4. The contractor's installation procedure should be reviewed to ensure that the PVC pipe is continuous after installation. Perhaps techniques such as a minimum set time, pop riveting the joints, or maintaining positive pressure on the PVC pipe while extracting the casing will be necessary to ensure this.

5. The contract should require the contractor to apply grease to the casing during drilling carefully so as to minimize the grease smear on the PVC drain pipe.

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Abridgment

Dynamic Compaction of Granular Soils

G. A. Leonards, W. A. Cutter, and R. D. Holtz

The densification of a loose granular fill by dynamic compaction is described. The effective depth of compaction was found to be described by the relationship $D \simeq \frac{1}{2} (Wh)^{1/2}$ when D and h are expressed in meters and W is expressed in metric tons. The degree of compaction achieved was found to correlate with the product of the energy per drop and the total energy applied per unit surface area.

This paper describes the use of dynamic compaction to densify a loose granular fill in preparation for the construction of a warehouse at the National Starch and Chemical Corporation's Indianapolis plant [further details of this work are described elsewhere (1)]. During the 1930s, embankments of granular material—a sand spoil from an adjacent gravel pit operation—had been placed along the northern property line and through the central portion of the development area. The two embankments merged on the east side of the property to enclose a triangular-shaped tract of land.

The original plans called for constructing the warehouse on a controlled granular fill entirely located between the two spoil embankments. However, subsequent to the filling and grading operations of this area, it was decided to enlarge the warehouse and to shift its location eastward. These changes meant that both the northeast and the southeast corners of the warehouse structure would be situated over the old spoil embankments, which had been constructed simply by end dumping. Because the project was being constructed as quickly as possible, the old spoil embankments had to be improved as expeditiously as possible.

Basically, the spoil materials were a loose, fine-to-medium sand (having thin gravelly seams) covered by a well-compacted sand whose thickness increased with increasing distance from the crest of the old spoil piles (see Figure 1). The percentage of fines (those passing a 75-μm sieve) ranged from 2 to 10 and was typically 5-6. The depth to the underlying original ground surface was about 5-6 m, and the groundwater table was 9-10.5 m below the current ground surface. After examining a variety of ways for dealing with the problem, it was decided that densification would be both the cheapest and the most expedient method. Estimates were made of relative costs and times to completion for excavation and replacement by controlled, compacted backfill versus deep compaction in situ, and deep compaction by a heavy falling weight was selected for trial.

PRELIMINARY TRIALS

Preliminary trials were carried out by using the weights, drop heights, and drop patterns shown in Figure 2. Based on measurements of crater depth after successive drops, it was decided to limit the number of drops at each point to seven. Standard penetration (N) and Dutch cone penetration (q_c) tests were obtained before and after completion of the pattern shown in Figure 2a, and the results were sufficiently promising to justify the second trial, in which the 5.9 [metric] ton weight was dropped 12 m in the pattern shown in Figure 2b. Except for the first 0.6-1 m, a large improvement in penetration resistance was achieved down to the underlying clay layer. The clay layer apparently absorbed energy remarkably well and prevented deeper densification. Because the clay layer was at an even greater depth in the area to be improved, it was concluded that dynamic compaction by using the weight, drop height, and pattern shown in Figure 2b at each footing location should be satisfactory.

RESULTS OF DYNAMIC COMPACTION

A grid was outlined at each footing location and compaction was carried out. Figure 3 is typical of the results achieved. In all cases, sufficient compaction ($N \geq 15$) was obtained to the desired depth (5 m) and the footings were proportioned by using a contact pressure of 168 kPa. The warehouse has now been in service for more than two years, and measurements show that the maximum total settlement has been less than 5 mm. Area compaction of lesser intensity was applied between the footings to support the slab on ground used for the warehouse floor. Although measurements have not been made on the floor slab, it has not settled noticeably.

VIBRATION EFFECTS

Because of the possibility of further extensions to the plant, the relationship between the distance of a drop point from an existing structure and the induced vibrations was evaluated. A seismograph was placed on an exterior footing (before the columns were cast), and the 5.9-ton weight was dropped 12 m at locations 3-24 m from the footing. The frequency of vibration was approximately 7 Hz, and the measured velocities were essentially ground motions. The peak particle velocity appeared to vary inversely with the logarithm of the distance from the drop point; on a drained granular soil, particle velocities of ≤50 mm/s at a distance of 3 m from the drop point were found.

Figure 1. Typical soil boring results.

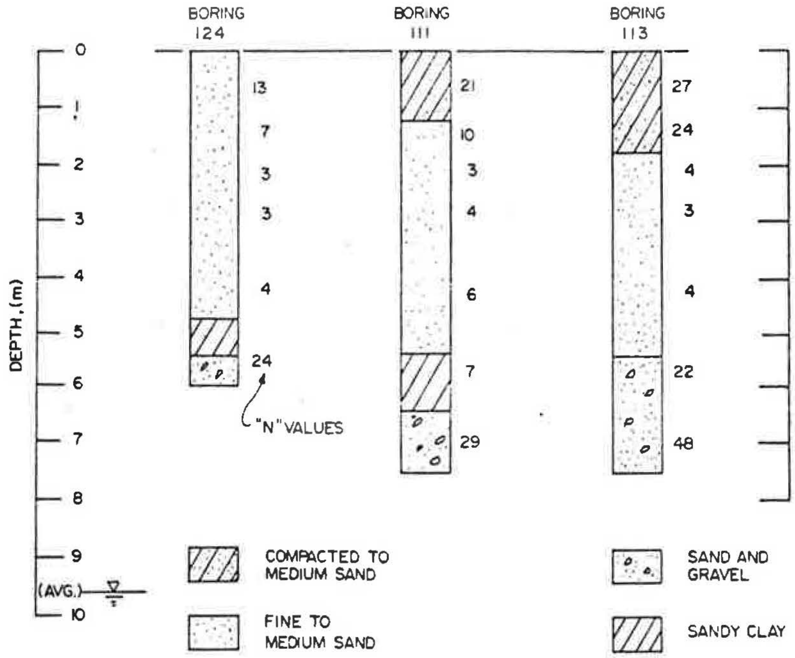


Figure 2. Number of drops and drop patterns.

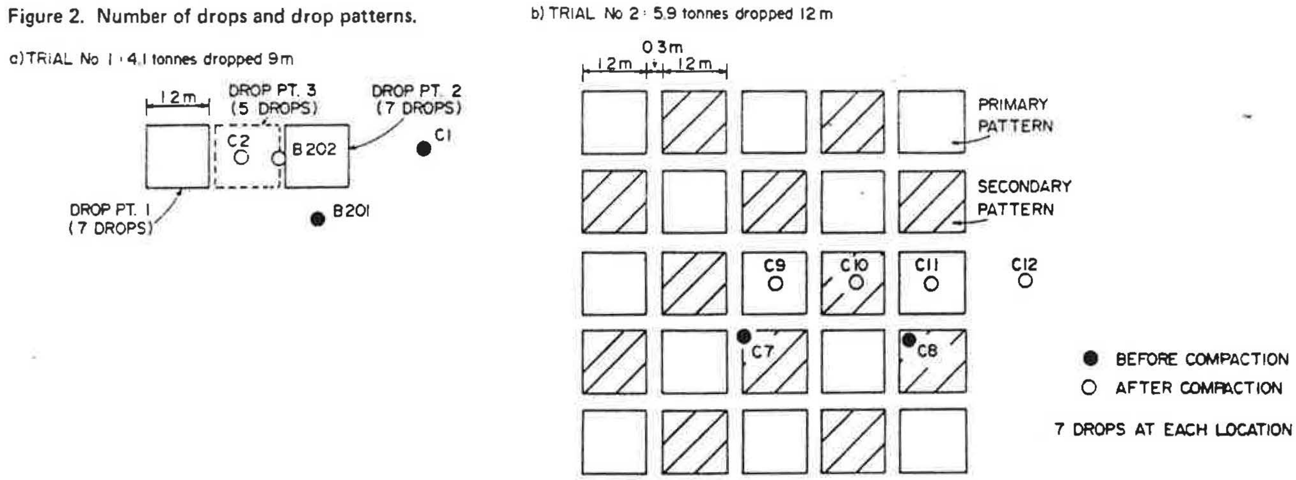


Figure 3. Relationship between cone penetration resistance and depth before and after dynamic compaction: footing H-1.

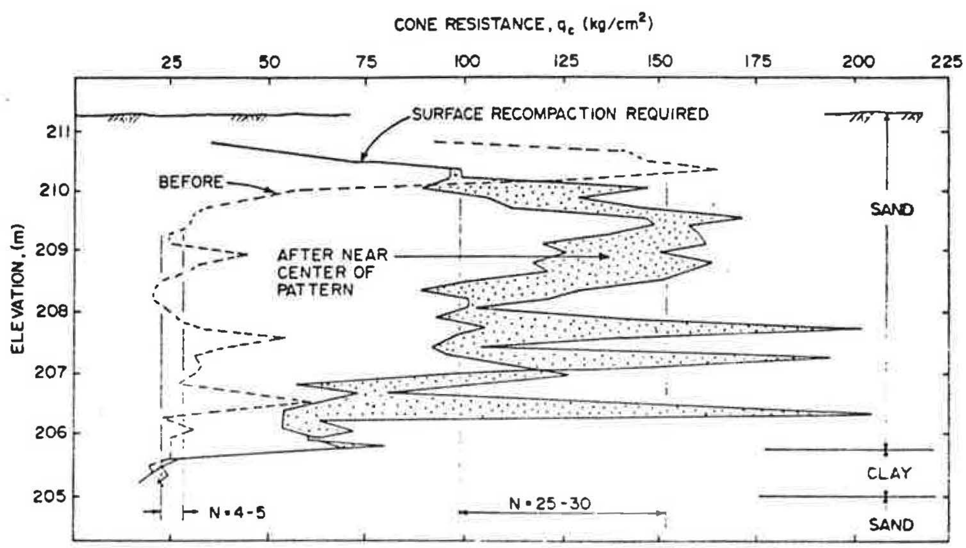


Figure 4. Relationship between depth of influence of compaction and square root of energy per drop.

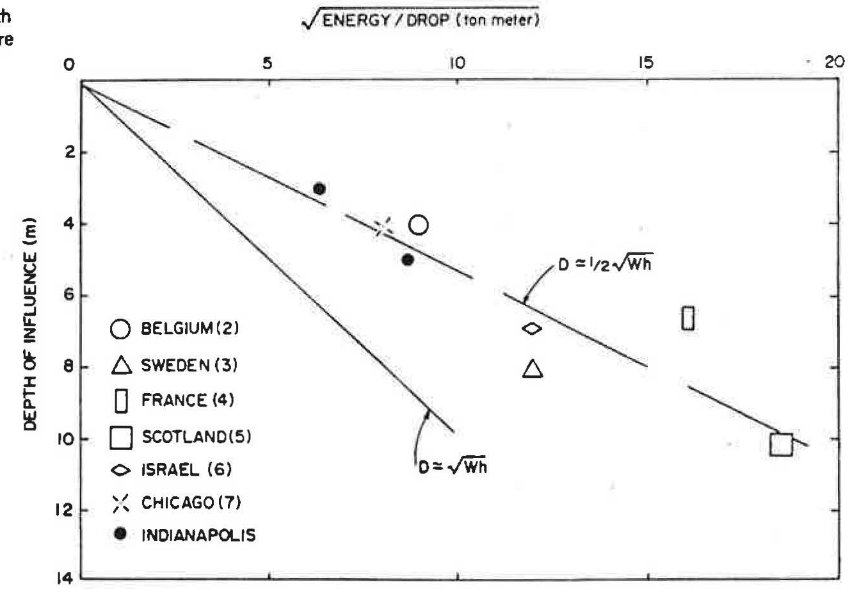
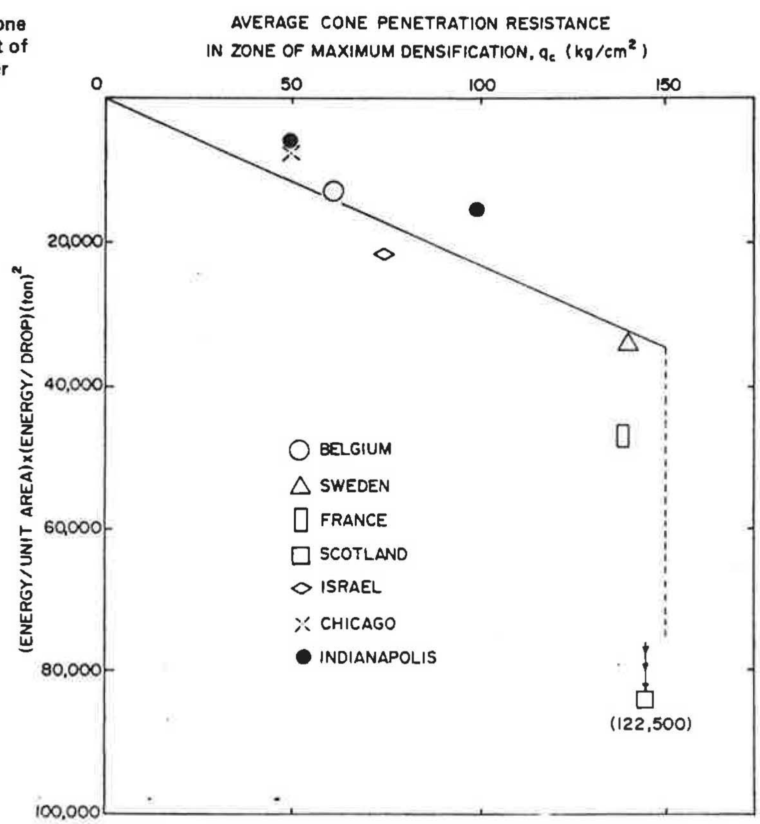


Figure 5. Relationship between cone penetration resistance and product of energy per unit area and energy per drop.



COMPARISON WITH PUBLISHED DATA

As a guide for future work, the results obtained in Indianapolis were compared with those available in the literature. Figure 4 shows the relationship between the energy per drop and the depth to which significant densification took place. A suitable criterion for the depth of influence would depend on the soil type and its initial state of compaction; for the work reported in this paper, the criterion was an increase in N-value of 3-5. A common rule of thumb (2) is expressed by the relationship

$$D \approx (Wh)^{1/2} \quad (1)$$

where

D = depth of influence in meters,
W = falling weight in tons, and
h = height of drop in meters.

It appears, however, that the use of this rule tends to overestimate the effective depth of compaction substantially and that

$$D \approx \tfrac{1}{2}(Wh)^{1/2} \quad (1a)$$

more nearly reflects available experience.

The degree of compaction attained depends not only on the energy per drop but also on the sequence of drop points and the number of drops at each point. Available data from Belgium (3), Sweden (4), France (5), Scotland (6), Israel (7), and Chicago (8), as well as these, suggest that, for dry granular soils, the degree of compaction (as measured by q_c) correlates best with the product of the energy per drop and the total energy applied per unit of surface area (Figure 5). It appears that there may be an upper bound to the densification that can be achieved, corresponding approximately to q_c = 150 kg/cm², but more data are needed to verify this result.

CONCLUSIONS

1. In granular soils, the depth to which densification is significant is controlled mainly by the energy per drop: Relationship 1a given above is recommended as a guide for preliminary trials. The presence of clay layers or seams will greatly attenuate the effective depth of compaction.

2. The upper meter of soil is usually left in a relatively loose state, and surface recompaction is required.

3. For dry granular soils, the degree of compaction achieved seems to correlate best with the product of the energy per drop and the total energy applied per unit surface area. It appears that there may be an upper bound to the compaction that can be attained and that this corresponds to $q_c \approx 150$ kg/cm² (N = 30-40).

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Construction of a Root-Pile Wall at Monessen, Pennsylvania

Umakant Dash and Pier Luigi Jovino

A case history of the design, analysis, construction, and performance evaluation of a root-pile wall is presented in this paper. The root-pile wall was contracted for construction by the Pennsylvania Department of Transportation to correct a landslide near Monessen. The structure consisted of four hundred and fifty-eight 12.5-cm (5-in) diameter cast-in-place concrete piles placed at different inclinations to both the vertical and the horizontal axes. The piles were connected at the top by a 76.2-cm (30-in) thick by 1.82-m (6-ft) wide cap beam constructed in two 30.48-m (100-ft) sections. The cap beam was constructed first, and the root piles were then installed by extending drill holes through the cap beam to bedrock at predetermined locations and inclinations, inserting a single no. 9 deformed reinforcing steel bar (grade 60) into each drill hole, and grouting the holes. Nine survey targets were marked at the top of the cap beam to measure both horizontal and vertical movements and seven slope inclinometers were installed at various points both upslope and downslope from the structure to measure horizontal movements of the structure and the surrounding soil. This paper describes the soil and groundwater conditions, soil test results, slope stability analyses, design of the root-pile wall, and the findings of the horizontal and vertical measurements of wall movement. The following summary, observations, and conclusions are made: (a) a root-pile structure provides a fast and economical alternative to many conventional structures; (b) before the installation of the root piles, the movements of the cap beam varied from less than 2.5 cm (1 in) at the north end to more than 45.7 cm (18 in) at the sound end—these movements were due to movements of unstable soil in the slide area; (c) after the installation of the root piles, there were significant movements [up to 5 cm (2 in)] in the cap beam as well as in the soil below it, which indicated that some movement of the root-pile structure was needed before resistance to earth pressure could be mobilized; (d) no significant soil movement through the root piles could be detected—the small-diameter piles and the soil between them appeared to work as a single composite structure; and (e) conventional design procedures for retaining walls appear to provide adequate overall design for root-pile walls (the geometry of the root-pile structure described in this paper is patented and may not be the optimum design for all situations).

During the construction of a four-lane highway along the Monongahela River, just north of I-70, a series of landslides occurred. One of these landslides, at the northern end of the project, involved the new highway construction, as well as two water lines and a city street above the slope about 76 m (250 ft) from the northbound lanes.

A root-pile wall was designed and constructed to correct the landslide along PA-306 in Monessen, Pennsylvania. Several alternatives (such as tieback, reinforced-earth, and concrete-gravity walls) were considered, but the root-pile method of correction was