Dynamic Compaction of Loose and Hydrocompactible Soils on Interstate 90, Whitehall-Cardwell, Montana

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The use of dynamic compaction to increase bearing capacity and reduce settlement of loose foundation soils at great depths is becoming more widespread in the United States. The use of dynamic compaction on a recent Interstate highway overlay project in Montana is described. Foundation soils treated were about 20 to 25 ft (6.1 to 7.6 m) of loose dry alluvial sands and silts having low *N*-values (blows per foot) by the standard penetration test (less than 4 in some cases). Correlation has been found between known and suspected deposits of hydrocompactible soils and semiarid areas having less than about 12 in. (30 cm) of precipitation a year in Montana. A 15-ton (13.6-metric ton), 6-ft (1.8-m) diameter weight with a drop height of 60 ft (18.3 m) was used for the dynamic compaction process. The compaction was performed even in winter months through frost layers broken up by ripping equipment. Dynamic compaction was found to be a very cost-effective method for improving deep deposits of loose sand and silt. Cost savings from this method of compaction compared with feasible alternatives was estimated at $2.6 million. The average increase in *N*-value of the loose materials following compaction was 5 to 12. Some complaints of structural damage were received from local citizens even though the closest residence was 170 ft (52 m) from the compacted area.

The Whitehall-Cardwell project began as an overlay to correct several miles of bumpiness of unknown origin. The severity of the bumps demanded that a foundation investigation be performed. This was subsequently done in the fall of 1982 and spring of 1983 (*1*).

Interstate 90 in the project area is located in an intermontane basin of Cenozoic deposits overlying Archeozoic bedrock as shown in Figure 1. The area is semiarid irrigated cropland and grazing land with generally less than 12 in. (30 cm) of annual precipitation (Figure 2).

Test drilling revealed that the soils beneath the areas of distress consisted generally of loose deposits of sandy silt and silty sand with scattered channel deposits of gravels to a depth of about 20 to 25 ft (6.1 to 7.6 m). Some clayey soils were also present. The water table in the area is generally greater than 60 ft (18.3 m).

Laboratory consolidation tests were performed that indicated that some of the sands and silts exhibited a collapsing or hydrocompactible structure. This type of soil structure has been reported variously in the literature (*2*). Primarily it consists of voids between soil grains held in place by clay bonding, bubble cavities formed by air entrapment, interlaminar openings, or voids left by decaying vegetation. Its environment of deposition is typically semiarid or arid.

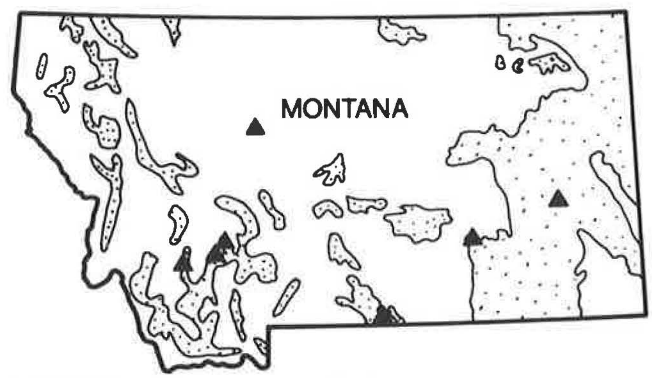

FIGURE 1 Known areas of hydrocompactible soils and Cenozoic deposits.

The most widespread deposits of collapsible soils are wind-blown sands and silts or loess. Deposits are also formed in alluvial fans such as the Whitehall project area, colluvial deposits, residual soils from a leaching process, and volcanic tuffs. These soils tend to settle or "collapse" from wetting, additional loading, or a combination of both.

Figures 1 and 2 show the correlation between known and suspected areas of collapsing-type soils and the areas of Cenozoic deposits and the areas of 12 in. or less annual precipitation. These correlations appear to indicate where to look or be aware of other possible deposits of collapsing-type soils.

Within the job limits, areas to be treated were delineated by mapping patched and overlaid pavement areas, and further verification was made by test drilling.

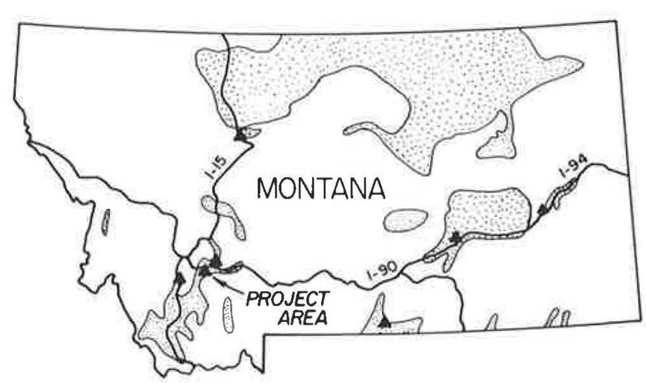

FIGURE 2 Known areas of hydrocompactible soils and areas of precipitation (12 in. or less per year).

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EXAMINATION OF TREATMENT ALTERNATIVES

Various alternatives for treatment of the Whitehall project foundation soils were studied. These included vibroflotation, subexcavation, prewetting, blasting, and others. In the summer of 1983, a dynamic compaction (D.C.) test project was set up with the cooperation of a local contractor. The project consisted of dropping a 6-ft (1.8-m) diameter, 15-ton (13.6-metric ton) culvert-encased concrete weight freefall from a height of 50 ft (15.2 m). A total of 56 drops was made on five crater locations on the project. Results indicated that this method would be effective for treatment of the soils. Contract specifications for the project, such as the drop pattern shown in Figure 3, were developed from the test project. The construction specification developed was basically procedural, requiring a minimum of five drops of the weight per drop point for the first pass and three drops for the offset second pass following site leveling of the first-pass craters. Provision was made for additional drops if weight penetration on the final of the five- or three-drop sequence exceeded 10 percent of the total penetration up to that point (*3*). With the necessary depth of compaction known, the weight and drop height were based on the following empirical formula (*4*):

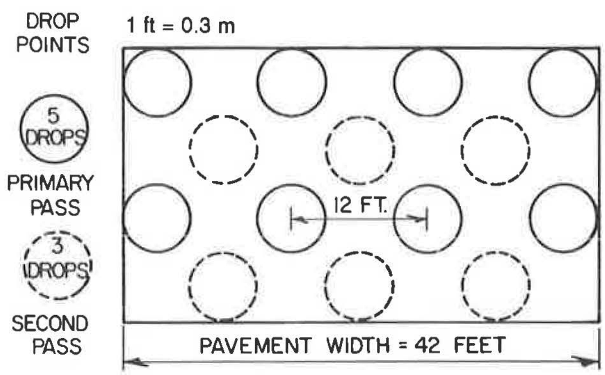

FIGURE 3 D.C. drop pattern.

$$D = E_D (W \times H)^{1/2}$$

where

D = depth of influence (m),
E_D = efficiency (½ or 50 percent for estimating),
W = falling weight (metric tons), and
H = drop height (m).

The project design depth of influence was 20 to 25 ft. With a 15-ton weight and 60-ft drop height,

$$D = \tfrac{1}{2} (15 \text{ tons} \times 60 \text{ ft})^{1/2}$$
$$D = 26 \text{ ft } (7.9 \text{ m})$$

Because 90 to 95 percent of the soils needing treatment extended to a depth of about 20 ft (6.1 m), with the remainder extending to about 30 ft (9.1 m), the added 6 ft of treatment depth in the 26 ft was considered a good safety margin.

RESULTS

Public Relations

Before the test project was performed in 1983, letters announcing the test drops were circulated to most of the residences within about ¼ mi (0.4 km) of the project area. Some public interest was generated by these letters. One party had recently purchased an office building and house and wondered why the walls were cracked and the floors not level. The possibility that the structures had been built on poor foundation soils was explained to him.

The "production" D.C. work started at the west end of the westbound lanes on August 9, 1984, and proceeded east until completion of these lanes on September 19, 1984. After winter shut-down, work started on the east end of the eastbound lanes on February 28, 1985, and proceeded west until completion of these lanes on April 3, 1985.

Soon after the production project started in 1984, at the west end of the job, local residents complained about the contractor's working hours of 4:00 or 5:00 a.m. to 10:00 or 11:00 p.m. These complaints were generally resolved by the contractor.

Studies of available literature were made before the start of the project to determine safe distances for D.C. work to prevent structural damage (*4*). Of particular concern at the time was a large, high-pressure gas pipeline located about 75 ft (23 m) from a portion of the east end of the project area. A decision was made to perform vibration monitoring in this area during the D.C. work using the U. S. Bureau of Mines criterion for peak particle velocity of 2 in./sec (50 mm/sec) or less to prevent damage to structures. A minimum distance from the impact points to the pipeline was calculated to be about 45 ft (13.7 m). Frequency of vibration was estimated to range between 5 and 20 Hz (*5*).

Because the D.C. work was to pass fairly close to several residences on the west end during compaction of the eastbound lanes, it was decided to perform crack and damage survey and vibration monitoring near these residences. In the fall of 1984, shortly after this decision was made, a claim for damages attributed to the D.C. period for the westbound lanes was made for one of these residences.

The crack and vibration surveys were performed in March and May 1985 by Northern Engineering and Testing (*6*). Peak particle velocities were within the recommended safe level for residents, as shown in Figure 4. After D.C. work, some cracks showed an increase and some a decrease in size as compared with their measurement before D.C. A majority of the cracks measured, however, showed no change or change within what was considered an accuracy of measurement tolerance for no movement. Cracks and damage found from the field surveys appeared to be preexisting and on-going at the three residences. Damage appeared to have been caused by hydroconsolidation of the foundation soils because of poor drainage around the structures. These residences, as well as other structures in the area, appear to have undergone distress primarily from the

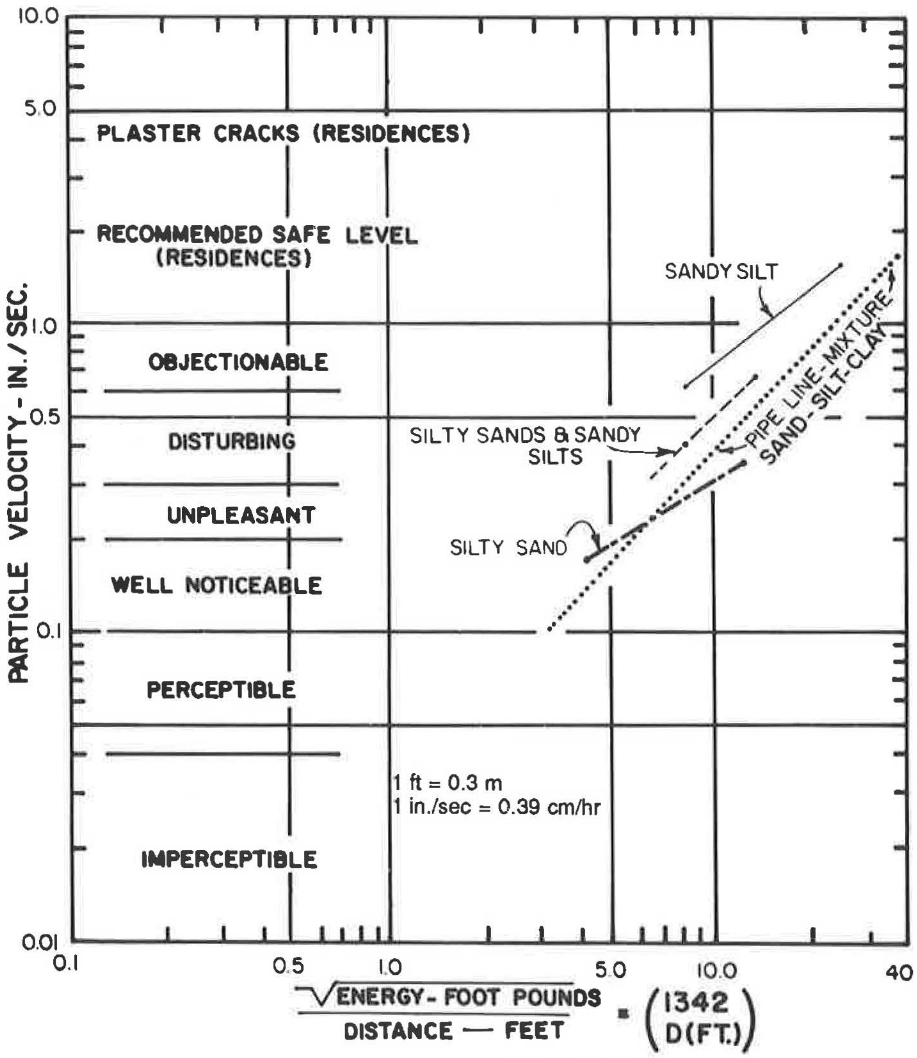

FIGURE 4 Scaled energy factor versus particle velocity.

same problems experienced by the Interstate highway in this area before the D.C. work.

Equipment Used

The D.C. equipment used on this project, with the exception of the weights, was manufactured or assembled by Neil F. Lampson, Inc., Kennewick, Washington. The two 15-ton weights were constructed by the general contractor, Hilde Construction Company.

The two compactors constructed by Lampson weighed approximately 114 tons (103 metric tons) each and were capable of lifting the weights from drop heights of almost 70 ft (21 m). The compactors moved forward and backward and turned by means of a tracked-base transporter unit. According to Lampson personnel, capacity of the transporter was 350 tons (317 metric tons) and capacity of the draw works, 500 tons (454 metric tons).

The weights were lifted by using a 1¼-in. (3.2-cm) diameter straight lay or "no spin" cable. The cable was attached to a swivel connected to two short chains, which in turn were connected to opposite sides of the top of the weight. The two connections to the weight, rather than one center connection, allow better control of the weight. The chains were threaded through several old automobile-size tires to reduce chain wear. Later it was discovered that swivel damage could be substantially reduced by threading the chains and automobile tires through a used large-diameter heavy-equipment tire. The combined weight of all the tires added approximately ¾ to 1 ton (0.7 to 0.9 metric ton) to the 15-ton compacting weight.

Craters

Craters, of course, are the immediate measurable results of D.C. Records of penetration depths and number of drops were taken for all craters on this project. Number of drops versus penetration can be plotted and indicate whether the energy input from the drops is being efficiently utilized. The D.C. was performed on 1.5 to 2.0 ft (0.3 to 0.6 m) of base-course materials left in place after removal of the existing pavement.

Figures 5 and 6 are drop curves for an area with and without 2 ft of base-course material and for an area with frost versus a frost-free area, respectively. The test area with base-course materials removed required extra drops by the special provision's 10 percent requirement, indicating the high resistance of the base-course material. Field testing, however, did not indicate any increase in the effective depth of improvement or density properties of the soils.

D.C. work on the eastbound lanes started in February 1985. Soil beneath the paved areas was frozen to depths of approximately 4 to 6 ft (1.2 to 1.8 m), whereas frost in adjacent open

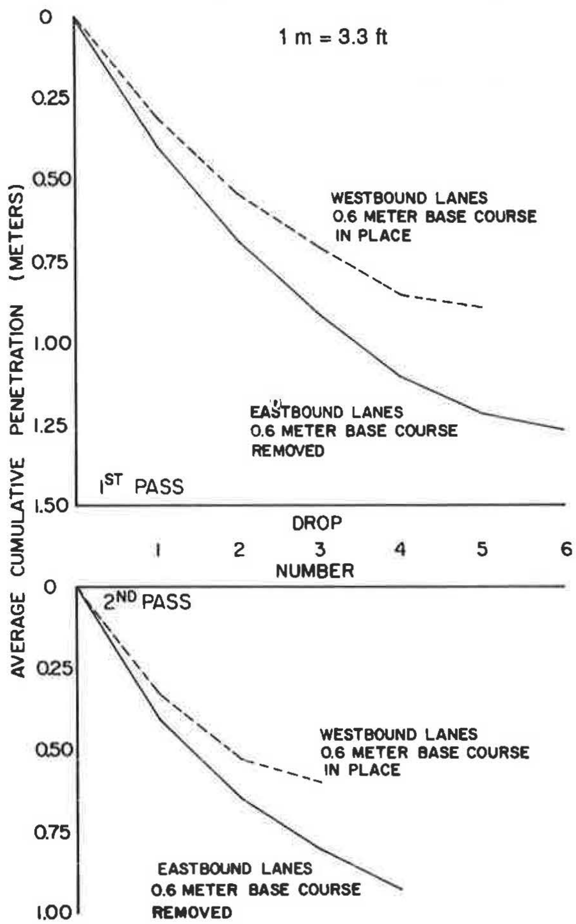

FIGURE 5 Crater depths as function of number of drops with and without 2-ft base course, Station 257± to 258±.

areas was generally less than 1 ft (0.3 m). The contractor was advised that the effects of the D.C. would be very limited in frozen ground. At the beginning of pavement removal, the contractor elected to rip the subgrade soils. This proved satisfactory, leaving only some pieces of frozen soil a maximum size of 10 to 12 in. (25 to 30 cm) nominal dimension. As pavement stripping continued and temperatures rose, the frost rapidly left the ground. The areas with frost required extra drops to meet the special provision's 10 percent requirement as compared with a frost-free area. Again, field testing did not

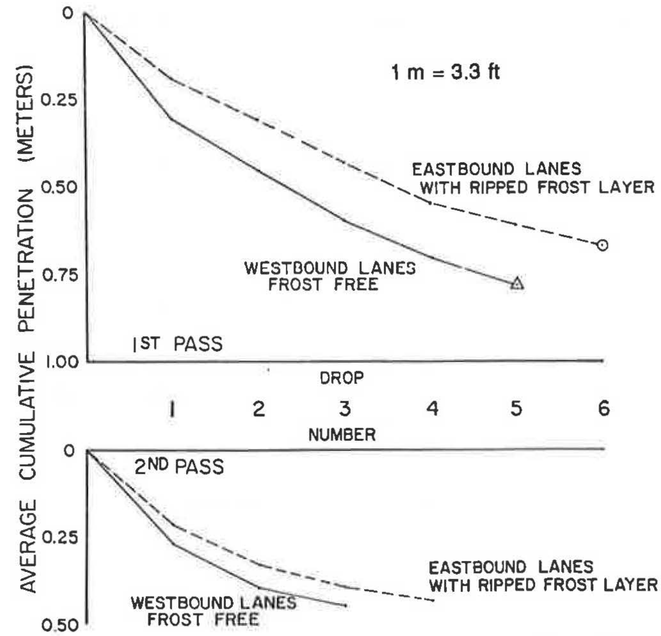

FIGURE 6 Crater depths as function of number of drops for area with frost versus a frost-free area, Station 272+00 to 272+50±.

indicate any differences in the depths of improvement or soil densities for areas with frost versus frost-free areas.

Field Testing Data

Besides the drop-penetration records, other drilling and field testing was performed. Standard penetration, dynamic cone penetration, and some Shelby tube density, as well as Dutch cone penetration tests were performed at various locations. Typical before-and-after D.C. results and qualitative statements of improvement are shown in Figure 7 and Table 1. Average increase in *N*-value for the improved soils was 5 to 12. Average lowering of the ground surface after D.C. and final compaction of the upper 2 ft (0.6 m) or so of loose material was about 0.7 ft (21 cm). A maximum of about 30 in. (76 cm) was originally projected from laboratory consolidation testing. As found during the initial drilling investigation, Shelby tube densities were very difficult to take and density results were inconclusive. Dutch cone penetration tests were also inconclusive, possibly because of the limited amount of Dutch cone testing done or types of foundation soils present, or both.

Time and Cost

A time-and-cost summary is presented in Table 2. Final treatment cost for the D.C. was \$6.97/yd^2. This does not include the ±\$0.30/yd^2 cost for standard compaction methods used on the upper 2 ft or so of loose material left for final grading after D.C. Approximately \$2.6 million was saved by using D.C. instead of other alternatives, as shown by the following cost comparisons (the actual cost per cubic yard for excavation was based on the

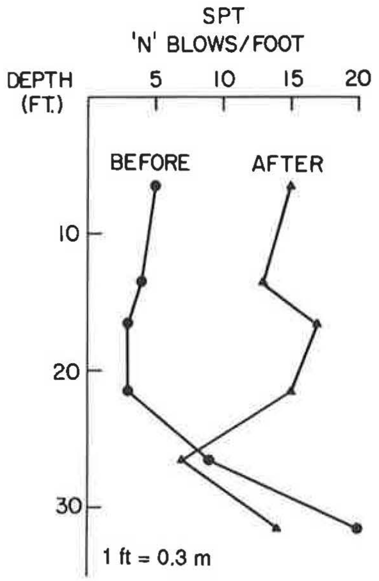

FIGURE 7 Typical field testing results before and after D.C.

cost from other jobs with similar quantities of excavation and placement):

	Engineers' Estimate	*Actual Bid or Cost*
Subexcavation (20-ft depth)	\$3,078,800 (\$2.15/yd^3)	\$3,393,840 (\$2.37/yd^3)
Dynamic compaction	−1,129,030 (\$10.00/yd^2)	−775,130 (\$6.97/yd^2)
Total savings	\$1,949,770	\$2,618,710

SUMMARY: MAJOR CONCLUSIONS AND RECOMMENDATIONS

The D.C. project in Montana was a result of technology transfer and encouragement from FHWA to try a new geotechnical procedure that apparently could be performed by almost anyone who uses the following guidelines, as well as those in the reference material (7). A D.C. project is particularly feasible in dry collapsible-type soils such as those found on this project.

Vibration Levels

Vibration monitoring for this project indicated that peak particle velocities were well within the recommended safe level for residences. The perceptible level of vibrations was high enough to be felt by those in the immediate project area and became a cause of concern to them.

Even though the particle velocities were within the safe range for this project, performance of vibration and crack-type surveys for any structures within the perceptible range of the D.C. work is recommended. These types of surveys would be mandatory for work in urban areas and should be done in conjunction with a general presentation to the public explaining the D.C. process, vibration perception and damage levels, and vibration and crack surveys.

Test Section

A small test section using the D.C. process in the project area proved invaluable in design of the production work. Results from this test section correlated with other examples. It is recommended that D.C. work be done on a test section unless good correlation from previous D.C. projects can be shown. This will demonstrate whether the procedure will work or not and is extremely useful in developing a procedural type of specification indicating patterns for craters and number of blows as well as weight, size, and drop heights. It is recommended that specifications allow flexible use of weight drops. For example, if five weight drops were set up and only two were used to achieve density, the remaining three would not be paid for unless used at a different location.

Resistant Layers

D.C. was effective on all but approximately 5 percent of the area compacted. The majority of materials treated on the

TABLE 1 IMPROVEMENT OF PENETRATION RESISTANCE AND SOIL PROPERTIES ON BASIS OF STANDARD PENETRATION TEST

	Silt			Sand				Clay		
	Type 1	Type 2	Type 3	Type 1	Type 2	Type 3	Type 4	Type 1	Type 2	Type 3
Initial N-value	0–4	4–10	10–30	0–4	4–10	10–30	30–50	4	4–8	8–15
Relative density or consistency	Very loose	Loose	Medium dense	Very loose	Loose	Medium dense	Dense	Very soft to soft	Medium stiff	Stiff
Avg increase in N	4	9	(−7)[a]	12	9	5	(−10)[a]	8	6	6
Improvement range	N.A.	13–19	10–18	12–16	13–19	15–30	18–23	8–12	10–14	14–19
Relative density or consistency	N.A.	Medium dense	Medium dense	Medium dense	Medium dense	Medium dense	Medium dense	Stiff	Stiff	Stiff–very stiff

Note: N = blows per foot.

[a]Represents data from only one test boring.

TABLE 2 TIME-AND-COST SUMMARY

	Westbound	Eastbound	Total
Starting date	08-09-84	02-28-85	08-09-84
Ending date	09-19-84	04-03-85	04-03-85
No. of working days (2- to 10-hr shifts/day)	30	25	55
Treated area (yd^2)	54,694	56,588	111,282
Total drops	29,366	31,146	60,512
Extra drops (by special provision)	1,236	1,718	2,954
Cost of extra drops ($) ($10/drop)	12,360	17,180	29,540
Cost of regular D.C. work ($) ($6.70/yd^2)	366,450	379,140	745,590
Total cost ($)	378,810	396,320	775,130
Final cost per drop ($)	12.90	12.72	12.81
Final cost per square yard ($)	6.93	7.00	6.97

project consisted of very loose to loose silty sands and sandy silts. The average increases in *N*-value for the improved soils varied between 5 and 12. Areas not improved by the actual D.C. performed included scattered pockets of loose soils located deeper than the design improvement depth, wet clay soils, and areas with scattered shallow dense gravel layers. Quality control testing did not indicate that the presence of a 2-ft thick base-course mat or a "ripped" frost layer was detrimental to the final results on this project. The weight size and drop height design appeared to overcome the resistance of these layers on this project. More energy (extra drops), however, was required at additional cost in the frost areas and in the test section where base-course gravels had been removed. It is recommended that hard layers be noted and a determination be made on the economy of removing them or designing the weight and drop height to penetrate them.

Correlation

It was determined that a correlation could be made between the semiarid unglaciated areas of Montana and the presence of known or suspected deposits of hydrocompactible soils. Improvement in the strength of these materials was accomplished on this project by the simple and economical process known as D.C.

The correlation of semiarid areas, material types, and the presence of hydrocompactible soils is known to exist in other states. As more construction projects are relegated to marginal use land, a rough land use planning map should be prepared for other states showing areas of detrimental hydrocompactible soils.

The D.C. process is available to anyone and is recommended as a safe, simple, and economical way to improve soil strengths at great depths.

ACKNOWLEDGMENT

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