straints will dictate the use of this method. In these cases, the state will design the treatment rather than making use of the design-construct concept. Bidding competition by the many contractors who are not staffed to develop a design-construct contract should result in lower bid prices.

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Embankment Stabilization by Use of Horizontal Drains

Stephen E. Lamb

An embankment 24 m (about 80 ft) high that traverses a narrow valley on I-81 about 56 km (35 miles) south of Syracuse, New York, began to fail several years after construction. In the spring of 1973, the pavement dropped several centimeters to bring the cumulative patch to 46 cm (about 18 in). Complete failure of the embankment was anticipated for the spring of 1974 because past movements had been largest during the spring and recent monitoring had indicated an increase in the rate of movement. A design for a stabilization project to counteract the effects of the hydrostatic pressure that was believed to be causing the failure was prepared that consisted of a berm and shear key at the toe of the embankment. Implementation of this design, however, could not be completed before spring. Therefore, it was decided to stabilize the embankment by decreasing the excess hydrostatic pressure by installing a system of horizontal drains, a project that could be completed before spring and at a cost saving of $1 000 000. The project was completed in early spring of 1974, and the embankment has been stable since then. This paper describes the design, construction, and postconstruction evaluation of the project. In addition, observations and comments are made that should be of assistance in evaluating this method of stabilization for future projects.

The problem discussed in this paper occurred in an embankment 24 m (approximately 80 ft) high that crosses a steep-sided valley 122 m (approximately 400 ft) wide. The embankment is part of I-81 in central New York State; in April 1973, about eight years after its construction, a severe crack was observed in the southbound roadway (see Figure 1). The pavement had dropped several centimeters and, by mid-July, approximately 46 cm (18 in) of asphalt paving had been placed to maintain the highway profile. A field review to determine the extent of the distress showed that stabilization would be beyond the scope of maintenance personnel.

DESIGN INVESTIGATION AND ANALYSIS

The investigation of the area consisted of a survey, a drilling program, and establishment of horizontal-movement-control stakes. The drilling program consisted of 21 borings. In 17 of the borings, observation wells were established to monitor groundwater fluctuations and to locate the depth of movement in the foundation soils (1). A plan of the site is shown in Figure 2.

Inspection of a 91-cm (36-in) diameter, corrugated metal pipe culvert in the center of the unstable area showed separations at several joints and a few separations between the joints. Also, the underdrains placed at the original ground surface during original construction were flowing.

The cross section shown in Figure 3 indicates the soil profile and observed static water table. The observation wells indicated a variable artesian pressure below the silt and clay layer, and measurements in these wells indicated significant lateral movement in the lower part of the silt and clay strata.

An analysis of the data obtained by the end of July 1973 suggested that a progressive wedge-type shear failure was occurring through the silt and clay. The most significant factor contributing to the failure was that, since construction, the static water table had risen 21 m (approximately 70 ft) in the embankment.

Analyses indicated that a counterweight berm would not be adequate to provide stability. One apparent solution was to form a key against sliding by a close-order sequence of excavation and backfilling and then construct a berm above the key. Analysis of this indicated that the slope would be stable but that the method would require an excavation for the key that extended approximately 9 m (30 ft) below the existing ground surface and cost about $1 million, because of limited access for equipment. Next, an at-grade structure spanning the failure area was considered, but the cost estimate for this was also about $1 million. Therefore, the decision was made to prepare plans to stabilize the area by using a shear key-berm solution, because a structure would require long-term maintenance. The design for this could have been completed by April 1974; however, the construction would not have been completed until several months later.

Because a major failure during the critical spring period was a significant possibility, a review was made of another method for solving this problem. In the western United States, groundwater is sometimes removed from slopes by installing horizontal drains. Such drains cannot be considered a permanent solution because of the complexity of underground water movements, but their cost is only approximately 4 percent (i.e., $40 000) of the cost of the more-permanent treatments. Consequently, a recommendation was made to attempt to achieve stabilization by installing horizontal drains.

A contract was negotiated, and work began in March 1974. Basically, the contract included a general description of the work to be performed and a 30-day completion date with a liquidated damage clause.

Because the schedule of the project did not provide sufficient time to complete the borings until just before beginning the contractual work, the description of the work to be performed was written before all the subsurface information could be obtained. However, the description was sufficiently general to allow making modifications after reviewing all the information.

The location and spacing of the drains was determined from an educated guess rather than a theoretical analysis. Basically, the drains were to be installed in a fan-shaped pattern from the toe of the embankment at two different inclinations. The upper level was to be installed above and parallel to the silt and clay strata to achieve the maximum possible drawdown in the embankment material and to prevent recharge during peak runoff. The lower level was placed so as to provide drainage for the lower permeable strata (see Figures 3 and 4).

CONSTRUCTION

There are several practical problems of installation that are important in the design and inspection of horizontal drain projects.

The equipment requires a level working surface approximately 6 m (20 ft) wide. The working surface at the site was established by constructing a small cut-fill section at the toe of the embankment.

Before drilling was begun, the equipment was leveled on a timber mat. The horizontal alignment was established by sighting along stakes located at the top and toe of the slope, and the vertical alignment was established by loosening the swivels and adjusting the leveling jacks by using a 15-cm (6-in) machinist's level on the initial length of drilling rod (see Figure 5; note scissor swivel and adjustable jack on ends of carriage). The 0.1-0.15 m^3/min (30-40 gal/min) water required for drilling was obtained by damming the culvert outlet.

The heavy-walled, flush-coupled steel drill casing had an expendable bit adaptor with a J-slot on the first section. One or two O-ring seals were placed in the grooves in the adaptor to prevent drill cuttings from lodging between the disposable bit and the adaptor and possibly freezing the bit to the adaptor.

The 3-m (10-ft) sections of drill casing were advanced with water and rotation until the planned length was reached. The disposable bit was then advanced without rotation or water until the bit jammed against firm material. To uncouple the bit, the rotation of the casing was reversed for approximately one-quarter to one-half turn and then the casing was hydraulically withdrawn to dislodge the bit.

Two problems were encountered during the drilling operation: (a) flowing silt and sand plugging the drill

Figure 1. Problem site: highlighted pavement crack.

Figure 2. Plan view.

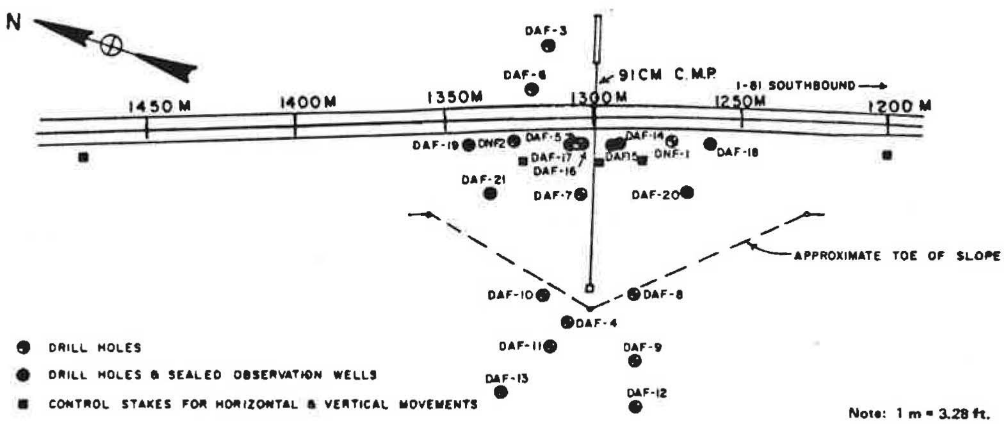


Figure 3. Cross section: typical soil profile.

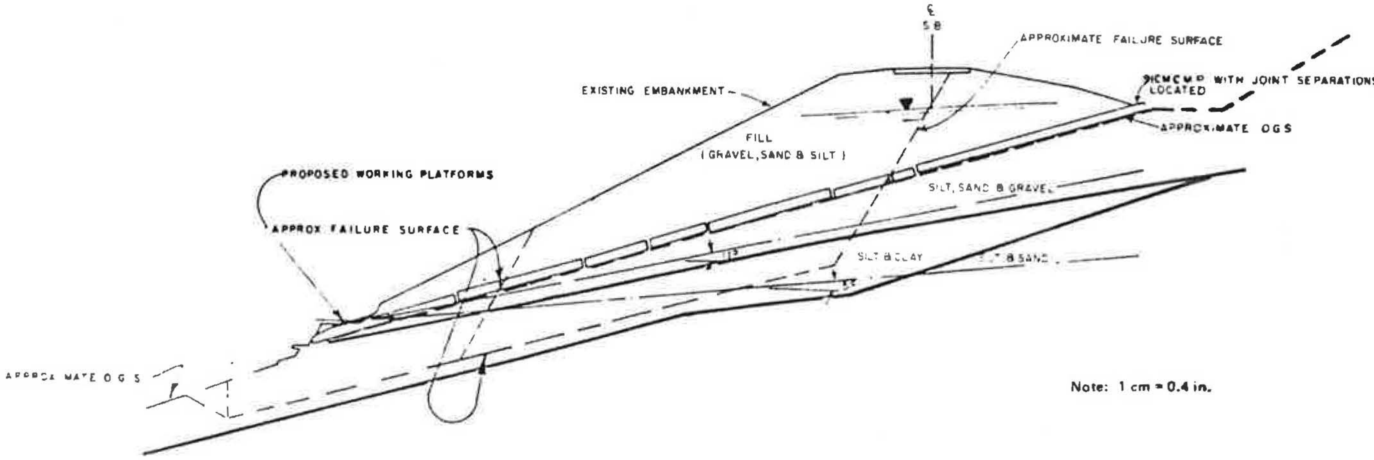


Figure 4. Drain layout: plan view.

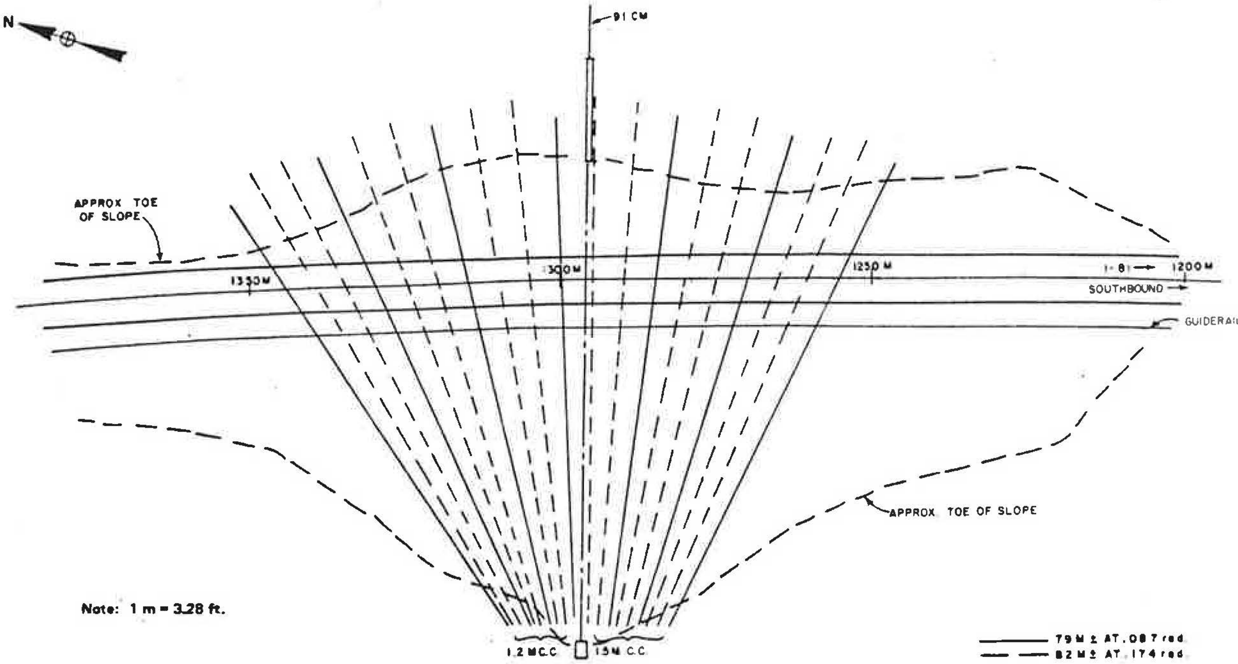
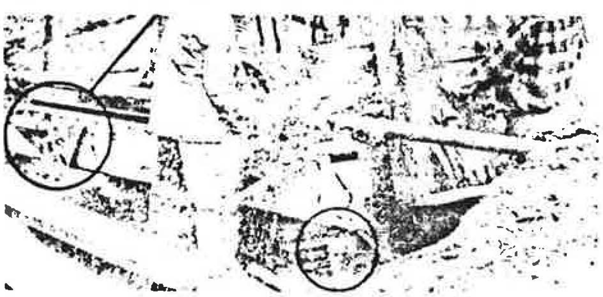


Figure 5. Vertical alignment of drilling equipment.



casing and (b) inability to dislodge the disposable bit. These problems can be avoided by using a one-way valve and spacer in the bit assembly when drilling in loose sand or silt and by attaching the bit and O-ring seals carefully.

The drain pipe was 5-cm (2-in nominal) diameter polyvinyl chloride (PVC) that had 0.25-mm (0.010-in) wide slots in a trislot configuration around the perimeter at 6-mm (0.25-in) intervals. Each drain consisted of joined 3-m sections of slotted pipe, except that the last two sections were unslotted to prevent piping at the drain outlet. The drain pipe was plugged at the upper end and inserted into the drill casing. As additional lengths of pipe were inserted into the casing, each joint was glued. The casing was extracted in 3-m sections from around the PVC pipe, which was held in place in the slope by a floating-lock piston device. This locking device is a one-way water-tight piston that, as the casing is withdrawn, is retained within the drill casing by water pressure. Occasionally, the piston bound in the drill casing, but this problem could usually be solved by rapping the casing with a sledge hammer as the casing was being withdrawn.

A few drains did not function after completion. A review of the installation sequence suggested that this problem was probably caused by the separation of a PVC pipe connection when the piston locking device bound in the casing. This situation was satisfactorily corrected by pop riveting some of the connections.

Grease was applied to the threads of the drill casing each time a section of casing was added, an operation that resulted in the formation of a grease ring on the inside of the casing as the connections were tightened. This excess grease then smeared over the drainage slots in the PVC pipe when the pipe was installed through the casing. Because the grease is not water soluble, this smearing significantly reduces the effective area available for water to enter the pipe. [For example, on a 76-m (250-ft) section of pipe that was removed from the casing because the drill bit could not be dislodged, approximately 30 percent of the effective drainage area was smeared with grease.] The grease cannot be eliminated because the lubricant is required for the drilling. Therefore, the amount of grease applied should be minimal, and its application to the casing connections should be carefully done.

The work involved the installation of 1585 m (5200 ft) of drains and was completed in 3 weeks.

POSTCONSTRUCTION DATA COLLECTION AND EVALUATION

After completion of the drain installation, data were first obtained weekly on the movement of the control stakes, the water elevation in the observation wells, and the flow rate from each drain. In October 1974, the recording interval was modified to annually for the movement of the control stakes and biweekly for both the water elevation in the observation wells and the flow rate from each drain. In July 1975, the reading intervals for the water elevations and flow rates were again modified, this time to four times per year—April, July, October, and January. No additional cracking has been noted during periodic visual inspections of the pavement in the area. Also, the survey data from the control stakes indicate that there have been no significant horizontal or vertical movements since the drain installation was completed.

The total flow rate and the flow rates from each level of drains are shown in Figure 6. Records from a nearby weather station indicate that the upper level of drains

Figure 6. Flow data.

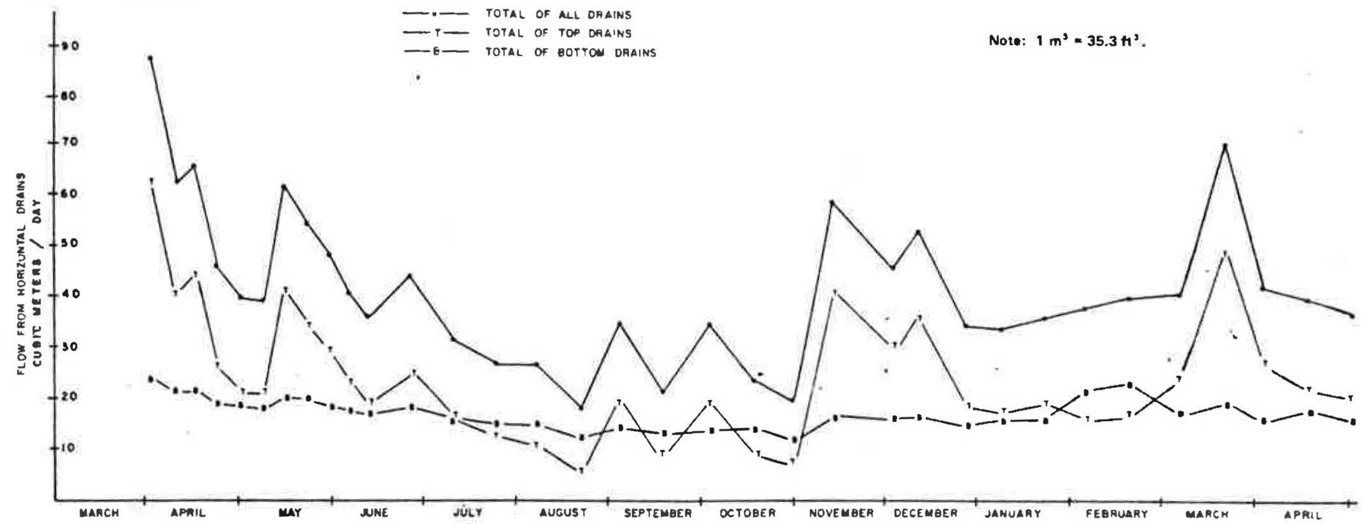
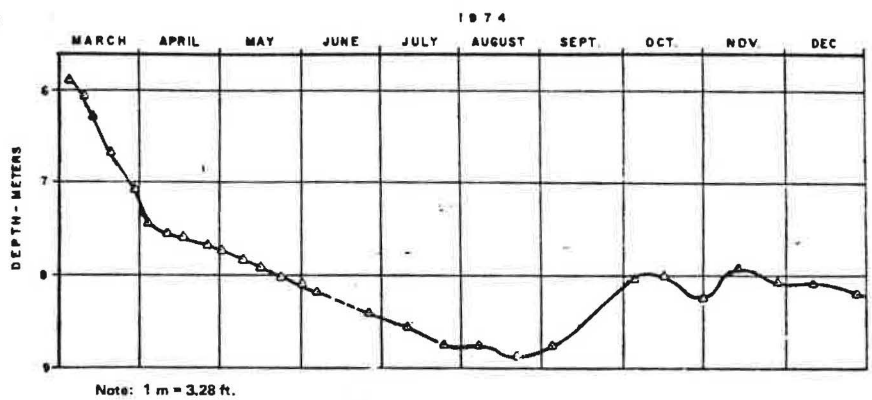


Figure 7. Typical drawdown curve: observation well DAF-14.



may be sensitive to rainfall. The flow data obtained after April 1975 indicate that the total flow is generally increasing. However, these readings were obtained during traditionally wet seasons. The data shown suggest that more information is desirable for a long-term performance evaluation of the drains.

The typical drawdown curve shown in Figure 7 indicates that the static water table above the silt and clay strata was generally lowered 3 m during 1974. This drop, coupled with a 1.5-m (5-ft) reduction of the water table in both the silt and clay and the underlying silt and sand was sufficient to provide stability. The data to date indicate that the drop in the water table will be long term.

The flow rates of the individual drains varied from dripping to approximately 0.03 m^3/min (8 gal/min).

Several of the completed drains were checked in September 1975 to determine their slopes. This check was initiated because, in a subsequent horizontal-drain project, all the drains rose above the desired inclination. The drilling equipment used in the two projects was similar but not identical. The elevations of the drains were checked at 15 and 30 m (50 and 100 ft) by using a small-diameter polyethylene tube filled with water as a level.

Any difference in the inclinations between the as-installed values and those found in the September 1975 check were attributed to the method used to set the initial inclination rather than to wandering of the steel casing. Basically, the information obtained indicated that the drains did not significantly rise on this project.

Water flowing from the drains did not freeze during the winter if the flow rates were more than 0.004 m^3/min (0.1 gal/min).

CONCLUSIONS

The drains have lowered the groundwater table and eliminated the factors that caused the pavement failure and the movements have stopped.

A 1979 survey indicated that there has been no movement in the past five years and that the flow from the drains is basically unchanged. Thus, the horizontal drains have apparently permanently solved the problem and also saved $1 000 000.

RECOMMENDATIONS

1. The site evaluation should include the installation of a complete monitoring system. This system should include provisions for monitoring (a) the water table in the area and (b) horizontal and vertical movements of the ground surface and should be permanent and protected from damage during construction and due to vandalism.
2. A prebid inspection should be held for interested prospective bidders during which questions could be answered. This would enable the attending bidders to more accurately estimate the cost of the work and result in lower bid prices.
3. For greater precision, the vertical inclination of each drain should be set by using a level having a minimum length of 0.6 m (2 ft).

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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Publication of this paper sponsored by Committee on Embankments and Earth Slopes.

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.