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Underground Investigation for Large Excavation at Victorian Arts Centre

BY
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I INTRODUCTION

The North End of the Victorian Arts Centre in St. Kilda Road Melbourne has been planned as one large building housing a complex of auditoria and related facilities. The Architect has placed the building underground, utilising the whole $1\frac{3}{4}$ acres available down to an average depth of 90 feet. An initial general site investigation had revealed the following underground strata in order of depth:-

Filling - of clay, rubble and gravel to an average depth of 25 feet.

Soft Organic Silty Clay - ranging from 40 - 48 ft. in thickness at depths of 40 - 100 ft. below ground.

Sand, Gravel and Cobble Aquifer - present only at the southern half of the site, thickness ranging from 0 - 25 feet.

Silurian Mudstone Bedrock - ranging in depth below ground from 75 ft. at the northern end to 100 ft. at the southern end. Some Basalt rock and some stiff clayey silt are also present overlying this bedrock at the northern end (See Fig. 4).

The nature of these materials, together with an arrangement of the auditoria that prohibits retaining wall supports at intermediate heights, has meant that the building must be housed within a massive perimeter retaining wall spanning between a strutting frame at the surface and the bedrock at the bottom.

Several types of wall and construction methods have been assessed. The detailed solution for each system requires a knowledge of specific water, soil and rock parameters, and this investigation has been planned and implemented to provide this data.

This paper will describe the various wall systems studied, details of the site investigation and its application to the analysis of forces acting on the structure.

II INITIAL INVESTIGATION

The general investigation for this portion of the Victorian Arts Centre site - the North End, was carried out in 1966 with the objective of determining general soil stratum profiles and properties for use in assessment of various foundation and excavation alternatives.

The investigation consisted of 11 boreholes of $3\frac{1}{2}$" diameter averaging 105 ft. depth including "N" size coring of Silurian rock for a minimum of 10 ft., and associated soil testing. The qualitative results of the investigation are summarised above.

III WALL SYSTEMS

Figs. 1 and 2 show just two of the various retaining wall/excavation systems devised to produce the large excavation space.

The following possible wall construction methods were then considered:-

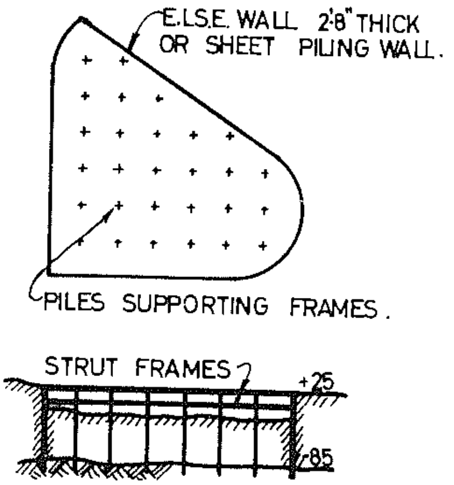
For interlocking caissons, wall excavation would be by caisson sinking assisted by ground freezing and/or pressure grouting.

For a Solid or Cellular Insitu Wall, the construction method could include -

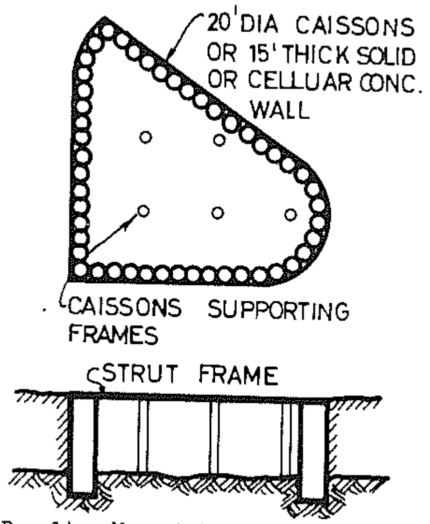
a) Soil excavation within sheeting formed by frozen ground or temporary concrete walls placed by bentonite displacement.

b) Soil excavation by mining techniques within sheet piling, assisted by ground freezing and/or pressure grouting.

Tenders were called for construction of the large excavation in August, 1970 allowing each Contractor to select one or other of the above methods, or any variations of his own initiative. At the time of preparation of this paper, the method had not been finally determined.



Result:- Walls would break &
bottom heave up when
excvn. to 50 ft.depth.
Hence: Reject system.
FIG.1. THIN WALL, THREE FRAMES



Result:- Max. int. free space for
theatres.Ret.walls built
first so no int. heave.
Hence: Method adopted.
FIG.2. THICK WALL,SINGLE STRUT FRAME

IV SPECIFIC UNDERGROUND INVESTIGATION

OBJECTIVES

The objectives of this investigation were to determine:-

1. The rock surface beneath the site, particularly on the perimeter.

2. The properties of the rock, i.e. dip and strike of bedding planes, jointing orientation, frequency and extent of clay filling, strength, density and permeability.

3. The properties of the insitu soil, e.g. strength, permeability and earth pressures.

4. Some properties of the frozen soil, e.g. strength compressive and tensile creep.

Total values $20M

SCOPE

In order to obtain maximum value for the investigation expenditure of $150,000, the work was divided into four separate contracts as described below, each being suited to one special function. The whole was co-ordinated and supervised by M.J & A. and took place between June 1969 & July, 1970.

Perimeter Drilling by Soil Mechanics Ltd.

Thirteen boreholes spaced at approx. 100 ft. centres were drilled around the perimeter of the site, as shown in Fig. 3 and penetrated 40 ft. into the Silurian bedrock. 4½" diameter undisturbed samples were taken of the organic clay stratum for laboratory testing and 5½" diameter rock cores were taken for detailed description and permanent record purposes.

Caisson Sinking by McDougall-Ireland Pty.Ltd.

Two 5'-9" dia. steel-lined shafts approx.100 ft. deep were sunk. One of these shafts, the North Caisson, penetrated the rock a distance of 0 - 2 ft. A chamber 11 ft. diameter was excavated below this level for conducting insitu rock strength tests and to permit wall tenderers to evaluate the rock en masse. The South Caisson was sunk to the surface of the rock to determine water flows in the sand stratum at that level. It also enabled the placement of pressure cells and piezometers.

Observation Wells by W.L. Sides & Son Pty.Ltd.

Four eight inch diameter boreholes with filter screens were drilled around the South Caisson to observe water levels.

Testing by Soilmech Pty.Ltd. in conjunction with Dr. J. Morgan, Reader, Department of Civil Engineering, University of Melbourne. Some specialised tests were also carried out by Professor Davis and Dr. Lee of the Department of Civil Engineering, University of Sydney.

A comprehensive programme of ambient and frozen temperature soil laboratory tests was conducted on undisturbed samples. Insitu horizontal shear strength tests on rock were performed in the North Caisson. Pressure cell installation and pump tests were made in the South Caisson.

FIELD WORK - DRILLING, SAMPLING, CORING AND TELEVISION INSPECTION

The 13 boreholes were sunk by the cable drop tool method inside 10" and 8" diameter casing. The sequence of strata revealed is summarised in Fig. 4. The 7 ft. above the location of any undisturbed sample was excavated by hand augering.

38 No. undisturbed samples were recovered in 4½" O.D. x 10g. x 3'-6" long sample tubes.

Diamond drilling was carried out with 2 - 7½" O.D 5½" I.D., 5 ft. long triple tube core barrels.

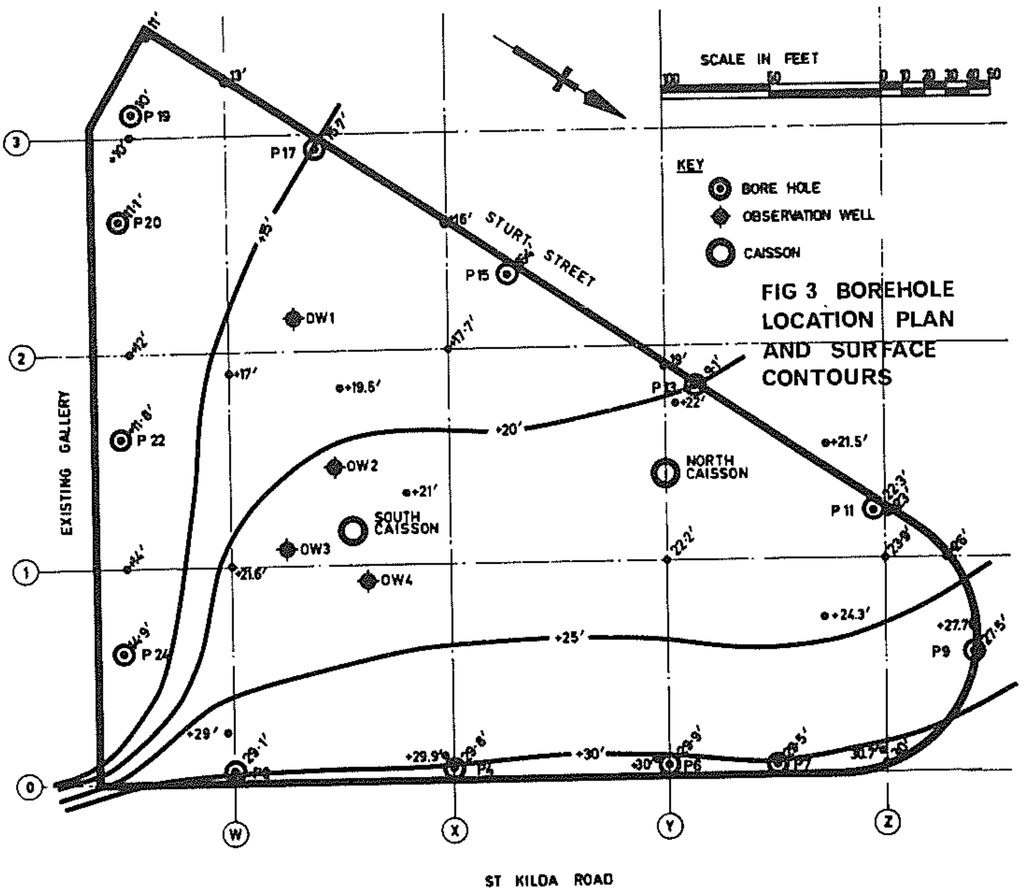


FIG 3 BOREHOLE LOCATION PLAN AND SURFACE CONTOURS

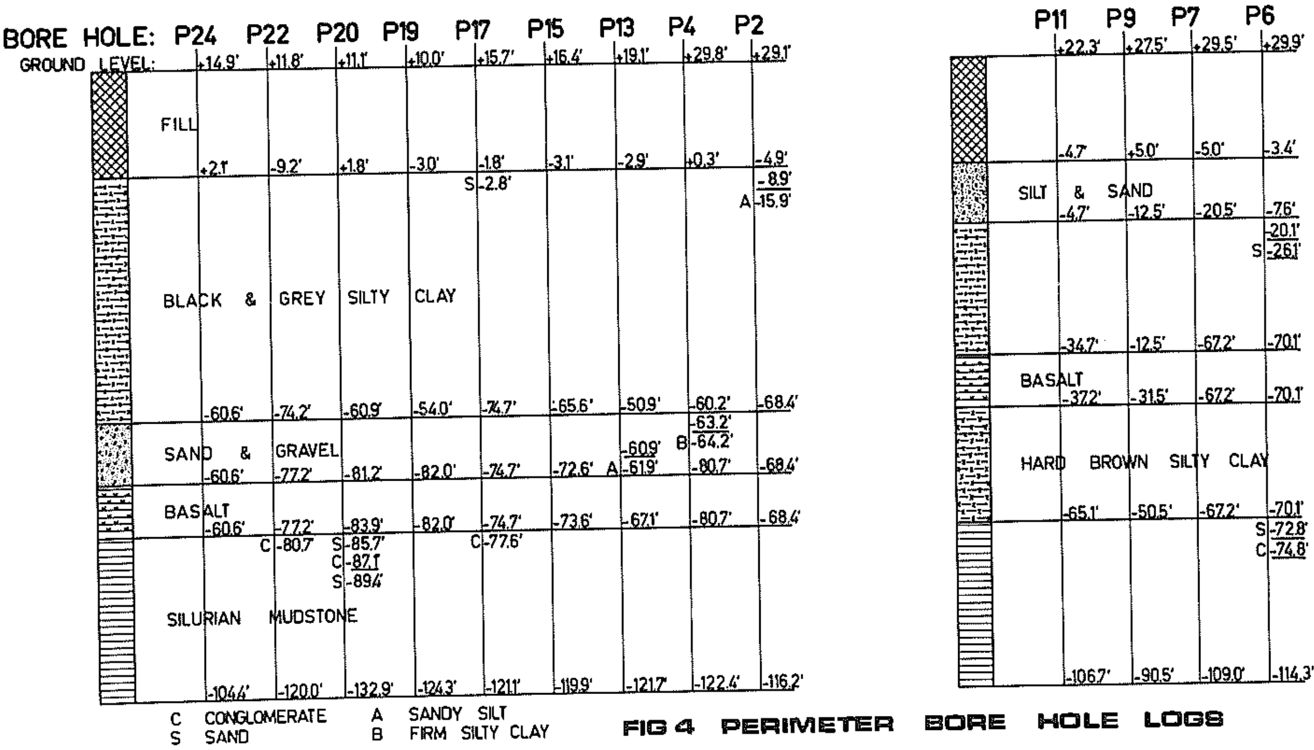


FIG 4 PERIMETER BORE HOLE LOGS

In order to determine the direction of dip of the Silurian rock bedding planes, an underwater television camera installation was developed specifically for this project and used in 8 representative boreholes.

The camera, which was 5¾" dia. and 2'-8" long was designed to operate underwater at depths up to 160 ft. Lighting was built into the camera. The camera lens was fixed, but side viewing in all directions was obtained by means of a metallic mirror which could be rotated about the longitudinal axis of the camera by remote control from the surface.

Directional orientation was obtained by a magnetic compass built into the camera. 6½" and 23" television screens were provided in the control caravan for picture monitoring and for photographing results.

LABORATORY TESTING AND STRATUM PROPERTIES

These tests were performed to provide information to design and evaluate any construction method.

Black/Grey Silty Clay

The samples were predominantly stiff grey and black silty clays, but did include clayey sands, sandy silty clays and these materials with lenses of fine sand. The group symbols were predominantly CH, Cl and OH with some CL and SC.

The programme of laboratory testing included the following tests. Typical results only are quoted, the detailed results appearing in Reference No. 1.

Tests on insitu samples:-

Water content 48% Bulk density 113 lb.per cu.ft.
Liquid Limit 81% Plasticity Index 55%.
Undrained Triaxial C_u 8 p.s.i., ϕ_u 4 deg.
Constant Axial Stress Triaxial Type P, Initial K'0.56
E_{min} 900 p.s.i., E_{max} 6700 p.s.i.
Consolidated Undrained Triaxial C' 2.5 p.s.i., ϕ 32 deg
A(1) 0.31 A(3) 0.08
K'_o 0.45
Consolidation M_v 0.016 ft^2/T at $\frac{1}{2} - 1$ T/ft^2.

Tests on Frozen Samples:
Creep tests - tensile strength ranges from 10 k.s.ft. to 25 k.s.ft. for temperatures of 14 deg.F and -4 deg.F at rates of strain between 0 and 1.5 x 10^{-4} in/min.

Creep tests - compressive ranges from 10 k.s.ft to 33 k.s.ft. at 14 deg.F for 0 to 1.5 x 10^{-4} in/min rate of strain and 47 k.s.ft. to 77 k.s.ft. at -4 deg. F for 0 to 0.5 x 15^4 in/min rate of strain.
E_u 65,000 p.s.i.

Heave test 2% weight and height increase with vertical stress applied - Test of short term duration.

Silurian Rock.

The salient features revealed by the cores are as follows:-

a) General Description of Rock Cores.

This general description was prepared by Mr. J. L. Neilson, Senior Geologist, Victorian Mines Department.

"Recent drilling at the site to test for the proposed peripheral retaining wall has found unusually fresh and hard Silurian mudstone (with lesser fine sandstone) extending down at least 40 ft. from the topographic upper limit of the Silurian bedrock. The rock is hard and mostly sound; very few unstable shear zones were discovered, and clay bands are rare.

Dip and strike of the beds are fairly constant over the site, and bedding planes contain small-scale irregularities which would make slip on them difficult. The beds intersect the line of the wall at large angles, which also inhibits slip. Only at the south-west corner of the site will beds dip into the excavation and strike along the wall alignment; there, care will be necessary, particularly if any clay seams should be present.

Jointing is not intensive and is confined to high-angle joints. It is low-angle joints (from horizontal to 30°) which would be of most significance in foundation failure of the wall by horizontal stresses, and such joints are tight and always irregular, never completely planar, which in addition would make failure along them more difficult. Some joints were limonite (iron oxide) cemented, and none was clay filled.

Several shear zones of unknown orientation do not appear to be serious weaknesses but should be carefully considered."

b) Core Recovery.

100% recovery was obtained in all boreholes.

c) Dip of Bedding Planes.

Apart from a few isolated occurrences the dip ranged from 15° to 40°. The dip at the north-west end of the site was $35^\circ \, {}^{+5^\circ}_{-10^\circ}$ (i.e. in boreholes P11, P13 & P15).

The dip for the remainder of the site perimeter was 25° ±10°. The direction of dip was 355° ± 20° rel. to mag. north.

d) Rock Quality Designation.

R.Q.D. is defined as:-

$$\frac{\text{The Aggregate Length of Core Pieces 4" long}}{\text{Total Length Drilled}} \times 100$$

This varied considerably within any one borehole in quite a random manner (e.g. 15% to 70% typical). The total range for this site was 0% to 100% with approximately 30% of the 0-33% R.Q.D. present, and approximately 30% of the 33-67% R.Q.D. present.

e) Rock Strengths.

13 unconfined compression strength tests ranged from 420 to 10,300 lb/in^2 with an average of 3,600 lb/in^2. The planes of weakness were flatter than 34° and failure generally was by vertical splitting.

Two undrained triaxial compression tests were done. These were stage tests on single samples and produced the following results:-

C = 34 lb/in^2 $\emptyset$ = 30^o(clay on failure plane at angle of 48^o).

C = 0 lb/in^2 $\emptyset$ = 30^o(clay on failure plane at angle of 47^o).

f) Density of Rock.

The average bulk density of the rock core samples tested in the laboratory produced the following results:-

= 160 lb/ft^3 with a range of 151-171 lb/ft^3.

Insitu Stresses.

In order to obtain some indication of the insitu stress conditions of the rock, it was decided to adopt a programme of direct measurement of forces in rock excavation struts at the time of excavation for the wall. This was influenced by the knowledge that the Railway Construction Board had used flat jacks in slots in the rock to measure these stresses in test shafts for the City of Melbourne Underground Railway, but in spite of reasonable techniques, did not arrive at any conclusive results.

Aquifer.

Tests conducted on disturbed samples taken during caisson sinking revealed material including silty, clayey fine sand, silty fine to medium sand, gravelly coarse sand, cobbles (3" to 8" size) with fine to coarse sand.

Analysis of ground water samples taken from the North and South Caisson included the following results. Sulphate (SO_4) -1,400 parts/million, Chloride(Cl) -12,400 parts/million which were acceptable for normal dense concretes.
During the sinking of the South Caisson, gases, principally H_2S, SO_3 and CO_2 were released from the ground water.

INSITU ROCK SHEAR TESTS.

The purpose of these tests was to carry out insitu large-scale horizontal loading shear tests of clay seams between bedding planes in blocks of Silurian rock, to give design data for specifying the depth of penetration of the base of the North End Wall.

The North Caisson rock chamber shown in Fig. 3 was excavated and a 3' x 3' x 3' (nominal) block was left intact on each of two floor levels for load testing. The test arrangement also included a steel framework for supporting the dial gauges used in monitoring vertical, horizontal and rotational movements. A "skate" was located below the vertical jack to ensure uninterrupted vertical loading as the movement of the upper block occurred.

Both tests were conducted in three stages as represented in Fig. 4; each stage involving an increase in horizontal load in 2T increments with a simultaneous reduction in the vertical load. The horizontal load being maintained constant until the movement ceased. The stage was terminated when the horizontal movement reached 0.1 in. or when the vertical load became zero.

The failure pattern and shear strength relation for the second block test is shown in Figs. 5 & 6.

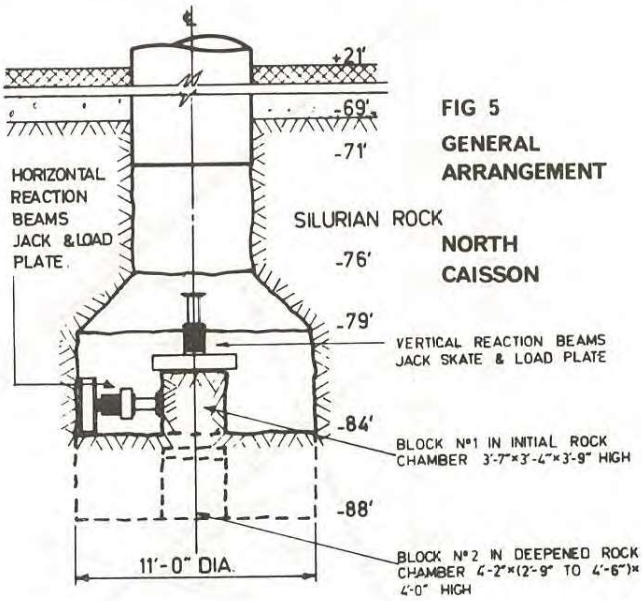


FIG 5
GENERAL ARRANGEMENT
NORTH CAISSON

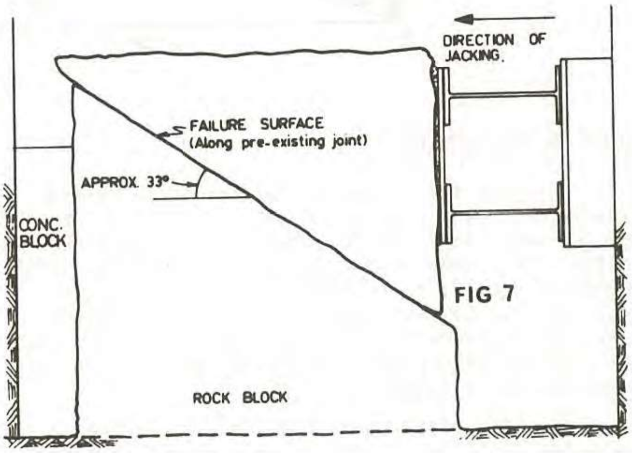


FIG 7

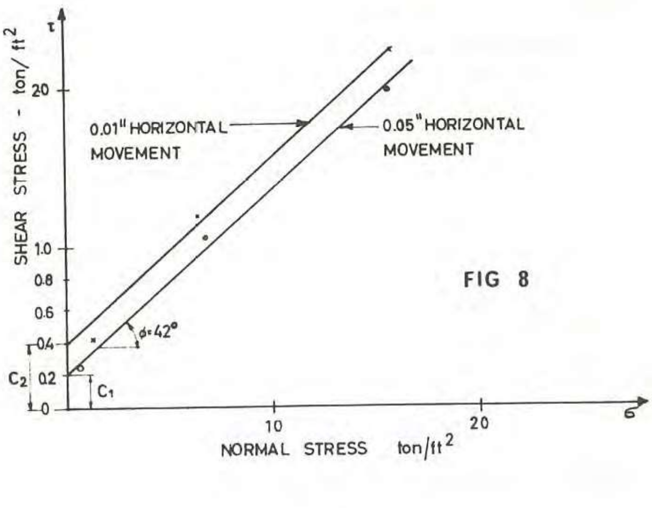


FIG 8

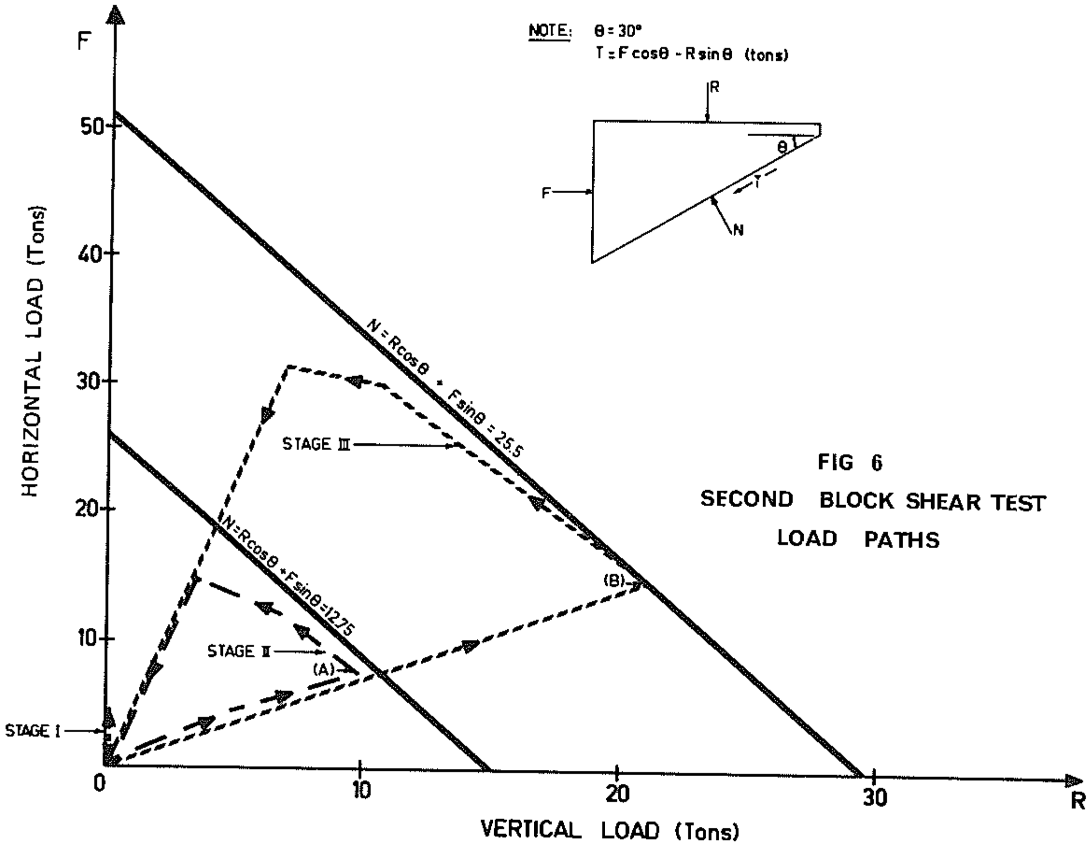


FIG 6
SECOND BLOCK SHEAR TEST
LOAD PATHS

FLOW IN THE AQUIFER

The pump test conducted in the South Caisson involved the observation of rate of drawdown of ground water level in the four surrounding observation wells. This enabled the average permeability of the confined aquifer to be computed. Observation of water levels without pumping enabled the maximum velocity of water to be computed. This information is required before the feasibility of freezing this stratum can be assessed.

The test arrangement included the pumping well, i.e. the 5'-9" dia. steel-lined South Caisson with its open base with a graded filter penetrating the aquifer to RL-70 ft., and four observation wells located as shown in Fig.1.

The graded filter to the South Caisson consisted of, from bottom to top -

3 ft. of ¼" screenings.
3 ft. of 2" - 3" screenings.
2 layers of A.S. No. 300 fabric.
Steel joists at 8" c/s welded to sides of caisson.

The four observation wells were $8^9/_{32}$" I.D. steel cased. The screens were of phosphor bronze material $5\frac{5}{8}$" I.D., 7" O.D. with holes ranging from 0.123" to 0.036" diameter.

A submersible pump was located at RL 62' and connected by a 6 inch P.V.C. delivery pipe to a section of steel pipe at the top of the caisson. This section contained a control valve, pressure gauge and $1\frac{1}{2}$" orifice plate and manometer. The discharge pipe was approximately 500 feet long and drained into a stormwater manhole in Sturt Street.

On the basis of previous trials, the discharge control valve was set at 55 gallons per minute and maintained constant for the duration of the test.

The permeability k of the aquifer was found to be:-

6.1×10^{-3} cm/sec. +30% - 20%.

The maximum velocity of water was found to be 0.7 ft/day which will not affect freezing in the aquifer.

V ANALYSIS OF ROCK STABILITY

Fig. No. 9 shows typical pressures acting on the external face of the perimeter wall together with the horizontal reactions provided by the top strut and the rock at the base of the wall.

For unit length of any wall e.g. Sturt Street, St. Kilda Road, etc. the horizontal resistance provided by the rock has been considered as being supplied by a reaction wedge bounded on its underside by joint planes parallel and perpendicular to the bedding and oriented in the most adverse manner possible (i.e. to give minimum capacity). Fig. 10 shows the forces acting on this wedge which are consistent with the following assumptions:-

1. Failure will not occur across intact rock since joints will be assumed to exist in the most adverse configuration.
2. The rock mass is assumed to be cut by three sets of orthogonal joint planes at any spacing.
3. One set of joint planes is assumed parallel to the bedding plane.

4. The other two sets of joint planes are defined by their intersection with the bedding planes being parallel and normal to the wall under consideration.
5. The shear force transmitted from the wall to the rock is limited to the buoyant weight at the wall; this force always being less than the shear strength of the joint, i.e. V = P - U.
6. Seepage forces on the underside of the failure wedge have been ignored.
7. The most adverse measured values of Dip and Strike for any particular wall have been adopted for the determination of the depth of penetration.
8. Hydrostatic uplift U under the wall is based on a flow net assuming homogenous rock.
9. Shear strength law on a typical joint plane is of the form -

$$\tau = K + \sigma_n \tan \phi$$

where K = constant

σ_n = normal stress

ϕ = angle of internal friction.

10. All soil and rock properties, dip, strike, etc. are based on the test results of the Underground Investigation, dated July, 1970.

viz: $K = 0.45\ K/Ft^2$.

$\phi = 38$ degrees $\quad \gamma_b = 85\ lb/ft^3$.

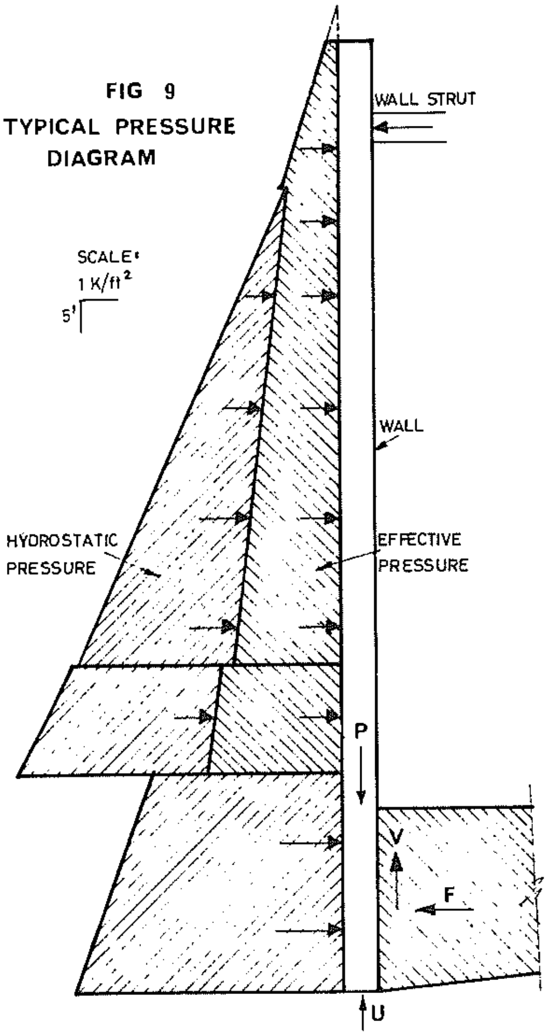


FIG 9
TYPICAL PRESSURE DIAGRAM

Critical Dip and direction of Dip of joint planes

Name of Wall	Dip Deg.	Angle between Dip & Wall
St. Kilda Road	20	55
Gallery	44	75

The capacity of the wedge to resist horizontal forces imposed by a wall was computed upon our G.E. 265 Computer E.D.P. for varying values of depth of penetration. This enabled the appropriate depth to be selected to satisfy the following criteria:-

i) Wedge capacity to equal 1.5 (F of S) times the working reaction of the wall, as supplied by the wall designer.

ii) A maximum of 25 ft., but if the factor of safety associated with this depth is less than 1.5 then the additional horizontal capacity is supplied by rock anchoring the wedge to the underlying rock.

iii) A minimum of 15 ft., to provide sufficient water cut off and to limit horizontal deflections.

The results of these analyses led to the adoption of penetration of 15 ft. for the Gallery wall and 25 ft. for Sturt Street and St.Kilda Road, with 20% of the horizontal load capacity at St.Kilda Road being provided by rock anchoring through the reaction wedge.

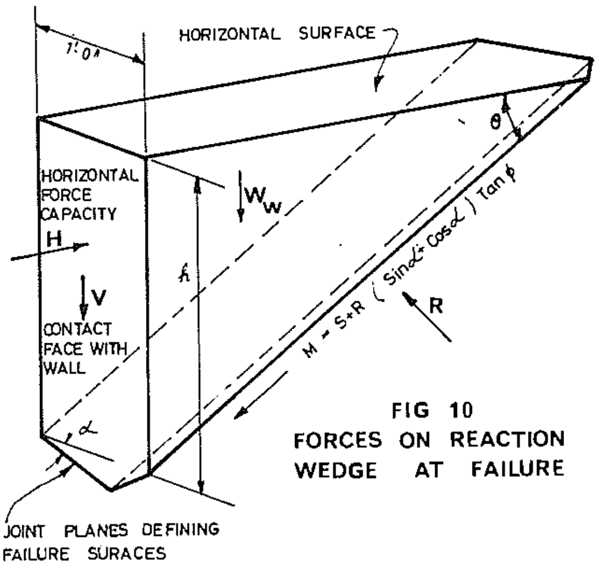


FIG 10
FORCES ON REACTION WEDGE AT FAILURE

VI ACKNOWLEDGEMENTS:

The Author records his thanks to Sir Roy Grounds and Mr.John Connell for granting permission to publish this paper, and to Mr.T. Langley for assistance in its preparation.

VII REFERENCE:

MILTON JOHNSON & ASSOCIATES
Victorian Arts Centre - North End, Underground Investigation Report, July, 1970.