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EQUIPEMENT EN INSTRUMENTS POUR L'ETUDE DU PHENOMENE DE FRICTION NEGATIVE AFFECTANT LES LONGS PILIERS ENFONCES DANS L'ARGILE MARINE SUPPORTANT L'AUTOROUTE DU QUEBEC

SOMMAIRE

Le passage supérieur de la grande route 41 en dessus de l'autoroute, à Berthierville, reposera sur des piliers de 39 pouces, munis d'une extrémité composite et enfoncés jusqu'à la roche de fond, à 270 pieds de profondeur. Les charges découlant des phenomènes de friction négative causés par la consolidation en profondeur des strates d'argile marine pourront atteindre les deux tiers de la résistance prévue des piliers. En vue d'étudier le phénomène de friction négative, on munit d'instruments de mesure deux piliers prototypes et on installa un pilier d'essais mobile. La série d'instruments comprenait des jauges de tassement, des piézomètres, des jauges de déformation, des télé-extensomètres, des thermocouples et un inclinateur à tube. L'auteur donne les détails de l'ensemble et rend compte de l'efficience des appareils.

INSTRUMENTATION FOR NEGATIVE SKIN FRICTION STUDIES ON LONG PILES IN MARINE CLAY ON THE AUTOROUTE DU QUEBEC

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M. Bozozuk* and P.M. Jarrett*

ABSTRACT

The overpass at Highway 41 on the Quebec Autoroute at Berthierville will be supported by 39-in. diameter composite end-bearing piles founded on bedrock 270 ft below the ground surface. The imposed loads from negative skin friction caused by the consolidation of the marine clay subsoils may reach two-thirds of the design capacity of the pile. To study the phenomenon of negative skin friction, two prototype piles were instrumented and a floating test pile installed. The instrumentation included settlement gauges, piezometers, deformation gauges, strain meters, thermocouples and a slope inclinometer tube. The design details of the instrumentation are given and their performance discussed.

The road building program of the Quebec Autoroute Authority includes a limited access toll road along the north shore of the St. Lawrence River. This route involves construction on the deep beds of compressible marine clay that exist throughout the St. Lawrence Valley. The first stage of construction lies betweeen Montreal and Berthierville. At Berthierville the autoroute crosses a railway spur line and Highway 41 within a few hundred yards. Two bridge structures are necessary, but both are integrated into one long fill that provides the bridge approaches and interconnection.

Such a fill on compressible clay has two inherent problems, the first being its stability against a sliding failure and the second, the settlements that occur in the subsoil owing to the weight of fill. The latter problem in this case could involve about 10 ft of settlement.

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Naturally such movements cause great difficulties for the structural designer, and for the Highway 41 overpass the consulting engineers decided that the most economical foundation method would be to drive end-bearing piles 300 ft to bedrock. Even allowing for their tremendous length, the major unknown factor in the design of such piles is the magnitude of negative skin friction.

The compressible clay subsoil, consolidating under the weight of the granular embankment, transmits an axial load to the piles through skin friction. It is the magnitude and distribution with depth of this load that is unknown and is so important to the economical design of the piles. The Soil Mechanics Section of the Division of Building Research, National Research Council, with the cooperation of the owner, designer, contractor and soils consultants for the project, designed and carried out an extensive instrumentation program to study the problem. To evaluate negative skin friction as related to piles, it is necessary to measure or define the stresses and strains occurring in the major elements of the system, i.e., the piles and the soil. For this project, therefore, the aim was not only to measure the loads developing on the prototype piles but also to examine the nature of the skin friction in the hope of improving later design practice. This paper describes the instrumentation that was installed during the period February to July 1966.

THE BRIDGE

The bridge consists of a deck simply supported by two abutments (Figure 1). Each of the two abutments is carried by six 39-in. diameter composite steel and concrete piles (Figure 2). The piles, consisting of a steel shell filled with reinforced concrete, are designed to carry a maximum load of 2,000 tons, about two-thirds of which is estimated to be imposed by negative skin friction. This percentage again emphasizes the importance of negative skin friction.

INSTRUMENTATION LIMITATIONS

In order to understand the problems of measuring stresses and strains in the prototype piles it is desirable to be conversant with the construction procedure used (1, 2). In brief, a 39-in. diameter 1-in. thick steel shell was first driven to bedrock in 40- to 60-ft sections. This was achieved using both a drop hammer and a Benoto machine which drives the shell by steady downward pressure and back and forth rotation. The interior of the shell was cleared out during driving using the Benoto grab. At bedrock a socket about 8 to 10 ft deep was made. The reinforcing cage was preformed in sections up to 65 ft long, and lowered into place. Concrete was then placed through an 8-in. diameter tremie pipe inserted through the centre of the reinforcing cage.

As may be imagined, these construction techniques immediately preclude several desirable approaches for measuring stresses and strains in the piles.

(a) The Benoto driving machine utilizes clamps wrapped around the outside of the shell. This prevented any attachments being made to the outside of the shell.

(b) Due to the grab method of cleaning and the close tolerance of the reinforcing cage, no projections were feasible on the inside of the steel shell.

(c) The central area of the pile had to be left clear for the tremie pipe to be installed.

These factors dictated that the instrumentation had to be installed as an integral part of the reinforcing cage. It also meant that no attempt could be made to take direct measurement of soil properties through the pile shell. This is unfortunate because they would have been extremely valuable. It was found impractical to take direct measurements of stresses or loads in the piles, and the methods used are based on strain measurements from which stresses must be calculated. For this latter step some value of the pile modulus must be found. This is difficult, because of the complexity of the relation between steel, concrete and soil. For this reason a simple 12-in. diameter hollow steel shell test pile was installed and instrumented at the site. Measuring techniques used in all these piles are described later in this paper.

INSTRUMENTATION OF PROTOTYPE TEST PILES

Of the twelve composite piles the two nearest the highway centrelines were selected for study (Figure 2). The 1-in. steel shell of B-2 was extended to the surface of the fill, whereas all the others, including that for B-3, were terminated at elevation +27 ft, the final elevation of the placed concrete.

Test Pile B-2

The instrumentation in B-2 consisted of deformation gauges (R-1 to R-5), three electrical strain meters (E-1 to E-3) containing thermocouples, and a differential movement gauge between the steel shell and the concrete. The deformation gauges and strain meters were located so that they coincided with changes in the steel reinforcing or with the cadweld joints as shown in Figure 3.

Deformation Gauges

As the loads are transmitted by skin friction, five deformation gauges were installed to measure the behaviour of the pile with time along its length. Each deformation gauge consists of two concentric iron pipes (Figure 4). The outer casing was cast into the concrete so that it became an integral part of the pile. The inner galvanized pipe stood free of the pile and acted as the stress-free reference point. The relative movement between the pipes provides a measure of the axial strain in the pile over the length of the gauge.

The 1-in. diameter pipe casings were installed between the vertical steel reinforcing bars onto the inside of the preassembled spiral cage reinforcing sections, then lowered into position. As the sections were formed in lengths up to 65 ft, the pipes were loosely clamped with "U" bolts and colour coded for identification. When the sections were hoisted into position for lowering into the steel shell, the pipes were connected, properly aligned, and firmly clamped into position (Figure 5). These operations gave little or no interference to the contractor, and at the same time allowed an unobstructed interior for the subsequent concreting operation.

Following the concreting operation, a pre-measured amount of SAE 20 oil was poured into the 1-in. pipes and the interior pipes were installed. The lower ends of the 1-in. iron pipes were fitted with machined caps so that they would mate properly, with the inside of the machined caps located at the bottom of the 1-in. pipes. These were installed in lengths not exceeding 10 ft, with the joints carefully spaced so that they did not coincide with those in the 1-in. casing. The couplings were machined to a loose sliding fit, with a clearance of 0.05 in., and their 10-ft spacing provided sufficient points of lateral support for the interior pipe, so that, together with the buoyant effect of the oil, the tendency for buckling from column action was overcome. This geometry served another useful function. The piston effect created by the couplings as they passed into the oil was sufficient to support the weight of the column and permitted it to sink slowly into position. The importance of this is evident when one realizes that the weight of the interior column of 1-in. pipe for R-5 is about 230 lb, too heavy to be supported by hand.

Once the interior rod was installed, the two pipes were cut off at the same level at the top. The brass pedestal (Figure 4) was driven carefully into the interior pipe, and the brass tube fitting mounted around it on the 1-in. pipe casing. The assembled unit formed the deformation gauge shown in Figure 4. Readings to ± 0.0001 in. are made with an Ames dial or micrometer gauge modified for the purpose as shown in Figure 6.

Strain Meters

The strain meters consist of four 120-ohm bonded strain gauges mounted inside a $1\frac{1}{2}$-in. diameter black iron pipe as shown in Figure 7. When cemented into position with the long axis parallel to that of the pile, any strains experienced by the concrete around the meters produce changes in resistance that can be measured at the ground surface. Three strain meters containing copper-constantan thermocouples were installed, one at the bottom, one midway along the length of the pile, and one near the top (Figure 3) to supplement the results obtained from the deformation gauges. When the elastic properties of the composite piles are known, the strain meters can be used to determine the actual loads at the three points in the pile.

The strain meters were assembled and calibrated in the laboratory prior to their installation. They were bolted onto the spiral reinforcing at pre-determined locations and lowered into the steel shells simultaneously with the deformation gauges. The electrical leads were protected by a 3/8-in. I. D. liquid-tight flexible conduit, which was slowly unrolled and fastened to the outside of the cage as the latter was lowered into position. One-in. diameter steel reinforcing bars were also fastened to the outside of the cage beside the flexible conduit to protect it from being crushed against the side of the steel shell. The complete unit was filled with No. 10 Voltaso transformer oil, an oil with high electrical resistance, to protect the electrical parts from moisture. A four-conductor shielded cable, type 8723, with a high resistance to the oil, was used to connect the four SR4 type C-6-141 strain gauges with the surface.

In operation, pairs of leads from the strain meter were used with pairs of leads from compensating (dummy) gauges at the surface, forming a two-arm bridge with 240 ohms on each arm. The units were read with a Baldwin BLH Strain Indicator, Type 20, correcting for the resistance and capacitance in the leads. The thermocouples were read with a potentiometer, using an ice bath for the reference junction.

Differential Movement Gauge

A differential movement gauge was installed at the top of the pile to measure the relative movement that may develop between the steel shell and the encased concrete. It consisted of two stainless machined bars, positioned one over the other. One bar was mounted on the steel shell, the other on the 1-in. pipe casing of deformation gauge R-5. Relative movements between the two were measured with the micrometer gauge to ± 0.0001 in.

Test Pile B-3

In test pile B-3, two deformation gauges, R-6 and R-7, and a PVC slope inclinometer tube were installed to depths shown in Figure 3. Because of the requirement that the interior area of the pile be kept clear for concreting purposes, they too were mounted on the inside of the spiral reinforcing cage between the vertical steel reinforcing bars, following the same procedures as for B-2.

Slope Inclinometer

The slope inclinometer tube, a semi-rigid polyvinyl chloride pipe with an internal diameter of $1\frac{1}{2}$ in. was set to a depth of 55 ft below the top of the concrete. Its slope was measured at 1-ft intervals of depth with a strain gauge inclinometer (3) to an accuracy better than ± 0.02 deg using the BLH Strain Indicator Type 20. The purpose of this instrumentation is to observe any lateral movements that may occur in the subsoil when the cut through the embankment is made for Highway 41.

12-IN. DIAMETER TEST PILE

A hollow, open-ended, 12-in. diameter by $\frac{1}{4}$-in. wall thickness circular steel pipe pile was instrumented with eight deformation gauges and driven through the embankment to a depth of 160 ft below the top of the fill at the location shown in Figure 2. Its purpose is to study the development of skin friction by observing the compression due to the loads imposed on the floating pile from both negative and positive skin friction.

The deformation gauges employed were similar to those described previously, but of smaller diameter. They were mounted on the outside of the test pile and spaced in such a way that the deformations could be observed at 20-ft intervals of length. To prevent any tendency for the pile to bend during the pile driving operation the gauges were mounted in pairs diagonally opposite each other in casings of equal length. In one, the inner rod extended for the full length of the casing; in the other, the inner rod terminated at an intermediate plug providing a gauge of the proper length. The details of the component parts used in the gauge are shown in Figure 8. The dial gauge and micrometer were used to measure the deformations.

The contractor installed the pile in five sections, with individual sections butt welded together. The pile was aligned vertically with two transits set 90 deg apart prior to butt welding and driving. After each section had been driven, it was cleaned out for its full length, eventually leaving a clean open-ended pile 160 ft deep.

The casings of the deformation gauges were welded to each section of pile. They were arranged symmetrically around the outside, 45 deg apart as shown in Figure 9, that is, at a spacing of exactly 5 in. measured around the circumference. It was most important that the gauges should be aligned straight and parallel to the long axis of the pile. Each pile section was first placed horizontally on the ground and a carpenter's level and square used to locate the highest point on the circumference at each end. A chalk line was then used to mark the position for the gauge. Once this line had been established, the positions of the other gauges were measured off along the circumference and marked with the chalk line. The 3/4-in. pipes were positioned over these lines and welded continuously to the pile in an alternating manner, shown in Figure 8, and colour coded to facilitate mating of adjacent sections. It was very important to ensure that the pipes were clean and clear of burrs. They were joined together on the pile with machined sleeves $1\frac{1}{2}$ in. long (Figure 9) welded in place. These pipes were kept 2 to 3 in. shorter than the pile sections so that they would not interfere with the butt welding and driving of the pile sections. After the butt welding of the pile sections the gauge casings were joined with the longer 8-in. sleeves, which were also welded in place.

PIEZOMETERS AND SETTLEMENT GAUGES

To study the rate and magnitude of consolidation in the subsoil in the vicinity of the test piles, "Geonor" type piezometers (4) and settlement gauges were installed at the locations shown in Figure 2. Piezometers P-1, P-2, P-3, P-4, and P-8 were located on the centreline of the autoroute. P-4 was adjacent to B-2 and P-5 was located midway between the four large piles, just east of B-2. Piezometers P-6 and P-7 were temporary ones installed about 475 ft east of the autoroute to determine the pore pressure gradient outside of the influence of the fill. The depths of the piezometer tips below original grade are given in Table I.

Ten settlement gauges (5) and a bench mark (6) were installed at the locations shown in Figure 2 to the depths given in Table I. Gauges S-1 to S-5 were located on the centreline of the autoroute, gauges S-6 and S-7 were adjacent to B-2, and gauges S-8 and S-9 at the centre between the four large piles just east of B-2. These four gauges were installed to provide information regarding the soil behaviour adjacent to the piles. Settlement gauge S-10 was installed down the centre of the 12-in. diameter test pile. Because of its great depth (175 ft below the top of the fill) and an unsupported length of 160 ft, special centring wheels (Figure 10) were installed at 20-ft intervals along its length to keep it centred within the pile. From a measure of the relative movements between this gauge and the top of the pile (Figure 10), and knowing how much the pile has compressed, the relative movement between the bottom of the pile and the surrounding soil may be obtained. Settlements are measured by running precise level surveys from the bench mark located 371 ft from the centreline of the autoroute to the individual settlement gauges. At the same time, the levels are run to a steel lug welded to the steel casing of B-2 in order to detect any vertical compressions that may be taking place.

PERFORMANCE

For the period April to November 1966 during which the instrumentation has been in service, its performance has been most satisfactory. The settlement gauges and piezometers are providing a good record of the magnitude and rate of consolidation of the marine clay subsoils beneath the embankment, and are expected to continue to do so for the life of the project. The deformation gauges on the 12-in. diameter steel test pile have proved extremely successful and this seems to be an excellent technique for studying pile deformations. Similarly, the deformation gauges and strain meters in the large 39-in. diameter composite piles are working well, although the magnitude and variation of the loads imposed by negative skin friction cannot yet be determined. This is due in part to the anticipated difficulty in determining the deformation modulus of the pile, and to obtaining the zero position for the instruments. The latter was obscured in the initial stages of construction by expansions in the pile caused by hydration of the cement, but the pile now appears to be reacting pr-perly to the loading from negative skin friction.

At this stage it is premature to publish any results concerning the loads. Nevertheless, this report should be a practical guide for anyone faced with the problem of instrumentation for a study on negative skin friction.

ACKNOWLEDGEMENTS

The help and cooperation of the Quebec Autoroutes Authority, Labrecque, Vezina and Associates, the consulting engineers for the bridge, Terratech Ltd., the soils consultants for the project, and Petrifond Ltd., the piling contractor, is gratefully acknowledged. The continued cooperation of all groups enabled the instrumentation to be successfully installed.

The authors also wish to thank their colleagues in the Soil Mechanics Section of the Division of Building Research, all of whom contributed at some time to the design or installation of the instrumentation.

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TABLE I

DEPTH BELOW ORIGINAL GRADE OF PIEZOMETERS AND SETTLEMENT GAUGES

Instrument			Depth Below Original Grade, ft
Piezometer		P-1	46
"		P-2	60
"		P-3	20
"		P-4	55
"		P-5	55
"		P-6	30
"		P-6 (Pushed)	70
"		P-7	10
"		P-7 (Pushed)	50
"		P-8	87
Settlement	Gauge	S-1	100
"	"	S-2	60
"	"	S-3	40
"	"	S-4	20
"	"	S-5	+1
"	"	S-6	10
"	"	S-7	61.5
"	"	S-8	16
"	"	S-9	61
"	"	S-10	145

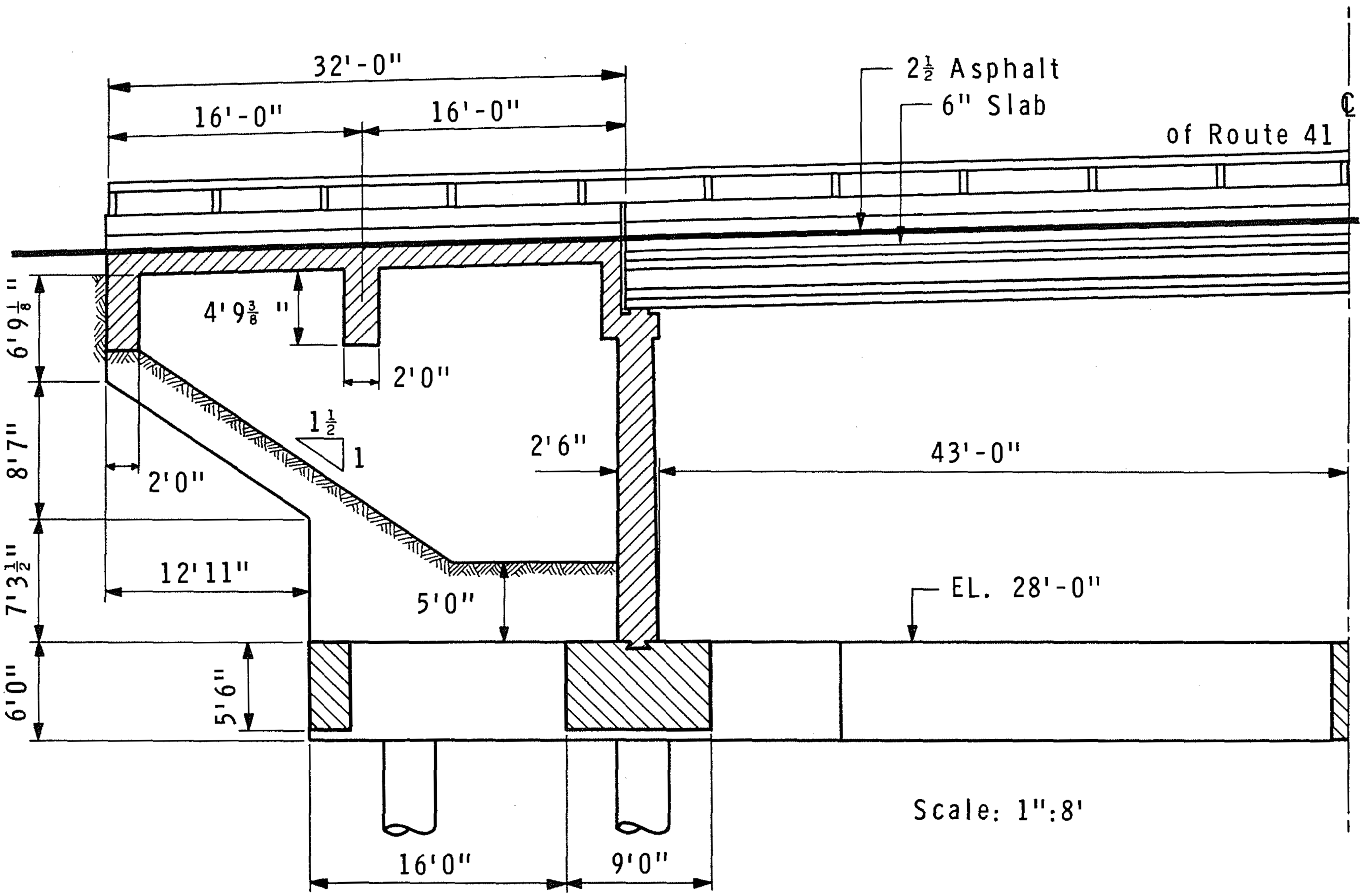
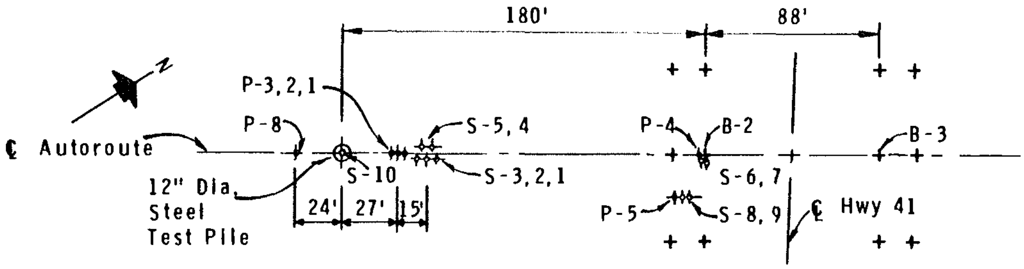


FIGURE 1 LONGITUDINAL HALF SECTION

BR 3642-2



PLAN

NOTE:

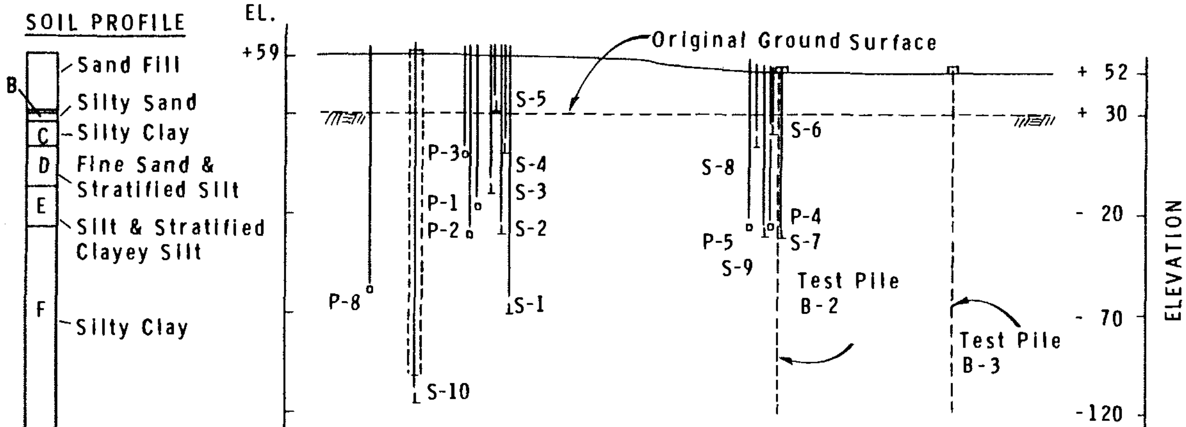
1 + 39" Dia. End Bearing Piles

2 • Piezometers (P)

3 ∘ Settlement Gauges (S)

4 BM Located 371' East of ℄ of Autoroute on N.E. Shoulder of Hwy 41

5 P-6, 7, Located 475' East of Autoroute and 110' South of Hwy 41



ELEVATION

NOTE:

1 ▫ Piezometers (P)

2 ⊥ Settlement Gauges (S)

3 B-2, in Bedrock at El. -248'

4 B-3, in Bedrock at El. -247'

FIGURE 2

FIELD INSTRUMENTATION FOR NEGATIVE SKIN FRICTION

BR 3790-1

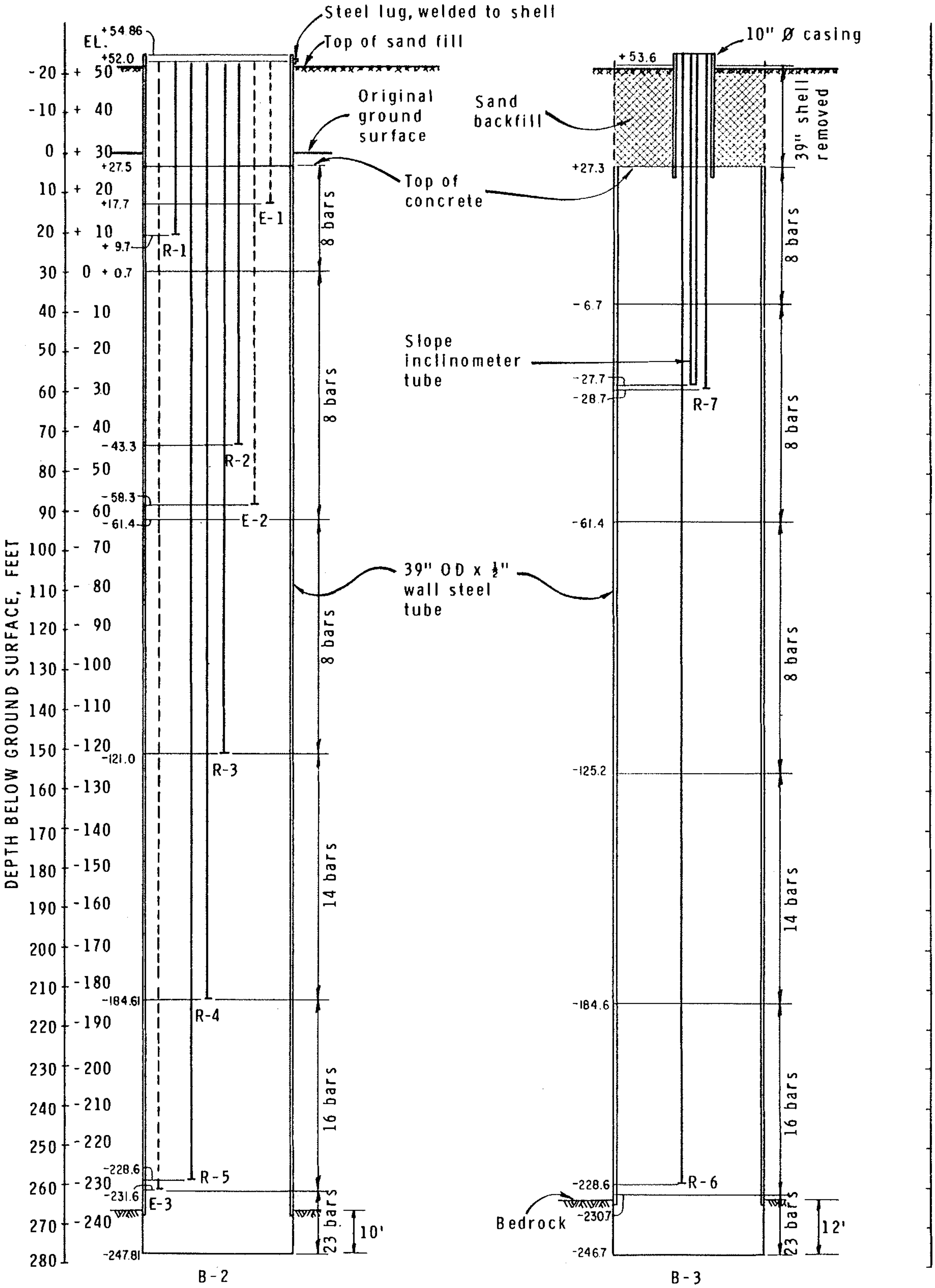


FIGURE 3

INSTRUMENTATION OF PILES B-2 AND B-3

BR 3790-2

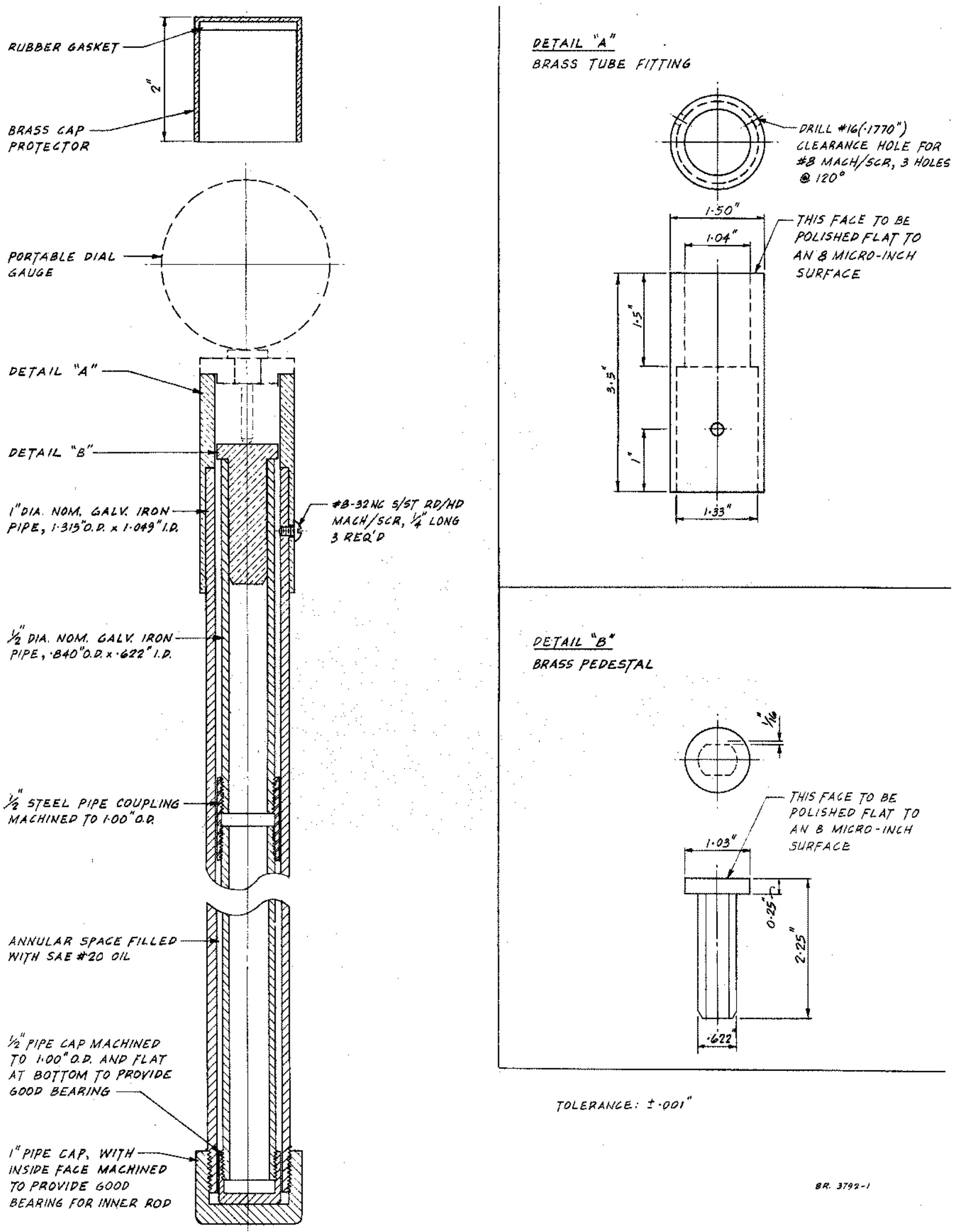


FIGURE 4   DEFORMATION GAUGE DETAILS FOR 39" DIA. COMPOSITE PILE

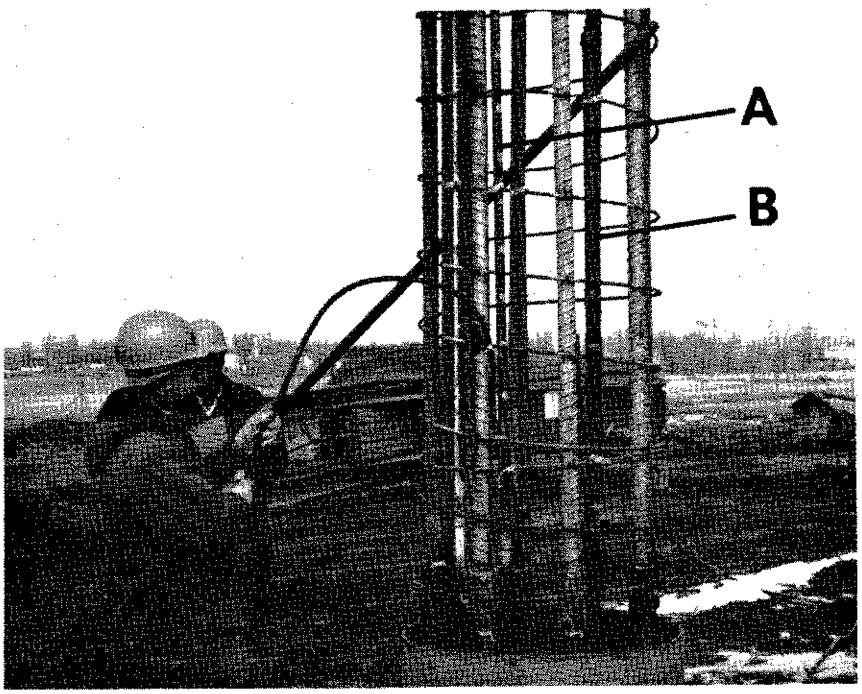


Figure 5 Instrumentation on Reinforcing Cage of Pile B-2.

A - Deformation gauge
B - Strain meter

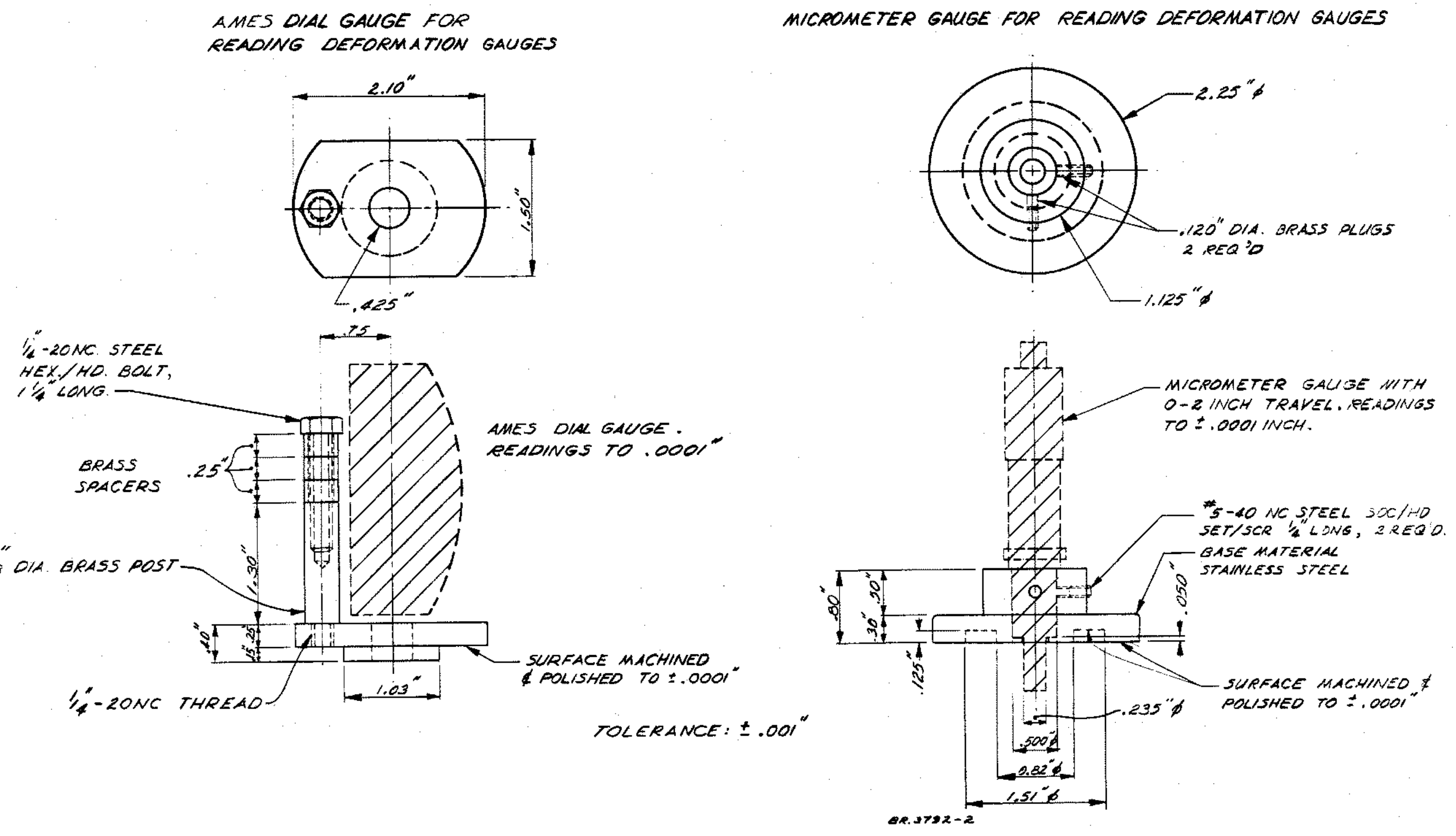


FIGURE 6 READING INSTRUMENTS FOR DEFORMATION GAUGES

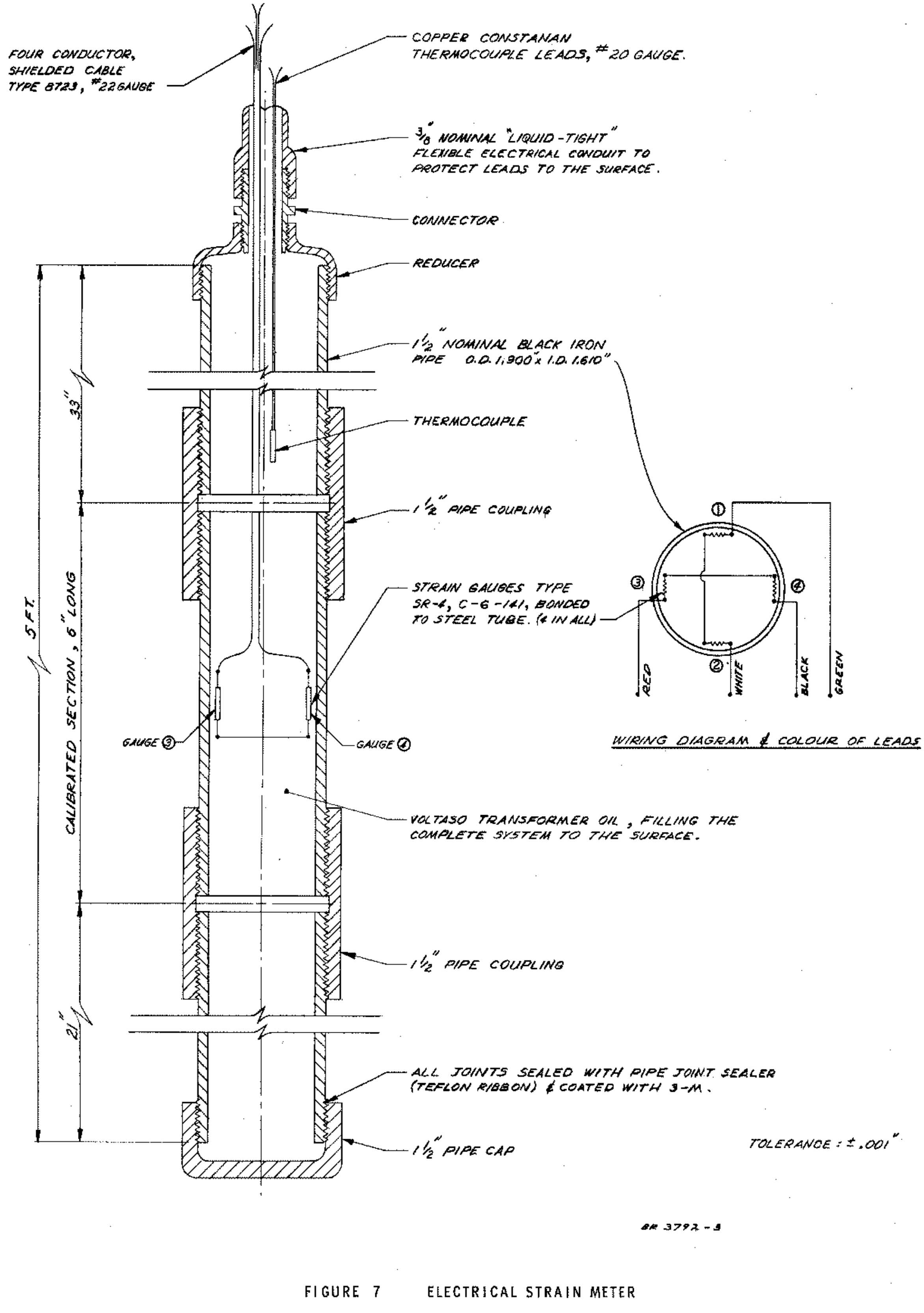


FIGURE 7 ELECTRICAL STRAIN METER

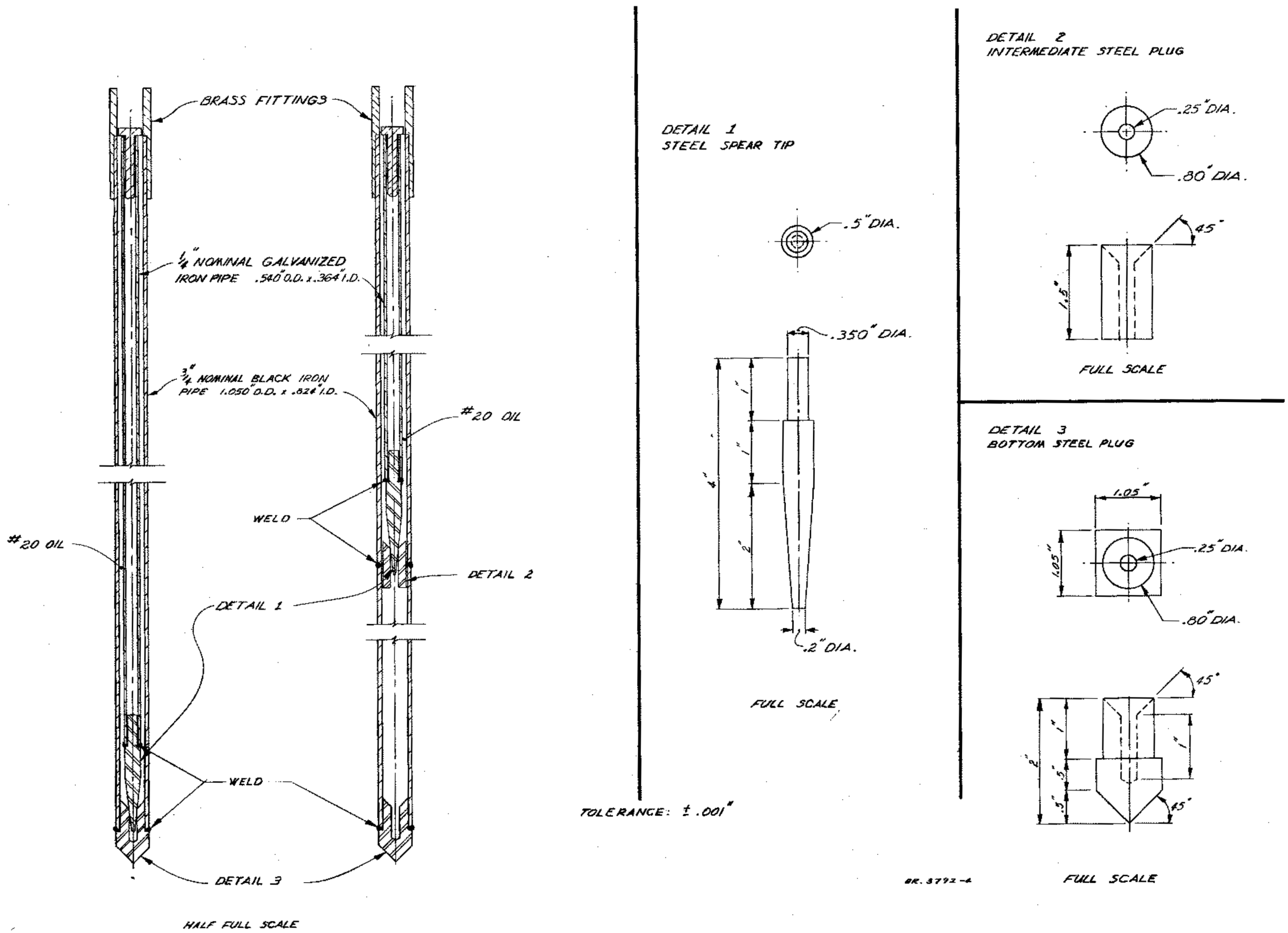


FIGURE 8 DEFORMATION GAUGE DETAILS FOR 12" DIA. TEST PILE

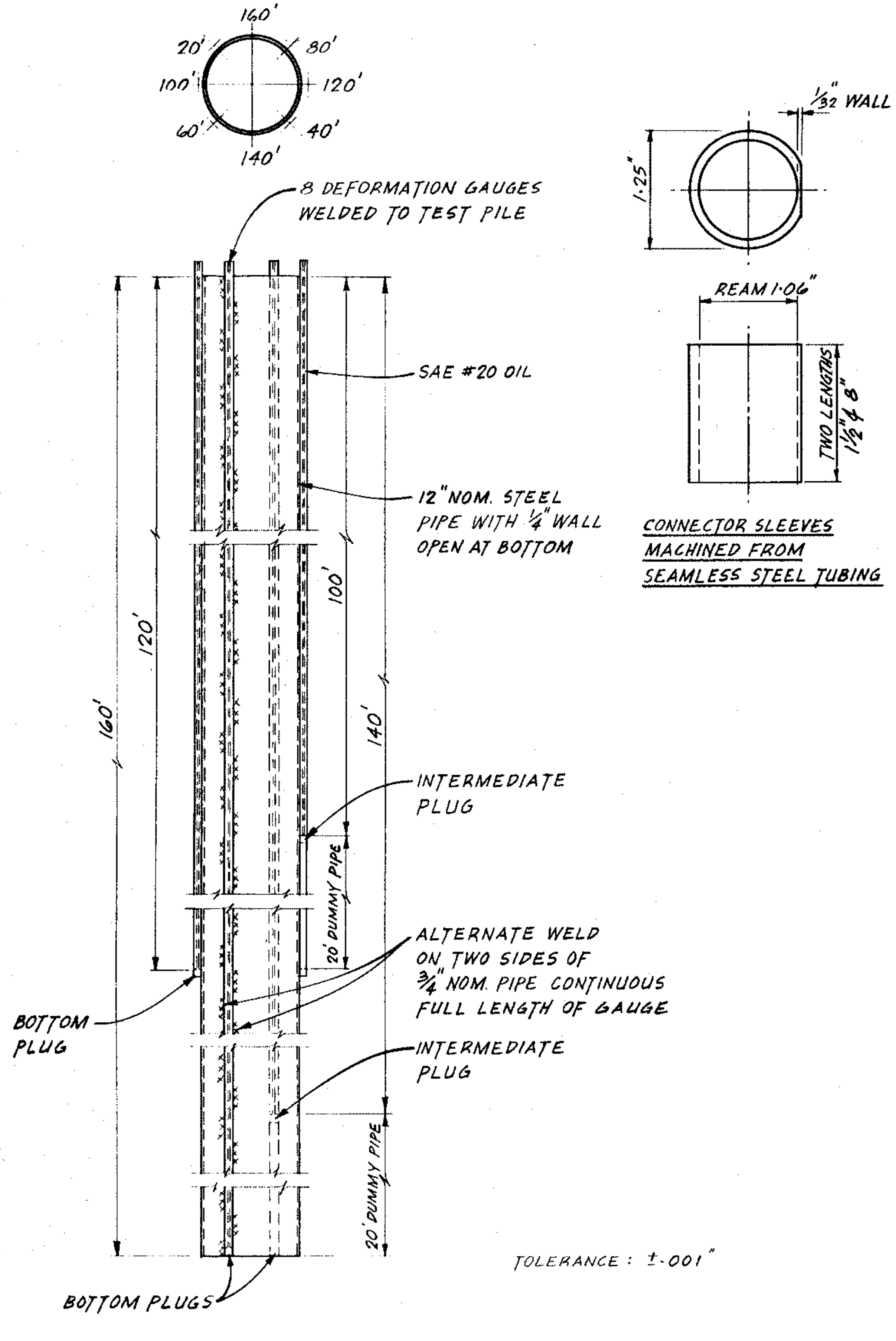


FIGURE 9

INSTALLATION OF DEFORMATION GAUGES ON 12" DIA. TEST PILE

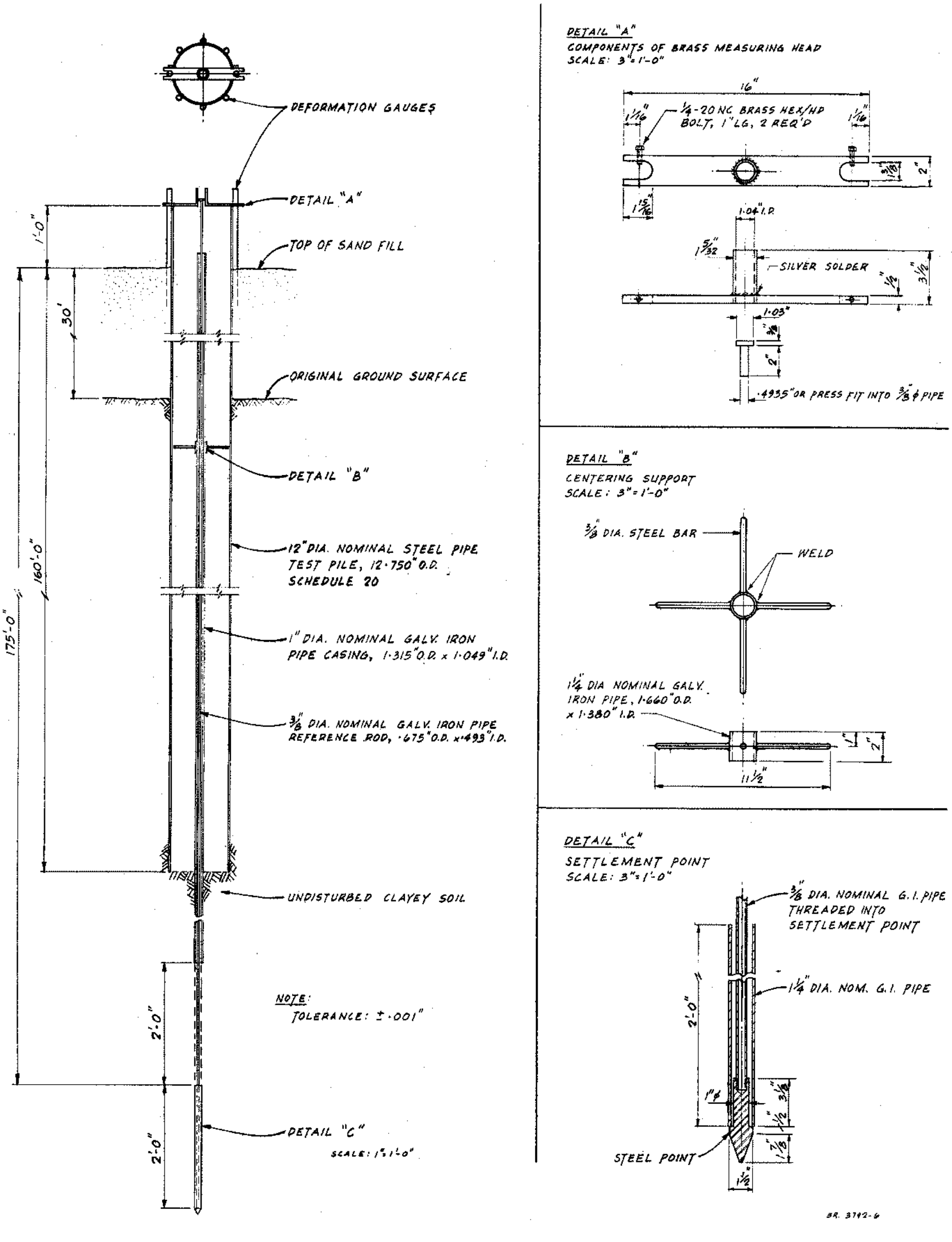


FIGURE 10 SETTLEMENT GAUGE FOR 12" DIA. STEEL TEST PILE