENT

Name of Compound	Shorthand Notation	% by Weight
tricalcium silicate	C_3S	50
dicalcium silicate	C_2S	25
tricalcium aluminate	C_3A	12
tetracalcium aluminoferrite	C_4AF	8
calcium sulphate dihydrate (gypsum)	$C\bar{S}H_2$	3

crete. The new compounds formed during hydration are referred to as hydration products.

The main hydration products, derived from C_3S and C_2S, are a calcium silicate hydrate and calcium hydroxide. The first one, known as C-S-H, is a variable compound, with a poorly crystallized structure which forms extremely small particles. The calcium hydroxide, by contrast, has a well defined crystalline structure and a fixed composition. These two compounds account for about 70 to 85% of the volume of a hydrated cement paste. The rest of the volume is occupied by calcium sulfoaluminates, other minor compounds and voids. Of these, the calcium sulfoaluminate, which is derived from the hydration of C_3A, is very important with regard to the resistance of concrete to sulphate attack.

As mentioned earlier, the hydration of cement particles liberates heat and leads to the eventual hardening of the mass. Studies of the hydration characteristics of pure cement compounds have shown the relative importance of each of the compounds in the areas of heat evolution and strength development. C_4AF is a minor compound and will not be mentioned here. The rate of hydration of the compounds in Portland cement increases in the order C_2S, C_3S, C_3A. Thus, C_3A is the most reactive compound followed by C_3S and C_2S a distant third. Since the hydration reactions are exothermic, it follows that heat evolution is higher for the more reactive compounds, C_3A and C_3S, as compared to C_2S. In this respect, it should be mentioned that the presence of gypsum in cement controls the strong initial reaction of C_3A with water which often leads to flash set. Strength development on the other hand is not completely related to rate of hydration. Even though C_3A is the most reactive compound, it is known that pastes of C_3S and C_2S gain much more strength than those of C_3A. Moreover, since C_3S is more reactive than C_2S, it develops strength much faster. With time, however, strength developments of C_3S and C_2S are comparable.

While it is recognized that reactions in Portland cement concrete do not take place exactly as they occur in the pure cement compounds it is assumed that, within certain limits, the hydration of each compound in the cement takes place independently of the others present. Indeed, it has been found that the products of hydration of cement are chemically the same as the products of hydration of the individual compounds under similar conditions (Neville, 1981), suggesting that the previous assumption is reasonable.

Types of Cement Described by the American Society of Testing and Materials

The actual quantity of the various compounds (C_3S, C_2S, C_3A....) in the cement varies considerably and is dependent in part on the composition of the raw materials. Because the properties and behavior of Portland cement are influenced by the quantities of these compounds, cement with different properties can be produced by a suitable proportioning of the compound composition. Thus, ASTM recognizes five distinct Portland cements, namely, ASTM Types I through V. Typical compound composition of these cements is shown in Table 3.2.

Type I, referred to as normal, is the most common and is used where no special properties are required. In the other types the quantities of C_3S, C_2S, and C_3A are varied relative to type I, so that the various cements perform satisfactorily under particular conditions. For example, when a high early strength is desired, type III is called for. In type III cement, the amount of C_3S has been increased over that in type I. It will be recalled that pastes of C_3S show the fastest strength development of all the pure compounds and thus, type III cement gains strength faster than type I. More important is the fact that type III cement is ground more finely thus increasing the surface area of the cement which will be in contact with water. The larger surface area leads to faster hydration and more rapid development of strength. For mass-concrete placement, where thermal cracking might be of concern, type IV is utilized. Because the rates of hydration of C_3A and C_3S lead to high rates of heat evolution, these compounds are greatly reduced in type IV cement while the percentages of C_2S are increased.

BEHAVIOR OF FRESH CONCRETE

General Description

Concrete is made by mixing together cement, fine and coarse aggregate, and water. Properly mixed fresh concrete forms a mass with a more or less uniform distribution of its constituents. The cement and water (and some air) form the paste or matrix of the concrete in which the aggregate particles are "floating." This matrix serves two functions: first, it holds the aggregate particles in a dispersed state thus reducing point-to-point contacts, and second, it works as a lubricating material between the aggregate particles. Thus, the amount and composition of the paste plays an important role in determining the properties and behavior of fresh concrete.

TABLE 3.2. TYPICAL COMPOUND COMPOSITION OF PORTLAND CEMENT

Type of Cement →	I	II	III	IV	V
C_3S	50	45	60	25	40
C_2S	25	30	15	50	40
C_3A	12	7	10	5	4
C_4AF	8	12	8	12	10

The behavior of freshly-mixed concrete will be examined in relation to the calorimetric curve shown in Fig. 3.1. When cement is first mixed with water, a relatively fast chemical reaction takes place with a rapid evolution of heat. The rate of heat evolution drops rapidly marking the end of the first stage. In the second stage there is a very low rate of hydration. This period of relative inactivity, called the dormant period, allows concrete to remain plastic for 60 minutes or so, depending mainly on the characteristics of the cement. The length of the dormant period can be extended well beyond 60 minutes by the addition of a retarding admixture, or it can be shortened to a matter of minutes by adding an accelerator admixture. In addition, an abnormal setting behavior of the cement might also be responsible for an early loss of plasticity.

A rapid increase in the rate of heat evolution marks the end of the second stage. During the third stage the rate of heat evolution increases rapidly, reaches a peak, and then decreases gradually to a very small amount. During the first part of this period, the cement particles are actively hydrating and the new products are being formed. The growth and interlocking of these hydration products lead to a loss in plasticity and an eventual hardening of the mass.

Workability

There is a series of terms used to describe the appearance and behavior of fresh concrete. None are standard and all are dependent to some degree on the subjective evaluation of the observer. Consistency, for example, is used by some to describe the degree of dryness or wetness of a given mix. The wetter mixes usually are easier to pour and place than drier ones. Consistency is sometimes used as a substitute for workability. The latter, however, is a more complex term and has a broader meaning. The literature on fresh concrete has many different definitions of workability. In general, these definitions of workability can be grouped into one of two classes:

(1) those that attribute to workability the ease of mixing, transporting, placing, compacting, and finishing, and
(2) those that define workability in terms of the amount of useful internal work required to produce compaction of the mix.

Some other definitions of workability also include resistance to segregation and bleeding. Whatever definition is used, it is important to recognize that the workability of a mix is determined by a number of basic properties of the

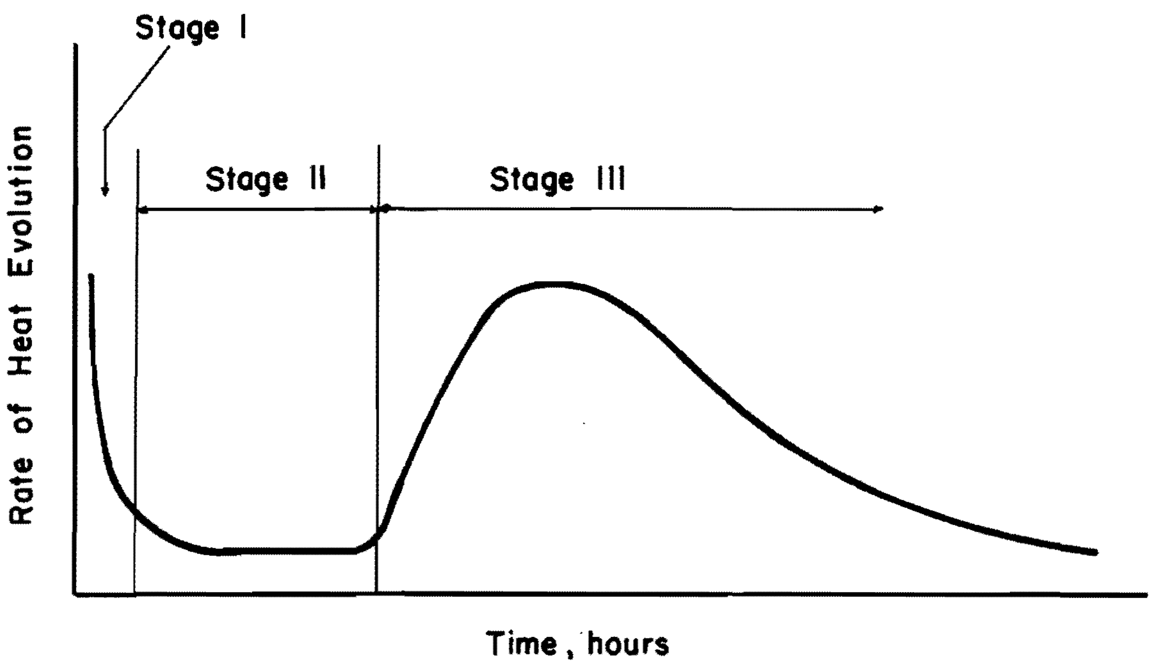


Fig. 3.1. Rate of heat evolution versus time for the chemical reaction between cement and water. (Mindess and Young, 1981)

mix such as the angle of internal friction, cohesion, viscosity, plasticity, and tendency for segregation and bleeding (Popovics, 1982). Furthermore, it must be pointed out that workability, as used now, is not only dependent on fundamental properties of the mix but also on external factors. For example, a mix can be designed which has a workability suitable for use in mass concrete but this workability would not necessarily be adequate for a tremie placement of the concrete. Thus, the intended use of the concrete, as well as the methods and equipment for placing it, have a bearing on whether the workability of a mix is adequate.

The single most important factor affecting the workability of concrete is the water content of the mix. Increasing the amount of water increases the workability. However, excess water might result in undesirable effects on the performance of the concrete. An increase in water content reduces the strength, increases the permeability, and can cause segregation and bleeding. There are other factors, in addition to water content, that have an influence on the workability and which must be considered when designing a mix. Among these are mix proportions (more specifically the amount of aggregate and the relative proportions of fine and coarse aggregates), the aggregate properties (size distribution, shape and texture), and the use of admixtures (air entraining agents, water reducing agents).

Measurement of Workability

A large number of methods have been developed to measure the workability of concrete. A few have been incorporated into standards while the rest have received very limited or no use at all. The proposed tests can be broadly classified as flow (such as the slump test), compaction, penetration, remoulding, two-point, and miscellaneous tests. For the purpose of this report, only the slump, the compacting factor, the tremie flow, and the two-point tests will be presented.

Slump Test. This is by far the most widely used test to measure the workability of concrete. The equipment required for the test consists of a hollow mold in the form of a frustum of a cone (12 in. high; 8 in. bottom diameter; 4 in. top diameter), a non-absorbant baseplate, and a tamping rod. The cone is placed on the base-plate with the smaller opening at the top and filled with concrete in three layers of equal volume, tamping each layer 25 times with the rod. After the excess concrete is removed from the top, the cone is lifted vertically and the concrete allowed to slump. The decrease in height at the center of the base of the specimen is the slump of the concrete. The details of the test are described in ASTM C142-74. (There are differences in the procedures for performing the test. For example, the British standard BS:1881:Part 2:1970, specifies four layers and measures the slump to the highest part of the slumped concrete.)

There are some difficulties associated with the slump test and its results. Two concretes with equal slump values do not necessarily behave in the same manner. For example, lean and rich mixes can be designed for a 4 in. slump. The lean mix (small amount of fines) will appear very harsh and with very little capability for plastic deformations while the rich mix will look cohesive and very plastic. Thus, a slump value is not a unique characterization of the behavior of a mass of fresh concrete. Another common objection to the test concerns its reproducibility. In this regard, Popovics (1981) has presented statistical evidence to show that the variations in the slump results are due to variations in the composition of the samples much more than to the lack of reproducibility of the test. In addition, the test does not work well with lean mixes nor does it differentiate among low workability concretes. Nonetheless, due to its simplicity, the slump test is commonly used. It is recommended for concretes with slumps in the range of 1 1/2 to 7 inches.

Compacting Factor Test. The compacting factor test was developed in England (1947) to measure the degree of compaction produced by a given amount of work. The standard amount of work is provided by allowing concrete to fall under gravity from a standard height. The apparatus used in the test is shown in Fig. 3.2.

To perform a test, the hinged doors at the bottom of the hoppers are closed and the top hopper is loosely filled with concrete. The door of the top hopper is opened and the concrete is allowed to fall successively into the middle hopper and then into the cylinder. The excess concrete is struck off and the density of the concrete in the cyinder is determined. The ratio of this density to that of the fully-compacted concrete is defined as the compacting factor.

This test does not differentiate well among concretes with slumps larger than about 3 or 4 inches. It is, however, sensitive in the low end of the workability scale where variations in the workability of dry mixes are reflected in the relatively large changes in the compacting factor.

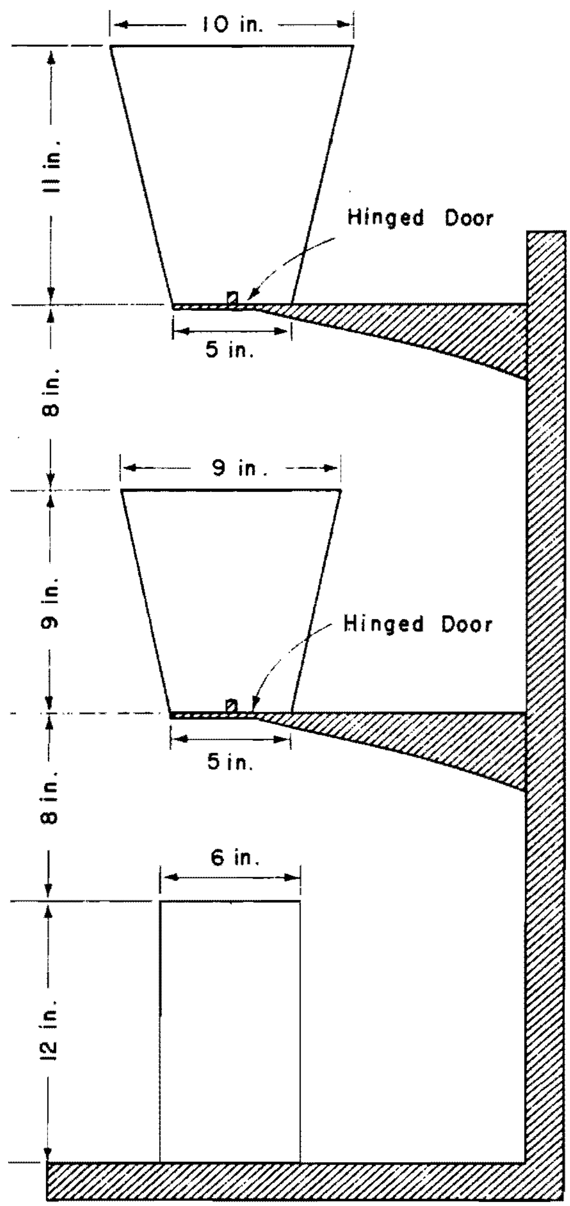


Fig. 3.2. Schematic of compacting-factor apparatus. (Mindess and Young, 1981)

Tremie Flow Test. In an attempt to simulate concrete flow through a tremie pipe, Gerwick et al, (1981) developed the tremie flow test during an investigation of underwater tremie placement of mass concrete. The apparatus used in the test is shown in Fig. 3.3. It consists of a 4 in. inside diameter tube held concentrically within a metal pail, 11.5 in. in diameter. The small tube is lowered to the bottom of the pail and concrete is placed in three layers, each rodded 25 times. After the concrete is in place, the tube is lifted until its bottom is 4.5 in. above the bottom of the metal pail. The concrete in the tube flows out into the pail. After the flow stops, the distance from the top of the tube to the top of the concrete inside the tube is measured and reported as the flow of concrete.

Two-Point Test. Tattersall (1976) has taken a rheological approach and advocates the use of a Bingham fluid to represent the flow of fresh concrete. He argues that workability should be measured as a unique characteristic of the mix independently of external factors. Thus, two concretes with the same characteristics would behave identically under any set of circumstances. Tattersall criticizes all the tests that have been proposed to measure workability arguing that they are all single-point tests and as such are unable to describe the behavior of a Bingham fluid. For such a material, flow does not occur until some critical stress is reached. The equation describing the behavior is:

$$\tau = \tau_0 + \mu D \qquad (3.1)$$

where

τ = shear stress

τ_0 = yield stress

μ = plastic viscosity

D = rate of deformation.

From Eq. 3.1 it is clear that two constants, μ and τ_0, must be evaluated. Thus, at least two measurements have to be made and hence, the name two-point test.

Tattersall used a food mixer equiped with a stainless hook and bowl. By measuring the electrical power input to the mixer and the speed of the hook, he was able to obtain values of torque, T, which were then plotted against speed. These yielded approximately linear relationships which he expressed as

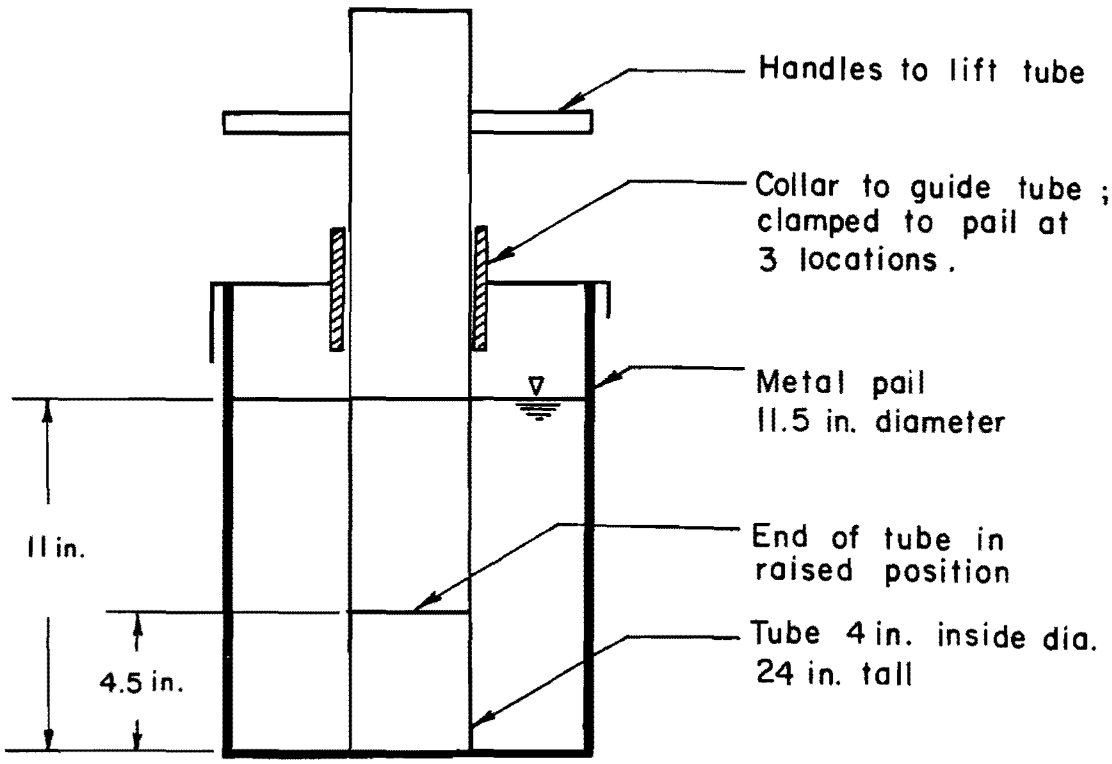


Fig. 3.3. Tremie flow device.
(Gerwick et al, 1981)

$$T = g + hN \tag{3.2}$$

where T = torque measured in N rev/sec and g and h were constants proportional to the yield value and the plastic viscosity. According to Tattersall, it is these two constants that provide a measure of the basic rheological properties of concrete.

Setting of Concrete

Figure 3.1 shows that when water is mixed with cement there is initially a high rate of hydration which drops sharply and is followed by the so-called dormant period. After the dormant period is over, a fast rate of hydration of the cement compound C_3S results in the stiffening of the mass. This process of setting can be viewed as a transitional period during which concrete is transformed from a fluid or semi-fluid to a rigid, hard material. The changes in the properties of concrete that take place during this time are gradual. Following the setting period the concrete starts to harden and gain measurable strength. This hardening should not be confused with the setting period.

Setting is usually described in terms of the inital and final sets. These are two arbitrarily defined points in the general relationship of strength-gain versus time. The initial and final sets do not have a special significance, nor do they bear any special relation to measurable properties of concrete. Of the two terms, initial set is generally more important. Typically, the initial set of commercial portland cements occurs in about 2 to 4 hours. However, under certain conditions an abnormal setting behavior can occur. Two major types of abnormal behavior are flash set and false set. Flash set refers to a rapid development of rigidity that cannot be disrupted simply by further mixing and which might result in adverse consequences on a job. It is caused by the fast hydration of C_3A, the most reactive compound in the cement. This type of problem has been eliminated by the addition of gypsum to portland cement to retard the hydration of C_3A. False set, on the other hand, is not a serious problem. It is also characterized by a rapid development of rigidity that, however, can be disrupted by further mixing without any adverse effects on the concrete. One way in which false set can occur is as follows. When gypsum is ground with very hot clinker, the high temperature can cause the gypsum to dehydrate to plaster. Upon the addition of water to cement, the plaster reverts to gypsum forming a rigid crystalline matrix. Because there is a small amount of gypsum, very little strength is developed and the rigidity can be overcome by mixing.

Measuring the Time of Setting

The time of setting of a cement paste does not correlate well with the time of setting of concrete because concretes have higher water-cement ratios. The effect of a high water-cement ratio is to increase the setting time. As a result, different tests have been established to measure the setting times of cement pastes and concretes. In general, the tests involve measuring the force required to penetrate a needle of given dimensions a certain distance into the material.

Shrinkage

Shrinkage refers to the reduction in volume which concrete undergoes at constant temperature and without the application of external loads. Investigators agree that shrinkage occurs as a result of the loss of water from the concrete. In the presence of an adequate supply of water, the inverse process of swelling might occur (Neville, 1981) although the volume change due to swelling is less than that for shrinkage. The term "plastic shrinkage" is used to describe the reduction in volume occurring while the concrete is in the fresh state while "drying shrinkage" is used when the concrete has hardened. In either case the shrinkage is associated with the loss of water from the concrete.

The shrinkage process starts at the surface exposed to drying and gradually penetrates into the interior of the concrete. Several factors affect the magnitude of the shrinkage which the concrete will experience. The relative humidity of the medium surrounding the concrete is the most important factor controlling water migration into and out of the concrete and hence the volume changes (Avram et al, 1981). The volume of aggregate as well as its stiffness play an important role in the shrinkage process. Because shrinkage occurs in the paste, the more the aggregate in the mix the less the shrinkage. In addition, because the aggregate restrains the shrinkage of the cement paste, the stiffer the aggregate the less the shrinkage. Other factors which have an effect on shrinkage are the initial water content or water-cement ratio, the cement content, and the specimen size and shape. The latter factor is easily overlooked but it can be very important. It is clear that thicker elements will lose water very slowly but in addition the shrinkage of a concrete member will depend on the ratio of its volume to the evaporating surface; as this ratio increases the shrinkage decreases.

ADMIXTURES

An admixture, as defined by ASTM C 125-79, is a material other than water, aggregates or hydraulic cement that is used as an ingredient of concrete or mortar and is added to the batch immediately before or during its mixing. Admixtures are used to modify the properties of concrete, for example, to improve workability, retard or accelerate strength development, or increase frost resistance. A given admixture might have more than one effect on the concrete. Thus, an admixture used to improve workability might also increase the strength and frost resistance of the concrete.

Admixtures can be broadly classified into the following groups:

(1) air entraining agents - added primarily to improve the frost resistance of concrete;

(2) chemical admixture - in concrete technology, this term is restricted to soluble substances, excluding air entraining agents, that are added to concrete with the purpose of controlling setting times and strength-related properties;

(3) mineral admixtures - these are finely divided materials added to concrete to improve its workability and durability or to provide additional cementing properties. Slags and pozzolanas are examples of mineral admixtures;

(4) miscellaneous - these admixtures have been developed for special purposes (grouting, corrosion inhibition, etc.) and are not included in one of the previous categories.

Chemical Admixtures

Chemical admixtures are classified into five groups: type A, water reducing; type B, retarding; type C, accelerating; type D, water reducing and retarding; and type E, water reducing and accelerating. Types A, B, and C will be treated briefly in the following paragraphs.

Type A, water reducing admixtures, as the name implies, reduces the amount of water required to achieve a given slump by about 5 to 15%. In practice these admixtures are used in three ways. Using the water reducing agent, the desired slump can be achieved by lowering the w/c ratio. This results in a general improvement in strength, impermeability, and durability. Alternatively, the desired slump may be achieved by reducing the cement content without any changes in the w/c ratio. This will result in economy since less cement, the most expensive ingredient in concrete, will be used. Finally, the water reducer may be used at any given w/c ratio, to improve the workability of a mix for ease of placement.

Type B, retarding admixtures, delay the setting of concrete through their action of slowing down the early hydration of C_3S. Retarders are commonly used in concreting operations in hot weather and also when delays are anticipated between mixing and placing. Such delays may result in early setting and losses of slump.

Type C, accelerating admixtures, hasten the normal processes of setting and strength development of concrete by acting in exactly the opposite way that retarders do. Accelerators are used in cold-weather concreting because they accelerate the rate of hydration of C_3S, thereby decreasing the period of time for which protection against damage by freezing is required. They are also used to speed construction by allowing earlier form removal.

Superplasticizers

A new class of water reducers, chemically different from the normal water reducers, can achieve water reductions of 15 to 30%. These compounds, variously known as superplasticizers, superfluidifiers, or high-range water reducers, were introduced in Japan in 1964 and later in Germany in 1972 (Ramachandran et al, 1981). They have been gaining widespread acceptance in recent years.

The development of superplasticizers has made possible: (1) the production of concrete with very high workability (7 to 9 in. slump) for difficult placements, using relatively normal mix proportions and without the occurrence of excessive segregation and bleeding; and (2) the production of high-strength concrete of normal workability because of the greatly reduced w/c ratio. The composition, properties and behavior of superplasticized concrete is beyond the scope of this report. For information on these aspects, the reader is referred to Malhotra et al, 1978.

CHAPTER 4. LATERAL PRESSURE OF FRESH CONCRETE

LITERATURE REVIEW

The knowledge of the magnitude and distribution of the lateral pressure of fresh concrete has been of interest to the construction industry for many years. This is due to the fact that the cost of formwork constitutes a large percentage of the total cost of the concrete structure. Therefore, any safe reduction in the amount of formwork is reflected in economy.

A large amount of research has been conducted to investigate the lateral pressure of fresh concrete. For the purpose of presenting and summarizing the research that has been done, it has been divided into two major areas: (a) theoretically oriented research, and (b) experimental research.

Theoretically Oriented Research

Theuer (1944), in a review of a paper by R. Hoffman written in 1943, presents the derivations that Hoffman made to arrive at an expression for the lateral pressure of concrete. By making the appropriate substitutions in the equations presented, the expression for the maximum lateral pressure of concrete is found to be:

$$S_{max} = \frac{\chi v \lambda_0}{ae} \tag{4.1}$$

where

S_{max} = maximum lateral pressure
χ = unit weight of mix
v = rate of placement
λ_0 = $\tan^2 (45 - \phi/2)$
ϕ = angle of internal friction
a = coefficient related to the time rate decrease of the lateral pressure
e = 2.718

In Eq. 4.1, λ_0 and a are considered to be constants which have to be determined experimentally. For a given set of conditions λ_0 is considered to depend on the nature of the aggregate, the water content, and the density of the mix, and a is dependent on the nature of the cement and the temperature. Hoffman performed several tests and found the limits (corresponding to before and after vibration) of 0.92 and 0.36 for λ_0, and 0.10 for a in both cases. Because for a given mix x, λ_0, and a are taken as constants, Hoffman's equation simply expresses a linear relationship between the maximum pressure of concrete and the rate of placement.

Rodin (1952) presented a rational explanation of the variation in form pressure based on the physical characteristics of concrete. His concept of concrete pressure against forms is presented in Fig. 4.1. According to Rodin, if concrete acted as a fluid, the lateral pressure at a given elevation would simply be the product of the head of concrete above that elevation times the unit weight of concrete as shown by curve I. He argued that because of arching phenomena alone, the pressure would increase according to curve II. Furthermore, since the shear strength and rigidity of the mortar were continuously increasing, dp/dh at a given elevation would decrease with an increasing head. The resulting pressure distribution corresponds to curve III which shows a maximum lateral pressure, P_m, reached at a depth, H_m, below the top of the concrete, and a decreasing pressure below. The decrease in pressure was explained as being caused by setting shrinkage, due either to bleeding and form leakage or to absorption of water by the aggregate and chemical changes (hydration of cement). If no shrinkage were to occur, the pressure distribution might have looked similar to curve IV. Therefore, whether the lateral pressure follows curve III or IV depends upon the presence or absence of excess water to replace that used in the hydration process. Rodin points out that curve IV might be typical of tremie placements where concrete is continuously saturated.

In addition to presenting his ideas on concrete-formwork pressure, Rodin gathered and reviewed in detail the available experimental data on the lateral pressure of concrete on forms. He discussed rate and method of placement, consistency and mix proportions, temperature of concrete, and size and shape of formwork as factors having an influence on the pressure developed by fresh concrete. From the analysis of the data, Rodin presented some empirical curves that give the maximum lateral pressure of concrete. The curves are expressed as:

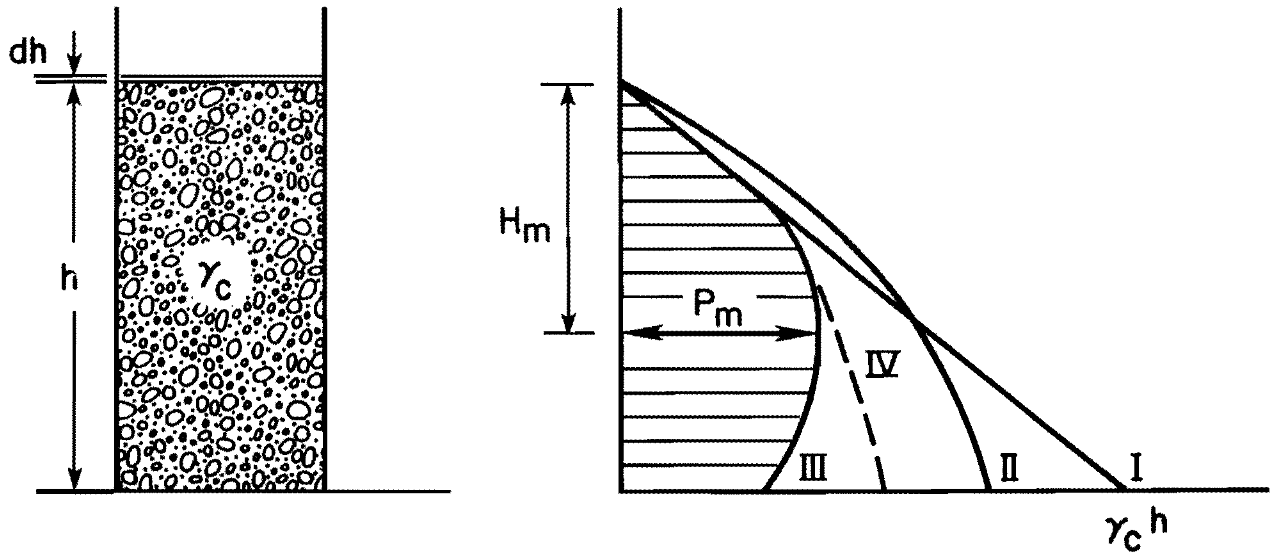


Fig. 4.1. Rodin's concept of concrete pressure against formwork.
(Rodin, 1952)

(a) for hand-spaded concrete

$$P_m = 110\ H_m \tag{4.2}$$

$$H_m = 3.6\ R^{1/3} \tag{4.3}$$

(b) for internally vibrated concrete

$$P_m = 540\ R^{1/3} \tag{4.4}$$

where

P_m = maximum concrete pressure, lb/sq ft
H_m = head of concrete at maximum pressure, ft
R = rate of pouring, ft/hr.

It must be pointed out that these equations represent curves which are approximate envelopes to the experimental data collected by Rodin. In addition, some correction factors were introduced to take into account the effects of mix proportions, and the slump and temperature of concrete. These are applied to the values calculated from the equations.

Starting with concepts concerning theories for each pressure, Schjodt (1955) mathematically developed a formula for the calculation of the pressure of concrete on formwork. He considered in his derivation those factors which he deemed important, including, among others, the rate of pouring, pore pressure, the depth of mix above the point in question, and the consistency, weight, internal friction and setting time of concrete. For the case of zero friction between the concrete and the forms, Schjodt derived the following equation for the maximum lateral pressure:

$$p_m = \left[\frac{\gamma\lambda}{2}\left(1 + \frac{h_1}{h_s}\right) + \frac{\gamma_0 K}{2}\left(1 - \lambda - \lambda\frac{h_1}{h_s}\right)\right]\left[h_1 + \frac{h_s}{2}\left(1 - \frac{h_1}{h_s} + \frac{K}{\lambda}\ \frac{\gamma_0}{\gamma + \gamma_0 K}\right)\right] \tag{4.5}$$

where

P_m = maximum lateral pressure
γ = unit weight of mix
γ_0 = unit weight of water
λ = $\tan^2(45 - \phi/2)$ = ratio of lateral to vertical pressure
ϕ = angle of internal friction of concrete at the start of test
h_1 = depth to which the effect of vibration reaches down
h_s = (rate of placement) x (setting time of concrete)
K = coefficient giving porewater pressure as a function of the height of concrete.

According to Schjödt, the friction angle ϕ for ordinary concrete ranges from 20° to 30° but may be even higher for dry concrete. For design purposes he recommends the conservative value of 20°. The values of h_1 vary with method of placing. For wall construction, a distance equal to the depth of the vibrator plus 2 ft is suggested. He finally states that for good concrete the porewater coefficient, K, will be in the range of 0.70 to 0.90. Schjödt also presented another equation applicable to the case where there was friction between the concrete and the forms.

The American Concrete Institute organized its Committee 622 in 1955 (later redesignated 347) with the objective of presenting a specification or code for recommended good practice in design of concrete formwork. The Committee made an extensive review of the existing literature and test reports concerning the lateral pressure of fresh concrete (from the early 1900's to about 1955). According to their evaluation of the data, the most important variables affecting the lateral pressure of concrete were the rate of placement, the temperature of the concrete mix, and the effect of vibration. These were combined in the following equation as applied to walls (with rates of placement smaller than 7 ft/hr) and column forms:

$$p = 150 + 9000(R/T) \tag{4.6}$$

[max of 3000 lb/sq ft for columns; 2000 lb/sq ft for walls].

where

p = lateral pressure, lb/sq ft
R = rate of placement, ft/hr
T = temperature of concrete mix, °F

The effect of normal vibration was included in the coefficient preceding the term R/T. The proposed equation assumed a concrete unit weight of 150 lb/cu ft, a slump of 4 in. or less, and little or no revibration of previously placed layers. The Committee further placed an arbitrary upper limit of 10 ft/hr on the rate of placement due to uncertainties about the pressures generated at the higher rates of placement. The report cautioned, however, that the use of slower setting cements, set-retarding admixtures, and sand-rich mixtures could result in higher lateral pressures because the concrete would remain fluid for a longer period of time.

In the latest revision by the American Concrete Institute (1981), an additional upper limit of 150 x h for the maximum lateral pressure of concrete was added. In the expression h represents the total height of the form. The revision responds to the fact that for short columns and high rates of placement the design equation gives pressures which are higher than the hydrostatic value for those particular conditions.

Levitsky (1973) presented an analytical method for determining the vertical distribution of form pressure. He argued that whenever a given height of concrete was poured, before hardening occurred, an approximate hydrostatic condition would develop in the mass of fresh concrete. However, when the rates of pouring were sufficiently slow, the bottom layers would begin to harden before the end of pouring and consequently they would resist the redistribution of additional weight into lateral pressure. Based on these ideas, he formulated the problem and derived a solution using a model for the hardening process and working within the framework of the theory of elasticity. Poured concrete was considered to be an elastic medium with the stresses, strains, and elastic coefficients being dependent on time and position. The hardening function selected was dependent upon the composition of the concrete mix. The rate of placement and strains due to volumetric changes were considered important variables while the internal friction within the concrete, pore pressure, yielding of the form, and other effects were considered to have a secondary influence. Levitsky compared results from his analytical solution with the experimental values obtained by Ritchie in his investigation and found good qualitative agreement between the two.

Olsen et al (1974) presented an equation based on the Rankine earth pressure theory for determining the lateral pressure of concrete on formwork. Rankine's equation was modified to take into account the lateral strain in the wet concrete. Three hundred triaxial tests were performed to evaluate the shear strength of concrete as a function of the set time. These tests, however, were for only one particular mix. A good correlation was found when the pressures predicted by the new method were compared to those calculated using the formulas recommended by the American Concrete Institute and the Civil Industries and Research Information Association (CIRIA).

Experimental Investigations

Rodin (1952) presented a summary of the experimental investigations that were performed prior to the 1950's concerning the lateral pressure of concrete. From his review of the published experimental data, he concluded that in many cases, because of the techniques used in measuring the pressures, the accuracy of the results was unreliable.

Ritchie (1962) conducted a series of laboratory tests under controlled conditions to investigate the influence of the rate of pour, formwork details and workability of the mix on the lateral pressure of concrete. Two mixes were used in the investigation with cement to aggregate ratios of 1:3 and 1:6 by weight. For each mix, low and high workabilities were used as measured by the compacting-factor test (corresponding slump values were less than 4 in.). The average temperature of the concrete for the tests was 68° F. The pouring rates varied from 10 to 70 ft/hr and the concrete was vibrated in the upper 12 inches. The formwork used in the tests was made of heavy timber and consisted of an 8 ft high column of section 6 in. x 6 inches.

The importance of the size of the formwork and the workability of the mix on the lateral pressures were shown in Ritchie's investigation. Although most of the tests were conducted with a form 6 in. x 6 in. in section, Ritchie made some measurements on forms 10 in. x 10 in. in section and in an 8 ft long by 2 ft wide section. As shown in Table 4.1, for approximately the same compacting factor and rate of pour, maximum pressures of 1.9, 2.4, and 7.0 lb/sq in. were measured for the 6 in. x 6 in., 10 in. x 10 in., and 2 ft x 8 ft sections, respectively. Ritchie argued that below a certain limiting size or in restricted formwork, there would be a falling off in the maximum pressure due

TABLE 4.1. EFFECT OF FORMWORK DIMENSIONS ON PRESSURES OF CONCRETE (Ritchie, 1962)

Mix	Size of Formwork (in.)	Water/Cement Ratio	Compacting Factor	Rate of Pour (ft/hr)	Maximum Pressure (lb/sq in.)
1:6	6 x 6	0.61	0.88	10	1.9
1:6	10 x 10	0.61	0.89	10	2.4
1:2:4	24 x 96	0.65	0.89	12	7.0

to arching in the concrete. Regarding the workability of the mix, his results consistently showed higher pressures with the 1:3 mix as compared to the 1:6 mix. He explained this difference, by referring to the workability. According to Ritchie, the true workability of the mix can be subdivided into the properties of compactability and mobility. The first one he refers to as the property of relative density, which is measured by the compacting-factor test. The mobility, on the other hand, is the characteristic which has the predominant effect on the build-up of pressure and is related to the angle of internal friction of the mix. The workability characteristics of the mixes used by Ritchie are shown in Table 4.2. According to Ritchie, for a given compacting factor, richer mixes would have a lower angle of internal friction than leaner ones because the particles would have more freedom to translate and rotate within the matrix. This lower resistance to internal deformation is reflected in a higher pressure for the same head of concrete.

CIRIA (Civil Industries Research and Information Association, formerly CERA, Civil Engineering Research Association) published a formwork design procedure in 1965, based on the results of a field investigation of the lateral pressures of fresh concrete. The research was sponsored by CIRIA and undertaken by the Cement and Concrete Association. The objective of the research was to measure the lateral pressures of concrete under actual site conditions. Because of this fact, it would have been impossible for the researchers to control all the variables which affect the pressure. The approach used by the investigators was to perform a large number of tests to accommodate all the influencing factors and to concentrate on sites which offered extreme conditions. In all, over 200 pressure measurements were performed. A wide range of conditions was covered by the tests as demonstrated by the following variations that were recorded: rates of placement from 1 to 120 ft/hr, minimum form dimensions between 5 in. and 8 ft, cement to aggregate ratios from 1:3 to 1:7, concrete temperatures from 38°F to over 90°F, and slumps between 0 and 6 in. In addition, heights of lifts were normally smaller than 10 ft, vibration was used, and no additives were added to the concrete.

The results showed that fresh concrete behaved almost as a liquid under the influence of vibration. Moreover, according to the report, this hydrostatic condition and, therefore, the maximum pressure from the concrete, was limited by one of two mechanisms; stiffening of the concrete and/or arching due to friction. Accordingly, CIRIA established two criteria for calculating the lat-

TABLE 4.2. WORKABILITY CHARACTERISTICS OF MIXES USED IN RITCHIE'S INVESTIGATION (Ritchie, 1962)

Mix	Work-ability	Compacting Factor	Slump (in.)	Vebe Time (secs)	Angle of Internal Friction, ϕ (degrees)	$\lambda = \frac{1 - \sin\phi}{1 + \sin\phi}$
1:3	H	0.93	3	2.5	10	0.70
	L	0.82	1½	4.0	14	0.61
1:6	H	0.95	3	2.5	28	0.36
	L	0.88	1	6.0	30	0.33

eral pressure of concrete. The limiting pressure according to the stiffening criterion was given by the equation:

$$p = \frac{\Delta Rt}{1 + C(t/t_{max})^4} + 12(8-R) \tag{4.7}$$

where

p = lateral pressure, lb/sq ft
Δ = density of concrete, lb/cu ft
R = rate of placement, ft/hr
t = time after start of pour, hr
C = vibration parameter, a function of the workability of concrete and the continuity of vibration
t_{max} = stiffening time, a function of slump and temperature of concrete.

CIRIA provided empirically-derived charts to obtain the values of C and t_{max} (Fig. 4.2). In the second criterion, the limiting pressure due to arching was calculated by the equation:

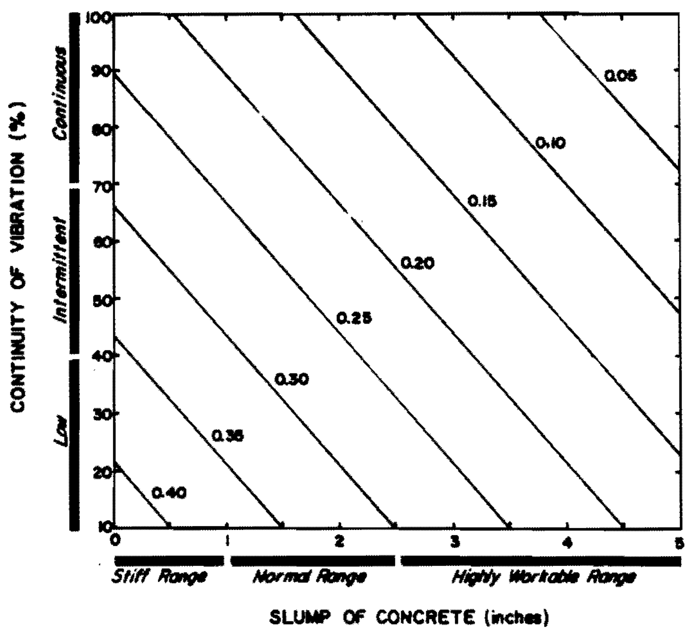
$$p = 30 + 50d + 20R \tag{4.8}$$

where

p = lateral pressure, lb/sq ft
d = minimum form dimension, in.
R = rate of placement, ft/hr.

This equation was restricted to sections with minimum dimensions smaller than 18 inches.

Peurifoy (1965) performed a series of laboratory tests to measure the lateral pressure of concrete using 16 in. inside diameter steel forms. His experimental setup allowed the measurement of the maximum lateral pressure but not its distribution with depth. The concrete mix was designed to give a 28-day compressive strength of not less than 3000 lb/sq in. and a slump of 5 to 6 in. Portland cement type I, river sand and crushed stone, 1-1/2 in. maximum size,



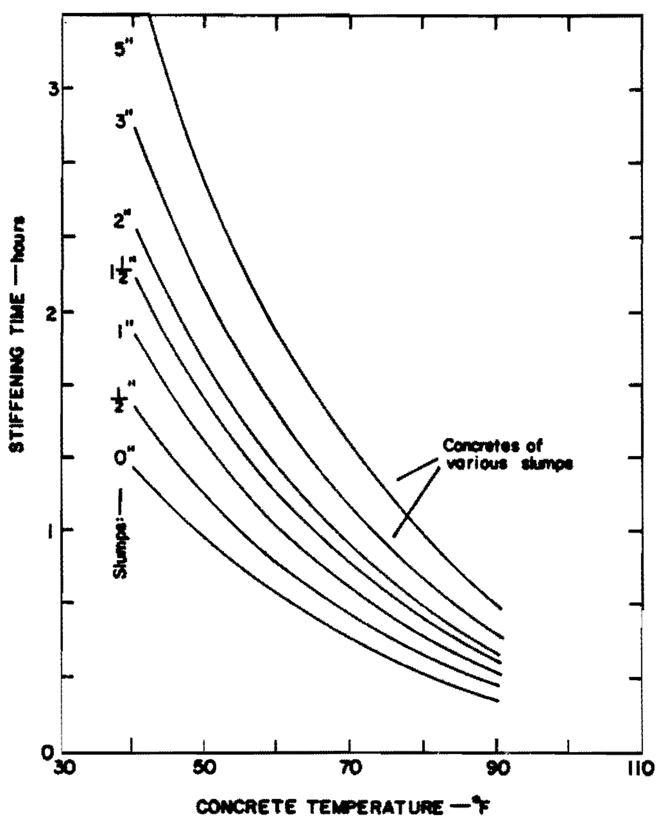


Fig. 4.2. Parameters C and t_{max} for use in CIRIA's equation. (CIRIA, 1965)

were mixed on site in proportions of 1:2.5 and 3.6, respectively. The data collected were for rates of placement of less than 10 ft/hr, and in a temperature range of 60 to 85°F. His results showed good agreement with the pressures calculated according to the ACI equation. Peurifoy also demonstrated that, for a given rate of placement and temperature, the pressure at a point increased with the height of concrete above the point, and that in general the maximum pressure increased with increasing rate of placement and with decreasing temperature.

Gardner and Ho (1979) conducted a laboratory study aimed at determining the effects on the lateral pressure of concrete of the rate of pour, strength of concrete, slump and consistency of concrete, formwork size, and movement of formwork. The experimental setup consisted of a steel open-ended box, 15 ft deep, 3 ft long and with a thickness that could be varied from 6 to 18 inches. Pressure cells were attached at different elevations along the length of the form so that the distribution of pressure with depth could be determined. The concrete used was of normal weight made with type I cement and supplied by a ready-mix company. The rates of pouring were 20 and 150 ft/hr, and the slumps were between 2 and 6 1/2 inches. Two maximum sizes of aggregates were used, 3/4 and 3/8 in., and the temperature of the concrete varied from 55 to 72°F. Based on the results, Gardner and Ho concluded that the maximum lateral pressure of concrete increased with increasing rate of placement, with slump, and with minimum form dimension. Regarding the effect of aggregate size and concrete strength, they concluded that these had no significant influence on the lateral pressure of concrete.

Gardner and Quereshi (1979) studied the effect of internal vibration on the lateral pressure of fresh concrete. The rate of pouring, the form dimensions, and the slump were kept constant while the temperature was measured and the vibration parameters were varied (power of vibration, depth of immersion, duration). From the data collected, they concluded that the lateral pressure was dependent upon the vibration parameters and that pressure varied inversely with the temperature of the mix. In a follow-up investigation, Gardner (1980) made a series of tests in which he standardized the vibration parameters, measured temperature, and varied the rate of pour, form dimensions and slump. His results showed that the maximum lateral pressure increased when the rate of placement, slump, and minimum form dimensions were increased, and decreased with an increase in concrete temperature. Based on his results, Gardner proposed the following equation to calculate the maximum lateral pressure:

$$p_m = 153\ h_i + \frac{2467\ HP}{d} + 13.26d + \frac{8305\ R^{1/2}}{T} + 53\ (\text{slump}-3)$$

or

$$p_m \leq 153h$$

where

p_m = maximum lateral concrete pressure, lb/sq ft
h_i = immersed depth of vibrator, ft
HP = power of vibrator
d = minimum form dimension, in.
R = rate of placement, ft/hr
T = temperature of mix, °F
h = total height of form, ft
Slump is given in inches.

It is worth noting that in this investigation Gardner made three tests with superplasticized concrete, and two with high-slump concrete (7 1/2 inches). For these tests he found maximum pressures which ranged between 65 and 85 percent of the full hydrostatic value.

Douglas et al (1981) measured the pressures developed by concrete during the construction of the walls for a large rectangular tank. The wall section, which was 20 in. wide at the base, tapering linearly to 13 in. at the top, was built using 26 ft x 20 ft steel-plywood forms that were reused. The measurements were done for rates of placement that varied from 9 to 18 ft/hr, using slumps of 3 ± 1 in., and for concrete temperatures of 74 ± 2°F. Their analysis of the data indicated peak pressures on the order of 40 to 50 percent of the equivalent hydrostatic value for rates of placement between 10 and 13 ft/hr. In another test with a rate of placement of 18 ft/hr, the observed maximum pressure was substantially higher than for the smaller rates of placement.

EVALUATION OF PREVIOUS RESEARCH

According to ACI Committee 622, prior to the 1950's, the control of concrete was not well defined producing a lack of uniformity in batching, mixing, and placing of concrete. The lack of control led to variations in the experimental measurements which were not taken into account in the anlysis of the results. In addition, as Rodin points out, some of the techniques of measurement used in these investigations were not very accurate and some doubt is cast upon the reliability of the results. However, the work done by the earlier investigators gives a general idea of the variations in pressure produced by changes in the rate of placement, slump, mix proportions, and temperature of concrete. These general variations are presented in Rodin's paper.

After the 1960's, better investigations concerning the lateral pressure of concrete were performed. The improvements were twofold: more uniform practices and a better understanding of the concrete behavior helped to exert a better control over the uniformity of concrete, and secondly, improved instrumentation systems were developed and used. These investigations were conducted using structural concrete and were aimed at providing guidance in formwork design. As a result, most of the data obtained pertains to rates of placement smaller than 20 ft/hr, vibrated concrete, and slumps less than 4 in. (except Peurifoy's investigation). The results (Ritchie, 1962; Gardner and Ho, 1979) seem to indicate that the shape of the concrete pressure diagram consists of a hydrostatic part, from the concrete surface to a certain depth, and decreasing from there on. In regard to the magnitude of the maximum lateral pressure, there is a lack of agreement as to its value and several equations have been proposed to determine it.

From the literature review it is apparent that there are numerous factors which affect the lateral pressure of concrete. It seems, however, that these factors can be divided into major variables and variables of unknown, probably minor, effect. Among the major factors are the rate of placement, the consistency and mix proportions, temperature of the concrete, minimum form dimension, vibration, and weight of concrete. On the other hand, variables of unknown effect include the type of concrete, use of admixtures, amount and arrangement of reinforcement (particularly in columns), the porewater pressure, maximum size of aggregate and yielding of the forms.

As a final comment, it must be pointed out that some of the equations that were presented, for example those of ACI and CIRIA, were derived as envelopes to the experimental data available to that particular investigation. They are intended for use as criteria for designing of formwork and not to predict the maximum lateral pressure of concrete.

OTHER MEASUREMENTS

Other measurements of the lateral pressure of concrete have been reported in the literature in relation to the construction of slurry walls. DiBiagio and Roti (1972) made observations of the pressure of concrete during the construction of a 65 ft deep, 3.28 ft wide slurry wall in soft clay. The measurements showed hydrostatic conditions in the upper 20 ft of the wall. At deeper locations the pressures were smaller than the hydrostatic pressure. The maximum pressure measured was around 6100 lb/sq feet. Uriel and Oteo (1977) made similar measurements during the construction of a circular trench wall. Hydrostatic pressures were measured in the upper 30 to 35 ft of the wall while the maximum total pressure was about 8300 lb/sq ft at a depth of about 95 feet. In both cases no information was given regarding the characteristics of concrete or the details of the concreting operation. It is assumed that 7 to 8 in. slump was used in both cases as this is the desirable slump range for slurry wall construction (Xanthakos, 1979).

A third case, referenced previously in Chapter 2, was presented by Reynaud and Riviere (1981). They measured the pressure of concrete during the construction of a cast-in-place wall made up of panels 3.9 ft thick, 23.6 ft long, and at least 82 ft deep. Hydrostatic conditions were developed in the upper 25 or 30 ft of the trench. The maximum pressure measured was about 6200 lb/sq feet. The slump of the concrete was measured as 7.9 in. while the rate of placement was 20 ft/hour.

CHAPTER 5. EXPERIMENTAL PROGRAM

INTRODUCTION

The literature review conducted has shown that there are numerous factors that affect the lateral pressure developed by fresh concrete. In any given situation, the maximum lateral pressure and its distribution with depth will depend on the interaction among these numerous factors.

Research studying the lateral pressure of concrete has been conducted under conditions typically encountered in the construction industry. These investigations have established the influence of such major variables as the rate of placement, consistency, mix proportions, temperature, vibration, and minimum form dimension. The influence of other variables such as reinforcement, type of cement, and use of admixtures, among others, is either minor or unknown.

The conditions existing during the construction of drilled shafts differ greatly from those under which the previous investigations were conducted. For example, rate of placement in the construction of drilled shafts may vary between 30 and 100 ft/hr, slumps are in the range of 4 to 9 in., and shaft diameters are seldom smaller than 24 inches. It is therefore obvious that the results and conclusions drawn from previous investigations do not necessarily apply to the case of drilled shafts and that some research which addresses the particular conditions existing during the construction of drilled shafts is necessary.

PROPOSED TESTING SCHEME

The objective of this study, is to determine the lateral pressure developed by typical concrete used for drilled shafts by the Texas State Department of Highways and Public Transportation (SDHPT). Theory and experience indicate that the ability of a drilled shaft to sustain axial load is strongly related to the lateral stress that exists at the interface of the concrete and the soil. The lateral stress at concrete placement will undoubtedly be higher for a high-slump concrete than for one with a low slump. However, the important lateral stress is that which exists after the concrete has hardened. While it is recognized that expansion or shrinkage effects might be important, these effects will not be treated in this study.

Several schemes were considered in the early phases of the project for the design of a test facility and for the design of a system for measuring the lateral stress of the fresh concrete. After some considerations it was thought advisable to measure the pressure distribution along the shaft rather than only the pressure at the bottom. In this way several measurements would be available in case the instrumentation at the bottom failed. The instrumentation used will be discussed in detail in Chapter 6. Houston was selected as the site for the testing system because it was found that the cost of equipment rental could be greatly reduced by performing the test in a construction yard. With this knowledge, contact was made with the president of Farmer Foundation Company and an agreement was reached which provided this study with heavy construction equipment, personnel, and the Farmer construction yard in Houston for the performance of the tests. Furthermore, the use of a possible site at the Balcones Research Center (BRC) was ruled out because of the extensive construction that was underway or planned at BRC.

DISCUSSION AND SELECTION OF PARAMETERS FOR THIS STUDY

Considering the multiplicity of factors influencing the lateral pressure of concrete, it is clear that any experimental investigation must be limited in scope to study the effects of those variables which seem to be the most important for the particular construction conditions. The variables influencing the lateral pressure of concrete will be discussed individually and the values of standards to be used in this investigation will be presented. Emphasis will be placed on the conditions existing during the construction of a drilled shaft. Where applicable, comments will be made concerning the standard practices of the SDHPT.

Rate of Placement

Investigators agree that this variable has a major influence on the lateral pressures developed by concrete. In drilled-shaft construction, the rates of placement can vary within a wide margin depending on the method of construction and other factors. For example, when drilling in stable formations above the water table, it is very easy to discharge the concrete truck directly in the hole and attain a very high casting rate. A concrete pump can also be used to obtain a high casting rate. On the other hand, when the tremie method is used, the truck discharges into a bucket which is raised to the top of the tremie, emptied and then brought down for another load of concrete. This considerably lowers the rate of casting. From data supplied by the SDHPT, from a review of previous studies, and from conversations with contractors it seems that reasonable limits for the pouring rate of concrete in drilled shafts are between 30 to 100 ft/hour. A rate of placement of between 30 and 40 ft/hr was selected for the experimental program.

Workability (slump)

Slump of concrete was not considered an important variable in some of the previous investigations for two reasons: (1) structural concrete typically had slumps of 4 in. or less, and (2) the use of mechanical vibrators, common in structural concrete, supposedly minimized the effects of slump. It should be intuitively obvious, however, that in the absence of vibration, the higher the slump of the concrete, the higher the lateral pressures produced. The present specifications of the SDHPT for concrete used in drilled shafts allow a range in slump of 4 to 7 in. depending upon the method of construction. Slumps of 4 to 5 in. and 7 in. as representatives of the extremes in slump conditions were tentatively selected for the experimental program with consideration given to the use of an even higher slump if early results so indicated.

Mix Proportions

As shown by Ritchie (1962), richer mixes have lower resistances to internal deformation than do lean mixes, and therefore, generate higher lateral pressures. The concrete specifications used by the SDHPT lead to mix proportions of cement, sand, and gravel which are approximately in the ratio of 1:2:3 1/2, respectively, by weight. A similar mix was used in this investigation.

Temperature of Concrete

Supposedly, the temperature of concrete will affect the rate of setting and therefore, influence the pressure developed. Limited experimental data available (Rodin, 1952; Peurifoy, 1965) indicate that within the range of 70 to 90°F the effects of temperature are approximately a change in ±100 psf/10°F. According to SDHPT specifications, concrete poured in drilled shafts should have a temperature lower than 85°F. For the purpose of this research, it was desirable to maintain the temperature of the concrete within the range of 75 to 85°F. Given the relatively small amount of change in pressure with temperature and the problems associated with maintaining a given temperature the approach used was to measure the temperature of concrete and make appropriate corrections to the measured pressures.

Minimum Form Dimension

Previous research has shown that as the minimum form dimension increases the measured lateral pressure increases (CIRIA, 1965; Gardner and Ho, 1979). It seems that in narrow sections, the frictional forces developed between the concrete and the form are capable of supporting a certain load of fresh concrete. As the section widens, the frictional forces become smaller relative to the mass of the concrete. The development of arching effects however, is dependent not only on the minimum form dimension but also on the mix characteristics, i.e., mix proportions and consistency. Thus, for a given form dimension, lean and stiff mixes lead to an increase in arching effects and therefore smaller lateral pressures, and vice versa.

In this investigation circular steel formwork, 42 in. in diameter, was used for two reasons. First, as was mentioned previously, drilled shafts are typically constructed with diameters larger than 24 in., so a formwork with a minimum dimension of 24 in. was desired, and second, this circular formwork was made available by Farmer Foundation Company at no cost to the investigation.

Vibration

Vibration of the concrete is sometimes performed in the upper 10 ft of the shaft. However, it is not a common practice and thus was not used in this investigation.

Unit Weight of Concrete

Research has shown (Ritchie, 1962; Gardner and Ho, 1979) that during the early stages of pouring, concrete behaves in a hydrostatic manner. Therefore, the lateral pressure developed is directly related to the unit weight of the mix. Normal weight concrete (about 145 lb/cu ft) was used.

Reinforcement

The amount and arrangement of the reinforcement seems to be an important variable which affects the magnitude of the pressure developed by concrete. No research has been conducted to determine its influence. No reinforcement was used in the tests.

Admixtures

No research has been conducted specifically to investigate the effects of admixtures on the lateral pressure of concrete. Gardner (1980) reports the results of three tests in which superplasticized concrete was used. He found an increase in pressures over non-plasticized concrete. In mixes designed for the SDHPT jobs, retarding admixtures are typically used.

Type of Cement

ASTM Type I Portland cement was used.

Time

The time variable is directly related to the gain in shear strength of the concrete. Time can be measured from the start of a placement, or from the start of mixing. The second alternative seems a more logical approach because the chemical reactions which lead to stiffening of the mix start from the moment that water is mixed with the cement. In relation to the time variable, the SDHPT has some requirements, concerning the time interval between the addition of cement to the batch and the placing of concrete in the forms. For example, for agitated concrete at a temperature of 85°F, the maximum time interval allowed is 60 minutes. If the shaft is cased and a set retarding admixture is used, this limit can be increased by 30 minutes. Thus, in this investigation a maximum time interval of 90 minutes was allowed between the start of mixing and the placing of concrete in the forms.

SUMMARY

From the previous discussion of the parameters that influence the lateral pressure of concrete, it has been shown that for this study most of them are fixed either by typical practices in the construction of a drilled shaft or by the standards used by the SDHPT. Thus, the study centered on the effect of consistency on the lateral pressure of concrete. The consistency was measured by the slump test, as it is the method used in practice.

A summary of the information discussed in the previous paragraphs is presented in Table 5.1 while the proposed testing arrangement is shown in schematic form in Fig. 5.1.

TABLE 5.1. SUMMARY OF TESTING VARIABLES

Variable	Previous Investigations (after 1950's)	Practice in Drilled Shaft Construction	This Study (tentative)	SDHPT
Rate of placement	20 ft/hr	very variable - depending on method of construction. Range: 25 ft/hr - +100 ft/hr	30 - 40 ft/hr	-
Slump	typically 4 inches	-	4 - 7 inches	4 - 7 inches
Mix Proportions	1:3, 1:6	variable	1:2:3½	1:2:3½
Temperature	range of 60 - 90°F	-	75 - 85°F	85°F
Minimum Form Dimension	6 in. up to 18 in.	typically not less than 24 in.	42 inches	variable
Vibration	most commonly used	maybe upper 10 or 15 ft of shaft, though not common	no vibration	no vibration
Unit Weight	normal weight	normal weight	normal weight	normal weight
Reinforcement	none	typically reinforced	no reinforcement	-
Admixtures	very little	set retarders, water reducers, superplasticizers	set retarder	usually set retarder
Type of Cement	Type I	Type I	Type I	Type I
Time	-	-	-	-

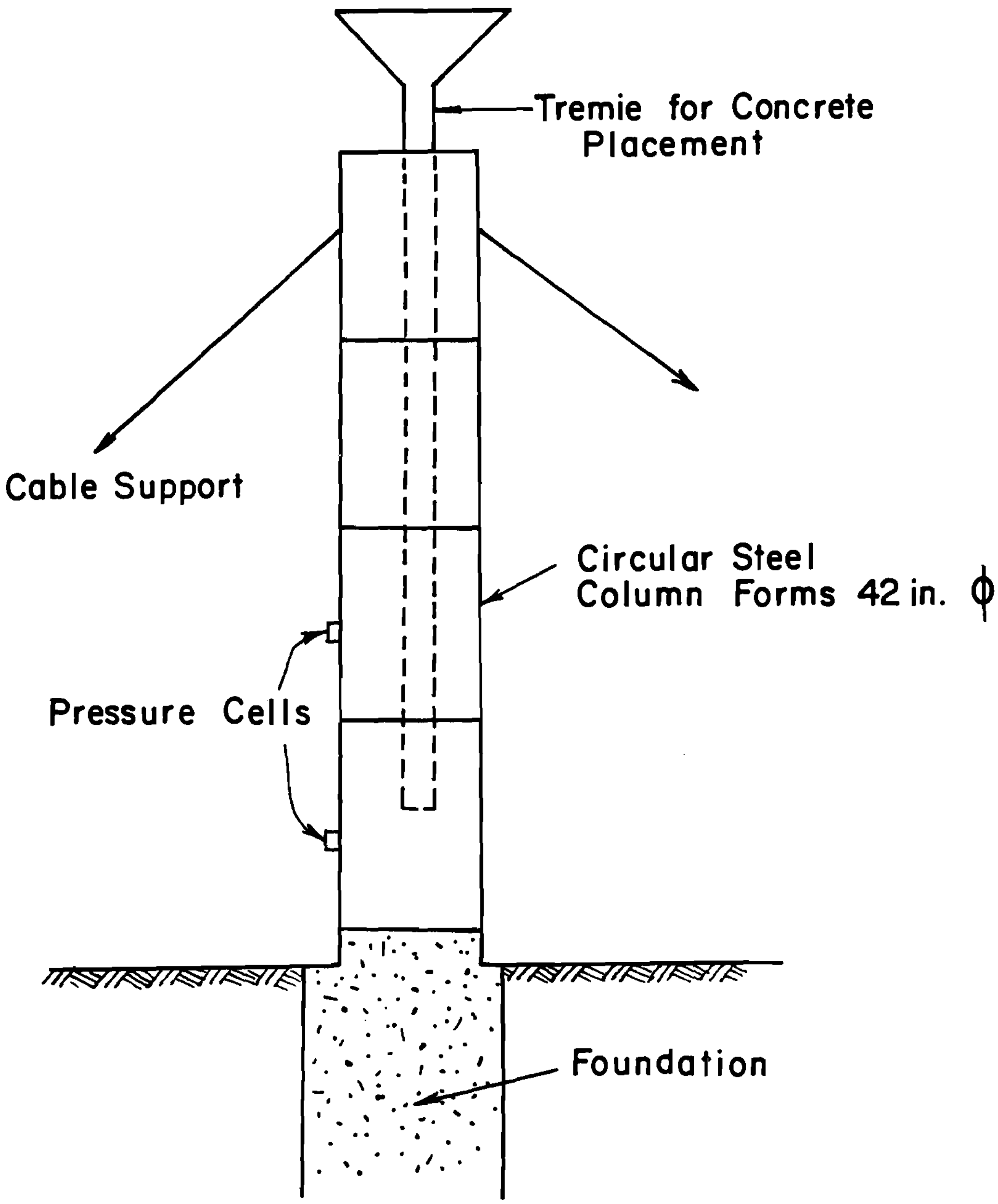


Fig. 5.1. Proposed test arrangement.

CHAPTER 6. INSTRUMENTATION

SCOPE

The purposes of this study are to measure the lateral pressure of fresh concrete used in drilled shaft projects and to relate such measurements to behavior of drilled shafts. Therefore, it is essential to have a reliable and accurate system for making such measurements. Fresh concrete is neither fluid nor a purely granular material but it has characteristics of both. Furthermore, the properties and behavior of a fresh mass of concrete vary according to its consistency and, in addition, with time. This dual nature of concrete was considered in designing the system to measure the pressure.

The aim of this chapter is to present the work done in order to arrive at an instrumentation system. The first step was to review the techniques used by previous investigators. Design and calibration of a pressure-measuring system was then accomplished as described in the following sections.

REVIEW OF METHODS PREVIOUSLY USED

Rodin (1952) described different techniques used by researchers before the 1950's to measure the lateral pressure of concrete. According to Rodin, Roby measured the deflection of a 7/16 in. steel plate, 6 in. wide, extending the full width of the form and resting in knife edges 28 in. apart. Shunk measured the load required to prevent a piston, fitted into a 9 1/4 in. diameter cylinder, from moving under the applied concrete pressure. Other investigators measured the strains in steel bars holding the formwork and from these were able to calculate the pressure of concreting. Still others used some kind of pressure cell. Rodin reports that Halloran and Talbot used a pressure cell of the type designed by the Waterways Experiment Station while Smith, Slater and Goldbeck, and Teller all used Goldbeck-type pressure cells. Rodin finally concludes that in many cases the methods used to measure the pressure were of uncertain accuracy.

Ritchie (1962) used a pressure cell of the deflecting-diaphragm type that had a sensitive diameter of 2 in. and a diaphragm thickness of 0.015 in.

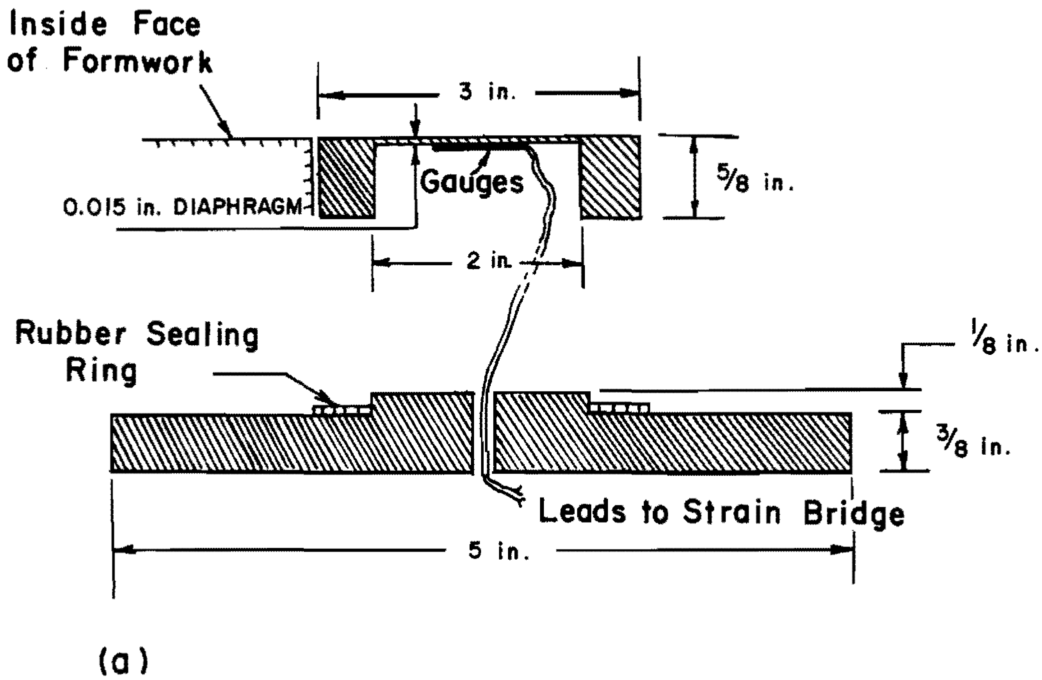
machined from mild steel (see Fig. 6.1). The strain in the diaphragm was picked up by standard, fine-wire electrical-resistance strain gauges. According to Ritchie, the deflection of the diaphragm under the maximum recorded pressure (5.2 lb/sq in.) was 0.0088 inch. Because of the small deflection, Ritchie assumed that the action of the gauge had no artificial influence on the build up of pressure within the formwork.

CIRIA (1965) designed and constructed for their investigation what they named the "formwork pressure balance" instrument. This is simply a piston and cylinder device (Fig. 6.2). The piston is housed in a chamber or cylinder and is connected by a rod to a 6 in. diameter, 3/8 in. thick, mild steel plate. Concrete pressure acts against this plate which is the only part of the cell in contact with concrete. Small displacements of the piston are detected by sensitive microswitches (0.0005 in. sensitivity) so that air pressure within the cylinder can be increased to balance the thrust of the concrete.

Peurifoy (1965) used 16 in.-inside-diameter steel forms in his laboratory experiments. The instrumented section, located 12 in. from the bottom of the column, consisted of two semicylinders 12 in. high, held together by two 3/8 in.-diameter steel bolts. The bolts were instrumented with strain gauges so that the loads on the bolts could be measured to allow pressures on the test section to be calculated. According to Peurifoy, several precautions were taken to insure that the test section was without any restraint or binding so that the pressures produced by the concrete were resisted entirely by the bolts.

Gardner and Quereshi (1979) made measurements using an earth pressure cell developed by Arthur and Roscoe at Cambridge University. The cell was machined from a solid piece of heat-treated, high-grade aluminum alloy. It is designed in such a way that the active face of the cell is supported by thin webs which are instrumented by strain gauges. An interesting feature of this cell is that it can measure both normal and shear stresses. Gardner and Quereshi attached circular, plexiglass face-plates to transmit the concrete pressure to the cells.

Douglas et al (1981) measured the pressures produced on concrete-wall forms on an actual construction project. The form ties were tapered steel rods having a minimum diameter of 1 1/4 in. and instrumented with a pair of foil electric-resistance strain gauges applied on opposite sides of the bar. Temperature-compensating gauges were attached to a small cylindrical sleeve which was positioned in the same location as the active gauges. From the measure-



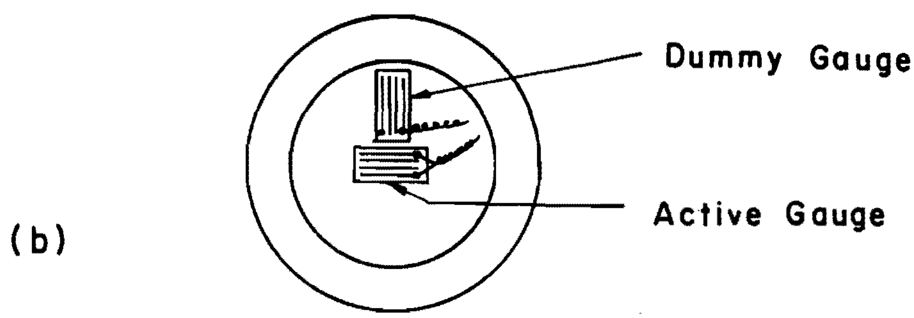


Fig. 6.1. Pressure cell used by Ritchie (Ritchie, 1962).
(a) Cross section of cell
(b) Inside face of diaphragm

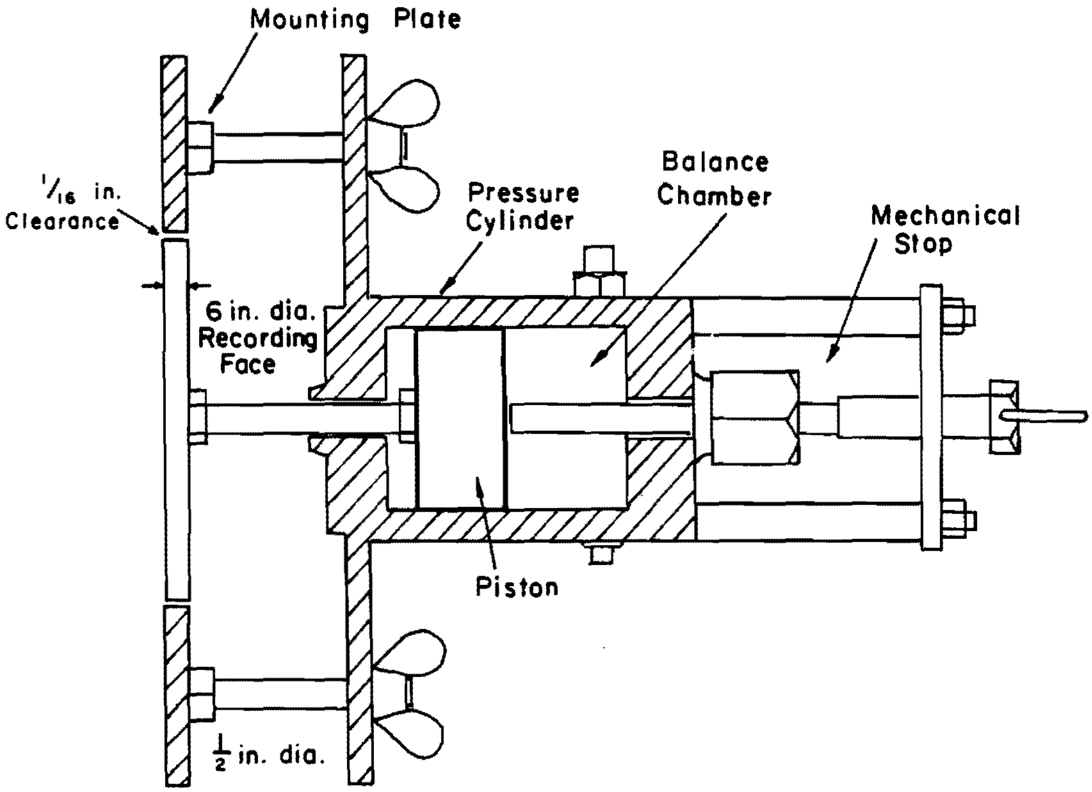


Fig. 6.2. Formwork pressure balance. Microswitches that detect piston movement are not shown. (CIRIA, 1965).

ments of the tie-rod tensions and with an assumption regarding the pressure distribution, they were able to calculate the pressure on the forms.

In summary, the techniques which have been described in the previous paragraphs can be classified as direct or indirect. Peurifoy, and Douglas et al, used indirect measurements to determine the pressure of concrete. The indirect methods seem to require a more complicated scheme and, in addition, an assumption must be made regarding the distribution of the lateral pressure. On the other hand, the techniques used by Ritchie, CIRIA, and Gardner and Quereshi seem to be more straightforward and give a direct determination of the lateral pressure of concrete.

SELECTION OF PRESSURE CELL

At the start of the project several alternatives were considered for measuring the pressure of concrete. One of the ideas was to design a system similar to that used by Peurifoy. The bottom section of a pipe would be split in two halves and held together either by bolts or some kind of steel strap which in turn would be instrumented. After considering the type and size of formwork to be used, it was realized that a simpler system was desirable. Thus, it was concluded that a pressure cell of the diaphragm type was the most practical. These pressure cells are simple to operate and should be easy to install. Furthermore, several pressure cells can be used so that the pressure distribution along the formwork can be determined. Also, if any of the cells do not work properly, data from the other cells would be sufficient for analysis.

DIAPHRAGM-TYPE PRESSURE CELL: DESIGN CONCEPT

A pressure cell of the diaphragm type consists of a thin, circular plate which is fixed or clamped at its edges. Timoshenko (1941) used the theory of elasticity to describe the behavior of the diaphragm. The resulting equations define stress, strain, and displacement and are presented in Appendix A, along with the assumptions made in the derivations.

The diaphragm movement under pressure can be sensed in a variety of ways. In the case of relatively large movements, mechanical instruments can detect the displacement. More commonly, however, strain gauges applied on the inside face of the diaphragm are used to detect the deflections of the thin diaphragm.

Two modes of operating the cell are possible. Pressure can be applied to the face of the diaphragm and the output from the strain-gauge circuit noted.

In this manner a calibration curve can be obtained and the system is referred to as a freely-deflecting diaphragm. The second alternative to operate the system involves the application of an internal pressure within the cell so that the strain gauge circuit remains in balance under the applied external pressure. In this method, the strain gauge serves to indicate the original position of the diaphragm. The internal cell pressure is then used as a measure of the applied pressure. This is referred to as a null-balance system.

Each mode of operation has its advantages and disadvantages. The deflecting diaphragm is simple to operate because only the strain readings from the gauge circuit must be monitored during a test. The null-balance system is more complicated because an additional system is required to apply and measure the pressure required to maintain the diaphragm in the null position. A second aspect which must be considered in comparing both methods of operation is the phenomenon referred to as "arching." Terzaghi (1943) describes arching as the action by which pressure is transferred from a yielding mass of soil onto adjacent stationary parts. The yielding causes the development of shearing resistance within the mass of soil and thereby reduces the pressure upon the yielding part. This phenomenon is relevant to the design of the pressure cell. In the case of the null-balance system, an attempt is made to keep the diaphragm from deflecting, thus minimizing any problems related to arching. The other system, however, allows the diaphragm to deflect and therefore, arching can develop. Effects of arching are less critical in a mass of fresh concrete than in soils because the fresh concrete is similar to a fluid. However, as time progresses and concrete starts to stiffen, the effects of arching will undoubtedly come into play. The final aspect which will be mentioned in comparing the two systems is concerned with the measurement of pressure after the maximum is attained. Once the diaphragm has deflected and the concrete acting against its face starts to stiffen, the diaphragm will no longer be measuring the lateral pressure of concrete. Rather, its output will be a result of the permanent deformation of its face. Theoretically, this problem does not occur in the null-balance system because by design, the diaphragm is kept from deflecting at all times.

From a consideration of the points described above, the null-balance method represents a better system to measure the lateral pressure of concrete, and the method was selected for use in this investigation.

DESIGN OF PRESSURE CELL

General Considerations

There are several considerations of a general nature which much be observed in the design of the measuring system. Instrumentation designed for field work must be rugged to withstand handling and site abuse yet it must be sensitive to detect the desired pressure variations. The behavior of the system should not be affected by temperature and humidity changes and reuse of the cells is necessary. In addition, the system must be simple to operate and easy to install.

Technical considerations include the type and magnitude of the pressure to be measured, the materials to use in fabricating the cells, requirements concerning the cell dimensions, and the desired response of the device. Concrete is a heterogeneous material whose properties vary with its consistency and its nature was considered in the final design. The expected maximum pressure was around 30 lb/sq inch.

A material was sought for the cells that would not corrode or react chemically with cement. This requirement ruled out the use of aluminum. In addition, the material had to be readily available and with appropriate mechanical properties.

Dimensions for the cell were selected using criteria presented in later paragraphs. General considerations included building a pressure cell which was easy to handle and install. In addition, the cell could not be so large that its flat face would interfere with the curvature of the wall producing changes in the stress distribution around the cell. Finally, availability of diaphragm gauge sizes was considered. Regarding the desired response of the cell, the sensitivity of the cell (output per unit of pressure applied) should be compatible with the instruments that will be used for recording the data, and a linear response is desired in the range of pressures in which the cell will be used.

Design Parameters

Much work has been done concerning the design of pressure cells for use in soils. These studies have produced criteria for the design of pressure cells based on the behavior of the soil acting against the face of the cell. These parameters were used as guidelines in the design of the pressure cell recognizing the fact that the measurement of pressures in soil is more critical than the measurement of pressure in fresh concrete of the consistency used in this investigation.

The design parameters are presented in Table 6.1. The first three are quoted from the work of Reese et al (1968). Criteria 1 and 2 come from the work on the Waterways Experiment Station (WES) which has conducted extensive studies into the measurement of pressure in soils. The first criterion is related to the effects of the compressibility of a cell which is flush with a rigid surface as opposed to being embedded in the mass of soil. For negligible cell effects, the ratio of diameter to deflections must be larger than 1000. Other investigators have suggested larger values. This reduces the movement of the diaphragm and, therefore, minimizes any arching problem or stress redistributions which might occur due to the deflection of the diaphragm. The second criterion refers to the distance a pressure cell will project above a rigid surface. For negligible cell effects the diameter to projection ratio must be larger than 30, according to WES.

The third and fourth criteria are related to the dimensions of the cell, more specifically to the total and the sensitive diameters. First of all, the area of the cell must be large enough to have a sufficient number of contacts with the aggregate particles. In this regard, Weiler and Kulhawy (1978) using uniform spheres showed that when the sensitive diameter is larger than about 5 times the sphere diameter, the measured pressures are in error by less than 5%. In addition, a ratio of sensitive area to total area smaller than 0.45 has been recommended by Peattie and Sparrow (1954) for cells smaller than 4 inches. A certain minimum ratio of sensitive to total area is needed, among other reasons to provide masssive sides which will produce conditions of fixity for the circular diaphragm.

Materials and Dimensions

Considering the recommendations shown in Table 6.1, the dimensions of the pressure cell were selected as shown in Fig. 6.3. The pressure cell has an outside diameter of 4.0 in., a sensitive diameter of 2.25 in., and a thickness of 0.75 inch. Stainless steel 304, a readily available, corrosion-resistant material was used to construct the cells. Because of its strength, relatively thin diaphragms can be used.

These dimensions produce a ratio of sensitive to total area of 0.32 which is within the design limit. Regarding the diameter to projection ratio, a value of 42 was calculated considering the geometry of a 4 in. cell mounted on a 42 in. diameter formwork. The value is larger than the minimum recommended by WES. To calculate the ratio of sensitivity to particle diameter an assumption

TABLE 6.1. PRESSURE CELL DESIGN CRITERIA

Criteria	Investigator
$\frac{\text{diameter}}{\text{deflection}} > 1000$	WES, for a cell flush with base
$\frac{\text{diameter}}{\text{projection}} > 30$	WES, for rigid cell projecting from base
$\frac{\text{sensitive area}}{\text{total area}} < 0.45$	Peattie and Sparrow
$\frac{\text{sensitive diameter}}{\text{sphere diameter}} \leq 5$	Weiler and Kulhay

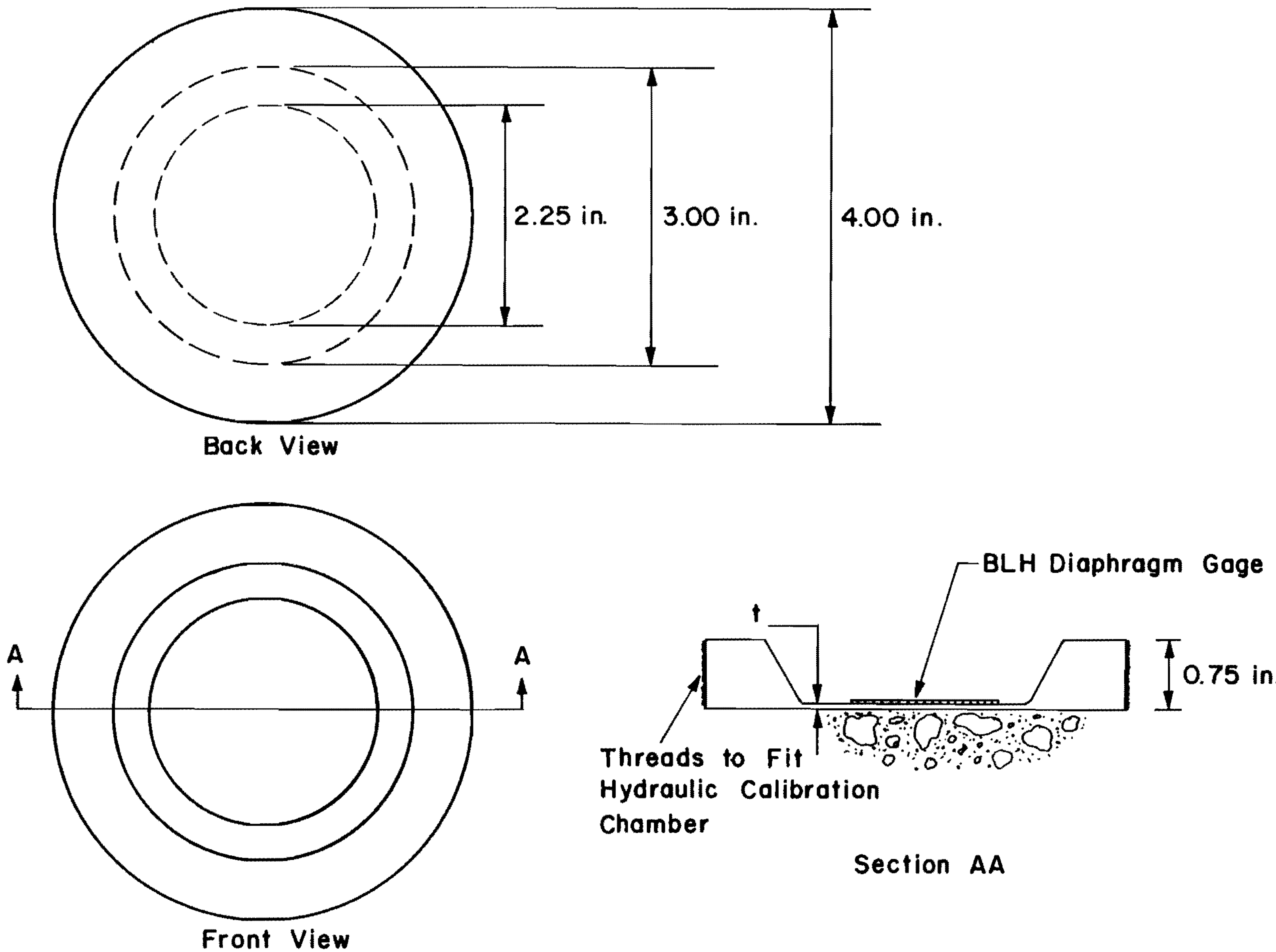


Fig. 6.3. Pressure cell design for present investigation.

was needed. In a mass of fresh concrete, particle size will vary widely, from 1 to 1 1/2 in. maximum size for coarse aggregate to the very fine particles of cement of maybe a few microns; therefore, either a representative value or a range of values must be considered. Assuming particle sizes of 3/4 and 1/2 in., the calculated ratios are 3.0 and 4.5, respectively. These values seem to be reasonable considering the semi-fluid nature of fresh concrete.

The diameter to deflection ratio was calculated as follows. Given the dimensions and material properties of the cell, the equation for the deflection of the center of a clamp-edge plate (Appendix A), and a design pressure of 30 lb/sq in., a minimum thickness of 0.050 in. is calculated which theoretically will produce a cell with a diameter to deflection ratio of 1000. The thicker the cell the larger the ratio. On the other hand, the sensitivity of the cell (output per unit applied pressure) is decreased by thicker diaphragms. Thus, we have two contradictory requirements and a compromise must be reached between them. In addition, the stresses in the diaphragm should be limited to a fraction of the elastic limit if the freely-deflecting diaphragm system is used. For the cell dimensions given in Fig. 6.3, a diaphragm thickness of 0.050 in. and an applied pressure of 30 lb/sq in., the maximum stress in the diaphragm is 11,400 lb/sq in. which is well below the elastic limit of stainless steel 304.

Construction of Prototype Cells

Two pressure cells were constructed to determine the effect of the thicknesses of the diaphragm on the behavior of the cells. The diaphragm thicknesses selected were 0.045 and 0.062 inch. The cells were built from a piece of 304 stainless steel rod, 4 in. in diameter according to the dimensions of Fig. 6.3. They were fitted with a full bridge diaphragm gauge ordered for this application from Baldwin-Lima-Hamilton (BLH). This special gauge has two semi-circular elements near the center of the diaphragm which measure the tensile tangential strain and two radial elements near the perimeter of the diaphragm which measure the compressive radial strains. The gauge size was selected for our particular diameter so that the point of inflection of the radial strain would lie in the open space between the tangential and radial elements of the gauge.

NULL-BALANCE SYSTEM

A detailed description of the null-balance system developed for this investigation follows. A sketch of the system is presented in Fig. 6.4. The system consists of a pressure cell of the diaphragm type attached to a backplate by four screws with a rubber O-ring between them. On the back of the plate, a T-connector is screwed. The two ends of the connector serve for different purposes. One end is connected by flexible tubing to a panel board where nitrogen gas pressure is applied, controlled, and measured by a Bourdon gauge. The other end of the connector leads the wires from the strain gauge to the strain indicators. This side has been sealed with silicon to prevent any leaks. The pressure cell and backplate assembly used in this investigation are shown in Fig. 6.5.

The system is intended to work in the following manner (refer to Fig. 6.4): as pressure p_0 increases, the diaphragm deflects inward and this is sensed by the strain gauge-indicator system. The internal pressure in the cell, p_i, is then increased to bring the diaphragm back to the original position as indicated by the needle of the strain indicator. The internal pressure, p_i, is closely controlled to maintain the original undeflected position of the diaphragm.

CALIBRATION

The pressure cells were calibrated in order to confirm their expected behavior. Two types of calibration were done. First the cells were calibrated against a fluid pressure to determine the behavior of the diaphragm and strain gauge installation (check the linearity of the curve). The second type of calibration was done against concrete. This calibration was performed since it cannot be assumed that fresh concrete will behave against the diaphragm as a fluid will.

Fluid Pressure Calibration

The hydraulic calibration apparatus used is shown in Fig. 6.6a. The pressure cell screws into this device and the face of the diaphragm seals against the O-ring within the calibration apparatus. The complete set up ready for calibration is shown in Fig. 6.6b.

The two pressure cells were calibrated by increasing the hydraulic pressure in increments until the maximum pressure of 40 lb/sq in. was reached. The results are shown in Fig. 6.7. Both cells produce a reasonable output. Note in

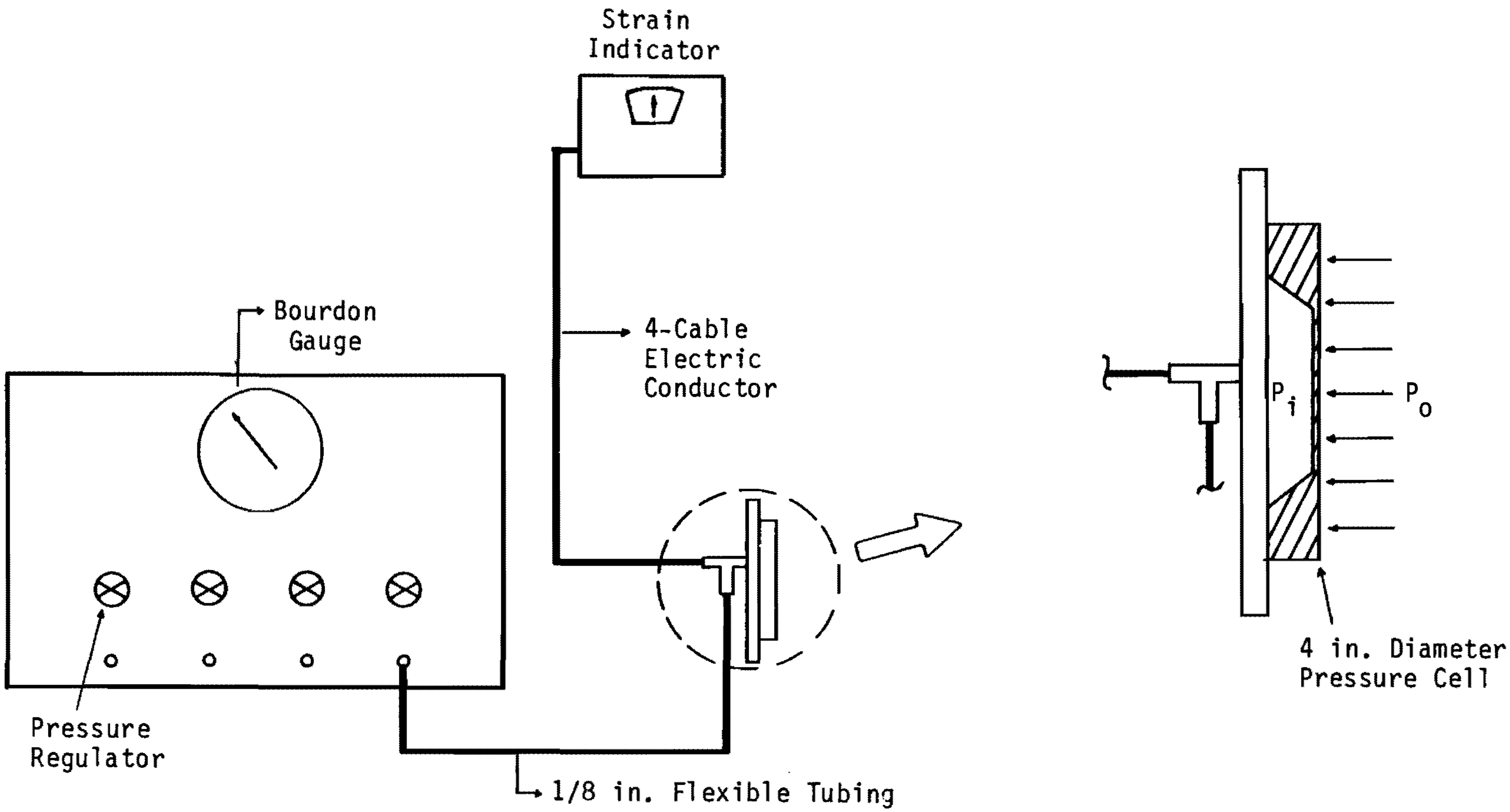


Fig. 6.4. Sketch of null-balance system.

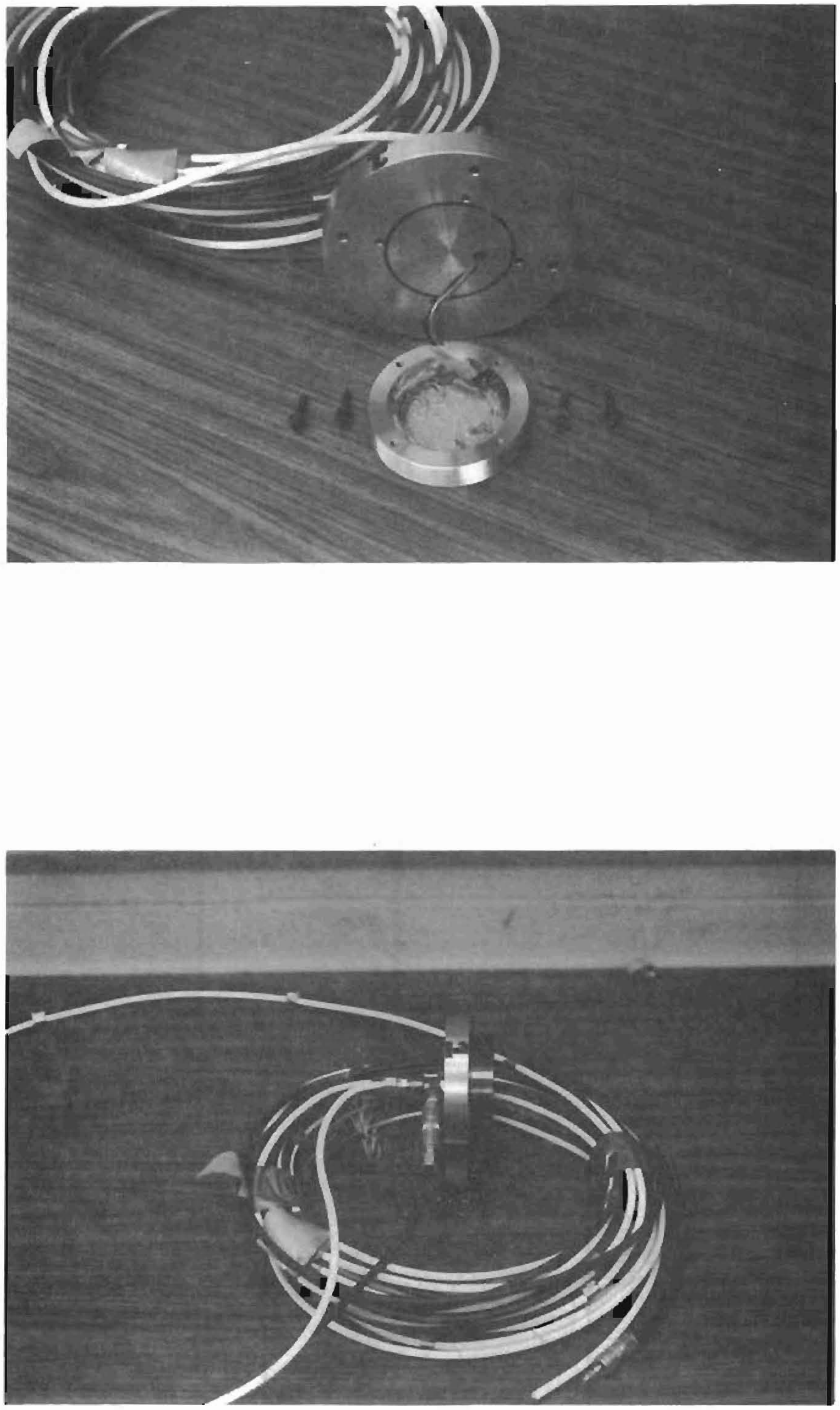
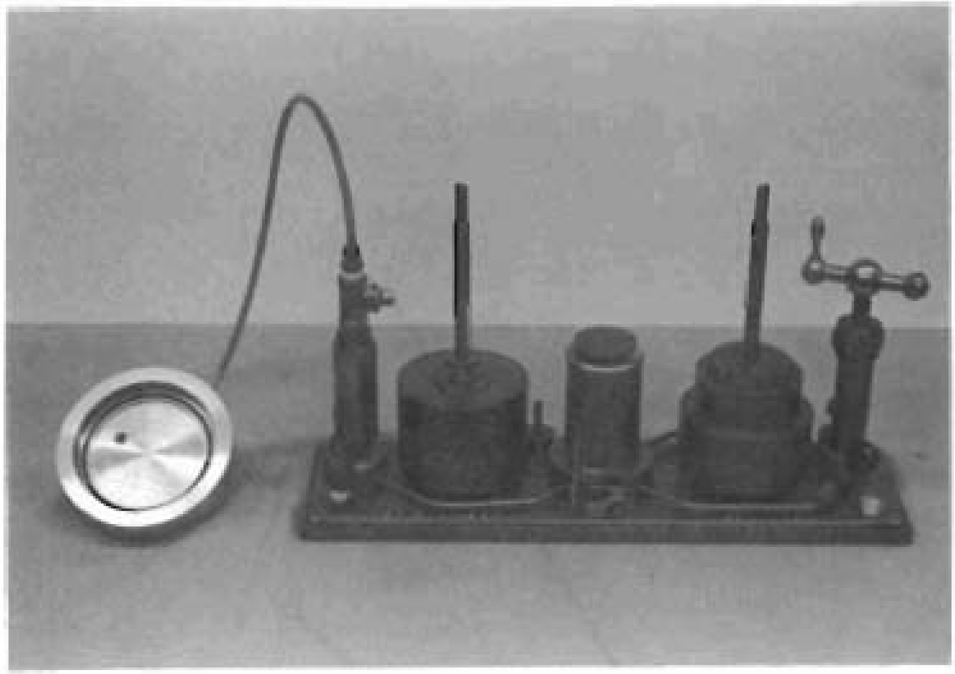
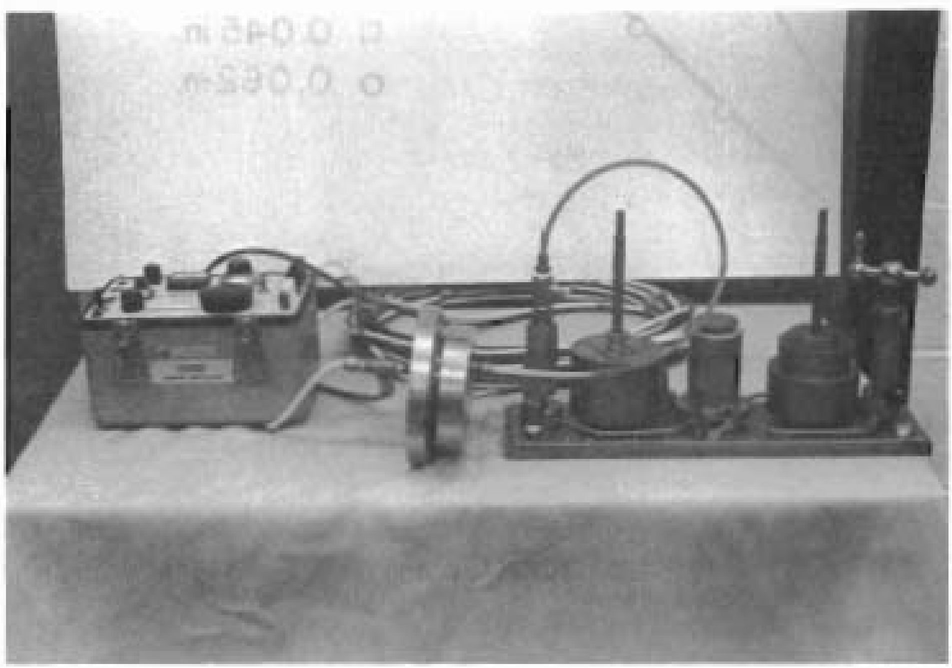


Fig. 6.5. Pressure cell and backplate assembly.



(a)



(b)

Fig. 6.6. Calibration of pressure cell against a uniform fluid pressure.

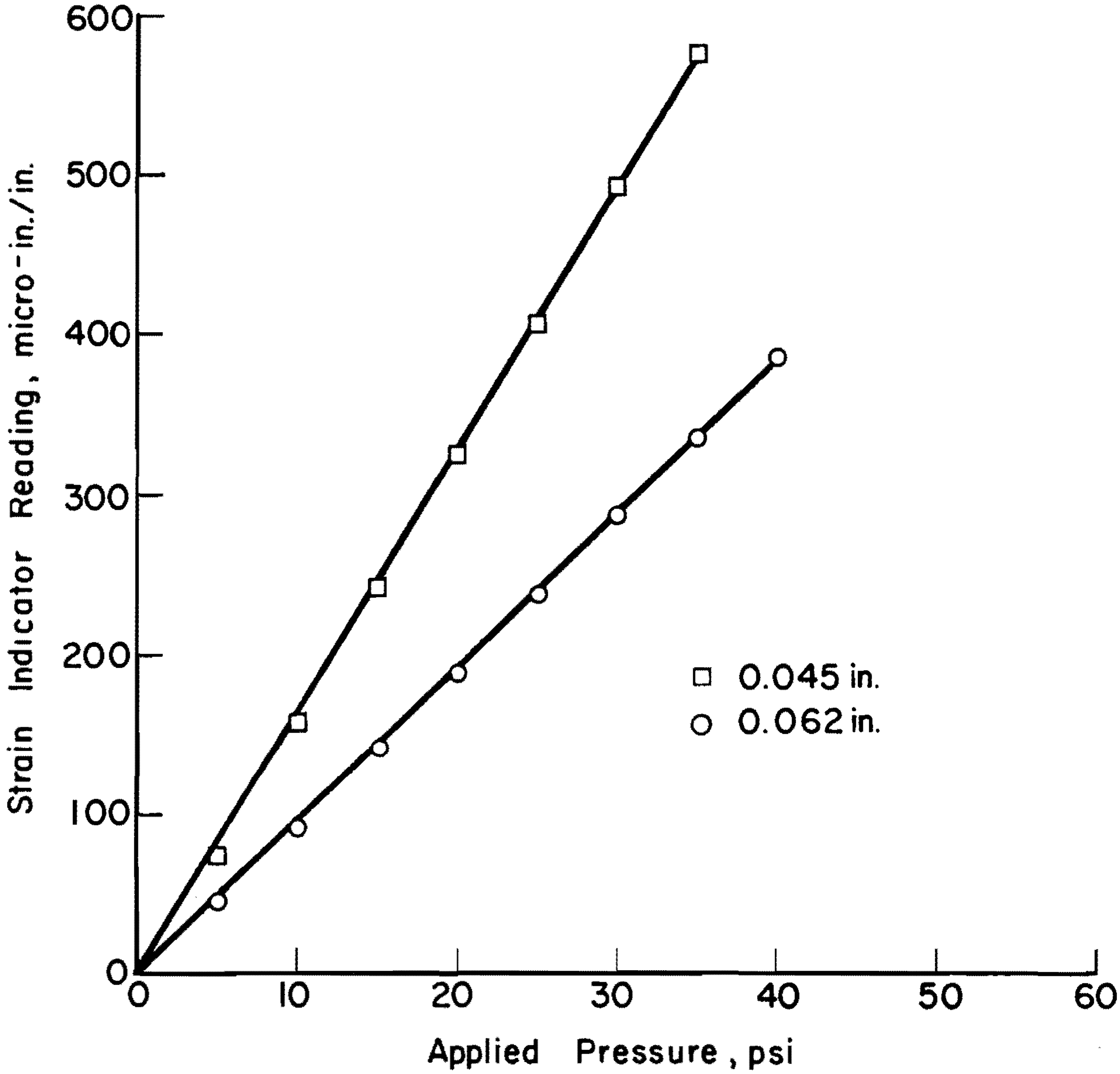


Fig. 6.7. Results of calibration against a uniform fluid pressure.

the figure that the vertical coordinates correspond to the output from the strain gauge circuit which means that the diaphragm is deflecting in this set of calibrations. A second set of calibrations was performed in which the null-balance system was used. As the fluid pressure was applied, the cell pressure was increased to keep the needle of the strain indicator in the initial position. The thinner cell (0.045 in.) overregistered the applied pressure by as much as 1 lb/sq inch. The data from the thicker cell (0.062 in.) fell on a line at 45° indicating that the internal cell pressure was equal to the applied pressure.

Calibration Against Concrete

To calibrate the pressure cells against fresh concrete a calibration chamber was built which permitted the application of a known pressure to the concrete and in addition allowed the face of the diaphragm to be in contact with the concrete. The chamber was large so that side resistance was minimized, and consequently, the applied pressure at the top of the concrete was the same as that at the bottom where the pressure cell is in contact with the fresh concrete. To construct the calibration chamber, the recommendations of the Waterways Experiment Station (WES) as quoted by Reese et al (1968) were considered. WES recommends that (1) the sample height be twice the diameter of the gauge and, (2) the sample diameter be at least four times the height of the soil mass. These recommendations are for a calibration chamber in which soil will be placed. In our case, this implied a chamber at least 8 in. high and 32 in. in diameter. It is believed that in the case of fresh concrete the problems of side friction and arching are not as critical as in soils so that a smaller calibration chamber should prove adequate.

Description of Calibration Chamber

The calibration chamber used in this investigation is shown in Fig. 6.8. It consists of a section of a steel pipe, 3/8 in. thick, 19 1/4 in. inside diameter, and 6 in. deep. A steel plate, 7/16 in. thick, was welded to the bottom side of the chamber and a 4 in. diameter hole opened in the center to accomodate the pressure cell. At the top, a cover bolts to the main body with 16-1/2 in. diameter bolts. This cover has an inlet port for the applicaton of air pressure.

Tests

To perform the calibration tests, a 5 in. slump concrete was prepared and placed in the chamber without compacting. The mix used was similar to the one

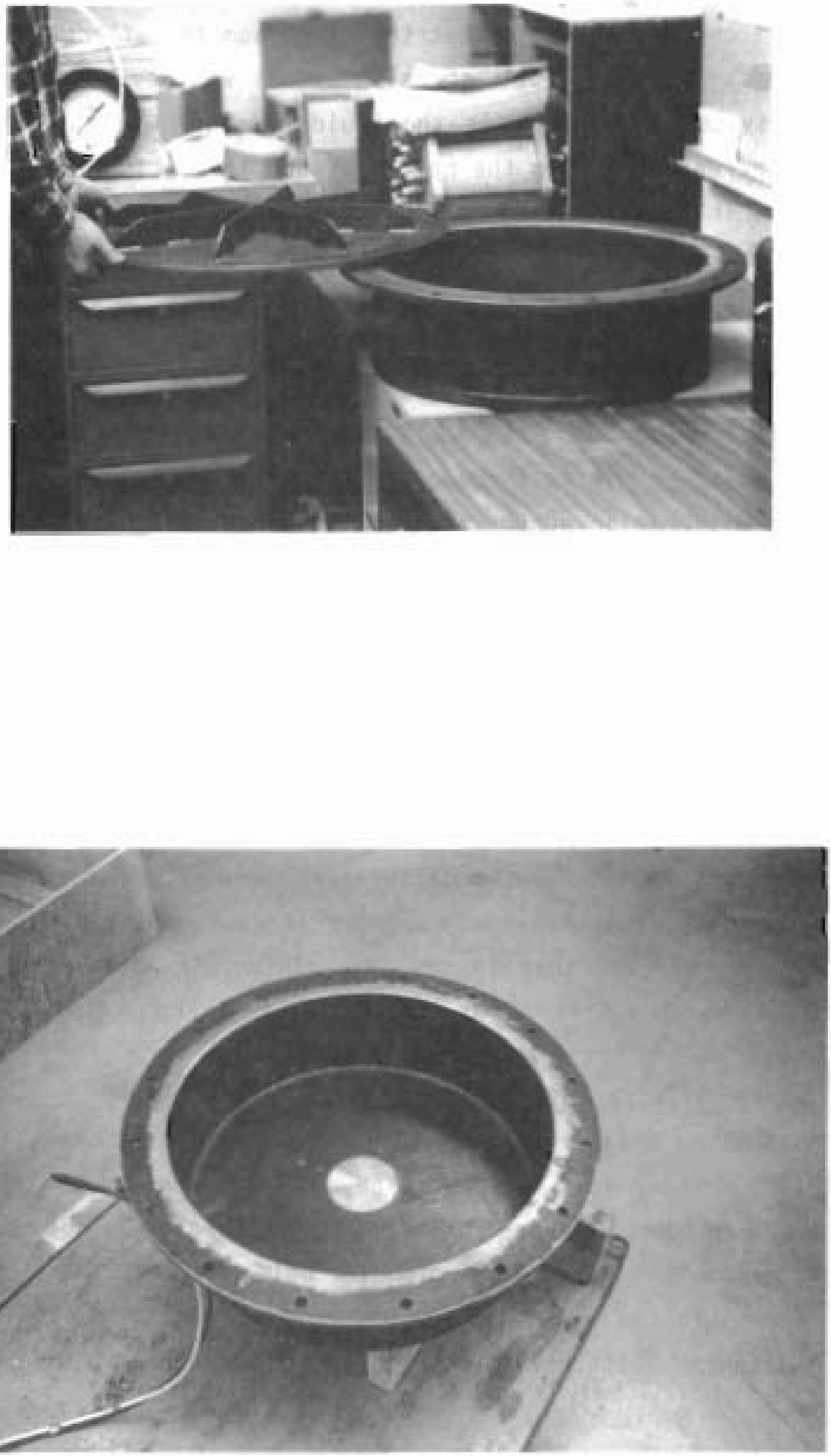


Fig. 6.8. Calibration chamber.

to be used in the field tests. The cover was bolted to the chamber and air pressure applied in increments. After each increment the pressure required to maintain the gauge circuit balance was measured and a calibration curve of the internal cell pressure vs. applied pressure was obtained.

To transmit pressure to the concrete two different systems were studied. The first involved pouring about 1/2 in. of water on top of the fresh concrete and then applying the air pressure on top of the water. The second consisted of placing a thin, flexible membrane (plastic polyethelene) on top of the fresh concrete, clamping it in place with the lid, and then applying the pressure. Care was taken to insure that the membrane had a slack in the chamber so that it could move freely. Eventually, the second method was adopted for calibration. Regarding the first method, there was concern that water acting directly on the fresh concrete would simply increase the porepressure and that this would be the pressure registered by the cell at the bottom. By placing a membrane between the concrete and the air pressure this problem was avoided. The setup used in these calibration tests is shown in Fig. 6.9.

The results of the first two calibrations on the 0.062 in. cell are shown in Fig. 6.10. For pressures less than about 10 lb/sq in. the internal cell pressure was equal to the applied pressure. Above 10 lb/sq in., however, the cells underregistered the applied pressure by as much as 10% at an aplied pressure of 30 lb/sq inch. To find out whether this was the true behavior of the cell in contact with fresh concrete or if some problem related to the behavior of the calibration chamber existed, it was decided to run a test with the chamber filled with water. Since side friction and arching would not be present, it was anticipated that the internal cell pressure would be equal to the applied pressure if there were no extraneous interferences. The results of the calibration are also shown in Fig. 6.10. As seen, even with water in the chamber the pressure cell was indicating a pressure lower than the applied one. These results confirmed the suggestion that the nonlinear behavior of the cells was related to some problem of the calibration chamber. It was reasoned that large deflections of the bottom plate could possibly interfere with the behavior of the cell since the pressure cell fitted very tightly in the hole of the bottom plate. Therefore, the deflections of the bottom plate were measured and plotted against the applied pressure. For purposes of comparisons, the theoretical deflections of a plate of the same dimensions were calculated using the theory of elasticity. Two end conditions were considered: simple support and

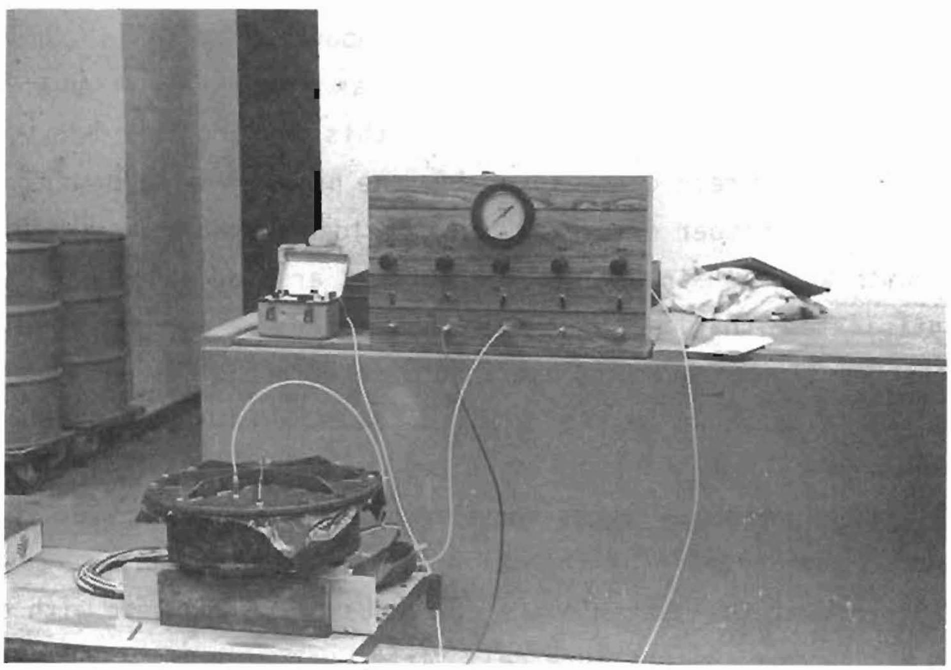


Fig. 6.9. Calibrating pressure cells against concrete.

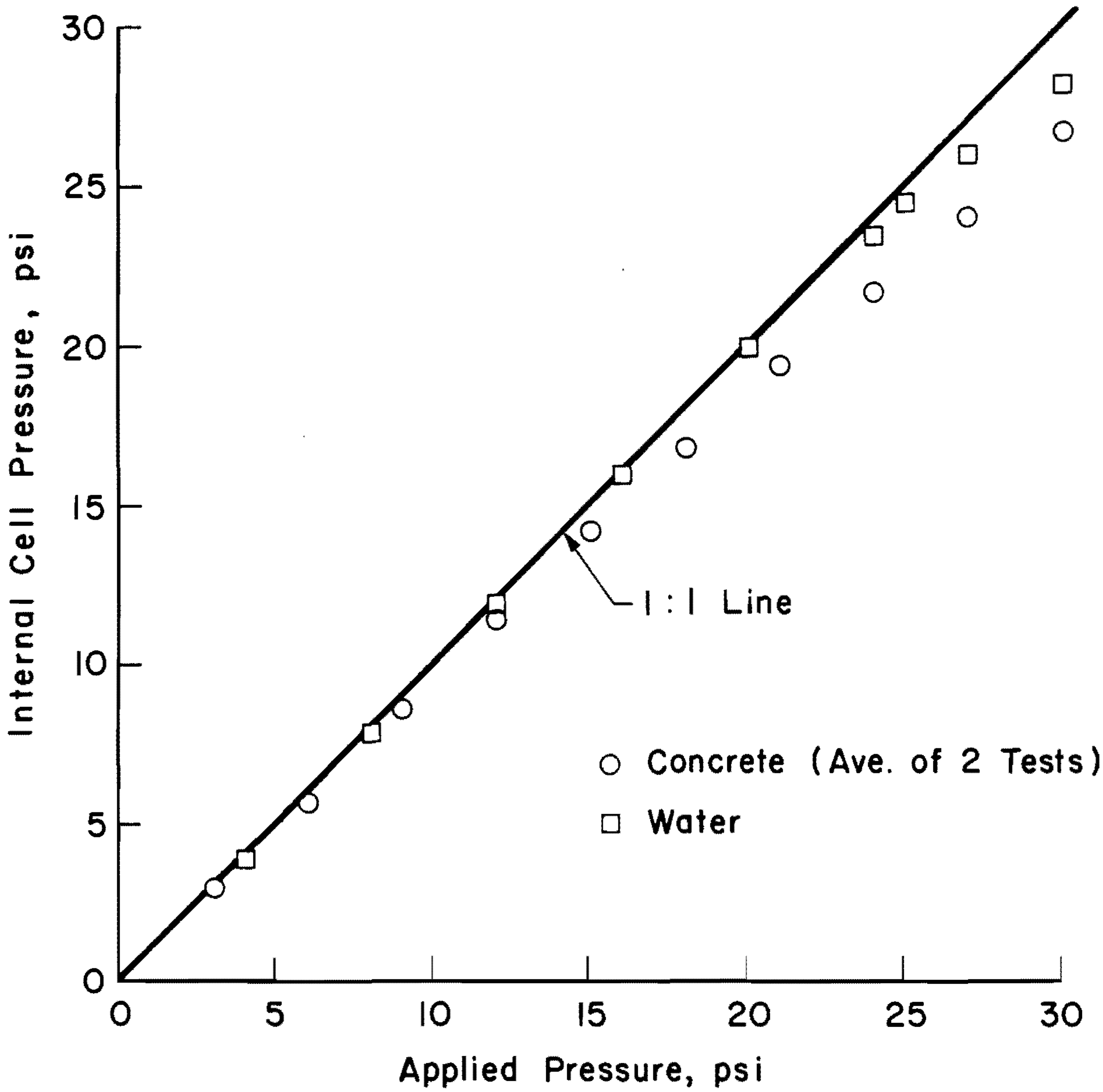


Fig. 6.10. Results of preliminary calibrations.

clamp-edge conditions. The results of the measurements and predicted deflections are shown in Fig. 6.11. The upper and lower solid lines represent the calculated deflections assuming a simply supported and a fixed ends plate, respectively. The dots represent the deflection of the bottom plate measured at the center. Even though the measured deflections were closer to the clamp-edge condition, the movements were still relatively large. To correct this situation, some stiffeners were welded to the bottom plate and the deflections were once again measured. The results are shown in Fig. 6.11 as the square symbols. The stiffeners added were able to reduce the bottom plate deflection by almost 50%.

Once the calibration chamber was stiffened, it was again filled with water and a calibration performed. In this case the internal cell pressure was found to be equal to the applied pressure. It was concluded that during the first calibrations the deflection of the bottom plate had been interfering with the proper performance of the pressure cell and causing the nonlinear behavior. Therefore, a new series of calibrations were performed.

The results of the new calibrations are shown in Fig. 6.12 for both the 0.045 in. and the 0.062 in. diaphragms. The thin diaphragm is seen to overregister the applied pressure by as much as 8% at 24 lb/sq inch. The thicker cell, on the other hand, registers a pressure which is basically the applied pressure. The reason for the behavior of the thin cell is uncertain.

Based on these results, a diaphragm thickness of 0.062 in. was selected for this investigation. Four more pressure cells were constructed for use in the field. The results of the calibration of all five cells against a fluid pressure are shown in Appendix B.

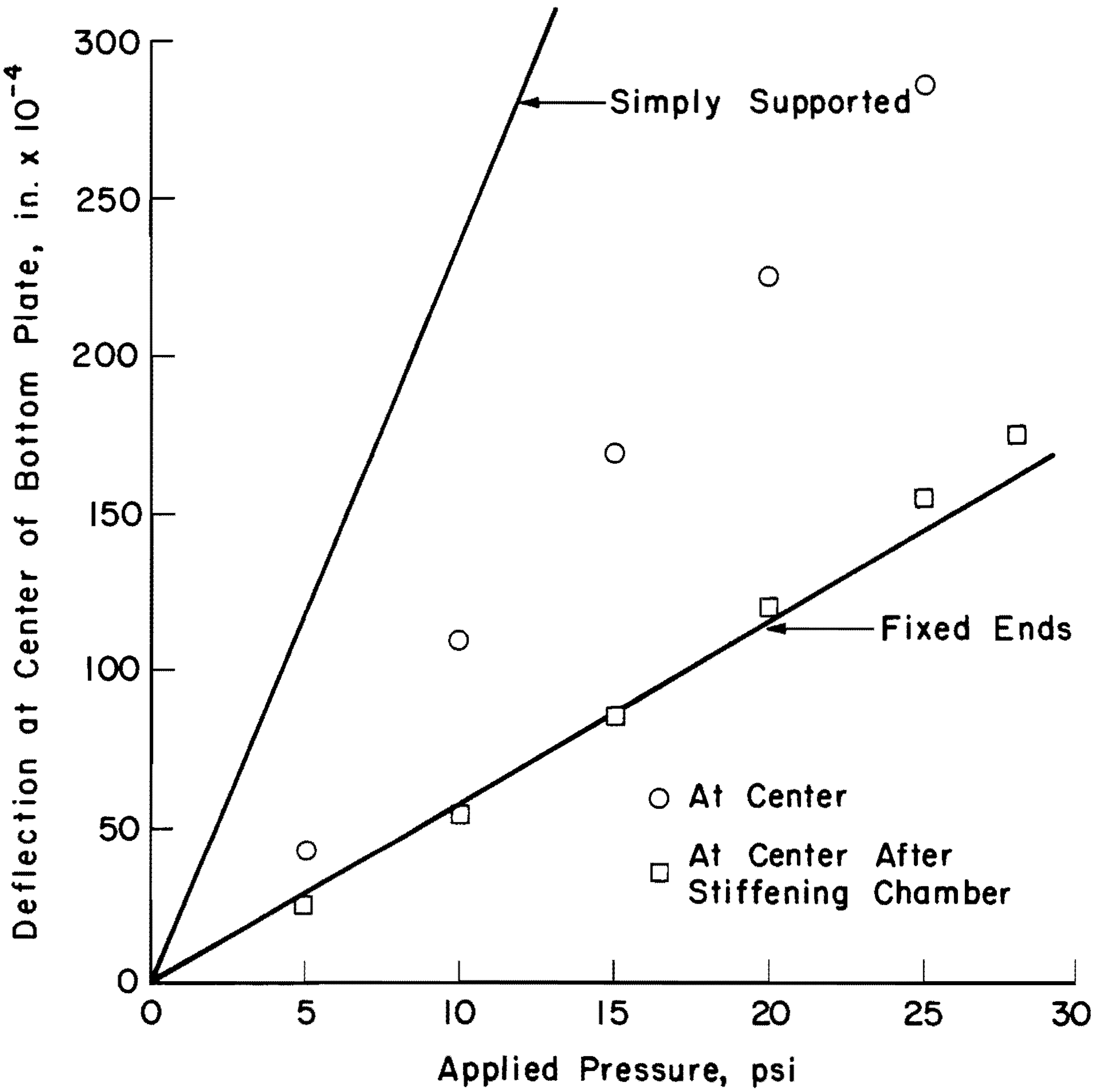


Fig. 6.11. Deflection of the bottom plate of the calibration chamber.

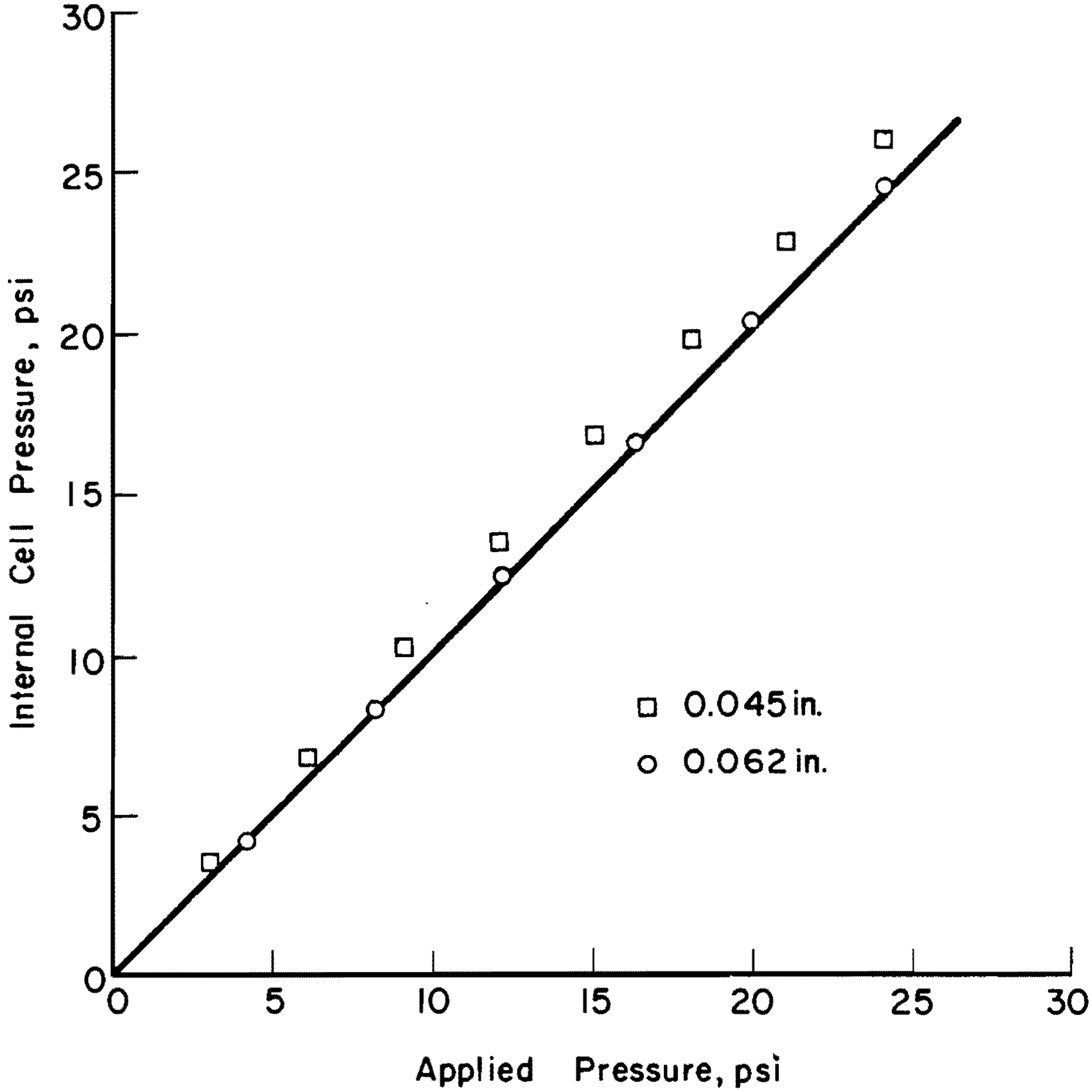


Fig. 6.12. Results of calibration against concrete.

CHAPTER 7. FIELD WORK

This chapter contains a description of the field work performed at Farmer Foundation Company's construction yard in Houston. Personnel from Farmer Foundation Company and from the University of Texas at Austin cooperated in performing the work.

TESTING FACILITIES

The location of the testing facilities used in this investigation in Farmer Foundation's construction yard in Houston provided easy access to construction equipment as well as personnel. The testing facilities per se consisted of the foundation element and the formwork assembly. The foundation was a drilled shaft, 48 in. in diameter. The top 6 in. of the shaft had a reduced diameter so that the 42 in. diameter formwork could be wrapped around it. The formwork consisted of semi-cylinder sections, 42 in. in diameter which were available in 4 and 8 ft long sections. These were assembled to form a column 32 ft high. Four wires were tied from the top of the column and anchored to a firm place in the ground. At the top of the column a small space was provided for a man who would open the gates of the buckets (see Figs. 5.1 and 7.1). Holes slightly larger than 4 in. were cut in some of the formwork sections to accomodate the pressure cells (Fig. 7.2).

TESTING PROGRAM

The consistency of concrete, as measured by the slump test, was the main variable investigated. Two values of slump were used initially, a 4 to 5 in. and 7 inches. The rate of placement selected was in the range of 30 to 40 ft/hour. For the resources at hand, it was estimated that only a few tests could be performed. Because of the small number of tests, no definite testing program was prepared in advance. It was thought proper to have a flexible testing program. The first test was done using a 4 to 5 in. slump. Additional tests were performed once the information from previous tests was analyzed.

Fig. 7.1. Field set up.

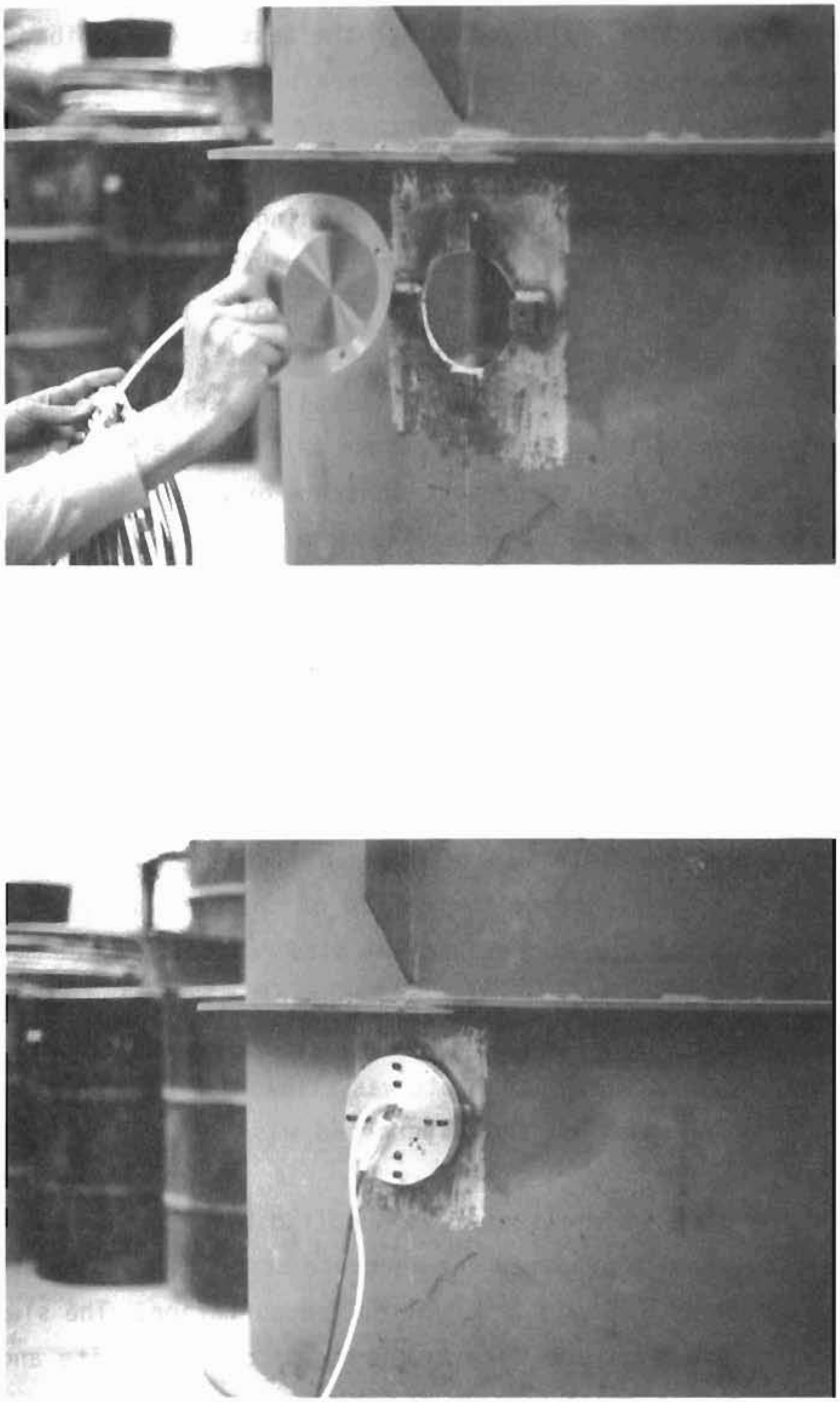


Fig. 7.2. Pressure cell assembly being attached to formwork.

TEST PROCEDURES

The general procedures followed during the tests are described in the next paragraphs. Significant deviations from these procedures or special problems encountered will be noted in detail during the narrative of each individual test.

A column 32 ft high was assembled by joining together sections of formwork. This operation was performed one or two days ahead of the testing day. Two pressure cells were attached at an elevation of 2 ft above the base of the column. Additional cells were located, one each, at elevations of 7 1/2, 13, and 20 ft above the base. The cells were installed early on the morning of the test. The annular space between the pressure cell and the hole in the formwork was filled with silicon glue to prevent concrete or water from leaking through the opening. On the average, the installation of the five cells took between 1 1/2 to 2 hours. The tremie was then lifted by a crane and placed in the center of the formwork. The particular tremie used in this investigation consisted of a 10 in. diameter pipe, 25 ft long which had staggered openings in the sides at about 5 ft intervals. This is referred to as a window tremie.

When the concrete truck arrived at the site, a test was performed to determine the slump, and water was added, as necessary, to bring the slump to the specified value. Concrete was discharged from the truck into a 2 cu yd bucket that was lifted by a crane to the top of the column. A workman at the top of the column opened the bucket and was also responsible for measuring the elevation to the top of the concrete by means of a weighted tape. When the height of concrete in the formwork reached about 20 to 25 ft, the window tremie was removed and the last of the concrete was allowed to fall free. The rate at which the buckets were filled and discharged was controlled to produce the desired rate of placement.

The concrete used in the tests was supplied by a local ready-mix plant. The plant was requested to prepared the concrete according to specifications of the Texas Department of Highways and Public Transportation. The slump and temperature of the fresh concrete were measured at the test site and cylinders were made for determination of compressive strength. In an attempt to maintain a constant slump, a slump test was performed on every other bucket and water was added as needed. The temperature of the concrete was measured every time a slump test was performed.

Before the start of a test, the instruments for reading the strain gauges on the pressure cells were balanced. The arrangement for reading the pressures is shown in Fig. 7.3. During the placement of the concrete the strain-gauge readings were maintained in a null position by changing the internal cell pressure. After the discharge of each bucket of concrete, a set of readings was taken and the elevation of the concrete was recorded. Readings were also taken immediately before and after the tremie was pulled so that the effect of pulling the tremie could be determined. After pouring was completed, the cells were monitored until the pressure appeared to remain constant. The data from all of the tests are presented in Appendix C.

DESCRIPTION OF INDIVIDUAL TESTS

Test 1

The first test was performed on the morning of April 14, 1983, a cool morning with the air temperature in the low sixties. Only three levels of instrumentation were used in the test. Cells were installed that morning before 8:00 a.m. No signs of drift were noticed during the short period of observation.

Concrete arrived at the job around 9:00 a.m. and the slump was measured as 2 1/2 inches. Water was added and the slump raised to 5 inches. The temperature of the concrete was 70 F. Placing of the concrete from the first truck started at 9:32 a.m. and was completed in about 20 minutes without any problem. At this time the column of concrete was 14 ft 2 in. tall. The second truck started discharging about 10:00 a.m. with a concrete slump of 4 1/2 inches. After the first bucket, the man at the top of the formwork noticed that concrete was not flowing well within the tremie. Attempts to free the concrete by bumping the tremie up and down were unsuccessful. Upon removal of the tremie it was observed that a plug of concrete had formed in the bottom 4 or 5 feet. A short tremie about 5 ft long was installed and a second bucket was discharged. At this time a test indicated a slump of 3 1/2; water was added and the slump was brought 4 1/2 inches. Pouring was completed by 10:36 a.m. without any other incidents. Readings of the instruments were taken until about 12:30 when they appeared to have reached an equilibrium.

During this test only one slump test and one temperature measurement were performed on the concrete of the first truck. Only two slump tests were done and no temperature measurements were taken on the concrete of the second truck.

Fig. 7.3. Arrangement for reading pressures in field.

Test 2

The second test was performed on May 13, 1983, and a 7 in. slump used. In this test, as well as in the following ones, an additional level of instrumentation was provided at 20 ft above the base. The concrete trucks arrived around 9:55 a.m. and the slump was found to be 6 1/2 inches. Five gallons of water were added and the slump brought up to 7 inches. The temperature of the concrete was 85°F. Pouring from the first truck started about 10:10 a.m. and was completed in about 15 minutes. The height of concrete inside the formwork was 16 feet. The second truck started discharging around 10:40. When the height of concrete in the formwork reached 25 1/2 ft the tremie was removed without any difficulty. The last few feet of concrete were free-falled from the top of the column. The pour was completed at 10:58 a.m. Readings from the instrumentation were taken until 2:00 in the afternoon.

The concrete suppliers made a mistake and an air entraining agent was added to this mix. The mistake is not thought to have caused any major problems.

Test 3

The third test was performed on May 25, 1983, and a slump of 5 in. was required. Test 3 is basically a repetition of the first test because it was felt that poor control of the concrete had been exercised in the first test.

The concrete trucks arrived at 9:39 and a slump of 7 1/2 was measured. Several alternatives, such as the addition of cement to the truck, were considered for lowering the slump. All of the possible solutions involved a considerable waiting time. Therefore, it was decided to return the concrete and order a new load.

The new concrete arrrived at 11:30 a.m. and water had to added to obtain a 5 in. slump. Pouring started at 11:40 a.m. When the second bucket of concrete was lifted to the top, it was not possible to open the gate of the bucket so that a second man had to climb up and help to open the gate. This involved a delay of about 15 minutes. The second truck started pouring at 12:12 p.m. When the concrete reached a height of 25 ft above the base the tremie was pulled without much difficulty. Pouring ended at about 12:33 p.m. and readings were taken until 2:00 in the afternoon.

Test 4

The fourth and last test was performed on July 21, 1983. A high slump concrete was attained by the use of a superplasticizer. The basic mix design used in previous tests was modified slightly to accomodate the superplasticizer. To control the use of the superplasticizer, Dr. Ramón L. Carrasquillo of the University of Texas at Austin was present for this test.

The test was scheduled to start at 10:00 a.m. but the concrete was delayed. At 11:30 a.m. one truck arrived with a concrete of 2 in. slump. The test was not started because the second truck had not arrived. Around 12:15 p.m. the second truck arrived and preparations were made to start the test. A second test was done on the first truck and the slump found to be about 1 inch. Six gallons of water and 512 ounces of superplasticizer (dosage of about 14 oz/sack) were added to the truck and mixed. A new test was performed and the slump found to be 9 1/2 inches. Pouring was started around 12:35 p.m. The air temperature was in the mid-nineties and the temperature of the concrete about 94°F. The first truck was emptied by 12:49 p.m. and pouring began from the second truck at 12:56 p.m. The slump of this second truck was 9 1/2 in. and the dose of superplasticizer used was smaller (about 10 oz/sack). When the height of concrete reached 25 ft the tremie was pulled without any difficulty. It came out easily and cleanly. Pouring of the column finished around 1:15 p.m. Readings were taken until about 4:00 p.m. when a heavy rain started. By this time, however, it was apparent that the pressure readings were stable.

CHAPTER 8. ANALYSIS OF FIELD DATA

PERFORMANCE OF INSTRUMENTATION

A series of tests were performed regularly to observe the behavior of the pressure cells in order to determine if they were functioning properly. A dead-load tester was used to check the calibrations of the pressure cells before and after every test. The calibrations for each cell checked within ±5 units of microstrain every time.

The resistance-to-ground of the strain gauges was checked during the test program for each cell and found to be in good order. The drift in the readings of the strain gauges was monitored during the time that elapsed between installation of the pressure cells and the placing of the concrete. The results of these tests showed that cell #1 (one of the two cells at the bottom) registered a considerable amount of change, in the range of 15 to 20 units of microstrain, during the time observed. The other cells showed either no change or a negligible amount of change and, in all tests, by 9:00 or 10:00 a.m. the readings had stabilized. Because at the time of construction of the cells only two had been checked for temperature stability, it was decided to test all five cells. They were put in an oven and the temperature raised in increments from 75°F to 140°F. The strain-indicator readings showed no variations for cells #2 and #4, and small variations for cells #3 and #5. Cell #1, however, exhibited a change of 21 units of microstrain when the temperature went from 76 to 113°F. Beyond 113°F, the readings from cell #1 were constant. Consequently, it seems possible that the drift experienced by cell #1 in the field can be attributed to temperature changes.

Another check of the cells was performed at the end of each field test. As the pressure cells were removed from the formwork, the strain indicator reading required to maintain the circuit balance was noted. Supposedly, gauges working properly should have registered a reading close to zero as the cells were not subjected to pressure either from the concrete or from the internal pressure system. A considerable deviation from a zero reading was taken as a sign of improper functioning of the cells. Pressure cells #2 through #5 recorded less than about 5 units of microstrain when they were removed from the formwork. Cell #1 consistently showed high readings (about 40 units of microstrain) after being removed from the formwork.

After analyzing the results of these tests, it was concluded that cell #1 was showing signs of improper functioning while cells #2 through #5 were apparently working correctly. Furthermore, pressure cell #1 was consistently recording high pressures during the field tests. In many cases the pressures measured by cell #1 were much higher than the equivalent hydrostatic head of concrete. For these reasons, a decision was made to exclude the readings from cell #1 in the analysis of the tests.

RESULTS OF FIELD TESTS

During the early stages of pouring (the first 10 or 15 ft) the measured pressures in all the tests were equivalent to the hydrostatic head of concrete. As the head of concrete increased beyond that height, different pressure distributions were recorded in each test. In only one case was a hydrostatic condition attained throughout the length of the column. The pressure distribution observed in each test will be discussed later.

Two other points need some comment. The first is the effect of pulling the tremie on the lateral pressures. Because some kind of pressure change was expected, it was decided to take readings from the pressure cells immediately before and after pulling the tremie. The pressures indicated in the results of the individual tests do not reflect the effects of the removal of the tremie. Tremie effects are discussed in a separate subheading in this chapter. The second point which needs some remarks concerns the final or equilibrium pressures that were measured. After reaching a maximum the measured pressures fell to a constant value after two or three hours. During this time the strain indicator was monitored continuously until no more changes in pressure were noted. These residual pressures are also discussed under a separate subheading.

The data for each test are presented by a figure which contains two different plots. On the left hand side of each figure a series of curves representing the distribution of concrete pressure along the length of the formwork at different stages of concreting is presented. For comparison, a line representing a hydrostatic distribution of pressure for concrete is also shown. The second plot, located on the right side of the figure, shows the height of concrete in the formwork versus time elapsed since the concrete truck left the plant.

Test 1

The first test was intended for a 5 in. slump. As mentioned in Chapter 7, only three levels of instrumentation were used, at 2, 7 1/2, and 13 ft from the bottom of the form. The results from this test are shown in Fig. 8.1. It can be seen that a hydrostatic condition developed in the concrete up to a height of concrete of less than about 15 ft (this coincided with the load of the first truck). Above 15 ft, the pressures started to decrease. A maximum pressure of about 11.5 lb/sq in. was recorded for this test at the lowest level of instrumentation. This represents only about 39% of the possible maximum pressure (the equivalent hydrostatic condition). It was unfortunate that an additional level of instrumentation was not available, say around 20 ft, to define properly the shape of the lateral pressure distribution.

From Fig. 8.1 it is seen that the pressures measured at the 13 ft level of instrumentation were indeed very low. This might have been due to either of two causes or a combination of both: setting of the concrete at this location and/or arching of the concrete in the upper part of the form. By referring to the plot of time versus height in Fig. 8.1, it becomes evident that by the time the height of the concrete reached 20 ft, over two hours had elapsed since the truck left the plant. Thus, it would not be surprising if stiffening of the concrete were already in progress. Arching of the concrete also could have occurred. By studying the information on Test 1 found in Appendix B, it is seen that a relatively stiff concrete (3 1/2 in. slump) was poured when the depth of the pour was about 20 feet. This concrete could have formed a plug in the formwork so the pressure from an additional head of concrete was not distributed to lower elevations but was taken by shear forces developed along the periphery of the formwork. The fact that a stiff concrete was poured at about 20-ft height was confirmed when the formwork was removed and the column of concrete was exposed. Thus, it seems that a combination of the setting of concrete and arching were responsible for the low pressures that were measured.

Test 2

A 7-in. slump was used for the second test and the results of this test are shown in Fig. 8.2. It is observed that the curves for the cases where the concrete was below a height of about 20 ft correspond to a hydrostatic condition. The curve corresponding to a height of concrete of 21 ft is linear, but not parallel to the hydrostatic line for a unit weight of 145 lb/cu foot. Calculations show that the line for 21 ft corresponds to a hydrostatic line for a

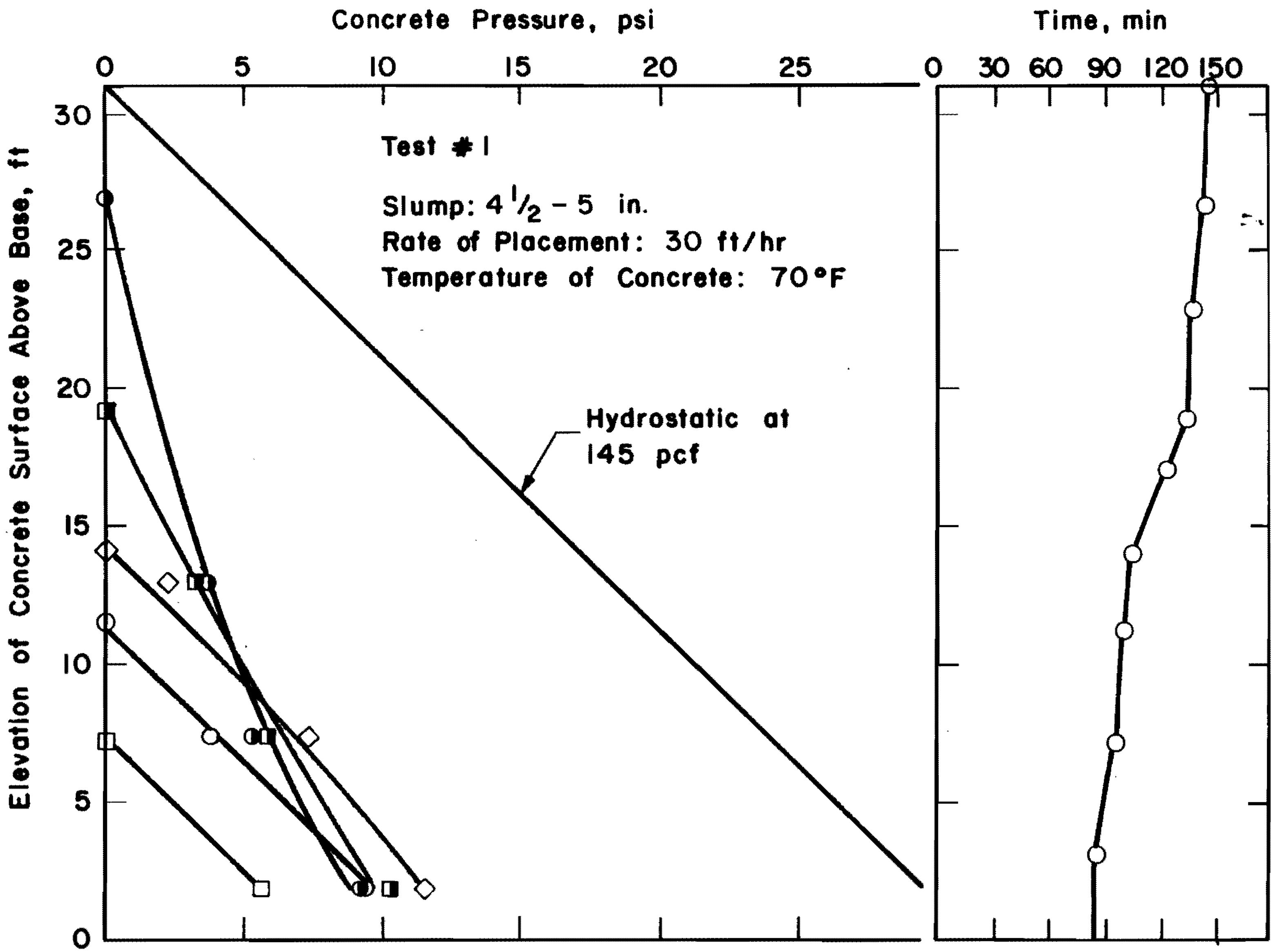


Fig. 8.1. Distribution of pressures from concrete at different stages of concrete, test 1.

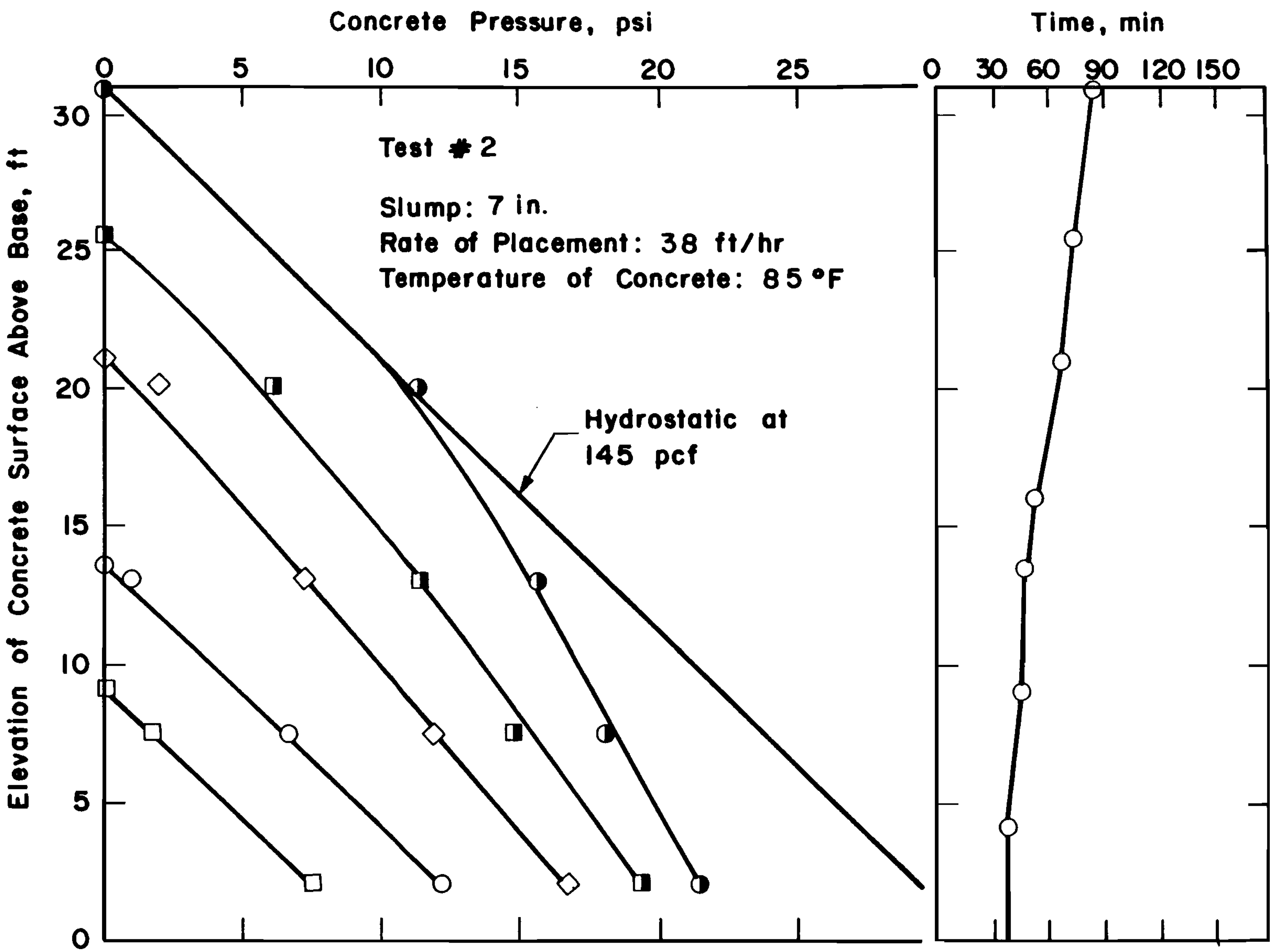


Fig. 8.2. Distribution of pressures from concrete at different stages of concreting, test 2.

unit weight of concrete of about 125 lb/cu ft while the lines for the heights of concrete of 9 and 13 ft correspond to a unit weight of about 151 lb/cu foot. Thus, the assumption can be made that the concrete acted hydrostatically to a depth of at least 13 ft and that pressures less than hydrostatic developed as the height of the column was increased to 21 feet. The distributions of pressure when the height of the concrete reached 25 1/2 and 31 ft show a hydrostatic distribution only for the top few feet. For the last lift of concrete (from 25 1/2 to 31 ft) a change in head of 5 1/2 ft of concrete produced only an increase in pressure of less than 3 lb/sq in. at the bottom level of instrumentation. If further increases in the height of the column were made, pressure at the bottom would possibly not have increased much more.

The maximum pressure measured in this test was 21 lb/sq in. which corresponds to 73% of the equivalent hydrostatic pressures for a unit weight of 145 lb/cu foot. It was measured at the end of concreting.

Test 3

After the results of the first two tests were analyzed a decision was made to repeat the first test because of the poor control which was exercised over the concrete. Therefore, Test 3 was made with a 5 in. slump.

The results from the third test are shown in Fig. 8.3. The curves in Fig. 8.3 show that concrete behaved hydrostatically when the height of the column of concrete was 11 feet. The distribution of pressure for a height of concrete of 20 1/2 ft is linear but not parallel to the hydrostatic line for a unit weight of 145 lb/cu foot. Calculations show that the line for 20 1/2 ft corresponds to a hydrostatic line for a unit weight of concrete of about 100 lb/cu ft. As the height of the concrete column is raised above 20 ft, the pressure at the bottom changes little. The maximum pressure measured was about 13 lb/sq in. at a head of 27 feet. This amount represents about 46% of the equivalent hydrostatic head for a unit weight of 145 lb/cu foot. In this test better control of the concrete produced a more uniform and faster rate of placement. Thus, these results can be more effectively compared with those from the second test. These comparisons will be done later in this chapter.

Test 4

Because a hydrostatic condition was not developed along the full length of the column for any of the first three tests, it was decided to perform the last test using a slump in the 9 to 10 in. range. Two methods can be used to produce such concrete. The first is simply to increase the water content of the mix until

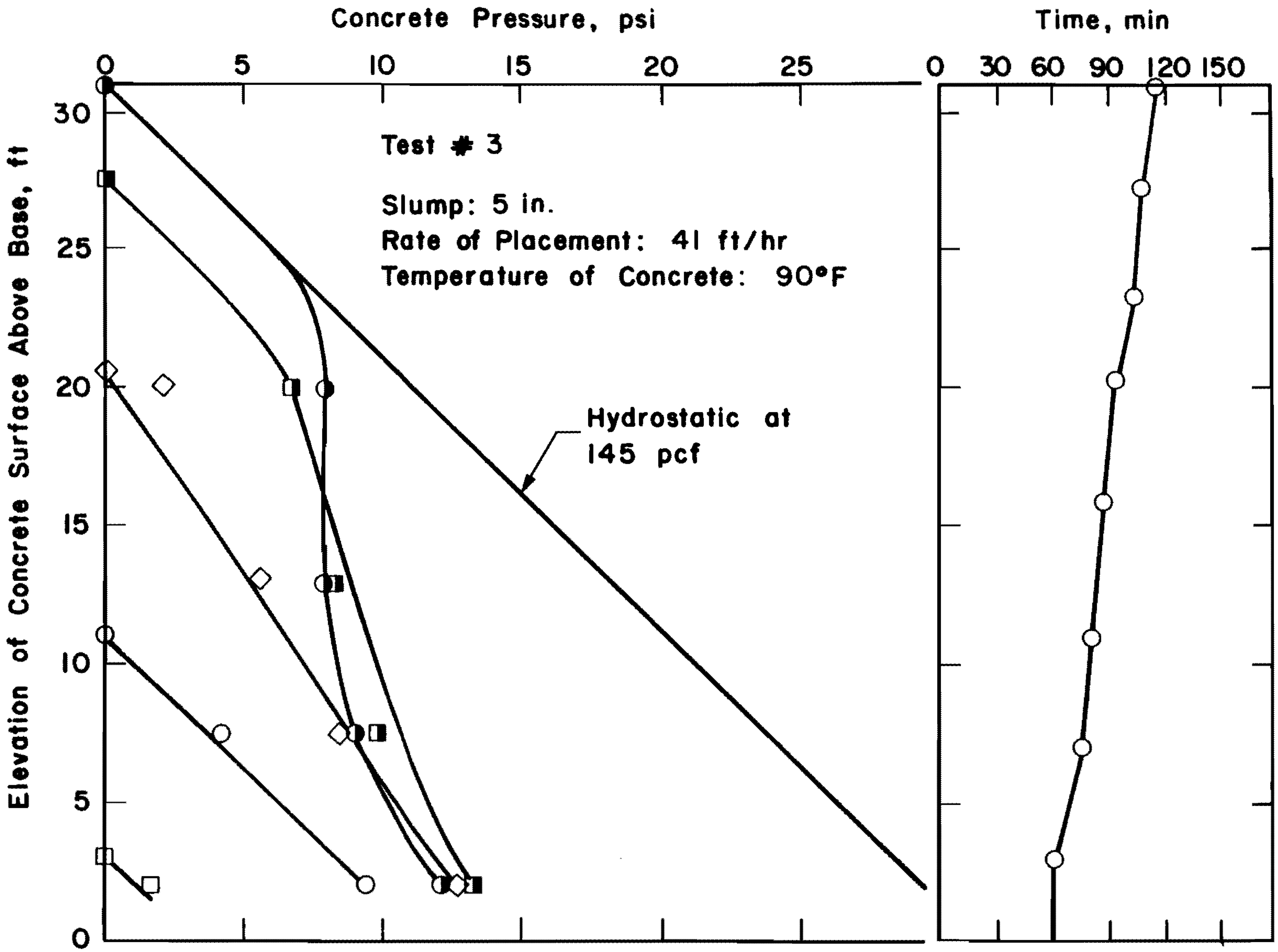


Fig. 8.3. Distribution of pressures from concrete at different stages of concreting, test 3.

the slump becomes 9 inches. The amount of cement would have to be increased to maintain the specified strength. The second method consists of making use of a superplasticizer. With this method a mix is designed for a 1 or 2 in. slump and the admixture is used to increase the slump to the required level. After a meeting with the sponsors, it was decided to use a superplasticizer to attain the high slump.

The results of the last test are shown in Fig. 8.4. As seen in the figure, a 9 1/2 in. slump was used in the test. All the pressure distribution curves are linear and parallel to the hydrostatic line for a unit weight of concrete of 145 lb/cu foot. The maximum pressure measured was 28.9 lb/sq in. which was equal to 99% of the equivalent hydrostatic pressure. From the plot of time versus height of concrete it can be noted that the superplasticizer was added to the first truck almost 1 1/2 hrs after the concrete truck left the plant. This, however, had no adverse effects on the performance of the test.

Tremie Effects

In an effort to ascertain the effect of pulling the tremie on the pressure of concrete, readings of the pressure cells were taken immediately before and after pulling the tremie. The measurements showed that the effect of pulling the tremie is variable and unpredictable. Maximum increases in pressure of 2.5, 4, and 8 lb/sq in. were recorded for Tests 1, 2, and 3, respectively. Test 4 showed no changes in pressure.

The effect of the tremie on the pressures of concrete can be related, at last conceptually, to the properties of the concrete at the time of pulling the tremie. A concrete with the consistency of a semi-fluid should not exhibit any changes in the magnitude of the lateral pressure upon withdrawal of the tremie. In this case the tremie is lifted without much difficulty as was the case in Test 4. As the concrete becomes stiffer, the bumping up and down of the tremie produces some compaction of the concrete and increases the lateral pressure as was the case in the other three tests. The magnitude of this increase in pressure is, however, totally unpredictable.

RESIDUAL PRESSURES

At any instrumentation level, the pressures measured started from zero, increased to a maximum, and then decreased to a constant value (residual pressure) after about two hours following the end of placement. The distribution

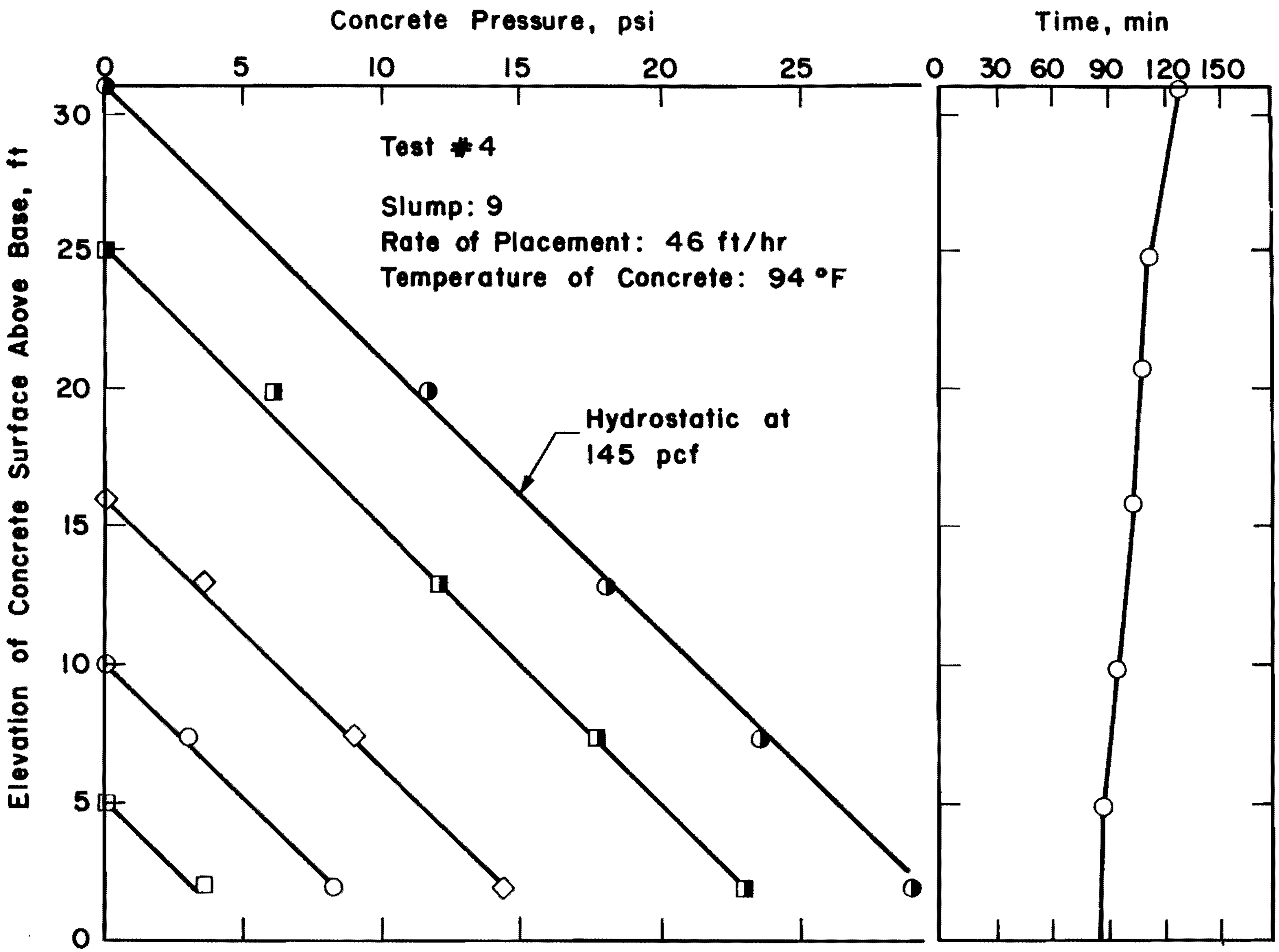


Fig. 8.4. Distribution of pressures from concrete at different stages of concreting, test 4.

with depth of these residual pressures is shown in Fig. 8.5 for each one of the tests. In general, these pressures amounted to between 1/2 to 2/3 of the maximum pressures measured. It is interesting to note that the residual pressures were higher for the higher slump concretes. The variation of pressure measured with time is shown in Appendix D for each test.

The drop in pressure measured in the tests is believed to be related to the loss of water from the forms. During the performance of all four tests, it was observed that water seeped out of the forms along the vertical line where the two half-cylinder sections came together. This drop in pressure behavior has been observed by other investigators. Rodin (1952) has attributed this phenomenon to shrinkage of the concrete due either to bleeding and form leakage, or to absorption of water by the aggregate and chemical changes.

SUMMARY AND COMPARISONS

A summary of all the test results is shown in Table 8.1. As seen in the table, there is information concerning the concrete properties, the variables affecting concrete pressure, and the results from the measurement of lateral pressure. The concrete properties are the unit weight, and the 28-day compressive strength. Both of these values were determined in the laboratory.

It is observed that the measured unit weights and compressive strengths for the concretes with the slumps of 5 and 7 in. (Tests 1, 2, 3) are very similar. As expected, the compressive strength for the 7-in.-slump concrete is somewhat smaller than for the 5-in.-slump concrete because the water-cement ratio for the 7-in.-slump concrete was higher. The small difference in strength might be due to the fact that an air entraining agent was used in the 7-in.-slump concrete. The concrete for Test 4, in which a superplasticizer was used, had a higher unit weight (by about 4 lb/cu ft) and a higher compressive strengths (about 1700 lb/sq in.) than the concrete for the previous tests.

Other information in Table 8.1 concerns the slump, the rate of placement, and the temperature of concrete. The values of these parameters for Test 1 are somewhat different than those of the other three tests. Therefore, the results of Test 1 cannot be compared directly to those of the other tests. However, Test 1 indicated the need for good control of the concrete consistency and showed that concrete with a 4 to 5 in. slump can stiffen under certain conditions and cause problems in placement.

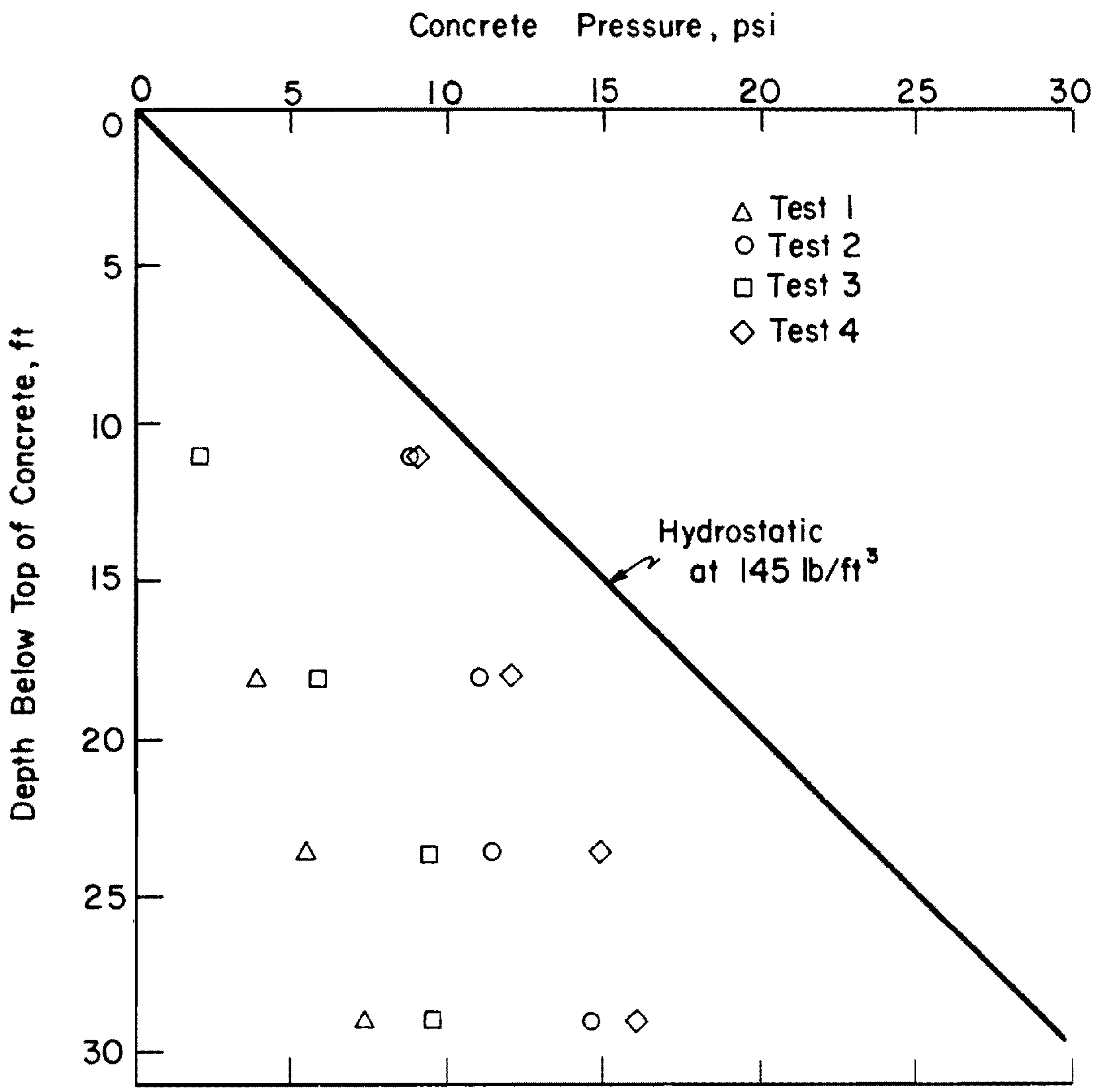


Fig. 8.5. Distribution with depth of residual pressures.

TABLE 8.1. SUMMARY OF TEST RESULTS

Test #	Date	Concrete Properties		Variables Affecting Concrete Pressure			Results	
		Unit Weight of Concrete (lb/sq in.)	f'_c [a] (28d, lb/sq in.)	Slump (in.)	Rate of Placement (ft/hr)	Temp. of Concrete (°F)	Max. Pressure[b] @ 85°F (lb/sq in.)	$K = \sigma_h/\sigma_v$
1	4/14/83	144.8	4900	4½	30	70	10.5	0.36
2	5/13/83	143.2	4500	7	38	85	21.3	0.73
3	5/25/83	144.2	4800	5	41	90	13.3	0.46
4[c]	7/21/83	148.2	6450	9½	46	94	28.9	0.99

[a] Average of four values

[b] Correction factor used ±0.69 lb/sq in./10°F

[c] Basic mix proportions slightly altered to allow the use of a superplasticizer

Tests 2, 3, and 4 were better controlled and the rates of placement and temperature conditions were more uniform so that a meaningful comparison between them is possible. In Fig. 8.6 the envelopes of maximum pressure that were measured for each test are presented. In the case of Tests 1 and 3, these maximum pressures did not occur at the same head of concrete. For Tests 2 and 4, on the contrary, all the pressures were measured at the maximum head of concrete of 31 ft, although at any given elevation the rate of pressure increase with head was smaller for Test 2 than for 4.

The values of the maximum pressures measured shown in Table 8.1 are modified to correspond to a temperature of 85°F. To perform the modification a factor of ±0.69 lb/sq in./10°F, obtained from the results of previous investigations (Rodin, 1952; Peurifoy, 1965), was used. Thus, if the temperature of concrete was 75°F at the time of the test, the pressure at 85°F was calculated by subtracting 0.69 lb/sq in. from the measured value. The last column in Table 8.1 shows the computed values of K, the ratio of horizontal to vertical stress in the concrete. The vertical stress was assumed to be equal to the unit weight of concrete times the height of the column of concrete.

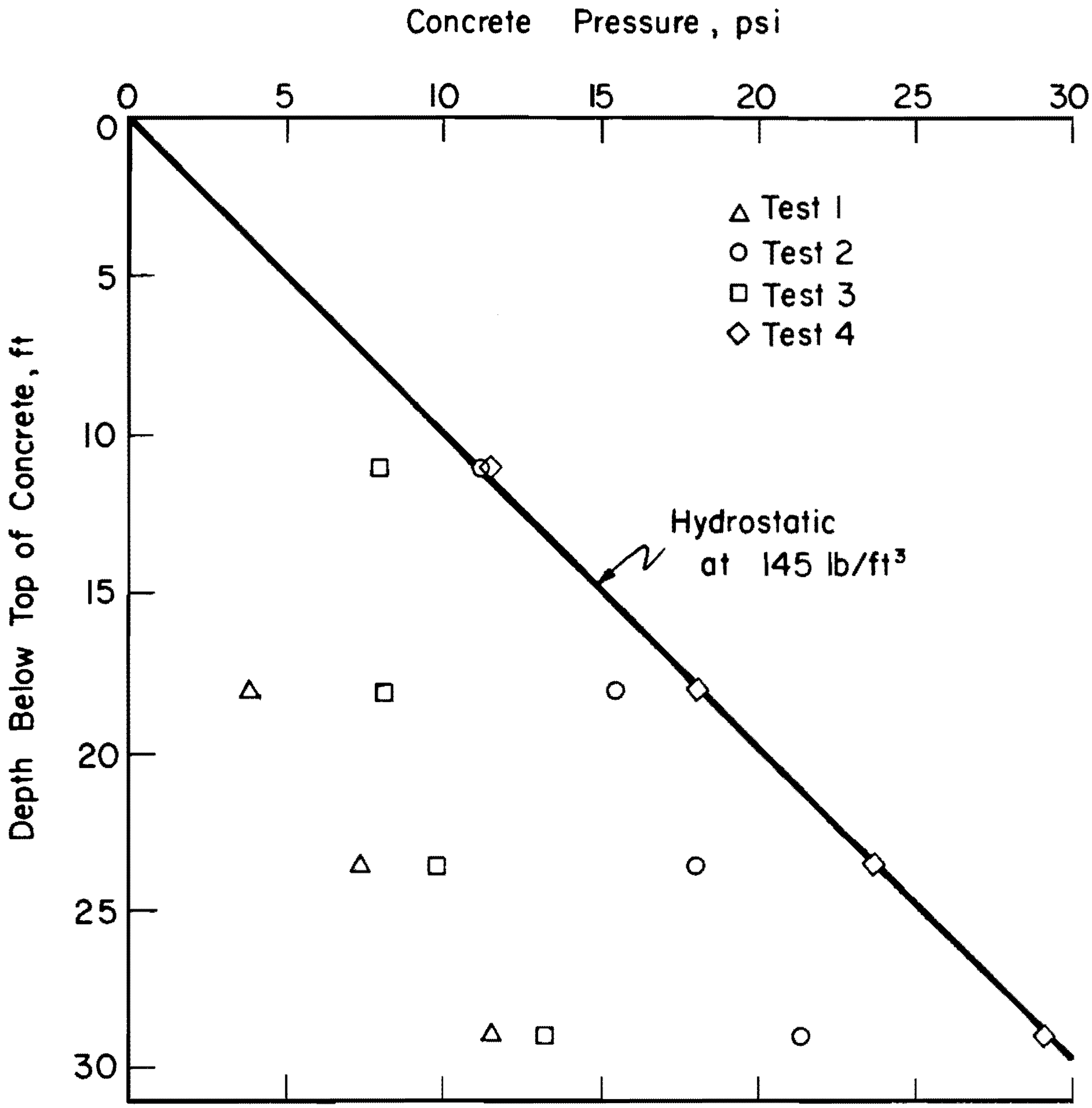


Fig. 8.6. Envelope of maximum pressure measured in each test.

CHAPTER 9. CONCLUSIONS AND RECOMMENDATIONS

The problem of predicting the lateral pressure of concrete is complex because of the large number of variables that are involved. Researchers have attempted to predict the lateral pressure from concrete using two different approaches. In the experimental approach, tests have been run under controlled conditions and the influence of a given variable on the pressure has been studied. In the theoretical approach, the behavior of fresh concrete has been modelled and a theory developed to predict the lateral pressure. The research conducted so far, using either of the methods, has been related to conditions that exist in building construction.

The experimental investigation described in this report has been concerned with the measurement of the lateral pressure developed by concrete used in drilled shafts. From the literature review and the results of the experiments, several conclusions have been drawn and recommendations are made.

CONCLUSIONS

- Fresh concrete is a complex material whose properties and behavior depend on the cement characteristics, the mix proportions, and the consistency at which the concrete is mixed. To complicate this picture further, the properties of concrete vary with time, as it changes from a semi-fluid or plastic material into a hardened state.
- The lateral pressure developed by fresh concrete will depend not only on the intrinsic properties of the material but also on the interaction of other factors such as rate of placement and minimum form dimension, among others.
- For the case of drilled shafts, the mix proportions and consistency of fresh concrete, the placement rate, and the length and diameter of the shaft are the important variables which control the pressure exerted by concrete against the sides of the hole.
- Other conditions being equal, an increase in the slump of the concrete will result in higher lateral pressures. During placement of fresh concrete for the construction of a drilled shaft, there is a certain distance below the

top of the column of fresh concrete over which hydrostatic pressure develops against the sides of the excavation. The hydrostatic pressure is equal to the total unit weight of the fresh concrete times the distance below the top of the column of concrete. An increase in the slump of the concrete, other factors being equal, will result in a greater length of shaft over which hydrostatic pressures will occur.

• The moving up and down of the tremie might produce some increases in the measured lateral pressure. These increases are unpredictable.

• The lateral pressures measured in the tests decreased with time and reached values which ranged from 50 to 70% of the maximum pressures that were measured. The amount of the decrease is consistent with the findings of other investigators. In the tests that were performed by the writers, it is believed that the reduction of pressure was due to: first, loss of water from the forms along the line where the forms were bolted; and second, shrinkage of the concrete due to loss of water.

• The magnitude of the loss of lateral pressure found in the tests is not directly relevant to the construction of drilled shafts. Because the steel formwork has a much higher stiffness than does soil, shrinkage will cause a greater loss of lateral stress in the concrete form than in a soil deposit. In addition, the presence or absence of water in the soil profile will affect the amount and nature of the volume changes in concrete and, therefore, the loss in lateral stress.

• From the results of the experiments described herein, and from previous investigations, it is concluded that an increase in the slump of the concrete will cause an increase in the load carried in skin friction. (Previous research has shown that the unit skin friction goes up as the normal pressure goes up at the interface of the concrete and the soil.)

• Future investigations on the behavior of drilled shafts should take into consideration the concrete characteristics and concreting operation because these factors will have an important influence on the pressure exerted by fresh concrete against the soil.

RECOMMENDATIONS

• The present specifications of the Texas Department of Highways and Public Transportation concerning the slump of concrete should be reviewed in the light of the results of the studies reported herein. A concrete with a high slump is desirable not only from the standpoint of increased lateral pressures (and increased skin friction) but also from the standpoint of construction. A high slump makes the concreting operation much easier and should result in a faster construction. The high slump should be attained by a proper selection and proportioning of materials and not by merely adding water to a given mix. In this manner other concrete properties will not be adversely affected.

• The use of superplasticizers to produce high workability concrete at reasonable water-cement ratios for use in drilled shafts should be given due consideration. The main drawback regarding superplasticizers is that their effects last for a limited time; however, the chemical can be added to the concrete at the site and many concrete pours for drilled shafts can be made in a short time. Also, as mentioned earlier, current research on superplasticizers is expected to increase the time for which the superplasticizers remain effective.

• Research has shown that in cohesive soils there is moisture migration from the concrete into the soil with a consequent decrease in the strength of the soil. In this regard, the use of a superplasticizer might have a beneficial effect besides increasing the lateral pressure. Because a relatively low water-cement ratio is used, there will be less excess water available for migration and less softening of the sides of the hole. Further research on moisture migration from concrete cast with a superplasticizer is desirable.

• The results of the investigation reported herein pertain to specific experimental conditions. It is known that the lateral pressure of fresh concrete is influenced by several other variables besides consistency. It is recommended that additional studies be conducted to establish the influence of the fresh concrete characteristics and the rate of placement on the lateral pressure of concrete. It is believed that in the construction of drilled shafts, these two variables will have a major effect. Fresh concrete characteristics of importance include consistency (investigated in this report), mix proportions and setting behavior. The latter is of utmost importance since it will control the length of time during which concrete will behave in a hydrostatic manner. The rate of placement in itself is of limited use unless the time spent in concreting or the length of the shaft are known.

• Field load tests should be carried out to ascertain the effect of the lateral pressure of concrete on the axial capacity of a drilled shaft. In light of the results of this investigation, special attention must be given to the concrete characteristics and the concreting operation.

• As a result of the present investigation, the information collected in previous field load tests of drilled shafts should be reanalyzed with the purpose of determining whether a relation exists between the lateral pressure of concrete and the load transfer.

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APPENDIX A

EQUATIONS DESCRIBING THE BEHAVIOR OF A CLAMP-EDGE UNIFORMLY LOADED CIRCULAR PLATE

APPENDIX A EQUATIONS DESCRIBING THE BEHAVIOR OF A CLAMP-EDGE UNIFORMLY LOADED CIRCULAR PLATE

The equations presented can be found in Timoshenko (1941).

The maximum deflection occurs at the center of the plate and is equal to:

$$y_c = \frac{3qa^4}{16Eh^3}(1 - \nu^2)\left[1 + \frac{4}{1 - \nu}\ \frac{h^2}{a^2}\right]$$

The maximum stress occurs at the edge and is equal to:

$$\sigma_r = \frac{3qa^2}{4h^2}$$

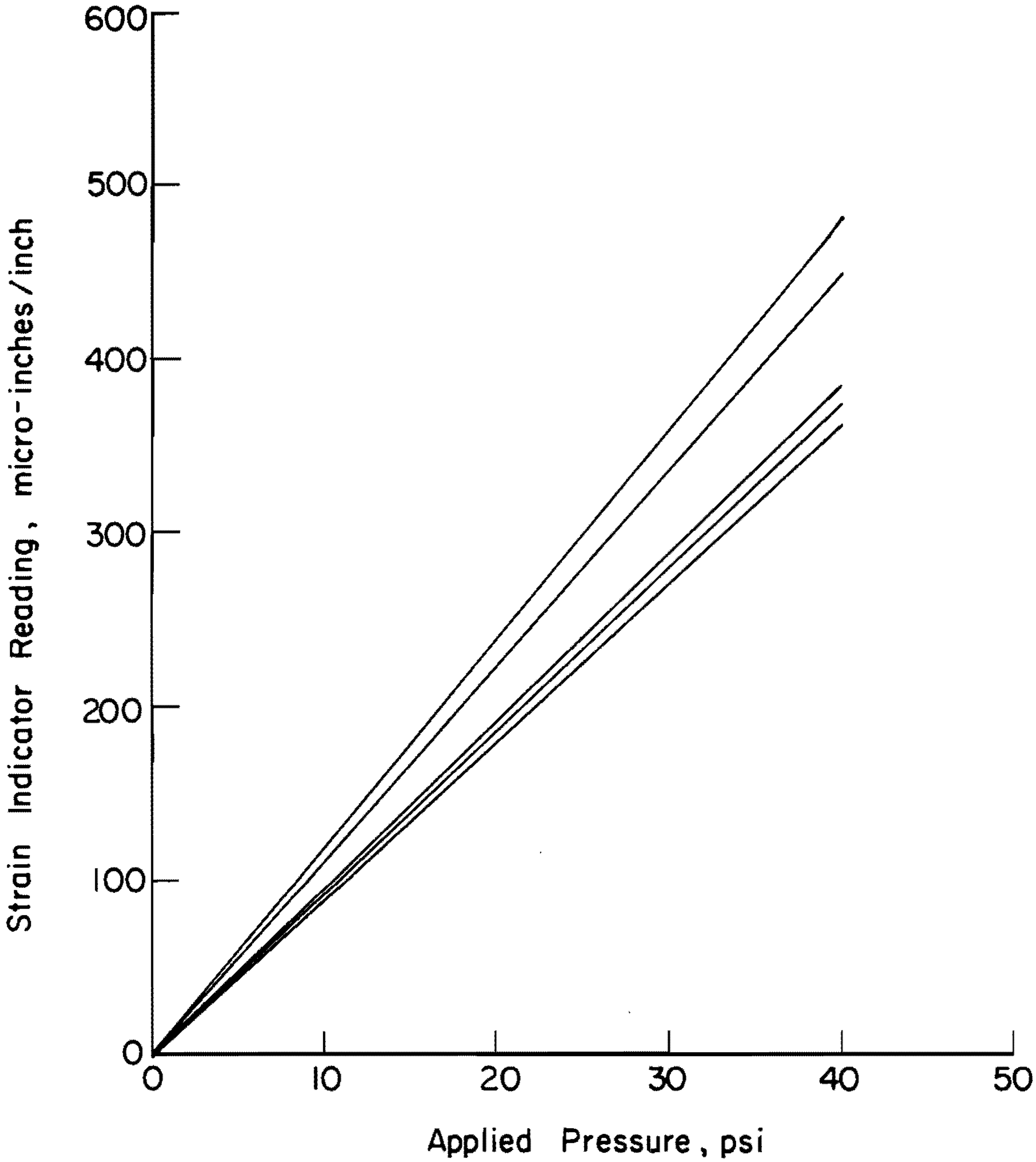
where a = radius of plate
h = thickness of plate
q = applied uniform pressure
y_c = deflection at center of plate
E = Young's modulus
ν = Poisson's ration
σ_r = radial stress

The following assumptions were made in the derivation of the equations:

- the plate is flat, of uniform thickness, and of a homogeneous, isotropic, and linearly elastic material
- all forces, loads and reactions are perpendicular to the plane of the plate
- deflections are small compared to the thickness of the plate
- only elastic action occurs.

APPENDIX B

CALIBRATION OF PRESSURE CELLS USED IN THE TEST PROGRAM


600
500
400
300
200
100
0
0
10
20
30
40
50
Strain Indicator Reading, micro-inches/inch
Applied Pressure, psi

APPENDIX C

DATA COLLECTED DURING FIELD TESTS AND CYLINDER STRENGTHS

Project 308
Report of Field Test #1
April 14, 1983

Time	Height of Concrete (ft)	Readings from cells (lb/sq in.) 1	2	3	4	Comments
8:55						Concrete mixer arrives at site. Slump 2½ in. Water added until slump goes to 5 in. Temperature of concrete 70°F.
9:32	3'-3"	1.9	2.0			First discharge
9:42	7'-4"	6.1	5.7			
9:47	11'-5"	10.3	9.4	3.9		
9:51	14'-2"	12.8	11.5	7.2	2.3	Last load from first mixer
9:58						Slump test second truck 4½ in.
10:00	17'-2"	12.5	10.6	6.5	2.4	First bucket second truck. Concrete won't flow well in tremie. Tremie bumped up and down.
10:10		22.4	18.8	10.3	4.5	Readings after bumping tremie
10:15	16'-2"	15.1	13.0	7.9	3.1	Readings after tremie was removed. Plug in bottom 5 ft of tremie. Short tremie installed.
10:20	19'	14.9	12.6	7.3	4.0	
10:25	23'	14.4	12.3	6.9	4.3	Slump test done after this bucket 3½ in. 5 gals H_2O slump raised to 4½ in. Short tremie removed.
10:33	26'-9"	13.6	11.6	6.7	4.2	Concrete free fall
10:36	30'-9"					Last bucket
10:45		12.1	10.4	5.9	4.0	

Time	Height of Concrete (ft)	Readings from cells (lb/sq in.) 1	2	3	4	Comments
11:00		10.3	8.0	5.6	4.0	
11:30		10.4	7.4	5.4	4.0	
12:30		10.4	7.4	5.4	4.0	Last readings

Slump and Temperature Record:

Time	Truck	Temp. (F)	Slump (in.)	
8:55	1	70	2½	34 gals H_2O added
9:30	1	-	5	
9:58	2	-	4½	35 gals H_2O added at site
10:25	2	-	3½	5 gals H_2O added
10:28	2	-	4½	

Compressive Strength:

Date cylinders taken: April 14, 1983

Date cylinders tested: May 11, 1983

		Unit Weight (lb/cu ft)	f'_c (lb/sq in.)
Truck 1	cyl. 1	145.6	4850
	cyl. 2	144.1	4920
Truck 2	cyl. 1	145.4	5020
	cyl. 2	144.1	4770

Nominal Mix: CA 2058 lb

FA 1176 lb

Cement 564 lb

Water 31 gals

3 oz retarder/sack

Project 308
Report of Field Test #2
May 13, 1983

Time	Height of Concrete (ft)	Readings from cells (lb/sq in.) 1	2	3	4	5	Comments
							Cells were installed by 8:00 a.m. No drift observed
9:55							Trucks arrive
10:10	4	3.8	3.0				Pouring started
10:18	9	9.8	7.4	1.6			
10:21	13½	16.2	12.1	6.5	0.9		
10:25	16	19.2	14.0	8.7	3.8		Last bucket from 1st truck
10:35		18.7	13.4	8.4	3.0		
10:40	21	22.2	16.6	11.7	7.1	1.8	First load 2nd truck
10:47	25½	26.0	19.2	14.6	11.3	6.0	After this bucket tremie was pulled
10:50		> 30	23.1	17.0	12.0	5.8	Readings after tremie was removed
10:58	31	> 30	25.2	20.3	16.1	11.2	End of pour
11:05		> 30	22.7	18.8	15.3	10.7	Pressure cells were monitored continuously to keep strain indicator in original position
11:30		23.6	17.0	15.1	12.9	9.3	
12:00		20.2	14.7	12.6	11.1	8.8	
12:15		20.2	14.4	11.4	11.0	8.9	
2:00		20.2	14.6	11.4	11.0	8.9	Last readings

Slump and Temperature Record:

Time	Truck	Temp. (°F)	Slump (in.)	
10:00	1	82	6½	5 gals H_2O added
10:05	1	83	7	
10:20	1	85	7½	
10:30	2	86	3½	20 gals H_2O added
10:37	2	85	6½	
10:45	2	85	6 3/4	
10:55	2	86	7	

Compressive Strength:

Date cylinders taken: May 13, 1983

Date cyclinders tested: June 9, 1983

		Unit Weight (lb/cu ft)	f'_c (lb/sq in.)
Truck 1	cyl. 1	143.1	4550
	cyl. 2	142.9	4450
Truck 2	cyl. 1	143.6	4560
	cyl. 2	143.1	4500

Nominal Mix: CA 2058 lb

FA 1176 lb

Cement 564 lb

3 oz retarder/sack

3 oz air entraining agent

Project 308
Report of Field Test #3
May 25, 1983

Time	Height of Concrete (ft)	Readings from cells (lb/sq in.) 1	2	3	4	5	Comments
9:39							Two trucks arrive. Slump 7½ in. Both returned.
11:30							New trucks arrive. V. dry concrete. H_2O added.
11:40	3	2.1	1.5				Slump 5¼ in. Start test.
11:45							Delay, gate of bucket will not open; 2nd man climbs & helps to open it.
11:55	7	7.8	5.6	0.3			
11:59	11	13.1	9.4	4.1			
12:06	16	17.3	12.2	7.9	3.2		End of 1st truck
12:12	20½	17.5	12.6	8.4	5.5	2.0	First bucket 2nd truck
12:15	25	--	12.9	8.9	7.5	4.0	Readings before pulling tremie
12:22	23½	28.6	20.6	15.7	10.5	2.2	Readings after tremie pulled
12:27	27½	29.5	21.2	17.1	13.2	6.7	
12:33	31	29.9	20.3	16.3	12.8	7.9	Last batch
12:45		23.2	16.1	14.8	11.0	4.6	
1:00		19.0	12.3	12.8	8.6	3.4	
1:30		16.4	9.5	9.3	5.8	2.0	
2:00			9.5	9.3	5.8	2.0	

Slump and Temperature Record:

Time	Truck	Temp. (°F)	Slump (in.)	
11:35	1	88	3½	15 gal H_2O added
11:40	1	88	5¼	
11:52	1	89	4 3/4	
11:45	2	-	-	Very dry concrete. 20 gals H_2O
11:55	2	90	2½	8 gals H_2O added
12:10	2	92	4 3/4	
12:15	2	92	3½	4 gals H_2O added
12:18	2	93	5	

Compressive Strength:

Date cylinders taken: May 25, 1983

Date cylinders tested: June 22, 1983

		Unit Weight (lb/cu ft)	f'_c (lb/sq in.)
Truck 1	cyl. 1	144.3	4700
1	cyl. 2	144.8	4880
Truck 2	cyl. 1	143.9	4700
2	cyl. 2	144.0	4880

Nominal Mix: CA 2058 lb

FA 1140 lb

Cement 564 lb

3 oz retarder/sack

31 gal water

Project 308
Report of Field Test #4
July 21, 1983

Time	Concrete (ft)	Readings from cells (lb/sq in.) 1	2	3	4	5	Comments
11:33							One truck arrived. Had to wait almost 45 min. before 2nd truck came.
12:35	5	4.6	3.5				Start pouring
12:42	10	11.9	8.2	2.9			
12:46	14½	18.7	13.3	7.7	2.2		
12:49	16	20.4	14.3	8.8	3.4		Last bucket first truck
12:56	21	-	18.6	13.0	7.6	0.9	Start second truck
1:00	25	-	23.0	17.6	11.9	6.0	Tremie pulled after bucket was poured. It came very easy and clean.
1:03		-	23.2	17.7	11.5	5.9	Readings after pulling tremie
1:06	27	-	25.6	20.0	13.6	7.9	
1:15	31	-	28.9	23.5	18.0	11.5	Last bucket
1:30			26.4	21.9	16.8	11.2	During these readings, once or twice the pressures increased suddenly by 2 or 3 lb/sq in. without any apparent reason, but only to decrease afterwards
2:00			23.1	19.7	15.6	10.4	
2:30			18.4	16.2	13.4	9.2	
2:45			16.8	15.0	12.7	9.0	
3:30			16.3	15.2	12.3	9.0	
4:00			16.0	14.8	12.0	9.0	Started to rain heavily. Stopped readings. Cells #3, 4, 5 remained with concrete pressure applied to their faces until the following day.

Slump and Temperature Record:

Time	Truck	Temp. (°F)	Slump (in.)	
11:35	1	92	1 3/4	
11:20	1		1½	6 gals H_2O & superplasticizer
12:30	1	93	9½	
12:42	1	95	9½	
12:50	2	92	2	3 gals H_2O & superplasticizer
12:54	2	94	9½	
1:12	2	94	9	

Compressive Strength:

Date cylinders taken: July 21, 1983

Date cyclinders tested: August 18, 1983

		Unit Weight (lb/cu ft)	f'_c (lb/sq in.)
Truck 1	cyl. 1	148.2	6540
1	cyl. 2	148.2	6400
Truck 2	cyl. 1	147.7	6440
	cyl. 2	148.6	6460

Nominal Mix: CA 1800 lb

FA 1450 lb

Cement 564 lb

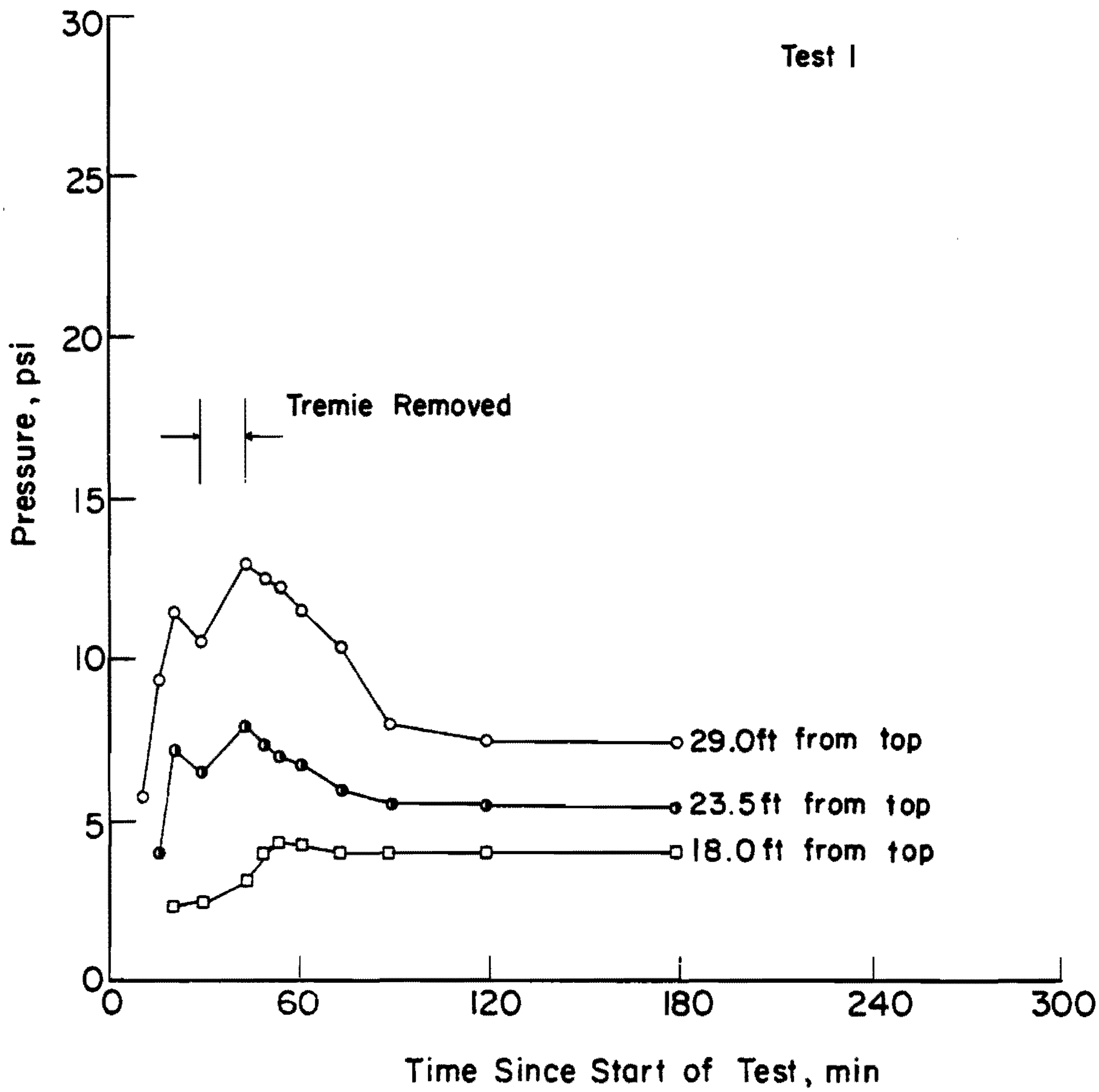
3 oz retarder/sack

Superplasticizer type - Pozzolith 400 N (Master Builders)

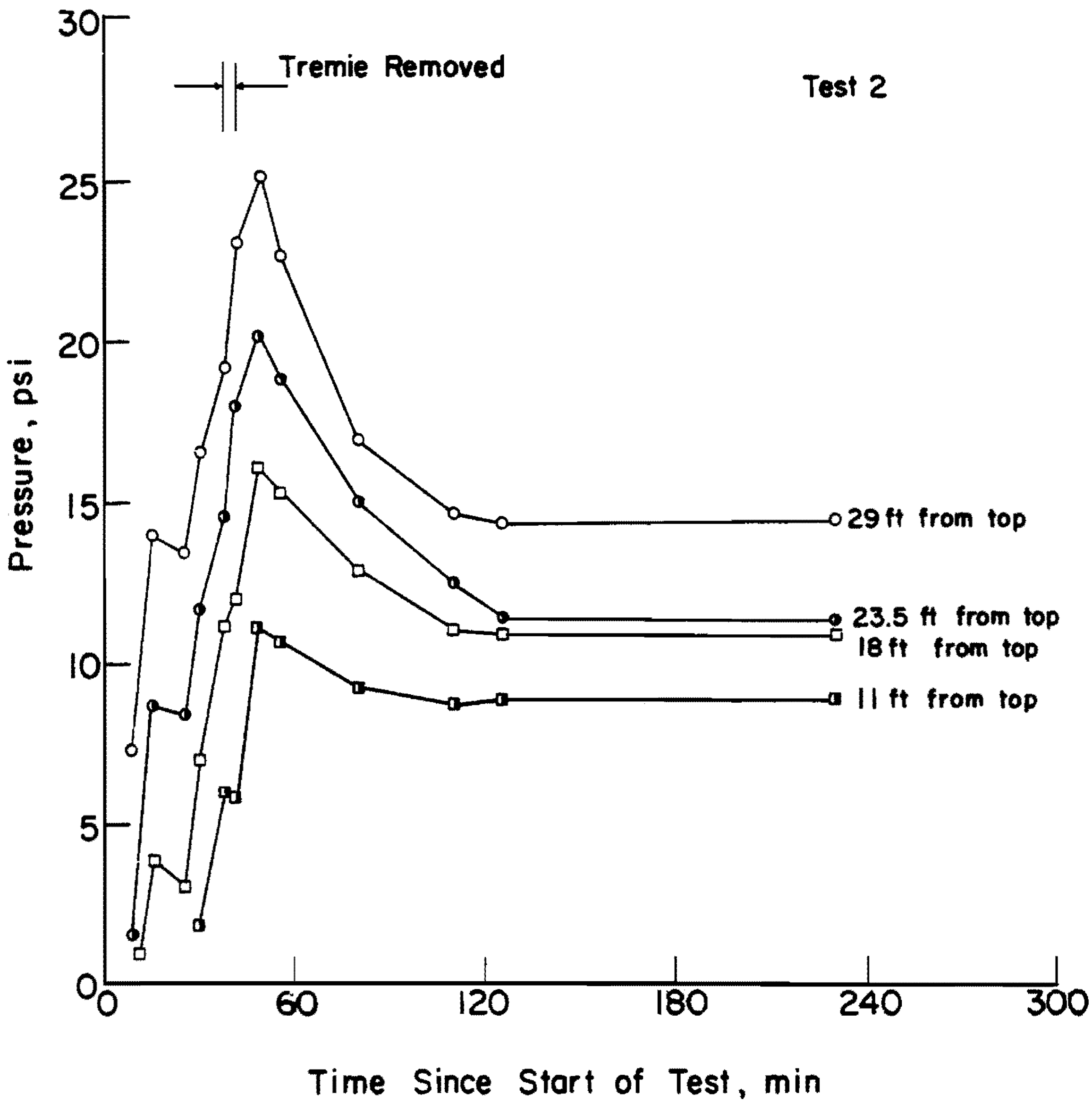
Dosage: Truck 1: 14 oz/sack
Truck 2: 10 oz/sack

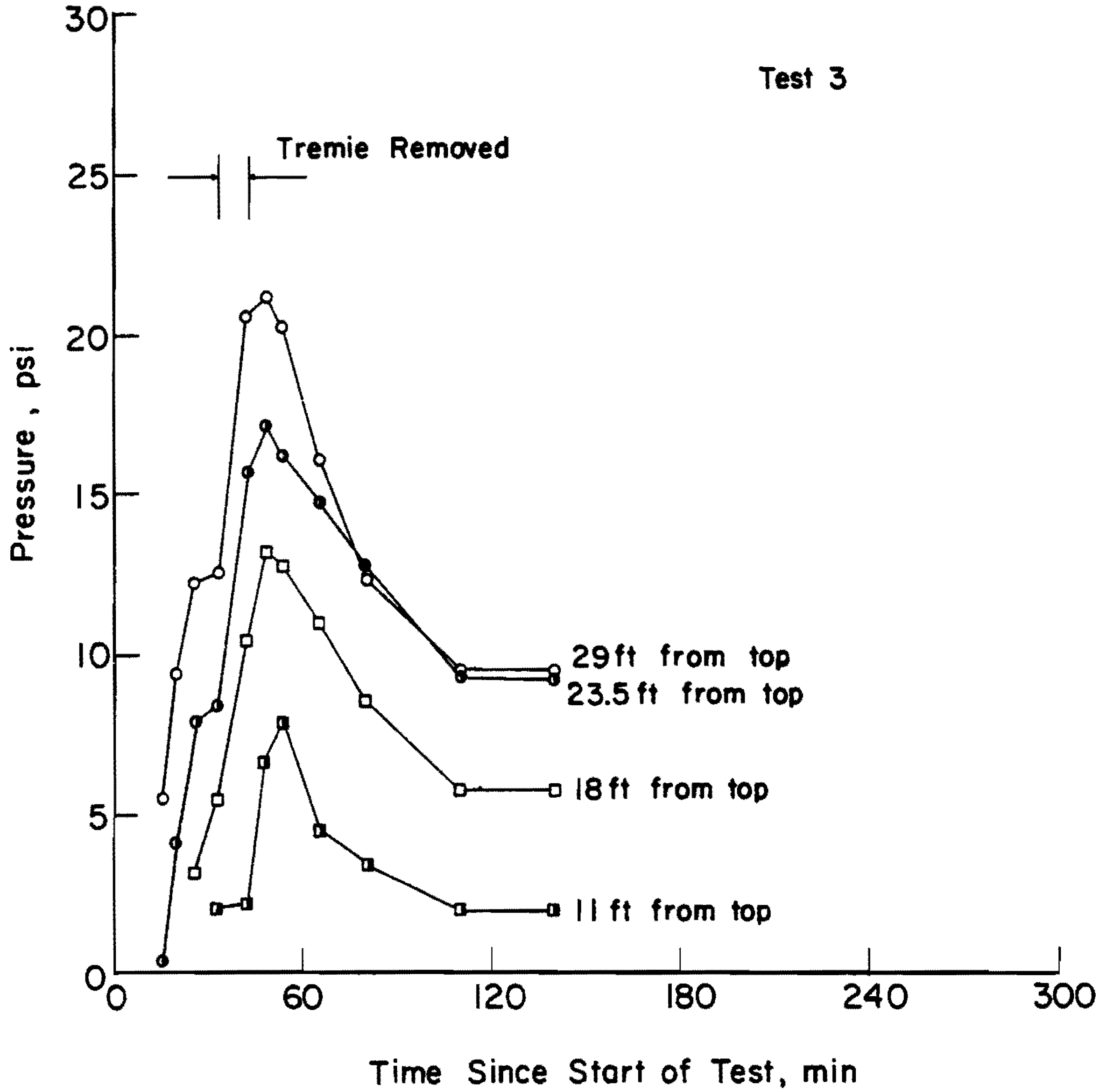
APPENDIX D

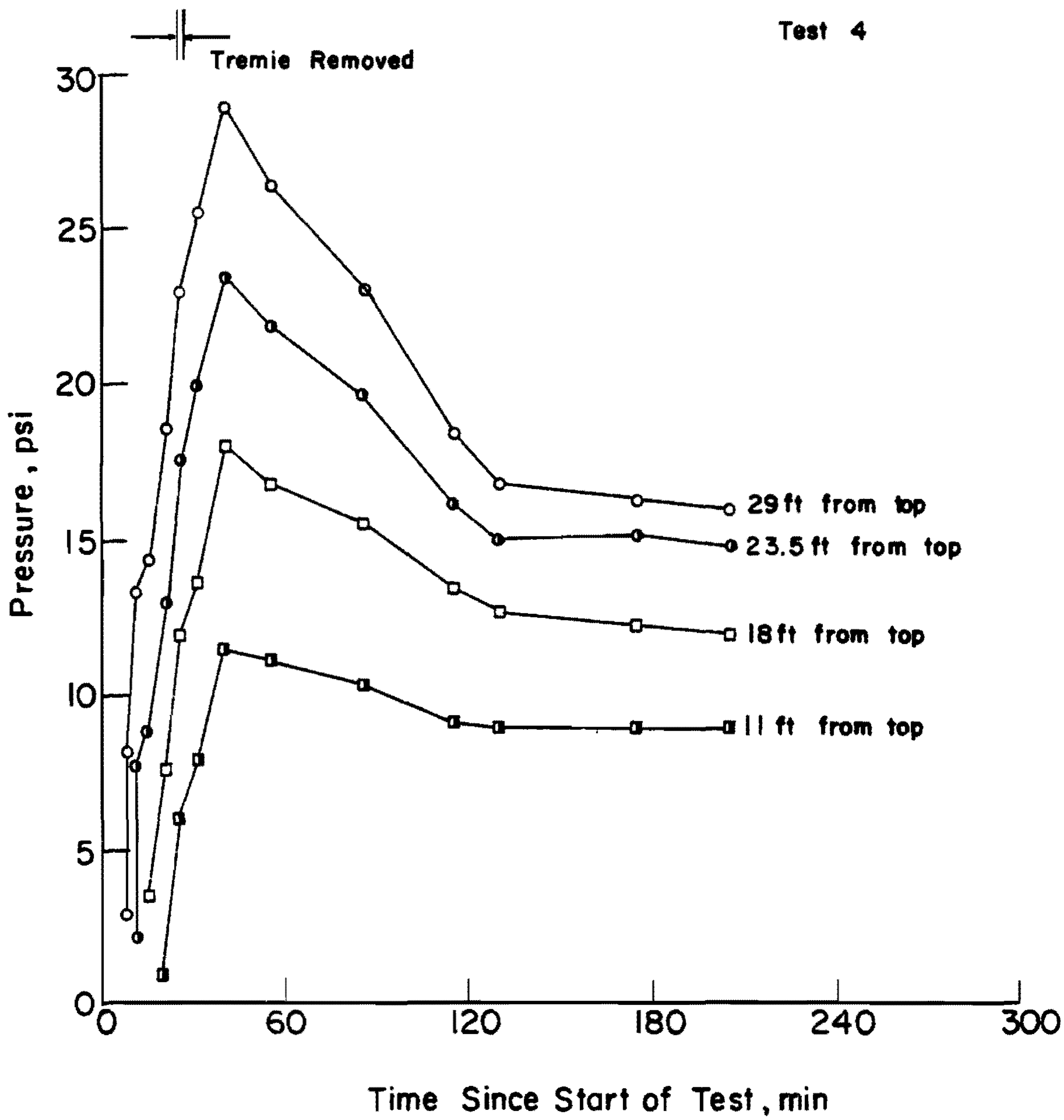
PRESSURES MEASURED VERSUS TIME



Test I
Tremie Removed
29.0ft from top
23.5ft from top
18.0ft from top
Pressure, psi
0
5
10
15
20
25
30
0
60
120
180
240
300
Time Since Start of Test, min


Tremie Removed
Test 2
Pressure, psi
0
5
10
15
20
25
30
0
60
120
180
240
300
Time Since Start of Test, min
29 ft from top
23.5 ft from top
18 ft from top
11 ft from top


Test 3
Tremie Removed
Pressure , psi
0
5
10
15
20
25
30
0
60
120
180
240
300
29 ft from top
23.5 ft from top
18 ft from top
11 ft from top
Time Since Start of Test, min



Test 4
Tremie Removed
Pressure, psi
Time Since Start of Test, min
29 ft from top
23.5 ft from top
18 ft from top
11 ft from top
0
5
10
15
20
25
30
0
60
120
180
240
300