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# USE OF VERTICAL DRAINS FOR SOIL IMPROVEMENT AT BRIDGE APPROACHES

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SYNOPSIS

The subsoils underlying the approaches for the Braddell Road flyover in Singapore include a surface layer of very soft peaty clay and marine clay varying in thickness from 3 m to 11 m. Embankments of 3.5 m and 4.0 m height were required at the two approaches.

Staged embankment construction and surcharge preloading with vertical drains was employed to allow construction without foundation shear failure and to reduce post-construction settlements. Prefabricated band drains were installed at spacings of 1.1 m and 1.3 m. Instrumentation was installed including piezometers, settlement plates and inclinometers. Stability during construction was controlled by monitoring foundation pore pressures and lateral deformations.

Total settlements during construction and preloading were in the order of 1.5 m, including 0.5 m of settlement due to undrained deformation in the soft clay. Observed consolidation settlements were similar to those predicted from conventional consolidation theory and laboratory test results. Pore pressures did not dissipate as rapidly as expected from this theory and the degree of consolidation indicated by pore pressures was less than that indicated by settlements.

Consolidation for 6 months under 10 per cent surcharge successfully eliminated post-construction primary settlement beneath the north embankment. Treatment for the south embankment was less successful because the design preload duration was not achieved and because of less effective drainage from the sand blanket.

NOTATION

| | |
|---|---|
| $\bar{c}$ | Effective cohesion intercept |
| $c_h$ | Coefficient of horizontal consolidation |
| $c_v$ | Coefficient of vertical consolidation |
| $m_v$ | Coefficient of volume change |
| N | Standard Penetration blow count |
| $s_u$ | Undrained shear strength |
| U | Degree of consolidation |
| $\Delta u$ | Excess pore pressure |
| $W_n$ | Natural water content |
| $W_p$ | Plastic limit |
| $W_l$ | Liquid limit |
| $\bar{\phi}$ | Effective angle of shearing resistance |
| $\delta$ | Settlement |
| $\gamma_w$ | Unit weight of water |

## I. INTRODUCTION

The new Central Expressway which runs approximately north-south through central Singapore crosses Braddell Road near the Kallang River. The dual 3-lane flyover which carries the expressway over Braddell Road is thus located in an area of soft alluvial soil which created difficult foundation conditions for construction of the 3.5 m and 4.0 m high approach embankments. Prior to construction of the expressway, the area was occupied by swamp and fish ponds. Preliminary boreholes drilled from an initial working layer of fill indicated the surface layer of soft clay to be 5 to 11 m deep. Preloading with artificial vertical drains was selected in design to allow construction of the approach embankments within the 16 months contract period.

## II. SUBSOIL CONDITIONS

Unconsolidated sediments at this site are of the Kallang Formation which has been described by PWD (1976), TAN and LEE (1977), PITTS (1983) and TAN (1983). Compressible soils of both the alluvial member (soft peaty clay) and the marine member (soft marine clay) are found at the site. Representative soil profiles at the north and south embankments are shown in Fig. 1.

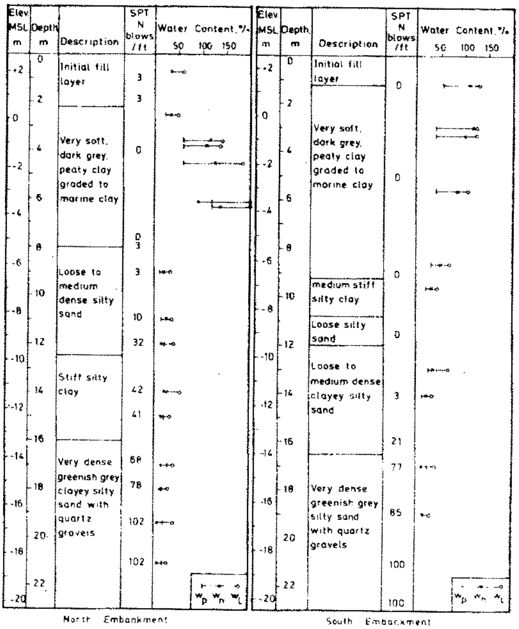


Fig. 1 : Representative Soil Profiles

At the south embankment soft compressible soil extends to a depth of 9 m and comprises a 7 m thick layer of soft greenish grey marine clay overlain by 2 m of soft peaty clay. At the north embankment, which is 450 m further inland, the compressible layer is predominantly soft peaty clay. Here the marine clay is less evident and is of erratic distribution in the boreholes. At the north embankment site the thickness of compressible material varies laterally across the roadway from 10 m on the west side to 3 m on the east. At both embankments the soft compressible soils are underlain by loose silty sand and multicolored stiff silty clay, presumed to be older sediments of the Kallang alluvial member subjected to some interdepositional weathering. These sediments are in turn underlain by stiff to hard clay and dense sand, presumed to be Old Alluvium (PWD, 1976).

### Geotechnical Properties

A program of laboratory testing was carried out on samples obtained from rotary wash drill boreholes by means of 75 mm diameter thinwall shelby tubes. Undrained shear strength in the soft clay was measured by unconfined compression test and by saturated unconsolidated undrained triaxial compression test. Effective stress strength parameters were measured by isotropic, consolidated undrained triaxial compression with pore pressure measurement. One-dimensional consolidation properties were measured in a conventional lever arm consolidometer. Separate specimens were prepared and tested with the applied stress in both the vertical and the horizontal direction. These tests indicated that the soft clay is normally consolidated having low shear strength and high compressibility. Table 1 lists the representative soil parameters of the soft clay layer used for analysis.

Table 1:

Representative Soil Parameters of the Soft Clay Layer Used in Analysis

| Parameters | North Embankment | South Embankment |
|---|---|---|
| Undrained Shear Strength $S_u$, $kN/m^2$ | 13.0-16.5 | 6.7-13.7 |
| Effective Cohesion Intercept $\bar{c}$, $kN/m^2$ | 11.4 | |
| Effective Angle of Shearing Resistance $\bar{\phi}$, deg. | 20.5 | |
| Coefficient of Volume Change $m_v$, $m^2/kN$ | $1.48 \times 10^{-3}$ | |
| Coefficient of Vertical Consolidation $c_v$, $m^2/day$ | $1.47 \times 10^{-3}$ | |
| Coefficient of Horizontal Consolidation $c_h$, $m^2/day$ | $2.19 \times 10^{-3}$ | |

Note: $m_v$, $c_v$, $c_h$ are average values measured for stress range imposed by embankment.

## III. DESIGN OF SOIL TREATMENT AND CONSTRUCTION

### 3.1 Design of Soil Treatment

Preliminary estimates of embankment settlement based on one-dimensional consolidation theory and the laboratory test results indicated that total consolidation settlements would be approximately 0.9 m and 1.5 m beneath the north and south embankments respectively, and that 20 to 30 per cent consolidation would be obtained within one year of construction to full height by normal vertical consolidation drainage, i.e., with no artificial drains. Stability analyses based on undrained shear strength indicated that the embankments could not be safely constructed to full height without an increase in foundation strength equivalent to approximately 25 per cent consolidation of the soft soil. It was thus evident that preloading alone would not be sufficient to allow construction within the available time, that staged construction with artificial vertical drainage would be necessary to construct the embankments safely without delay, and that additional preloading would be necessary to eliminate excessive post-construction settlement.

In order to suit the proposed construction program it was decided that embankment construction and preload placement should take place over a period of 4 months from installation of paper drains and that the surcharge should remain in place for a further 6 months. A minimum safety factor of 1.3 against foundation shear failure during construction was specified. Primary consolidation settlement was to be completed before opening of the expressway.

OV drains, manufactured by the Japan Vilene Co Ltd, were selected for vertical drains. These are 100 mm wide by 3 mm thick synthetic filter strip drains. Vertical drain layout and magnitude of preload were designed using the conventional theory of vertical consolidation with radial drainage. The value of $c_h$ used in design was the average value obtained from oedometer tests on horizontal samples by the log t method at applied stresses just above the critical pressure. The effect of vertical drainage in the soft clay was ignored.

A triangular pattern of drains at 1.1 m spacing was selected at the 4.0 m high south embankment, with surcharge of 26 kN/m$^2$. For the 3.4 m high portion of the north embankment 1.1 m drain spacing with 20 kN/m$^2$ surcharge was selected and for the portion below 3.0m height a drain spacing of 1.3 m and surcharge of 40 kN/m$^2$ were selected. Weathered granitic soil of compacted total density 19.6 kN/m$^3$ was used for embankment construction and preloading. The design of soil treatment is summarized in Table 2. The plan of embankments and layout of drains are shown in Fig. 2.

### 3.2 Construction

Construction of the embankments commenced in December 1981 with placement of an initial layer of fill approximately 1 m thick to form a working platform. The vertical drains were installed from this platform using twin 150 mm diameter mandrels mounted on a crawler crane. The mandrels were pushed without vibration to the full depth of the soft layer with the strip drain material fed through the mandrel centre and secured at the bottom by a disposable 150 mm x 100 mm flexible anchor plate. After full insertion the mandrels were withdrawn leaving the anchor point and strip drains in place.

Table 2: Design of Vertical Drains and Preloading Surcharge

| Required Final Embankment Height (m) | Allowance for Final Consolidation Settlement (m) | Final Fill Thickness (m) | Final Imposed Loading (kN/m$^2$) | Surcharge Loading (kN/m$^2$) | Total Fill Thickness During Surcharge Loading (m) | Vertical Drain Spacing (m) |
|---|---|---|---|---|---|---|
| North | | | | | | |
| 3.4 | 0.9 | 4.4 | 86 | 20 | 5.4 | 1.1 |
| 3.0 | 0.8 | 3.8 | 75 | 40 | 5.8 | 1.3 |
| South | | | | | | |
| 4.0 | 1.4 | 5.4 | 106 | 26 | 6.7 | 1.1 |

The strip drains were cut off approximately 0.5 m above ground level and a drainage layer placed over the area comprising 300 mm thickness of clean sand. The top ends of the vertical drains were connected to 300 mm wide strip drains laid transversely in the sand blanket. These strip drains were in turn connected to gravel drains laid beneath each toe of the embankment which discharged to open surface drains nearby.

Construction of the main embankment fill commenced in February 1982, the fill being spread in layers and compacted by a vibrating smooth drum roller.

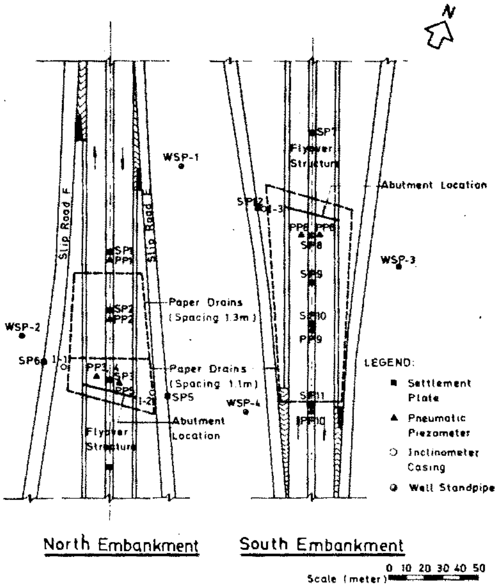


Fig. 2 : Layout of Approach Embankments

### 3.3 Instrumentation

Geotechnical instruments were installed prior to the embankment construction in order to monitor their performance. Three inclinometer casings were installed adjacent to the embankment toe to measure lateral deflections in the soft clay. Five plate settlement markers were installed at original ground level on the embankment centreline within the area of vertical drains. Additional plate settlement markers were installed beyond the toe of the proposed embankments to measure possible heave and beneath adjacent lower sections of embankment which were constructed without vertical drains or preloading. Eight pneumatic piezometers were installed in boreholes at different levels within the soft clay layer beneath the embankment centreline within the area of vertical drains. Two pneumatic piezometers were also installed within the soft clay layer beneath adjacent lower sections of embankment without vertical drains or preloading. Four open standpipes were installed some distance from the embankments to measure static groundwater levels. All instruments were installed after installation of vertical drains and before placing of embankment fill.

Figure 2 shows the locations of the installed instruments.

## IV. RESULTS AND DISCUSSION

### 4.1 Observations During Construction

Typical records of embankment height and measurements of settlement and pore pressure are shown in Figs. 3 and 4.

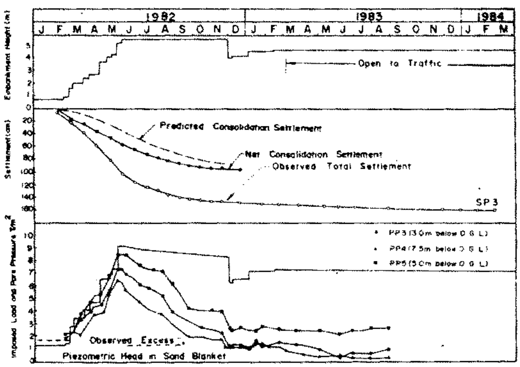


Fig. 3 : Results of Monitoring at North Embankment

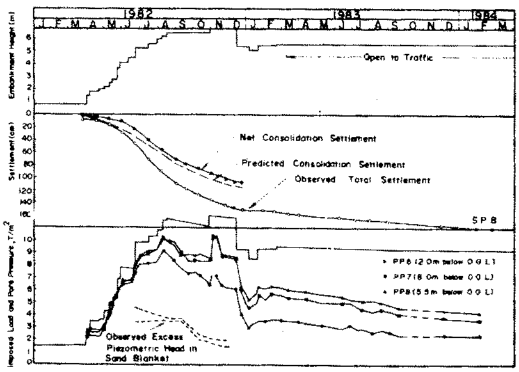


Fig. 4 : Results of Monitoring at South Embankment

Monitoring of observation wells indicated little fluctuation in general groundwater levels, which remained close to the original ground surface level.

Safety against foundation shear failure was monitored by analysis in terms of effective stress, with embankment height, height of adjacent slip road and observed pore pressure as input parameters. The safety factor was calculated from readings taken shortly after each loading increment. The rate of construction was able to proceed as planned at the north embankment. Construction did not proceed as rapidly as intended at the south embankment due to change of construction program for a nearby drainage canal. The minimum safety factor of 1.32 was calculated for conditions at the end of the initial preload construction period, during which 6.3 m of fill were placed in 4½ months. If foundation stability had been the only constraint a slightly more rapid rate of construction could have been used and the full preload could have been completed in a shorter period.

## 4.2 Lateral Movement

Significant lateral deformations were measured in the soft clay as the embankments and preload were constructed. Profiles of lateral movement at end of surcharge are shown in Fig. 5. The

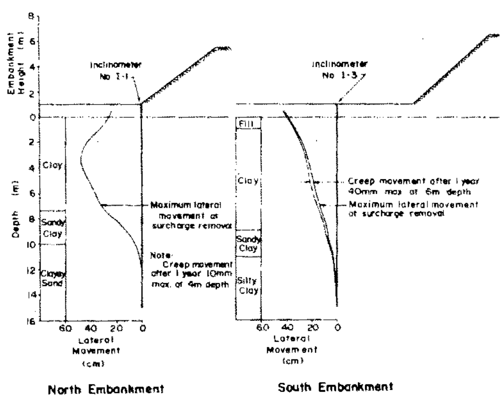


Fig. 5 : Profile of Lateral Movement due to Preloading

greatest lateral movements were recorded at I-1 and I-3, where the thickness of soft clay was greatest (10 m and 9 m respectively). In excess of 400 mm maximum lateral movement was measured at these locations whereas at I-2, where the soft clay is only 5 m thick, only 75 mm was measured. The majority of this lateral movement occurred rapidly as the embankment loading was increased, and the rate of lateral movement became slight one to two months after addition of loading ceased. Some continued creep movement did occur however, in spite of subsequent removal of preload. In the 12 months following preload removal approximately 10 mm additional movement was measured at I-1 and 40 mm at I-3. The greater movement at I-3 presumably reflects the less complete preloading obtained at the south embankment. At I-2 lateral movement was halted abruptly by construction of the adjacent slip road two months after completion of preload construction and no subsequent creep movement was observed.

## 4.3 Settlement

In order to evaluate observed settlements the total settlement was divided into components: a) consolidation settlement in the soft clay; b) undrained settlement in the soft clay associated with lateral deformation; c) immediate settlement in the underlying sand and stiff silty clay.

Component c) was adequately estimated from laboratory test data since it only represented a small percentage of the total settlement. Component b) was estimated by observing the volume of lateral displacement indicated by the inclinometers. The actual consolidation settlement in the soft clay was then derived by subtracting the calculated components b) and c) from the observed total settlement. Typical net consolidation settlements derived in this manner are compared with consolidation settlements calculated from laboratory test results in Figs. 3 and 4.

Although the duration of constant loading is not great, a comparison of the observed and calculated time settlement curves suggests that settlement occurred more rapidly than predicted at the north embankment and slightly more slowly at the south embankment. Revised values of $c_h$ and $m_v$ were assessed from each observed time-settlement curve using a least squares curve fitting method. The effect of settlement in reducing the net imposed loading of an embankment is usually ignored but when the settlement is large as in this case the effect is significant. In assessment of consolidation parameters the imposed loading was reduced by an amount $\delta x \gamma_w$ to allow for the submergence effect where $\delta$ = measured settlement, $\gamma_w$ = density of water.

The assessed values of $c_h$ and $m_v$ are compared with those derived from laboratory one-dimensional consolidation tests in Table 3. The comparison suggests that actual values of $m_v$ were similar to those measured in the laboratory and that actual values of $c_h$ were higher than the laboratory values. It must be noted however that the assessment assumed unhindered discharge from the vertical drains. As discussed below, slow drainage of the sand blanket was suspected and it is possible that the true values of $c_h$ were even higher than the assessed values.

Table 3: Assessed Values of $m_v$, $c_h$ Based on Monitoring Results

| | $m_v$ $m^2/kN$ | $c_h$ $m^2/day$ |
|---|---|---|
| Average Laboratory Values | $1.48 \times 10^{-3}$ | $2.19 \times 10^{-3}$ |
| Assessed Values: | | |
| North Embankment SP3 | $1.96 \times 10^{-3}$ | $5.05 \times 10^{-3}$ |
| South Embankment SP8 | $1.41 \times 10^{-3}$ | $2.78 \times 10^{-3}$ |

## 4.4 Pore Pressure

Measured pore pressures corrected for piezometer settlement and expressed as excess pressure with respect to measured static groundwater level are also shown in Figs. 3 and 4. The response of the piezometers to load increase is quite variable but the average increment in pore pressure after compensating approximately for the effects of dissipation during loading is in the order of 0.8 times the increment in applied total stress. This is less than influence factor for stress distribution which is between 0.9 and 0.98 for most piezometers. Detailed interpretation of the measured pore pressures was made difficult by the effects of secondary influences, particularly short term changes due to transient loading associated with the nearby bridge construction. Generally however, it is apparent that pore pressures dissipated during the period of constant loading and dropped further on removal of preload. Some further dissipation occurred thereafter but at a much reduced rate. The degree of consolidation at the end of the preload period calculated from the measured pore pressures is compared with that calculated from settlement in Table 4.

In all cases the degree of consolidation indicated by pore pressure dissipation is significantly less than that indicated by settlement. This may indicate a similar effect to that described by MESRI and CHOI (1979) and CHOA ET AL (1981) whereby organic clays susceptible to significant secondary consolidation may continue to settle under imposed load at a nearly constant value of excess pore water pressure. To compare the effect of drain spacing, Table 5 lists the pore pressure dissipation rate at nearly constant surcharge loading. As seen at the north embankment, all the piezometers installed in the area with drain spacing of 1.1 m show measured pore pressure dissipation rate reasonably close to the predicted values. For a drain spacing of 1.3 m, the measured rate is lower. At the south embankment in general, the measured pore pressure dissipation rate is slower than the predicted values. This may be due to the short period of loading as compared to the north embankment.

Table 4: The Degree of Consolidation at End of Preloading Period

| Location | Degree of Consolidation, U% | |
|---|---|---|
| | From Settlement | From Pore Pressure |
| North Embankment SP3 | 96 | - |
| P3 | - | 78 |
| P4 | - | 80 |
| P5 | - | 61 |
| South Embankment SP8 | 78 | - |
| P6 | - | 62 |
| P7 | - | 37 |
| P8 | - | 31 |

Table 5 :
Average Rates of Pore Pressure Dissipation Under Surcharge Loading

| Embankment | Drain Spacing m | Piezometer No. | ΔU at Full Surcharge $t/m^2$ | ΔU at End of Surcharge $t/m^2$ | Surcharge Loading Period, Month | Measured Dissipation Rate, $t/m^2$/month | Predicted Dissipation Rate, $t/m^2$/month |
|---|---|---|---|---|---|---|---|
| North | 1.1 | PP-3 | 7.4 | 1.2 | 6 | 1.03 | 0.88 |
| | | PP-4 | 6.4 | 1.1 | 6 | 0.88 | 0.76 |
| | | PP-5 | 6.4 | 2.5 | 4 | 0.98 | 1.00 |
| | 1.3 | PP-2 | 5.2 | 5.6 | 5.7 | 0.46 | 0.86 |
| South | [illegible] | PP-6 | [illegible] | 6.4 | 2.7 | 1.00 | 1.46 |
| | | PP-7 | 10.6 | 8.1 | 2.7 | [illegible] | [illegible] |
| | | PP-8 | 10.2 | 6.5 | 2.7 | 0.63 | 1.63 |
| | | PP-9 | [illegible] | 4.7 | 1.5 | [illegible] | [illegible] |

## 4.5 Performance of Sand Blanket

During construction of the embankments high standing water levels were observed in the plastic casings surrounding the settlement marker rods. Observation wells were then installed on the embankment centreline to intercept the sand blanket and piezometric levels 3 to 4 m above the static ground water level were measured. This indicated that the sand blanket was not providing the degree of drainage that was required. In September 1982, 4 one-metre diameter wells were drilled on the centreline of each embankment to just below the sand blanket level and these were emptied by pumping on a regular twice-daily basis through the remainder of the preloading period. A volume of approximately one cubic metre was withdrawn daily from beneath each embankment which represented approximately one-third of the total calculated void water expelled due to consolidation. This measure was partially successful in lowering piezometric levels in adjacent observation wells but did not show a significant effect on the overall rate of settlement or pore pressure dissipation.

Exploratory excavation revealed that the water flowing up the vertical band drains was contaminated with dark brown organic matter, which appeared to have partially clogged the sand blanket. The free drainage of the sand blanket is considered to have been affected due to this contamination.

## V. CONCLUSIONS

The use of vertical drains together with staged construction was successful in allowing construction of the embankments to proceed rapidly without foundation shear failure. At the north embankment preloading with vertical drains has been successful in eliminating primary consolidation settlement after construction but approximately 75 mm of settlement due mainly to secondary consolidation has occurred in the first year of use. At the south embankment the soil treatment was only partially successful in eliminating post construction settlement, due mainly to insufficient preload magnitude and duration because of other construction constraints.

The effectiveness of vertical drains will be directly reduced by any excess piezometric pressure in the drains themselves with respect to the static ground water level. Special attention should be given to detailed design and installation of the components of a vertical drainage system to reduce the excess pressure in the drains as much as possible. The drains themselves should be checked for adequate hydraulic conductivity under the maximum flow rate expected due to consolidation drainage. The drainage blanket, drain connections and discharge collection system should also be proportioned for minimum head loss under this maximum flow rate, with due consideration of the distortion which will occur due to settlement. The drainage blanket and discharge points should be placed as low as possible, although generally it will necessarily be some distance above the static water table because of the need to provide a working platform for drain installation. This unavoidable excess piezometric head in the drainage system should be compensated by additional preload.

It is suggested a good practice to install a few piezometers in the sand blanket so that its effectiveness can be checked.

In selecting the type of vertical drain to be used it is important to ensure that the external filter surrounding the drain is adequate to exclude fines which could clog the drainage system.

The smear zone created around each vertical drain during installation also introduces head losses into the consolidation drainage system due to its reduced permeability. The importance of reducing this effect has been discussed frequently. The mandrel used for installation on this project was considered larger than desirable and better drainage would probably have been obtained if a mandrel of smaller section more closely conforming to the shape of the drain had been used.

Careful attention to detail is necessary if piezometers are to provide maximum information in a vertical drain installation because of the relatively small scale of the consolidating zone around each drain. Verticality during installation and accurate plan position of both the piezometers and the drains in the immediate vicinity are important so that the position of the piezometer relative to drains is known accurately. Local variations in permeability, particularly the presence of small sand layers intercepted by drains, may cause large local variations in pore pressure. It is recommended that a piston sample be obtained from the location of each proposed piezometer so that the local soil fabric may be examined to ensure it is representative of general conditions. During site investigation, piezoprobe tests can be carried out to determine the presence of sand seams.

## ACKNOWLEDGEMENTS

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