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Improvement of a Dumped Rockfill Foundation by Dynamic Consolidation

Adrian Wightman
Klohn Leonoff Ltd., Richmond, British Columbia, Canada

Nelson F. Beaton
N. F. Beaton Consulting Ltd., Richmond, British Columbia, Canada

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Adrian Wightman

Manager, Geotechnical Division, Klohn Leonoff Ltd., Richmond, B.C., Canada

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SYNOPSIS The paper describes an Industrial Park development, on the British Columbia coast, where Dynamic Consolidation was chosen to densify a loose rockfill prior to construction of a modern high capacity sawmill on shallow foundations. The design and execution of the compaction are described. The in situ tests used to monitor the compaction are described and results presented. It is concluded that the desired result was achieved, and this is supported by a survey of foundation elevations taken four years after construction.

INTRODUCTION

The Duke Point Industrial Park is a project of the British Columbia Development Corporation (BCDC), just south of the coastal city of Nanaimo on Vancouver Island, British Columbia. The Park was created by quarrying bedrock material from two ridges, Jack and Duke Points, and placing it as fill to form a reclaimed platform extending into Northumberland Channel.

Of the 155 ha total area, 12.2 ha forms the site of a new sawmill owned by Doman Industries Ltd. The major part of the property occupied by the mill buildings is underlain by coarse, end-dumped, rockfill up to 15 m thick. After considering alternative foundation proposals, the Dynamic Consolidation (D.C.) process was chosen for compacting the fill to limit settlement and allow foundation support on spread footings.

This paper describes the design and execution of the D.C. work. The methods used to monitor the improvement in the compacted rockfill are described and discussed. Footing elevations taken four years after construction are presented to demonstrate that the performance to date has been satisfactory.

SITE DESCRIPTION

Location

The Duke Point Industrial Park occupies the site of two north to northeast trending ridges of bedrock, Jack Point and Duke Point. The Nanaimo river estuary lies to the west, and the east boundary adjoins Northumberland Channel. See Fig. 1.

Geology

The site consisted mostly of exposed bedrock with a very thin cover of topsoil. Bedrock consists of massive sandstone of the De Courcy Formation having an average thickness of 300 m.

The sandstone ranges from coarse to fine grained. It is moderately to weakly indurated, well compacted, and of moderate strength. The bedding is usually thick and massive.

Unconfined compression tests on seven samples free of natural discontinuities gave results ranging from 45 MPa to 110 MPa with an average of 74 MPa.

Sawmill Development

The mill covers an area of 12,800 m^2. It is a steel frame building with simply supported spans and bay spacings typically 6 m x 6 m. It contains some heavy vibrating equipment including a Hog and a Barker. When operating at full capacity, the mill produces 220,000 board feet of dimensioned lumber per shift.

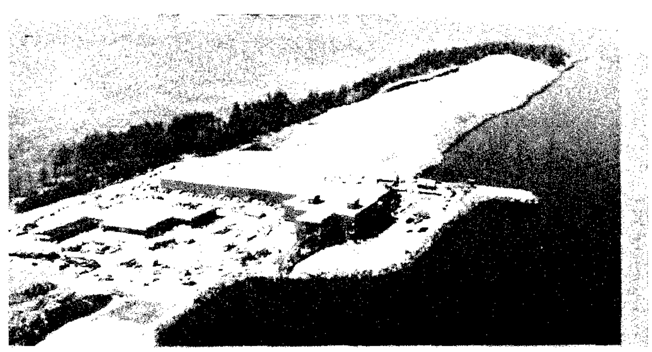


Fig. 1. Completed Mill, Looking North

The ground conditions beneath the sawmill can best be seen on section A-A, Fig. 2, which shows that most of the mill structures are located on rockfill varying in thickness from 0 on the west side to 15 m at the shoreline.

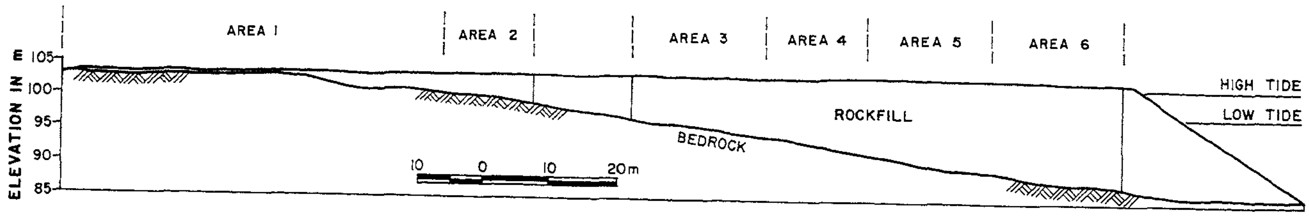


Fig. 2. Subsurface Profile, A-A.

FILL MATERIALS

Fill for extending the site beyond the original shoreline to establish a level platform at El. 104 m was obtained by quarrying sandstone bedrock from adjacent higher levels of the development. The actual rockfill gradation produced by controlled blasting was reasonably close to the specified range shown in Table 1, except that the actual maximum size was about 1 m.

The rockfill was loaded into trucks in the quarry and dumped at the advancing fill face where it was graded with a D-9 bulldozer.

TABLE 1 Gradation Of Rockfill

Sieve Size mm	Percentage Passing
600	100
300	50 - 100
200	40 - 100
75	15 - 50
37.5	10 - 35
19	5 - 25
4.75	0 - 10

FOUNDATION OPTIONS

The Industrial Park site development contract called for compaction of the rockfill placed above mean tide level in maximum 1 m thick layers. This would leave an uncompacted zone, below mean tide level, varying up to 12 m thick. See Fig. 2.

During the early planning stages of the sawmill project, three foundation support options were considered.

a) Drilled cast-in-place piles socketted into sound bedrock.

b) Spread footings founded on the rockfill compacted in layers above mean tide.

c) As for (b) above but with the addition of a preload-surcharge fill above site grade for a short period before construction.

The objection to (a) was the high cost. Option (b) raised the question of the long term settlement performance of the uncompacted loose dumped rockfill zone below mean tide. Sherard et.al. 1963), report that the post construction setlement of sluiced, high lift rockfills is highly variable, depending on the type of rock, gradation, and details of construction. They quote observations that, for fill heights of up to about 30 m, gave post construction settlements varying between about 0.6% and 1.3% of the fill height. If compression of 1% is assumed for the uncompacted fill zone below mean tide it leads to an expected maximum total settlement of 130 mm and a settlement gradient of 1/700. These figures were outside the range of values considered by the owner and the mill designers to be tolerable for the mill structures and equipment.

Option (c) was considered as a means of minimizing post construction movements, but the cost, logistics, and scheduling problems were formidable.

An alternative approach to site development and foundation support was found in the Dynamic Consolidation process. The technique was thought to have the ability to rearrange and compact the rockfill pieces and generally improve the future settlement performance of the fill. The fact that the fills were young improved the effectiveness of treatment by D.C. since no energy was expended in breaking through denser near surface soils often found in older fills. Furthermore, since it was decided to use D.C. to treat the entire fill profile, fill was end dumped to the finished grade elevation with no compaction above the tide level and the Owner was granted a considerable rebate by BCDC. The technique was therefore considered to be economically attractive as well as a technically sound solution to the problem.

THE DYNAMIC CONSOLIDATION PROCESS

The Equipment

Compaction was carried out using a 20 tonne tamper which was raised and dropped through a height of 30 m with an 1800 Model Lima Crawler Crane. A special tamper was built of hexagonal steel plates held together with large diameter steel studs. The tamper was approximately 4 ft high and 6 ft in diameter.

The tamper was sized on the basis of material type, maximum extent of fill to be treated and the empirical relation $D=K\sqrt{W.H.}$ where D is the effective depth of compaction, W is the weight of the tamper in tonnes, and H is the drop height in metres. The factor K varies with soil type from 0.5 for fine grained soils to 0.8 for coarse grained materials.

TABLE II Summary of Compaction Applied

Zone No.	Area m^2	Ave Depth of fill -m-	Phase 1 Energy $T\text{-}m/m^2$	Phase 1 Crater Grid -m-	Phase 1 Blows per Crater	Phase 2 Energy $T\text{-}m/m^2$	Phase 2 Crater Grid -m-	Phase 2 Blows Per Crater	Phase 3 Energy $T\text{-}m/m^2$	Phase 3 Crater Grid -m-	Phase 3 Blows Per Crater	Phase 4 Energy $T\text{-}m/m^2$	Phase 4 Crater Grid -m-	Phase 4 Blows Per Crater	Total Energy per Zone Tm/m^2
1	7088	0–3	80–190	3x3	2–3	–	–	–	–	–	–	–	–	–	80–190
2	3218	3.9	202	5x5	9–7	146	3x3	2	–	–	–	–	–	–	348
3	3405	9.0	102	13x13	20	172	6x6	10	115	3x3	3	–	–	–	389
4	2781	10.8	66	15x15	20	164	7.5x7.5	20	38	7.5x7.5	3	167	3x3	3	435
5	2872	13.7	44	18x18	25	116	9x9	20	134	9x9	20	185	3x3	3	479
6	1842	15.3	77	18x18	40	154	9x9	30	129	9x9	20	216	3x3	3	576

Compaction Design and Execution

A specialist Contractor, Geopac Inc., was engaged to carry out the final design and execution of the work. The project requirements were for compaction to achieve a uniformly dense fill such that individual post-construction footing settlements should not exceed 13 mm, and the differential settlement gradient across a structure bay should not exceed 1/1000.

When compacting to a significant depth the compactive energy must be applied in stages. The initial grid of compaction points must consist of widely spaced high energy impacts. For granular fills the initial stage generally has a grid spacing equal to the depth of fill. Subsequent stages follow at progressively smaller spacings with, possibly, lower energy input. In this way, the fill is in effect compacted from the bottom up as columns of compacted fill are created on gradually decreasing spacings. Creating a hard surface crust that would prevent subsequent compaction at depth must be avoided.

The site was subdivided into six zones based on the average fill thickness in each zone. Each zone was then subjected to up to four phases of treatment with compaction equipment.

The initial choice of energy input per phase is based on experience and often is adjusted based on site testing. Since no excess pore pressures could be generated in rockfill, there was no limit to the amount of energy which could be applied in each phase. Test craters, where base elevations are taken after each drop of the tamper, give an indication of the optimum energy level per grid point for a given compaction phase. Fig. 3 shows a zone 6 crater depth versus impacts plot, and illustrates that diminishing returns are realized beyond about 25 blows.

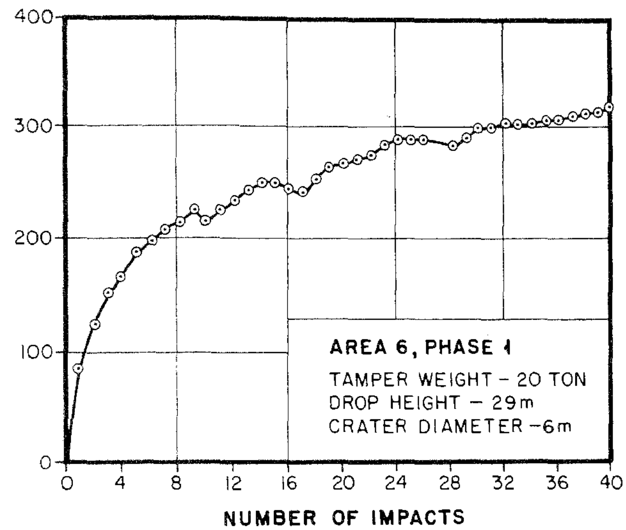


Fig. 3. Typical Crater Depth-Blows Relation

Fig. 4. Example Zone 6 Crater

The zones, number of passes, energy per pass and total energy input are summarized on Table II. The locations of individual compaction points are shown on Fig. 5.

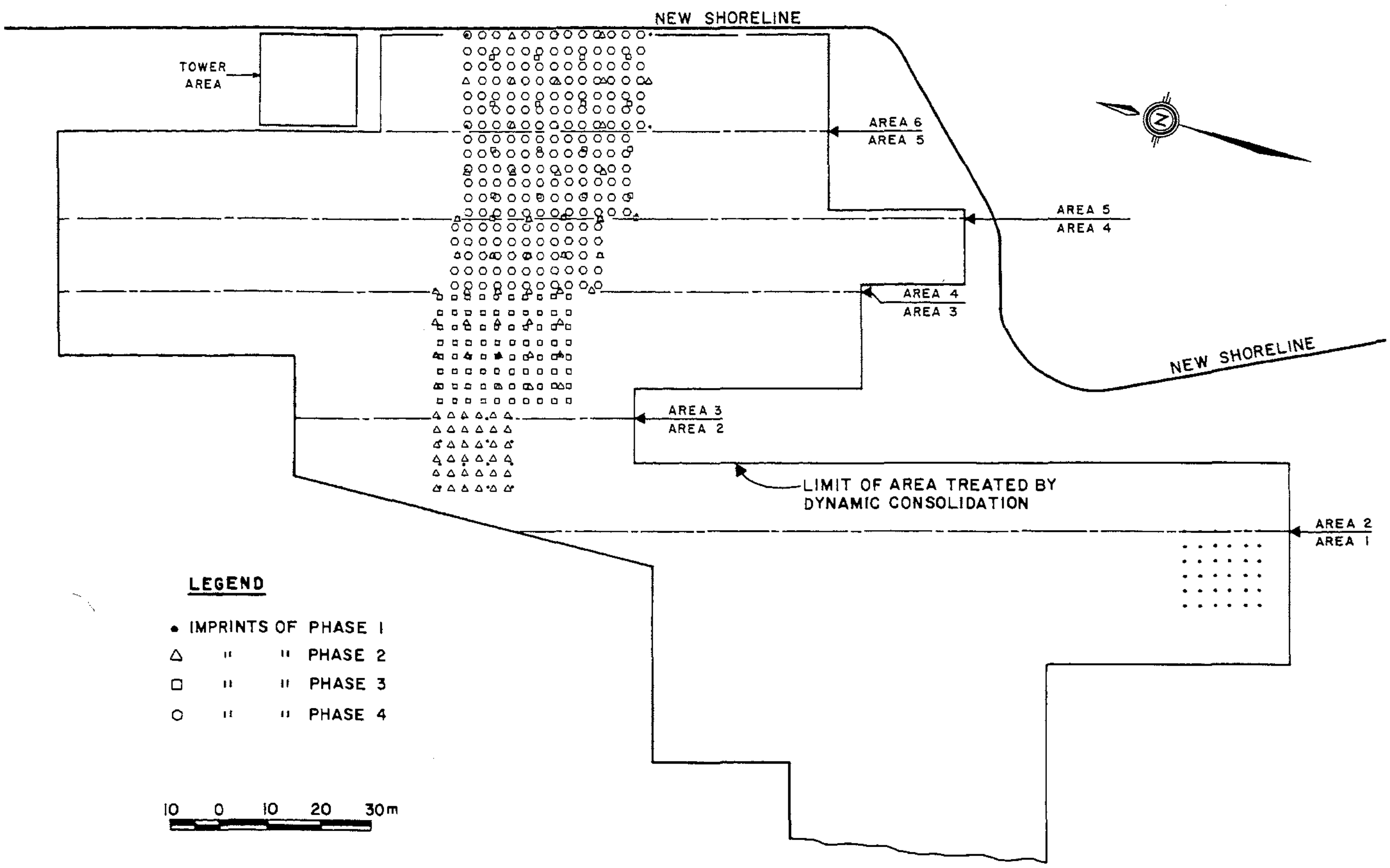


Fig. 5. Plan of Compaction

CONTROL TESTS

Rockfill is a difficult material to test using conventional geotechnical methods. Consequently, several methods were tried.

Pressuremeter

Prior to any compaction work, twelve boreholes were drilled through the rockfill and cased with BW casing. Specially slotted AW casing was then inserted and the BW drill casing withdrawn. The AW casing was slotted at specific intervals rather than over its full length in order to preserve enough strength to prevent collapse during compaction. Before and after compaction pressuremeter tests were run at the level of the slotted sections. After compaction, additional tests were performed at intermediate depths by partially withdrawing the casing from the hole. Prior to the down hole testing, calibration tests were run on unconfined sections of slotted casing to allow interpretation of the 'net' pressuremeter curve of the rockfill. Sample pressuremeter results are shown on Fig. 6. Fig. 7 shows the test locations.

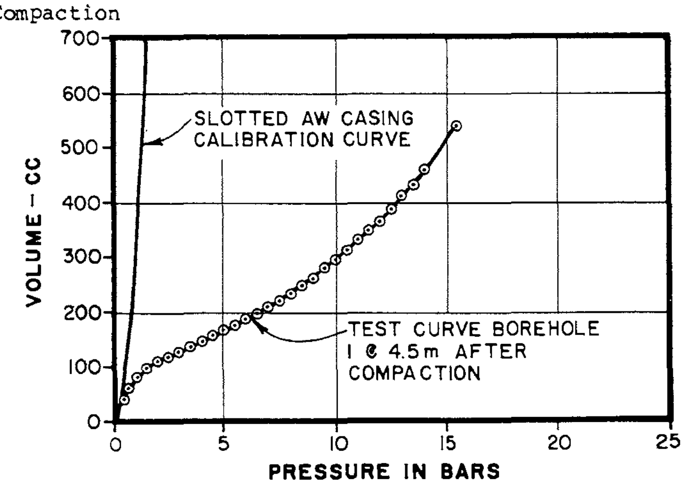
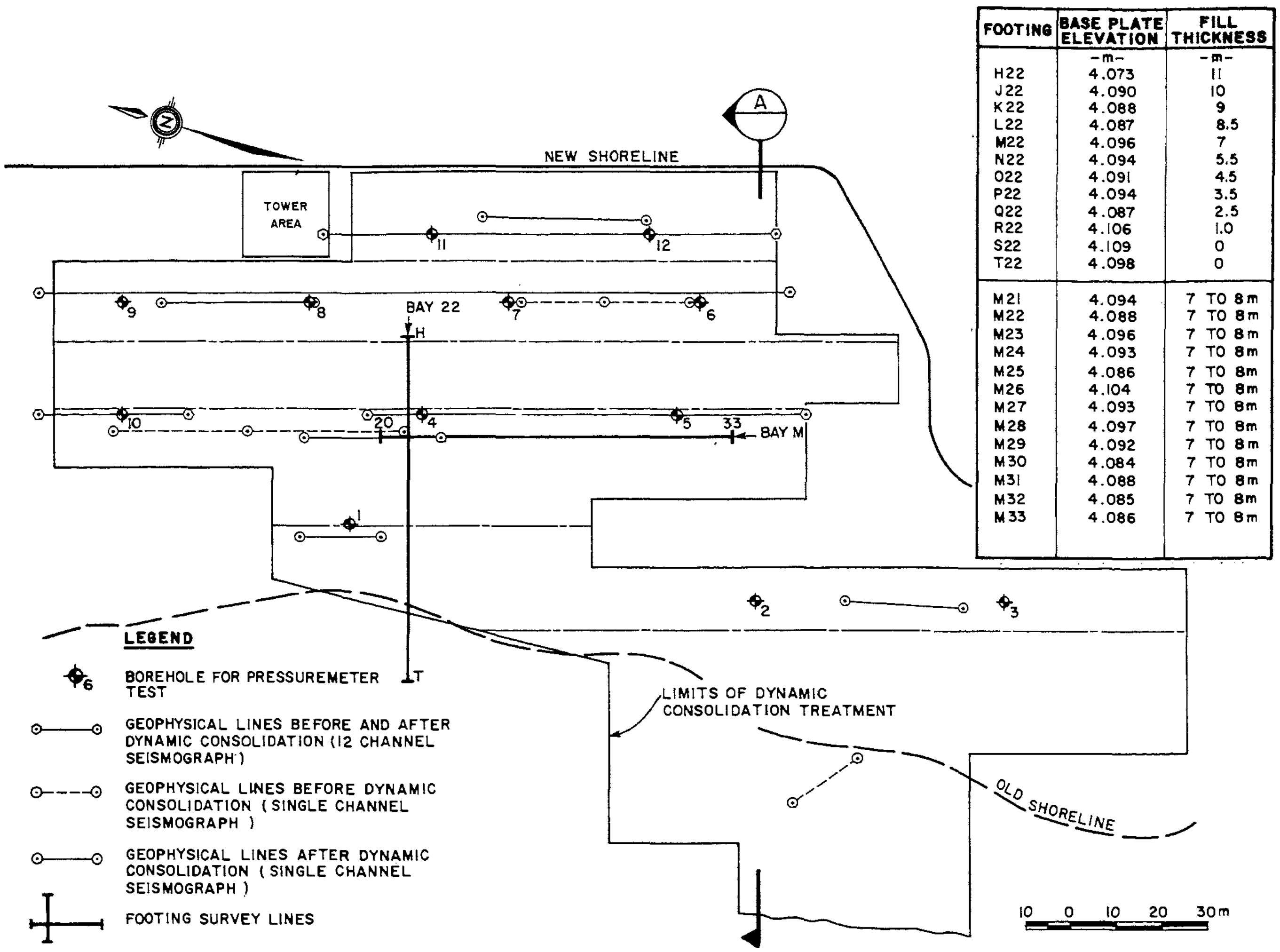


Fig. 6. Example Pressuremeter Test After Compaction

Geophysical Tests

Measurements of shear and compression wave velocities were made on the rockfill before and after compaction using seismic techniques.

Two sets of testing were done, one with a twelve channel signal enhancement seismograph configured to measure surface, Rayleigh, wave



FOOTING	BASE PLATE ELEVATION	FILL THICKNESS
	-m-	-m-
H22	4.073	11
J22	4.090	10
K22	4.088	9
L22	4.087	8.5
M22	4.096	7
N22	4.094	5.5
O22	4.091	4.5
P22	4.094	3.5
Q22	4.087	2.5
R22	4.106	1.0
S22	4.109	0
T22	4.098	0
M21	4.094	7 TO 8m
M22	4.088	7 TO 8m
M23	4.096	7 TO 8m
M24	4.093	7 TO 8m
M25	4.086	7 TO 8m
M26	4.104	7 TO 8m
M27	4.093	7 TO 8m
M28	4.097	7 TO 8m
M29	4.092	7 TO 8m
M30	4.084	7 TO 8m
M31	4.088	7 TO 8m
M32	4.085	7 TO 8m
M33	4.086	7 TO 8m

Fig. 7. Plan of Control Tests and Survey Line Locations

propagation, and the other with a single channel signal enhancement seismograph measuring refracted S waves. Locations of seismic lines are shown on Fig. 7.

Measurements of Crater Imprints

Throughout the dynamic consolidation work, records were maintained of the crater depths, the crater volumes and the number of blows of the tamper. A typical relation between crater depth and number of blows is shown on Fig. 3. From this it is apparent that little extra compaction is being achieved beyond 25 blows. By periodically monitoring this compaction relation, the optimum energy was maintained as the fill thickness and compressibility varied across the site. See Table II for blows per crater in each zone.

This procedure yielded an unexpected benefit in one small area of the site. The tamper behaviour changed dramatically indicating soft ground. Upon investigation with trial pits a narrow gully in the bedrock was found infilled with soft clay, concealed by a thin covering of glacial soil and rockfill. This area was given special treatment by D.C. whereby columns of compacted rockfill were formed on close spacings. During this work the soft clay was extruded from the ground between the columns.

Measurement of Induced Settlement

The average settlement induced in the fill by the compaction was calculated for each compaction zone by observing the initial and final fill surface elevations and making allowance for the amount of material imported to fill the craters and bring the site to final grade. The final induced settlement is illustrated on Fig. 8 expressed as a percentage of fill thickness. The average value is 5.6% which lies within the range of 3.5 to 10% observed for some dozen or so rockfill compaction projects around the world.

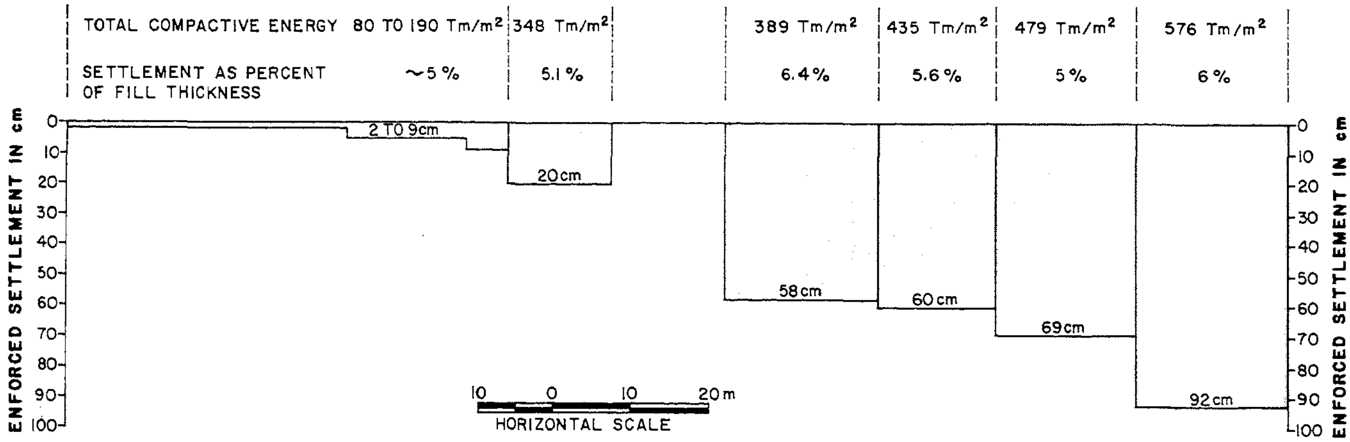


Fig. 8. Induced Settlement at End of Compaction

EVALUATION OF CONTROL TESTS

Pressuremeter

Eighty-seven pressuremeter tests were performed, distributed over 10 boreholes. Twenty-six of these were done in the loose fill prior to compaction and sixty-one in the fill after compaction. The fact that only forty-seven of these tests are interpreted as being useable is a measure of the difficulty of pressuremeter testing in rockfill. But even this low success rate might be considered good given the relative size of the rockfill particles compared to the testing instrument.

The forty unusable tests can be classified as follows:

No. of Tests	Interpretation
30	Slotted casing not in contact with the walls of the drill-hole, or test interval adjacent to a large void. This problem affected all of the pre-works tests.
3	Tests carried out inside boulders.
7	Irregular pressure deformation curves due to relative movements of blocks adjacent to the test hole.

Even the thirty-seven tests considered usable can be rated for reliability based on the shape of the test curve and the volume of fluid required to achieve initiation of the linear portion of the volume pressure curve. A relatively high value for initial volume, greater than 200 cc, indicates the presence of some voids adjacent to the probe, or some degree of borehole disturbance probably caused during retraction of a bent casing. Only eight of the thirty-seven tests were considered to give test data that were truly representative of the strength and deformation properties of the fill. The remainder are believed to understate these properties. One of the better test results after compaction is shown on Fig. 6.

Fig. 9 shows profiles of limit pressure and Pressuremeter modulus before and after compaction. All test data are plotted. It is significant to notice that even though many of the tests taken after compaction are of poor quality they still indicate that the compaction tightened up the rockfill against the walls of the casing to depths up to 13 m.

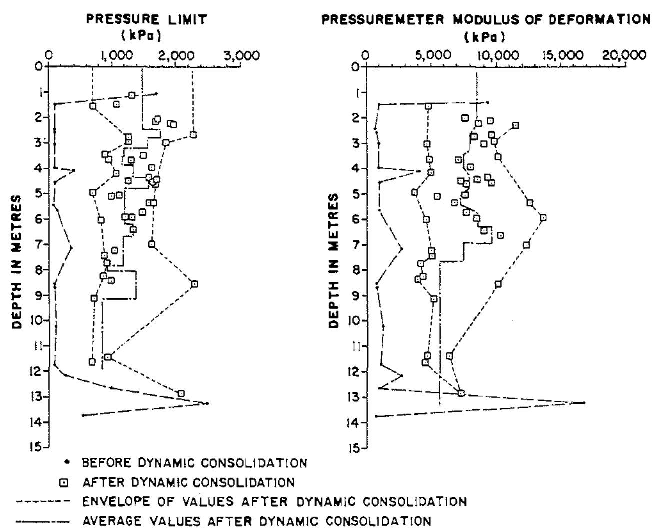


Fig. 9 Pressuremeter Results Profile

The results of the pressuremeter tests were used to estimate footing settlements according to the empirical methods presented by Menard (1975). The settlement estimated for a 3 m x 3 m footing loaded to 300 kPa was 10 mm. Given the specified tolerance of 13 mm and knowing that moduli resulting from the pressuremeter tests were conservatively low, this result was taken as an indication of satisfactory compaction.

Geophysical Test Results

Since the shear and Rayleigh wave velocities, V_s and V_R, in an elastic medium are related to the shear modulus, G, and mass density, ρ, by:

$$G=\rho V_s^2 \approx 1.1\rho V_R^2 \qquad (1)$$

The objective of the geophysical program was to detect increases in V_s that would signify a satisfactory increase in G.

The test results suggest that there is a near surface zone in which the seismic velocity did not change significantly from before to after compaction. Before compaction an average of V_R=298 m/s was measured, compared to 292 m/s after compaction. It is not clear how thick this surface zone is. By measuring refracted shear wave velocities, an indication of improvement at depth was obtained. The average shear wave velocity measured before compaction was 250 m/s. After compaction, an average value of 460 m/s was obtained. This suggests that the modulus was improved by a factor of 3.4.

The foregoing indicates, qualitatively, that the compaction produced significant stiffening of the rockfill. A quantitative guide to foundation performance can be obtained as follows.

Using rockfill gradations and void ratios reported in the literature as a guide, an estimate of void ratio, e = 0.5 was made for the Duke Point rockfill. This leads to the result that;

$$G_{seismic} = \rho V_s^2 = 1.77 \times 300^2 = 159 \text{ MPa} \qquad (2)$$

$G_{seismic}$ corresponds to a very low level of shear strain, considered to be about 10^{-6}, and can therefore be considered equivalent to G_{max}. It is well documented that for soils the value of G varies with shear strain, Hardin & Drnevich (1972), Seed and Idriss (1970). Ishihara (1982) shows a modulus reduction curve, in the form of G/G_{max} vs shear strain γ, for crushed rock. See Fig. 10. Assuming the shape of this curve is representative of the Duke point materials, the modulus to be applied to foundation design can be estimated.

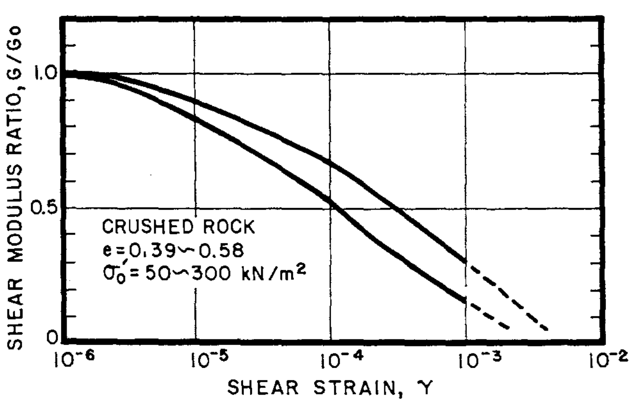


Fig. 10. Modulus Reduction Curve for Crushed Rock [After Ishihara (1981)]

A result from elastic theory states that settlement of a rigid plate, δ, is given by:

$$\delta = \frac{P\,(1-\nu^2)}{\beta\sqrt{BL}\;E} \qquad (3)$$

where P = total load
ν = Poisson's ratio
β = a constant dependant on B/L
B,L = footing dimensions
E = Young's Modulus = $2G(1+\nu)$

Scott (1981) shows that for soils this equation is likely to overestimate the settlement by a factor of about 3 for footings of dimensions 3 m. A comparison of shear and compression wave velocities, above the water table, suggests that ν=0.4 for the compacted rockfill. By estimating a value for G, and therefore E, from Fig. 10, an estimate of footing settlement can be made using equation (3). After considering several trial values of strain, close agreement was obtained between the average strain derived from predicted settlements and the assumed strain, when G/G_{max} = 0.1. The calculated settlement of a 3 m footing by this method is 10.5 mm.

Using a similar iterative procedure to obtain strain and modulus compatibility, the elastic compression of the 13 m thickness of rockfill under self weight can be estimated at 14 mm.

STRUCTURE PERFORMANCE

The settlement estimated from the elastic type analyses above are encouraging but they do not address the concern over time dependent post construction settlement which is a plastic type phenomenon. Confirmation that this had been eliminated was sought from an elevation survey of mill foundations.

An elevation survey was carried out in May 1983, three years after the mill construction was completed. Fig. 7 shows the survey line locations along with a tabulated summary of footing base plate elevations. The East-West line runs along the north end of the Sawmill Building and subfooting fill depths vary from zero in the west to 11.0 m in the east. The North-South line runs through the centre of the Sawmill Building and into the Barker Room to the south. Fill depths vary between 7 and 8 m. Since "as built" base plate elevations were not taken during construction, a detailed check of individual footing performance is not possible because there is a construction tolerance compared to design, of several mm. Foundation performance as a whole however can be evaluated based on the consistency of base plate elevations. For the East-West line note that footings S22 and T22 are on bedrock. Allowing for reasonable "as built" variation in base plate elevations it is clear that no significant differential settlements have occurred along the East-West survey line even though the fill depth varies uniformly from zero to 11 m.

The North-South survey line elevations are consistent for fill depths of 7 to 8 m. It is noteworthy that footings in the Sawmill and Barker are subject to cyclic stresses and vibration.

CONCLUSIONS

1. The Dynamic Consolidation process produced a compacted fill to meet the project specifications. This is borne out by the results of pressuremeter and geophysical tests before and after compaction, and by footing elevation survey four years after construction.

2. The compaction produced significant densification to the full 15 m depth of the fill as confirmed by the in situ tests.

3. While the pressuremeter program and the geophysical testing work yielded valuable results, neither one is an ideal quality assessment tool for rockfill. There are many physical difficulties involved in providing access for the pressuremeter probe, and significant scale effects that make the results suspect. Likewise for geophysical tests there are difficulties in applying enough energy to the ground to generate a recognizable signal at the recorder, and interpretation of the data requires an assumption of elastic behaviour that likely does not apply. Perhaps one of the most promising monitoring tools for rockfill compaction, one not attempted at Duke Point, is measurement of the deceleration of the tamping weight, interpreted as a dynamic plate test, as suggested by Hansbo (1977) and Mayne et al. (1983).

4. While in situ tests such as pressuremeter and seismic work can give valuable insight into the quality of the end product of compaction, the results and evaluations from these tests are generally not available until after the work is complete. It is important to have some measure of the compaction being achieved during the progress of the work. For this reason, it is important that an experienced contractor be retained to execute the work, and that a continuous check be made of induced settlement and optimum energy input, to ensure efficient and successful foundation treatment.

5. The Dynamic Consolidation method is particularly capable of detecting buried weak zones since the crater size and tamper behaviour immediately warn of changing ground conditions that can be treated appropriately.

ACKNOWLEDGEMENTS

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REFERENCES

Hansbo, S., Dynamic Consolidation of Rockfill at Uddevalla Shipyard. Proc. 9th Int. Conf. on Soil Mech. and Fndn. Engrng., Tokyo, Vol. 2.

Hardin, B.O., Drnevich, V.P., (1972) Shear Modulus and Damping in Soils: Design Equations and Curves. Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 98, SM7.

Ishihara, K. (1982), Dynamic Response Analysis. Proc. Int. Symp. on Numerical Models in Geomechanics, Zurich.

Mayne, P.W., Jones, J.S., (1983), Impact Stresses During Dynamic Compaction, Journal of Geotechnical Engineering, ASCE, Vol. 109, No. 10.

Menard, L. (1975), The Interpretation of Pressuremeter Test Results. Sols-Soils, No. 26.

Scott, R.F., (1981), Foundation Analysis, Prentice-Hall, Inc.

Seed, H.B., Idriss, I.M. (1970), Soil Mechanics and Damping Factors for Dynamic Response Analysis, Earthquake Engineering Research Centre, Report No. 70-10.

Sherard, J.L., Woodward, R.J., Gizienski, S.F., Clevenger, W.A., (1963). Earth and Earth-Rock Dams, John Wiley and Sons, Inc.