# Development of an Impact Cone Penetration Device for Backfill Evaluation

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The results of a series of preliminary studies that were conducted to develop a penetration device to be used in the assessment of the in situ condition of backfill materials behind the conduit wall of soil-steel structures and in other buried conduits are described. In particular, the methodologies associated with the design of an impact cone device and the results of a series of preliminary tests conducted on dense sand and loose granular soils are discussed.

Large-span metallic soil-steel structures and buried flexible conduits are used extensively as alternatives to conventional bridge structures for highway and rail overpasses. These structures are composed of corrugated metal and can occupy spans of up to 45 ft (*1–4*). The corrugated metal plate sections are typically ⅛ to ¼ in. thick, depending on the size of the arch, with 6- × 2-in. corrugations. The mechanical response of these structures in situ is dominated by the interaction between the flexible corrugated metal and the relatively compressible compacted soil placed adjacent to it. The soil medium is designed and placed to provide the relative stiffness necessary to generate full capacity of the soil-steel structure predominantly through shell action. Although the backfill material is usually placed to a given compaction specification, during the lifetime of the structure it can experience either compaction or loosening because of additional nonuniform surcharge imposed by compaction equipment or traffic loads. In addition, processes such as groundwater flow, frost action, and the attendant removal of fine particles can lead to reductions in the density of the backfill. In certain older soil-steel structures, inadequate material specification or compaction control has led to in situ backfill with loose regions of both cohesive and granular soils. The changes in the condition of the backfill can have a significant influence on the interaction behavior of an embedded flexible soil-steel structure. In extreme situations, reduction in deformability characteristics of the backfill has led to excessive deflections of the soil-steel structure, causing loss of serviceability or collapse of the structure by buckling (*5*). Evaluation of the in situ condition of the backfill adjacent to flexible soil-steel structures is therefore of fundamental importance to the assessment of the integrity of such structures.

Because of the cohesionless nature of conventional backfill materials and their free-draining granular particulate structure (e.g., dense sands, crushed granular A aggregate, etc.), it is not possible to retrieve samples for purposes of laboratory testing. Conventional testing philosophies (*6–9*) strongly advocate the use of in situ testing techniques for evaluation of in situ properties of granular materials such as backfill. Four types of in situ testing techniques, applicable predominantly to granular soils, are cone penetration testing, flat plate dilatometer testing, screw plate testing, and pressuremeter testing.

Cone penetration devices have been successfully used in geotechnical engineering practice for the evaluation of in situ properties of granular soils (*10–16*). In cone penetration methods static or dynamic testing techniques are used to evaluate either elementary in situ properties, such as relative density and void ratio, or, through empirical correlations and theoretical developments, the deformability and shear strength characteristics of granular materials. Both dynamic and static cone penetration techniques have been successfully used in the in situ measurement of properties of dry granular soils, cohesive soils, and frozen soils. No attempt is made herein to provide a complete bibliography on the subject of penetration testing. Other sources (*10–16*) and, where relevant, the conferences associated with them cite in excess of 500 further sources dealing with penetration testing. Extensions of conventional cone penetration testing are achieved through the piezocone test (*17*).

Pressuremeter testing techniques (*18–20*) have also been successfully applied to determine the deformability and shear strength characteristics of granular soils such as sands, silty sands, and, on certain occasions, sandy gravels. Pressuremeter testing techniques [either full displacement (*21*) or self-boring (*18*)] are expensive and are considered nonroutine. Furthermore, the relative particle sizes associated with granular backfill compacted adjacent to a soil-steel structure (the ratio of the diameter of a pressuremeter to largest particle size can be of the order of 3 to 5) are such that conventional pressuremeter testing would be of limited reliability. For this reason pressuremeter testing can be advocated for situations where the backfill consists of fine granular materials such as sandy silt or sandy clay.

In recent years flat plate dilatometer testing has gained popularity in connection with in situ testing of both granular and cohesive soils. The details of this test procedure are given elsewhere (*22,23*). In the flat plate dilatometer test, a highly flexible diaphragm is activated by internal pressurization of the device. The deflections of the membrane are then interpreted to estimate parameters such as relative density, void ratio, deformability characteristics, and so forth. Despite the highly speculative nature of the interpretation schemes, the device would be of limited practical use in in situ testing of backfill. The reliability of the membranes and their deformations would be limited in tests of granular materials con-

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taining relatively large particles with dimensions on the order of the diameter of the flexible diaphragm used in the test device.

The screw plate test is versatile and has been used extensively for determination of the in situ strength and deformability characteristics of both granular and cohesive soils (*24–30*). The device involves the insertion of a helical plate into the soil with a minimum of soil disturbance and its subsequent load testing. Here again, the granular structure of a typical backfill restricts the application of screw plate testing to situations in which mechanical devices (e.g., power augers) are available for the insertion of the screw plate and appropriate reaction devices (e.g., anchorages) are available for the application of the loads.

Owing to the preceding factors, in situ testing of backfill material has to be performed in such a way that the condition of the backfill can be determined at a variety of locations adjacent to a soil-steel structure with relative ease. As a preliminary study toward achieving this objective, the adaptability of in situ penetration testing techniques for the evaluation of the backfill condition adjacent to soil-steel structures was investigated. The objective of the laboratory testing program was to develop a procedure whereby the relative density of the backfill could be evaluated by performing a penetration test. It is assumed that with a knowledge of the in situ relative density characteristics of a backfill, its strength and deformability characteristics can be estimated through laboratory tests on samples of the granular material that are compacted to the measured relative density. The stages in the development of the laboratory device for impact penetration testing are summarized. The dynamics of projectiles penetrating geological media has attracted the attention of engineers and scientists for almost the past three centuries (*31–35*). Certain preliminary results of impact penetration tests conducted on moist dense sand and loosely placed crushed granular A aggregate are presented.

## EXPERIMENTAL INVESTIGATIONS

The primary objective of the study was to develop a relatively simple testing technique wherein a cone penetration device is inserted into a compacted granular soil adjacent to a rigid or flexible structural boundary. The boundary represents the conduit wall of a soil-steel structure. The constraints imposed on the penetration testing methodology were the following: (*a*) the orientation of the penetration device should be approximately normal to the plane of the rigid boundary, (*b*) the penetration device should be driven without the aid of a reaction frame, and (*c*) the energy imparted to the penetration device should be standardized and controllable. Two types of penetration tests were performed with these constraints in mind. The test procedures are outlined in the ensuing sections.

### Cone Penetration Device

A number of cone penetration devices are used in in situ testing of both granular and cohesive soils. The devices are characterized in relation to the angle of the cone device, which can range from 30 to 60 degrees. In this investigation, the cone angle was set at 60 degrees, and the diameter of the cone device was 1.4 in. Details of the stainless steel cone penetration device are shown in Figure 1. The cone section of the device was attached to a load cell to measure loads applied to the cone tip during an experiment. In this series of preliminary tests, however, no attempt was made to measure the loads induced at the conical tip of the penetrometer. The device has appropriate couplings that allow the connection to loading devices.

### Loading Procedures

Two types of horizontal cone penetration tests were conducted with the device. In the first type of test, the cone penetration device was attached to a horizontally mounted MTS servo-controlled hydraulically driven actuator, which can apply a constant rate of penetration of 0.1 in./min. Details of the experimental procedure are shown in Figure 2. In this loading procedure a load cell attached to the actuator is used to measure the total load acting on the cone penetration device. The tests were carried out in a concrete test tank 60 in. long, 36 in. wide, and 48 in. deep (Figure 2). The reaction frames required for the application of the loads were attached to the test tank. The second series of tests involved the application of an impact load to the penetration device. The device used for the application of the impact load was a Low

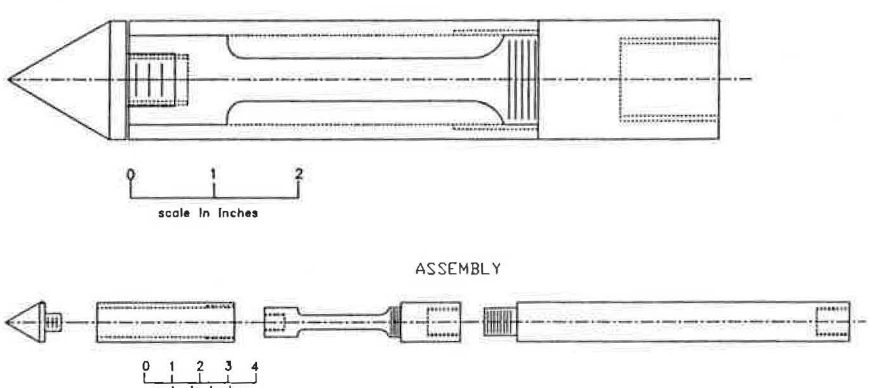



**FIGURE 1 Details of the cone penetration device.**

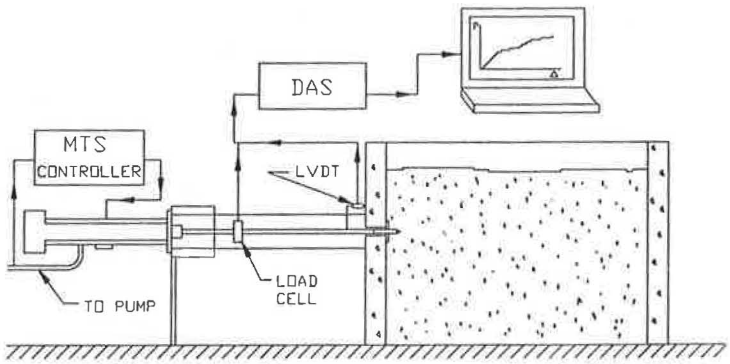



**FIGURE 2 Penetration testing facility: constant rate of penetration of the cone device.**

Velocity Powder Actuated Tool (HILTI Model DX 36M). The energy input to an impact was controlled by the selection of a charge type (HILTI Cartridge #2; Brown Extra Light; No. 000502906 CAL. 27 Short). Special couplings were designed to guide the energy of the impact to the penetration device. Figures 3 and 4, respectively, show the facilities and the stages in the application of the impact load to the cone penetration device.

### Testing Facilities

The quasi-static penetration tests that used the MTS device were performed in the test facility shown in Figure 2. The inside of the tank was lined with stainless steel sheets, and a metal plate was incorporated at the region where the cone penetration tests were performed. The entry point for the cone penetration device was cut out in the metal plate, and an aluminum foil barrier was installed. To prevent extrusion of the soil through the aluminium foil barrier during compaction, a solid cap was installed flush with the inner face of the metal plate. Using this procedure, adequate compaction of the soil was achieved in the vicinity of the rigid plate without damage to the soil retention barrier and the attendant loosening of the soil in the cone penetration zone. The impact cone penetration tests were performed in a test tank measuring approximately 95 × 83 in. in plan area and 72 in. in depth. The details of the test facility are shown in Figure 5. The end sections of the facility were fabricated of steel channel sections. A channel section near the base of the test tank was provided with circular cutouts for conducting the cone penetration tests. Altogether three slots were provided on each side of the test tank. The locations of the penetration test are shown in Figure 5. Here again, the apertures in the channels were provided with the metal foil–metal plate restraining system to allow retention of the soil during compaction and penetration testing. In the impact cone penetration tests, the cone device was guided through a teflon-lined aluminum housing. The housing was directly attached to the end-channel sections of the test tank.

### Material Preparation

Two types of granular materials were used in the investigations. The first was classified as a mortar sand ($D_{10}$ = 0.01 in., $C_u$ = 3, and $C_c$ = 0.95). The mortar sand was used in both test tanks shown in Figures 2 and 5. Because of the small dimensions of the tank shown in Figure 2, it was not feasible to use mechanical methods for the compaction of the sand. The sand was compacted manually using a tamper weighing 12 lb and a standard number of impacts applied by a free fall from a height of approximately 6 to 9 in. This compaction scheme has been used effectively in other experiments, and the resulting density can be controlled reasonably accurately (*36,37*). In the experiments involving constant rate of penetration of the cone penetration device, the mortar sand was compacted to a bulk unit weight of 170 lb/ft$^3$ at a moisture content of approximately 4.5 percent. The density of the compacted mortar sand was measured using a nuclear density meter (TROXLER 3401). The compaction of the sand in the larger test tank was carried out by using a vibratory plate compactor (Vibroplate MIKASA 52G), which makes it possible to attain good compaction of the moist sand at the end regions of the test tank, where the cone penetration tests will be conducted. Using the vibratory plate compactor, the moist sand was compacted to a bulk unit weight of 117 lb/ft$^3$. Again in this case the nuclear density meter was used to ascertain the in situ density of the compacted moist sand. A crushed granular A material ($D_{10}$ = 0.008 in., $C_u$ = 50, and $C_c$ = 3.1) was also used in this series of preliminary investigations. The material was placed in a loose state and manually compacted with a tamper. Because of the loose placement of the crushed granular A material, it was not possible to attain good control of density within the compacted regions. The bulk unit weight of the granular A material varied from approximately 107 to 112 lb/ft$^3$. These two extreme density variations were observed at the end regions of the test tank, the locations for the penetration tests.

## EXPERIMENTAL RESULTS AND DISCUSSION

Three experiments were performed in which cone penetration into the compacted sand was achieved at a constant rate of 0.1 in./min. Figure 6 shows the variation of the load-displacement response for the cone penetration device. The penetration resistance has a characteristic shape indicated by a near-bilinear form. The first linear portion roughly corresponds to the complete penetration of the conical part of the device into the compacted sand. The second linear portion of the load-displacement curve is indicative of the frictional resistance

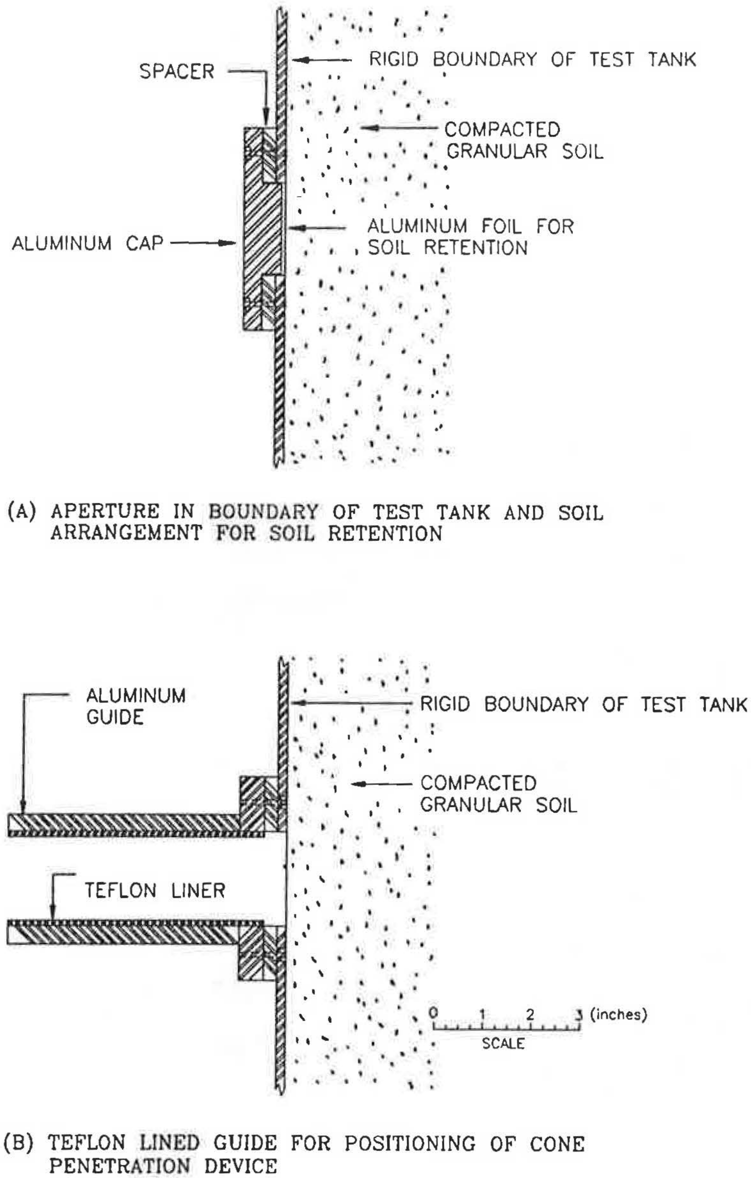



**FIGURE 3 Entry point arrangement in the impact cone penetration testing.**

generated along the cylindrical portion of the penetrating device. The maximum load measured in the test was about 1,600 lb and occurred at a penetration of approximately $4D$, where $D$ is the diameter of the cone.

The results of the impact cone penetration tests conducted on both dense moist sand and loosely placed granular A material are shown in Figure 7. The results for the normalized penetration values are plotted as a function of the number of impacts applied to the device. The results derived from tests conducted on compacted mortar sand indicate a great deal of uniformity. In these tests the maximum cone penetration for tests conducted in dense sand was about $2D$, and for tests conducted in loosely placed granular A it was about $3.8D$.

## CONCLUSIONS

The evaluation of the condition of a backfill behind the conduit wall of a soil-steel structure is an important consideration in the assessment of the load-carrying capacity of the soil-steel structure. Accurate assessment of the backfill condition can lead to an improved estimate of the integrity of the soil-steel structure system. In situ testing techniques offer the most reliable means for estimating the condition of the backfill. The most convenient method for evaluating the condition of the backfill nearest the conduit is to introduce the testing device through the conduit wall. The alternative method of introducing the testing device through the top of the embankment is both time-consuming and expensive. Furthermore, the interpretation of such a test must take into consideration the influence of a flexible soil-steel structure. The technique of introducing the testing device through the conduit wall limits the use of many of the conventional methods of in situ testing, such as the pressuremeter, screw plate, flat plate dilatometer, and so forth.

Cone penetration tests offer a suitable alternative to the aforementioned in situ tests. The conventional modes of cone

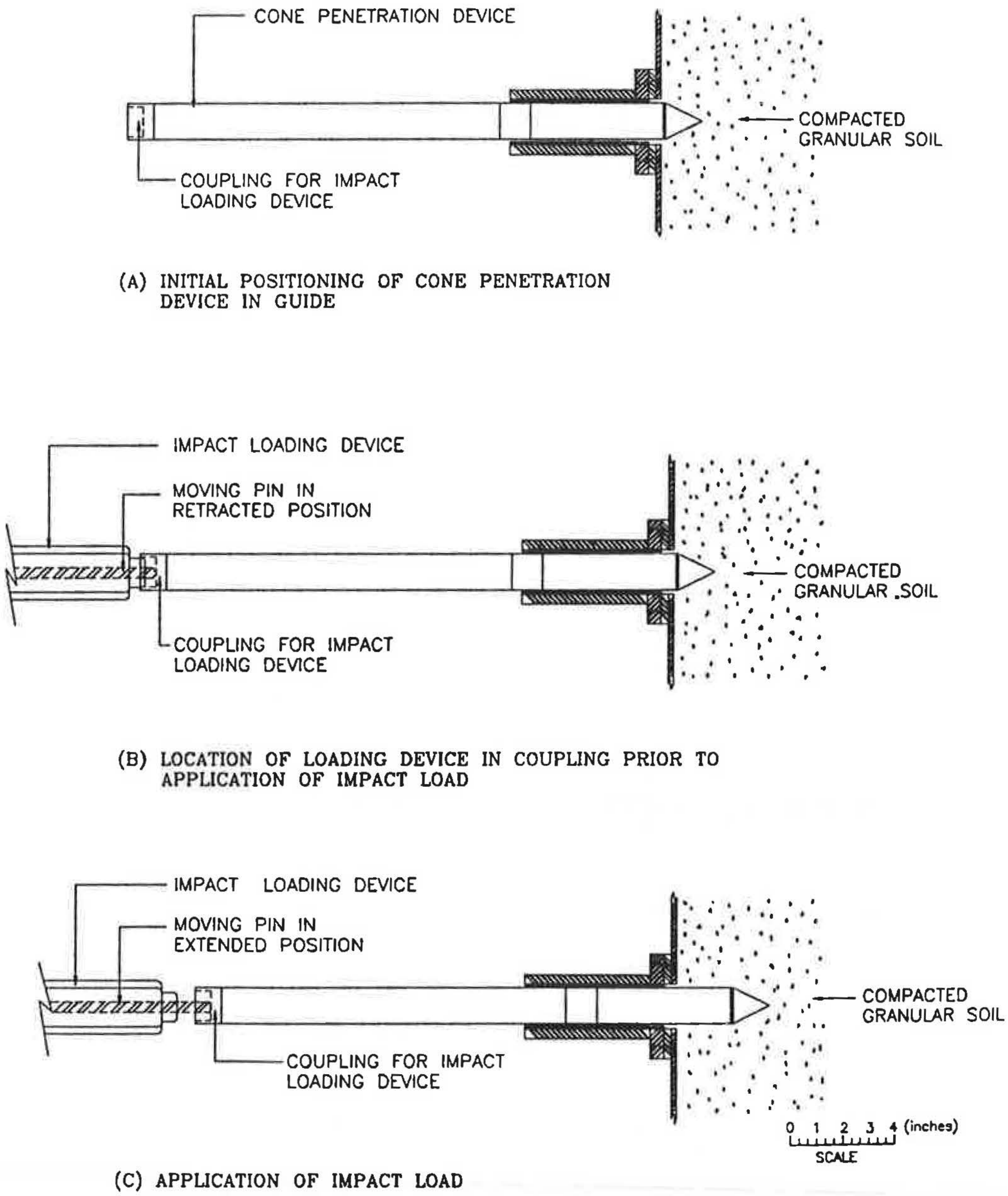
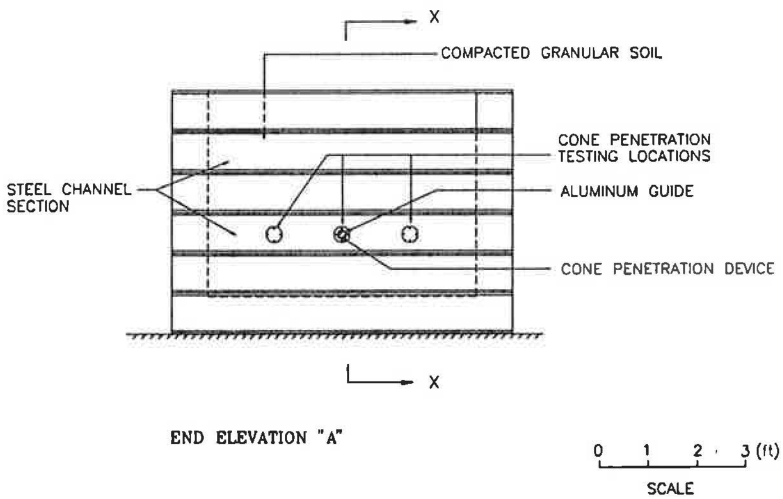



**FIGURE 4 Stages in the impact cone penetration testing.**

penetration testing invariably focus on techniques in which the cone penetration is vertical and suitable reaction frames are available for the application of the loads. With regard to backfill testing through the conduit wall of the soil-steel structure, these procedures are of limited applicability. Furthermore, the constraints of ease of use and portability of testing equipment necessitate development of an impact cone testing device. The primary objective is to ascertain the condition of the backfill by correlating the penetration resistance of an impact cone device with the relative density of the backfill. The stages in the develpment of an impact cone penetration device tested under laboratory conditions have been documented. The laboratory studies included the design of a cone penetration device and the identification of a method of applying impact loads to the cone device. The results of the preliminary laboratory studies indicate that the impact penetration resistance, as indicated by the penetration ratio $\Delta/D$ (where $\Delta$ is the penetration and $D$ is the diameter), at impacts in excess of 20 exhibit a reasonably linear variation, and the slope of the plot of $\Delta/D$ versus $N$ can be used as an indicator or penetration resistance index for the relative density of the granular material.

As emphasized previously, the impact penetration testing scheme described is a preliminary laboratory investigation that requires further laboratory study and field verification. The future research program could include the following:

1. Development and standardization of the dimensions of the cone and its materials,
2. Standardization of the impact energy input to the cone penetration device and the evaluation of the rate of energy input by force-velocity measurements,



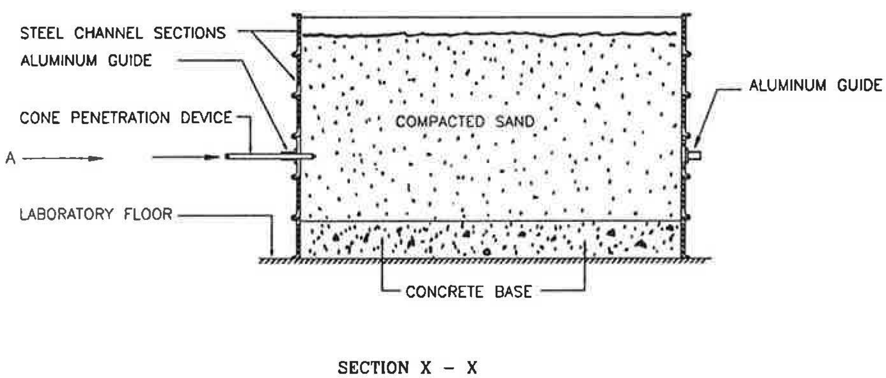




**FIGURE 5 Penetration testing facility: impact penetration of the cone device.**

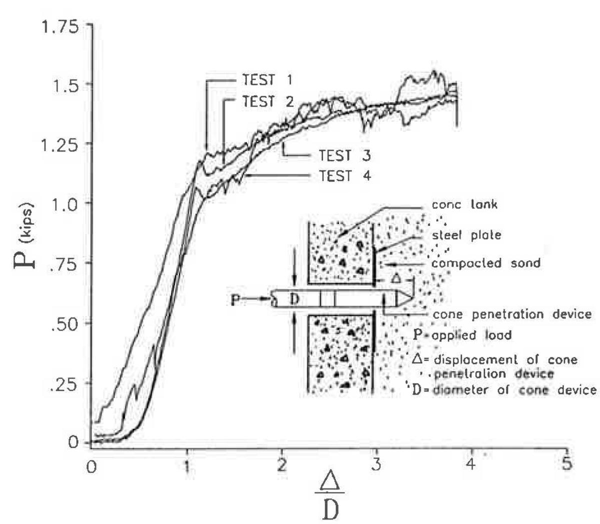



**FIGURE 6 Load-penetration data for constant rate of penetration tests conducted on compacted sand.**

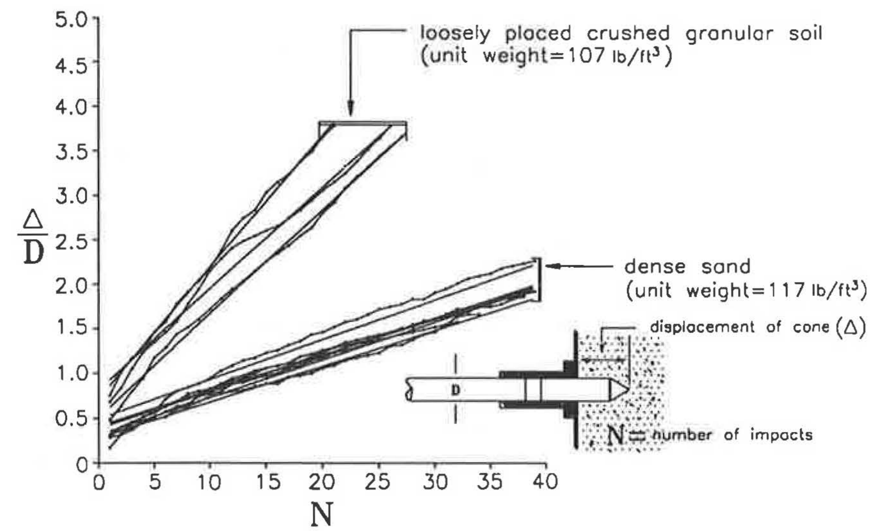



**FIGURE 7 Penetration-impact number data for penetration tests conducted on densely compacted sand and loosely placed crushed granular A.**

3. Extensive laboratory studies involving sand and crushed gravel that are compacted to specified densities at known moisture contents,

4. The development of correlation between relative density and penetration resistance for various granular backfill materials,

5. Extension of the studies to include poor-quality backfill materials,

6. Adaptation of the impact cone penetration device for in situ use,

7. Field trials involving the impact cone penetration device and existing and newly constructed soil-steel structures, and

8. Theoretical evaluation of the penetration mechanics of the impact cone penetration device.

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