The Influence on an Adjacent Building of Pile Driving for the M.I.T. Materials Center

Influence du battage des pieux pour le centre des matériaux du M.I.T. sur un édifice adjacent

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SUMMARY

This paper presents the results of field measurements of the heave and settlement of Building 10, and the pore water pressures developed under Building 10 as the result of pile driving at an adjacent site. Maximum heaves between 0.02 and 0.03 ft were measured during pile driving and maximum net settlements between 0.02 and 0.03 ft were recorded after completion of pile driving. Values of excess pore pressure head in excess of 40 ft of water were measured. A theoretical analysis based on the "stress path method" correctly predicts a small net settlement but considerably overestimates the actual value.

SOMMAIRE

On présente les résultats des mesures du soulèvement et du tassement prises sur le chantier de l'Edifice 10 ainsi que les pressions de l'eau interstitielle dues au battage des pieux dans un chantier adjacent. Les soulèvements maximums mesurés pendant le battage des pieux furent de 0,02 à 0,03 pieds, et les tassements maximums nets, à la fin du battage, ont atteint des valeurs entre 0,02 et 0,03 pieds. L'augmentation de la pression de l'eau interstitielle atteignit plus que 40 pieds. L'analyse théorique basée sur la méthode des traces des contraintes prédit un petit tassement net mais les valeurs obtenues par cette méthode sont plus grandes que le tassement mesuré.

IN 1962 THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY embarked on the largest expansion programme in its history. This programme involves more than $40,000,000 worth of buildings to be constructed during the period 1962 through 1967. The very poor subsoil conditions on the M.I.T. campus have resulted in a sizable portion of the cost of new M.I.T. buildings going into components below ground surface, a considerable part of construction time being devoted to work below the surface, and the general planning of the campus development being influenced by subsoil conditions.

The Department of Civil Engineering initiated in 1962, a project known as FERMIT (Foundation Evaluation and Research—M.I.T.). The purpose of this project is to attack the foundation problem at M.I.T. using the principles of modern soil mechanics in an attempt to: (1) ensure that future building foundations constructed on the campus will perform satisfactorily; (2) reduce the chances of foundation construction damaging existing structures; and (3) reduce the cost and construction delays associated with foundations. Even though still in its initial stages, FERMIT has obtained enough subsoil, groundwater, and building performance data to aid in the planning of the M.I.T. campus, and in the design and construction of new buildings. This paper presents results obtained from a FERMIT investigation of the effects of foundation construction on adjacent buildings.

The two buildings involved are No. 10 and No. 13 shown in Fig. 1. Building 10, constructed in 1915, is the central structure in the main M.I.T. building complex. Building 13 is the Center for Materials Science and Engineering and was constructed during the period 1963–64.

SUBSOIL CONDITIONS

The geology and properties of the M.I.T. subsoils have been described elsewhere (Horn and Lambe, 1964). Subsoil conditions below Buildings 10 and 13 vary somewhat over the site; an average subsoil profile for the two buildings is shown in Fig. 1. The groundwater table is about 8 ft below ground surface.

The soil of primary concern to the investigation described in this paper is the Boston blue clay between elevations −20 ft and −80 ft. The upper half of this clay layer is overconsolidated while the lower half is normally consolidated. In the normally consolidated portion of the clay, the undrained shear strength is 0.2 to 0.4 kg/sq.cm.; the compression index for virgin compression is 0.4 to 0.6; the liquid limit is approximately 50; the plastic limit 20; and the natural water content about 35.

FOUNDATION CONDITIONS

Fig. 1, which shows plan and section views of Buildings 10 and 13, indicates the types of foundations for the two structures. Building 10 is founded on timber piles (design load of 10 to 12 tons per pile) which terminate in the upper portion of the soft clay. A settlement survey conducted in late 1963 (Horn and Lambe, 1964) shows that Building 10 has settled approximately 8½ in.

The Materials Center is supported on 537 cast-in-place piles driven into the till and/or rock. Each pile has a design load of 70 tons, and consists of a concrete-filled steel shell having a 12¾ in. outside diameter and a ¼ in. wall thickness.

Because of its settlement record, central location, aesthetic importance, and close proximity to the site of construction for the Materials Center, there was considerable concern for the safety of Building 10 during construction. To reduce the chances of disturbing Building 10, the piles for Building 13 were pre-augured to elevation −72 ft, as indicated in Fig. 1.

FIELD INSTRUMENTATION

In order to check on the behaviour of Building 10 during the construction of Building 13, various devices were installed including displacement reference points on Building

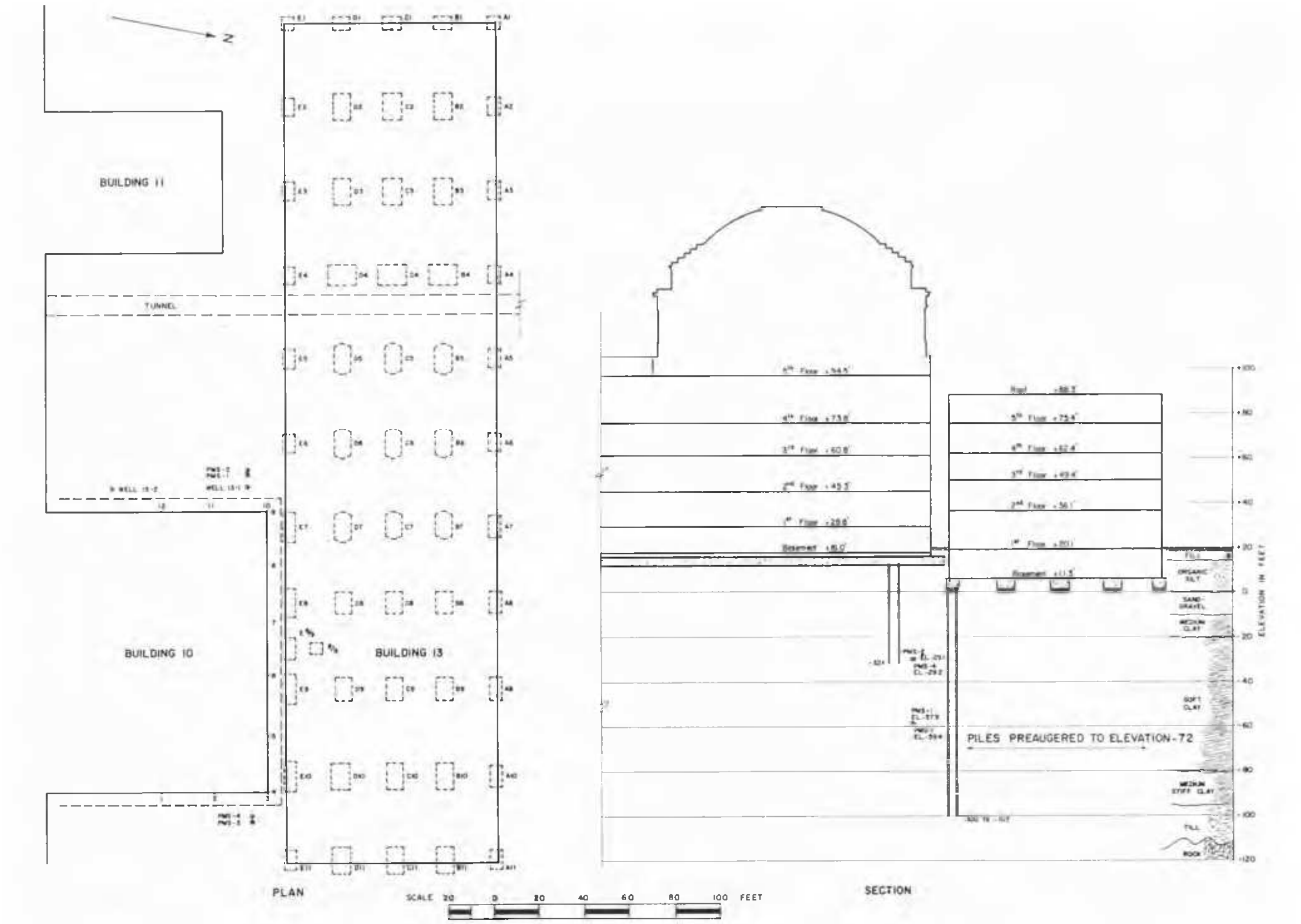

FIG. 1. Plan and section views of Buildings 10 and 13.

10 and on other nearby structures, groundwater observation wells extending into the sand-gravel stratum, and four piezometers in the Boston blue clay. The locations of these various devices are noted in Fig. 1. The devices were observed before and during construction. During the driving of some of the piles immediately adjacent to Building 10, the field pore pressures were recorded around the clock.

PORE PRESSURES AND BUILDING MOVEMENTS

Figs. 2 and 3 present some of the pore pressure, building movement, and observation well data obtained during construction. The phreatic surface, as indicated by Well 13-1, was initially at elevation +11.4. In the middle of September, 1963, the phreatic surface dropped, as shown in Fig. 2, because of shallow dewatering at the Materials Center and deep dewatering at the nearby Student Center.

As can be seen in Fig. 2, extremely high excess pore pressures developed in the two deep piezometers, the maximum value of excess pore pressure head being approximately 48 ft of water. The two shallow piezometers (PMS-2 and PMS-4) did not develop as high excess pore pressures as the deep ones. The maximum excess pore pressure head recorded in the shallow piezometers was only 17 ft of water.

As shown in the upper portion of Fig. 2, the two near corners of Building 10 at first heaved 0.025 ft and then settled. At the end of construction, the maximum net settlement was less than 0.03 ft. Fig. 3 shows pore-pressure data as a function of time for a period of intense pile driving activity.

ANALYSIS OF FIELD DATA

The data in Figs. 2 and 3 show the following very interesting characteristics: (1) very high values of excess pore pressure were developed by pile driving in the clay below the pre-augured zone. (2) pore pressures built up and dissipated at a very rapid rate. (3) Building 10 first heaved and then settled an amount exceeding the heave. (4) building movements were much smaller than would be expected from the large excess pore pressures which were developed.

As would be expected, the closer the pile driving to a piezometer the larger the pore pressure build-up. There was, however, significant pore pressure built up at distances of as much as 100 ft away from the pile driving.

An accurate theoretical analysis for pore pressures and building movements cannot be made because of the lack of data. Having pore-pressure data on only four piezometers, one cannot develop accurately the spatial pattern of excess pore pressures at any given time. Further, the influence of the structural rigidity of Building 10 on the settlements cannot be accurately determined. In spite of these difficulties, considerable insight into the behaviour of Building 10 and its foundation can be obtained by employing the "stress path method" proposed by Lambe (1964).

The stress path method consists of estimating the effective stress path for an "average" element in the compressible soil

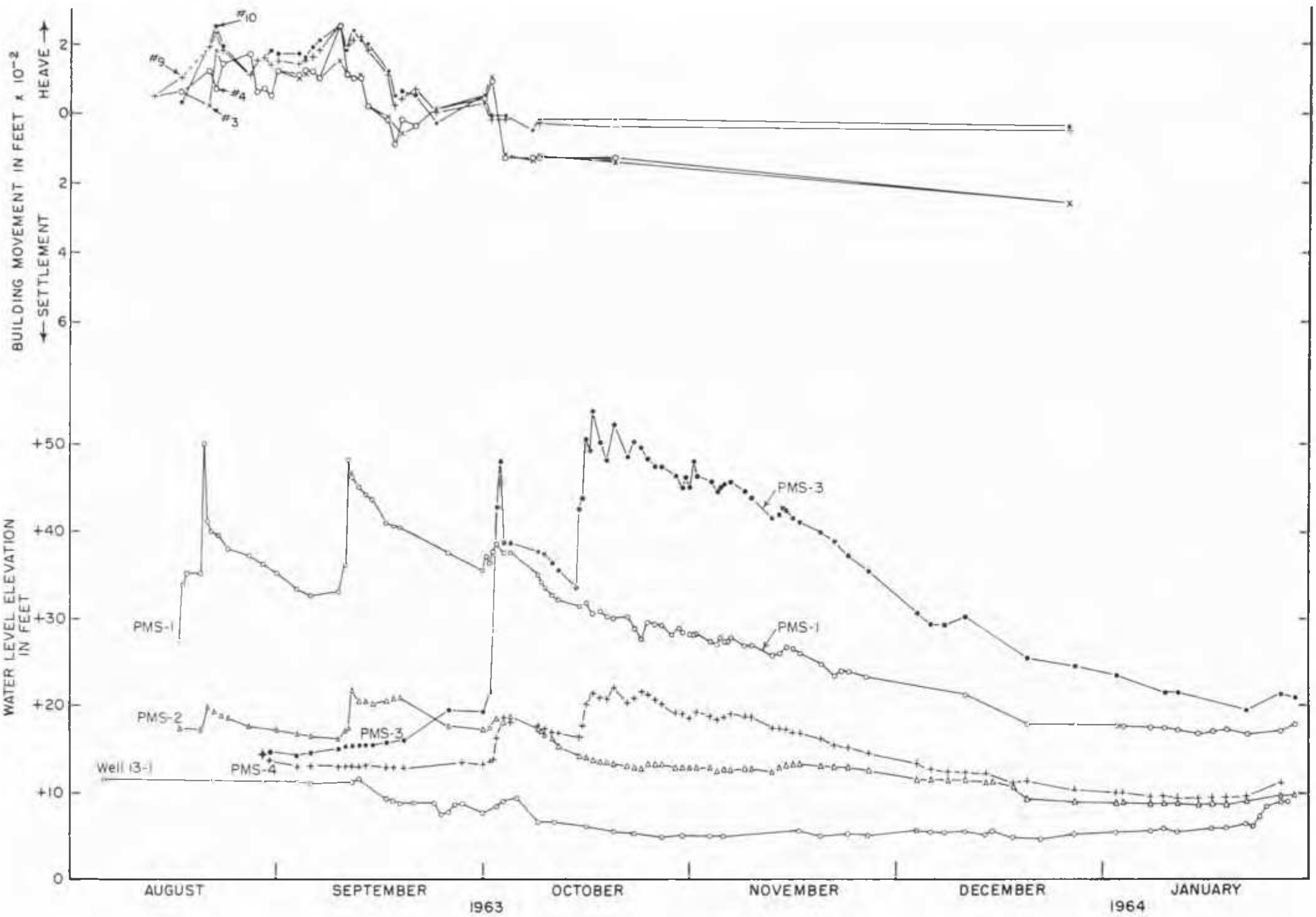

FIG. 2. Pore pressures and movements, Building 10.

layer, running a laboratory test on a sample of soil from the compressible layer such that the loading follows the stress path estimated for the field element, and using the laboratory measured value of vertical strain to estimate the building heave and settlement. Table I gives the five steps in the

TABLE I. STRESS-STRAIN HISTORY OF A SOIL ELEMENT AT ELEVATION −70 FEET

Step number	Description
1	Consolidation of element to overburden stresses under K_0 conditions. Initial stresses: $u_s = 2.49$ kg/sq. cm.; $\sigma_{vo} = 4.99$ kg/sq. cm. $\bar{\sigma}_{vo} = 2.58$ kg/sq. cm.; $\bar{\sigma}_{lo} = 1.42$ kg/sq. cm.
2	Construction of Building 10. Application of $\Delta\sigma_1 = 0.264$ kg/sq. cm. and $\Delta\sigma_3 = 0.01$ kg/sq. cm. in undrained loading.
3	Consolidation of clay under weight of Building 10. Total stress held constant and pore pressure dissipated.
4	Application of lateral stresses by driving piles. Total vertical stress held constant. Lateral stress increased sufficiently to give a field measured excess pore pressure of 1.28 kg/sq. cm.
5	Consolidation of clay under applied stresses with lateral dimensions of clay element held constant (i.e., K_0 conditions).

stress-strain history of a soil element at elevation −70 ft. Step 1 consists of the consolidation of the element during formation of the clay layer. Step 2 consists of the undrained loading of the soil element during the construction of Building 10. Step 3 consists of the consolidation of the element under the stresses imposed by Building 10. Step 4 consists of an increase in total lateral stress at constant total vertical stress until the pore pressure in the element equals that measured by the field piezometers. Step 5 consists of the dissipation of the pore pressures developed by the pile driving. It seems reasonable to assume that during Step 5, the lateral dimension of the soil element might remain approximately constant.

Fig. 4 presents the stress path for the laboratory test run on a sample of clay taken from the upper portion of the normally consolidated clay from a location near Building 10. The steps noted in Fig. 4 are those described in Table I. As can be seen in Fig. 4, the lateral total stress had to be built up to a value exceeding the vertical total stress in order to develop the field measured pore pressure. The vertical strain was −0.4 per cent during Step 4 and +0.5 per cent during Step. 5.

Fig. 5 shows what is considered to be a reasonable estimate of the distribution with depth of excess pore pressure at the location of piezometers PMS-1 and PMS-2, i.e. the northwest corner of Building 10. The estimated pore-pressure distribution curve was replaced with an "equivalent" uniform excess pore pressure head of 35 ft of water over a stratum of clay going from −45 ft elevation to −80 ft. Using these numbers and the results of the laboratory tests (the strains were reduced in proportion to the pore pressure head of 35

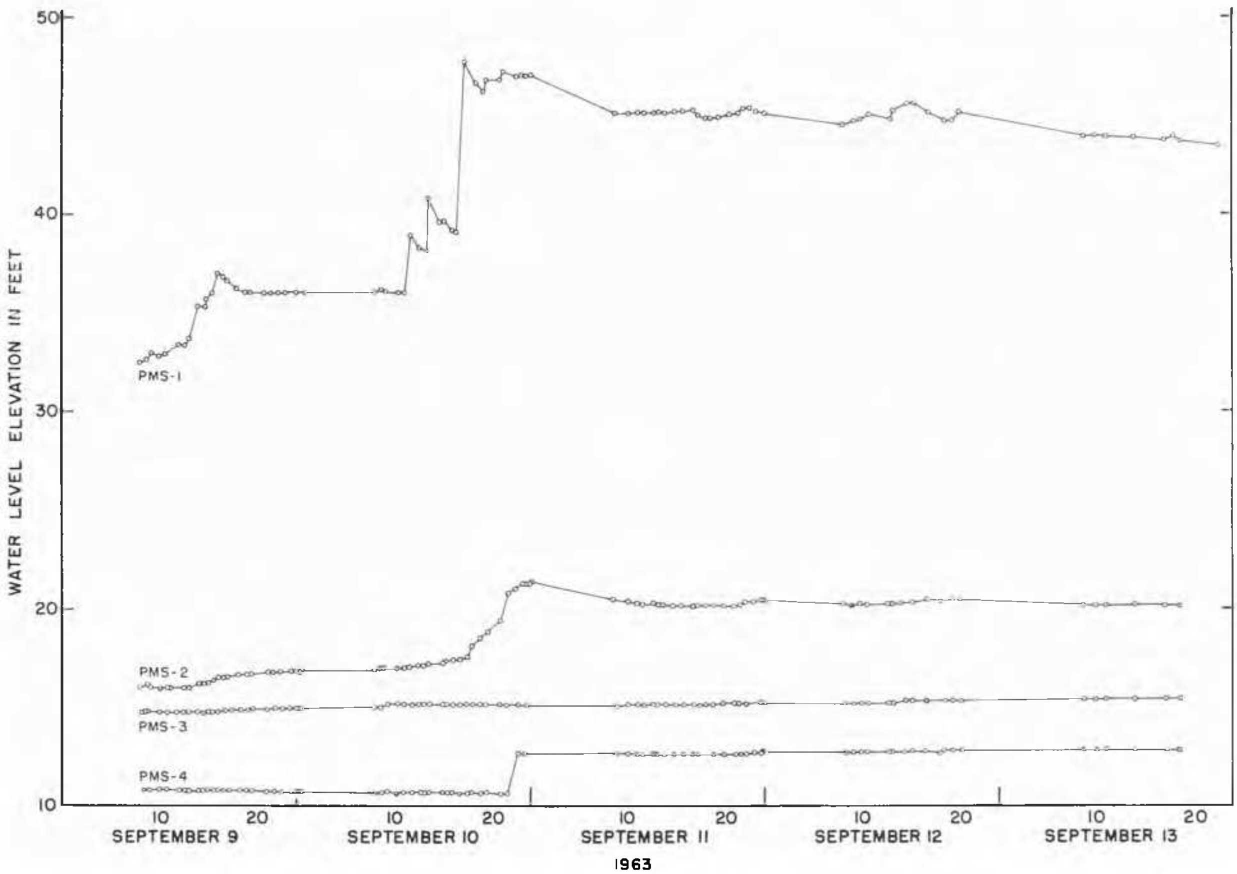

FIG. 3. Pore pressures, Building 10, September 9–13, 1963.

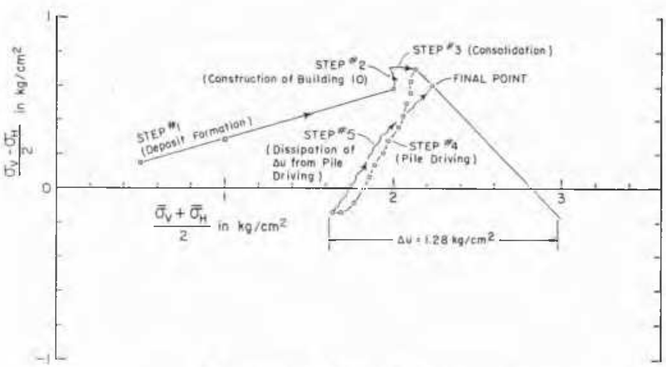

FIG. 4. Stress path for laboratory test.

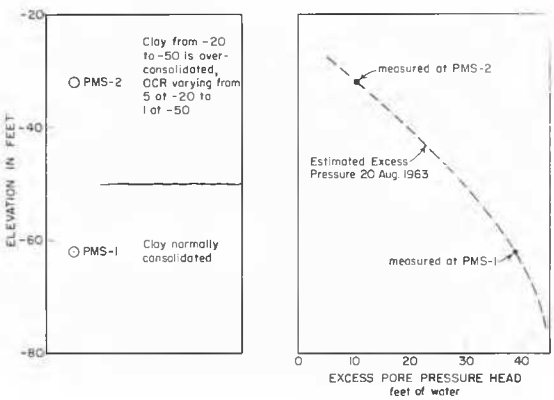

FIG. 5. Vertical distribution of excess pore pressure head.

divided by the head of 44 ft used in the laboratory test), the calculated values shown in Table II were obtained. A comparison of the calculated and measured values shows that the analysis correctly predicts small movements but considerably overestimates the actual values. The analysis also predicts a net settlement, as in fact occurred.

The three major reasons for the calculated movements of Building 10 being much larger than the measured ones are thought to be the following.

TABLE II. MOVEMENT OF NW CORNER BUILDING 10

	Calculated value	Measured value
Heave of building	0.14 ft	0.025 ft
Settlement of building	0.18 ft	0.030 ft
Net settlement	0.04 ft	0.005 ft

1. Soil disturbance during sampling resulted in a great change in the stress-strain modulus and compressibility (Ladd and Lambe, 1963; and Ladd, 1964).

2. The structural rigidity of Building 10 and of the buildings into which it frames, and the rigidity of the overlying foundation soils, especially the stratum of sand-gravel, aided in reducing the movements which might have occurred. A measure of the effect of building rigidity on the magnitude of heave is given by the performance of the steam tunnel which runs across the construction site. A point on the top of the steam tunnel at the south side of the site heaved over one

inch, a value which is about three times the maximum heave measured on Building 10.

3. The pore pressures measured just outside of Building 10 were higher than those under the structure.

Even though the calculated movements of Building 10 turned out to be much larger than those which occurred, they were most valuable to the engineers controlling construction of the Materials Center. The analysis showed that, in spite of the extremely high pore pressures developed under Building 10, only a small settlement should be expected. The analysis further indicated that the rate of settlement would be very rapid* as it turned out to be.

ACKNOWLEDGMENTS

A number of the authors' assistants participated in the investigation described in this paper. Mr. Walter R. Beckett, Mr. Alton P. Davis, and Mr. Richard S. Ladd helped with the field measurements. Mr. William A. Bailey ran the laboratory tests. The encouragement and support by the M.I.T. Office of Physical Plant is gratefully acknowledged. The help of Mr. Philip A. Stoddard, Administrative Vice President of the Institute, and Mr. William R. Dickson, Administrative Assistant to the Director of Physical Plant, has been particularly effective.

*The value of the coefficient of consolidation, c_v, estimated from Step 5 (Fig. 4) was approximately seven times the value obtained from a standard oedometer test on a normally consolidated specimen of the clay.

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