THE USE OF AIR BUBBLERS TO PREVENT
SHOALING AT WHARVES IN NAVIGABLE RIVERS

by

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B.A.Sc., The University of British Columbia,
Vancouver, B.C., Canada: 1960

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THE UNIVERSITY OF BRITISH COLUMBIA
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Addendum


The Fraser-Surrey Wharf, situated on the left bank of the Main Arm of the Fraser River just downstream of New Westminster, B.C., has been subject to severe shoaling during annual freshets ever since its construction in 1926. This has seriously hampered shipping and caused a loss in revenue. In the summer of 1965, Professor E. S. Pretious of the Civil Engineering Department, the University of British Columbia, was approached by the Fraser River Harbour Commissioners (owners of the wharf) to investigate the feasibility of employing air-bubblers to prevent shoaling in the approaches to the wharf. The project study was chosen as a thesis topic by the author, under the supervision of Professor Pretious. The research which was subsequently undertaken involved the theory underlying the interaction between air-bubbles and water; laboratory experiments to measure upward water velocities induced by rising air-bubbles; settling velocities of the bed-sand found in front of the wharf, and the critical tractive shear stresses for impending motion of the bed material. In the field, the hydraulic slopes of the water surface at the wharf was measured to determine if the conditions for bed-load movement existed; river current velocities were measured (surface and sub-surface); weekly, controlled, sounding surveys were carried out in addition to a number of sediment sampling surveys and float studies, to determine flow patterns. Bubbler hoses were designed and prepared in the Hydraulics Laboratory of the University of British Columbia and
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subsequently installed at the wharf. They were kept operating throughout the major part of the 1966 freshet (May, June, July). Recommendations for improved designs and further research are also made.
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The project could not have been carried out without the unfailing support and cooperation of certain Federal Government agencies, in particular the Water Resources Branch of the Department of Energy, Mines and Resources, who provided a suspended-sediment sampler and a current meter together with operators. They also made their sedimentation laboratory facilities available for analyzing the sediment samples. The cooperation of Mr. T.F. Smith of this Department, who coordinated the field work and organized the compilation of survey information, as well as the assistance and advice given by Mr. R. Keene and his staff in the New Westminster sedimentation laboratory, are gratefully acknowledged. The local District Office of the Federal Department of Public Works generously provided vessels and personnel to carry out regular hydrographic surveys of the project area and to install and remove the air-bubbler. Thanks are due to Mr. R. Wallace, P.Eng., of this Department for his cooperation in planning the sounding surveys and to Messrs. Faulkner and Gilmour, the Officers in Charge of the M.V. "Sounder", and their crews.
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The author wishes to express his gratitude to Captain H. R. Johansen, of the Fraser River Harbour Commission, Harbour Master for New Westminster Harbour, who originally suggested the possibility of applying an air-bubbler to the alleviation of shoaling at the wharf. His continuous and enthusiastic support of the field work, together with the cooperation of Captain Tom Groxier of the M.V. "Port Fraser", were indispensable to the operation of the air-bubblers, the sediment sampling and other integrated operations.

The experimental work described in this thesis was carried out in the Hydraulics Laboratory of the University of British Columbia. The use of these facilities is gratefully acknowledged, and thanks are due to Messrs. H.W. Schmitt and E. M. White of the instrument shop, Department of Civil Engineering, U.B.C., for their valuable suggestions and help in carrying out phases of the laboratory experiments and field tests.

The National Research Council of Canada generously supported the project with a research grant and the Fraser River Harbour Commission undertook all expenditures in connection with the rental of the air compressors besides providing free use of wharf facilities.

April, 1967.

Vancouver, British Columbia.
INTRODUCTION

The Fraser-Surrey Wharf, previously known as Pacific Elevators Wharf, was built in 1928 together with a grain elevator, silos and a large unprotected storage space. Situated in the municipality of Surrey, about one and one half miles downstream from New Westminster, B.C., on the left bank of the Main Arm of the Fraser River, the wharf is rather isolated from the main port facilities of New Westminster Harbour. However, being the only grain elevator wharf in the harbour area, and easily accessible to large cargo ships, it has grown in importance, particularly during the present boom in the export of grain and lumber.

Although New Westminster is the only fresh-water port of Western Canada, having certain transit advantages over the salt-water ports of Vancouver and Prince Rupert, it still has to struggle with a problem much less vital to Vancouver and of no concern whatsoever to the almost perfect natural harbour of Prince Rupert. The harbour of New Westminster, including the Fraser-Surrey Wharf, is subject to heavy siltation during freshet periods occurring annually between May and July. The Fraser River at New Westminster trifurcates into the North Arm, Annacis Channel and Annieville Channel (vid. Plate 1), with a consequent decrease in flow and increase in sedimentation in Annieville Channel, the navigation channel for deep-sea vessels. The major portion of New Westminster's harbour facilities is located at the junction of this trifurcation and is
therefore vulnerable to shoaling. The Department of Public Works, Canada, carries out an annual dredging program between the Port of New Westminster and the mouth of the Fraser River, about 21 miles downstream of the Port. This program includes Annieville Channel and the approaches to the Fraser-Surrey Wharf.

As Plate 1 shows, the wharf being located on the left bank of the River on the slightly convex side of the bend, the river's cross-section there would naturally tend to be shallow. Although the wharf would normally have created a local constriction in the width of the channel and might thus have improved local depths, it was built slightly downstream of an existing groin and thus did not contribute appreciably to any beneficial changes in the river bed. The natural depths of 16 to 22 feet below local low water in the immediate approaches to the wharf, were inadequate for the rapidly increasing draughts of modern shipping and since neither the presence of the elevator wharf, nor a training wall between the groin and the wharf appeared to improve the depths, annual dredging was inevitable ever since the wharf was put into service.

To maintain a required depth of 27 feet below local low water during the period 1928-1933, an annual average of 84,000 cubic yards of sand had to be dredged from the approaches to the wharf(1). From 1934 to 1948, a depth of 27 to 30 feet was maintained by an average annual dredging of 44,000 cubic yards. After the abnormally high 1948 freshet, 343,000 cubic yards were dredged
from the same area, followed by only 91,000 cubic yards in 1949 and an annual average of about 177,000 cubic yards between 1949 and 1954. In 1954, an increase in shoal of about three feet in front of the elevator was caused by excavating the river bed on the west side of the regular, navigation dredge cut in Annieville Channel, to provide fill for Annacis Island. This resulted in a stronger flow along the right side of the river and a weaker flow along the left side (approaches to the elevator wharf), increasing the shoaling tendency there (1).

The gradual increase in draughts of cargo vessels necessitated a dredged depth of 30 feet below local low water, which involved a general increase in dredging quantities to 300,000 cubic yards annually, from 1954 to the present year.

Sounding charts and shipping records indicate that the wharf has remained open to deep-sea ships annually from September to May without the help of dredging. Unfortunately, attempts to extend this period by dredging during the freshet periods were frustrated by the swollen sediment-laden river, causing the excavated area to fill in almost immediately after the dredge had moved away. Furthermore, dredging operations were made hazardous by the strong river currents, the dredge having to avoid the close proximity of the wharf for fear of damaging her equipment. Last but not least, a dredge would be in the way of shipping.

These limitations in the use of a conventional dredge clearly called for other methods to provide the Port of New Westminster with a wharf which could handle grain ships throughout the year.
from remedial works in the form of river-training structures which had been designed and tested in the Fraser River Model at the University of British Columbia, the installation of an air-bubbler on the river bottom in front of the Grain-Elevator Wharf was suggested as a possible shoal inhibitor. In comparison with a dredge, the air-bubbler would enlist, rather than avoid, the strong river currents and would operate at maximum efficiency during freshet periods. Furthermore, it would not get in the way of shipping.

In principle, the method of using an air-bubbler to prevent sedimentation can be explained as follows: The vertical, velocity profile of river flow generally shows a distribution of velocities increasing from zero near the river bed to a maximum at a depth approximately equal to 0.2 of the total depth. A screen of rising air-bubbles induces a vertical upward flow of water in the immediate vicinity of the rising bubbles. If this upward flow is strong enough to overcome the downward settling motion of the suspended sediment or bed load, then the sediment, while travelling downstream, would be forced up into the flow layers of higher velocities. Sediment, which would normally settle in front of the wharf, would thus be carried further downstream, provided that:

a) the upward flow, locally induced in the river by the bubble screen, is capable of lifting the sediment, and

b) the horizontal river flow along the wharf is sufficiently strong to transport the sediment past the wharf, once
it has been lifted to a higher level in the flow.

It is clear that a strong, upward, water-flow induced by air-bubbles is a necessary, but not sufficient condition for prevention of shoaling.

Without a sufficiently strong horizontal flow to carry the sediment away, the bubbler would be ineffectual.

Therefore, to investigate the possibilities of an air-bubbler to prevent shoaling at the Fraser-Surrey Wharf, laboratory research had to be combined with a study and analysis of the field conditions.
SECTION I

Review of Practical Applications of Air-Bubblers under Water

The use of compressed air to create a screen of rising air-bubbles in water and thus induce a circulation in the water, has been recognized by engineers since the turn of this century and, particularly in the last ten years, many research projects have been undertaken to investigate applications of this basic concept.

Probably the best known application of the property of rising air-bubbles to create a convection current in the surrounding water is the de-icer, a perforated pipe lying on the bottom of a lake or river and releasing compressed air. The rising air-bubbles entrain the relatively warm water at the bottom, carrying it upward to the frozen surface thereby melting the ice. This method originated in Sweden in 1953, where a great many ferry routes are now kept open during the winter. In Canada, de-icers have been installed at Prescott, Ontario, to keep the ferry slip open; in Manitoba, to prevent ice formation at the Slave Falls Dam; in British Columbia to keep a log pond near Castlegar operating during the winter; and in Quebec, where a lake near Kilmar is made ice-free by a bubbler, which also maintains the oxygen content of the lake, thus producing a healthier environment for the fish population. In this connection, mention should be made of the air-bubbler at Lake Lakelse in British Columbia, where rich, nutrient-laden bottom
water was forced to the surface where the fish reside.

In Tuktoyaktuk, N.W.T., near the mouth of the Mackenzie River, the wharf of the Northern Transportation Company, which is a vital link in the supply route along the DEW line, suffered heavy damage when the ice sheet in the bay moved up and down with a two-feet tide and dislocated the piles. An air-bubbler was installed underneath the wharf and the continuous flow of air-bubbles kept the ice away from the piles, saving the wharf. However, this method is only possible when the bottom water is warmer than the surface layers, which is the case in Tuktoyaktuk Bay with its brackish water (salinity 0.5% at 32°F). The same bubbler was ineffectual under the wharf at Cambridge Bay, 700 miles east of Tuk, where the salinity (2.9% at 29°F) is much higher and the water temperature consequently close to freezing all the way from the surface to the bottom.

A de-icer therefore makes use of the property of a stream of air-bubbles to induce a convection current in the surrounding water, which carries relatively warm water to the frozen surface. A similar application is the pneumatic barrier. In many rivers and tidal estuaries, salt-water intrusion from the sea threatens the fresh-water intakes of factories, industries and cities and the fertility of the surrounding land. The salt water moves upstream under the out-flowing fresh water in the form of a wedge. If the fresh-water outflow is much greater than the tidal inflow, a bubble barrier across the river would force the salt water upwards where
the river flow can carry it seaward. Salt water intrusion is a particularly aggravating problem on the East Coast of the United States, where in the near future the water supplies of New York and Philadelphia may have to depend partly on fresh river water. The Hudson and Delaware rivers have mixed estuaries; the tidal flow there is much larger than the river flow, resulting in a mixing of salt and fresh water during incoming tide and consequently a small, vertical salinity-gradient. Although an air-bubbler obviously would work better in a large, vertical salinity-gradient, experiments carried out in 1965 in the Hudson River with a number of bubblers showed a salt water reduction of 70% at a point 30 miles upstream from Manhattan. Encouraging tests with pneumatic barriers to reduce salt water intrusion have also been conducted in the harbour of New Castle, Australia (2).

Recent reports from Holland (3) describe how the salinity of deep saltwater pockets in the Schelde estuary is being reduced successfully by air-bubblers, after this estuary was closed off from the sea by the "Delta" dams.

Another recent development in Holland as well as in England is the pneumatic barrier to prevent silt-laden bottom water from penetrating river locks when the gates are opened to admit ships.

The operation of both the de-icer and the pneumatic barrier make use of the convection current in water, created by air-bubbles. The rising water current spreads out into a strong horizontal current
which has inspired the design of another device, the pneumatic breakwater. The idea that oscillatory, progressive surface-waves could be attenuated by a curtain of air-bubbles was put forward in 1902, by the American Civil Engineer, P. Brasher. The principle underlying the mechanism in the behaviour of this pneumatic breakwater has been a subject of scientific controversy for many years. Originally it was thought that the introduction of big air-bubbles would disturb the harmonic particle motion of the surface waves and reduce their amplitude. An article in "Compressed Air" (November 1959) by G. R. Smith mentioned a reduction in amplitude of 50%. The energy of a progressive wave system of amplitude "a" can be represented by $E = \frac{1}{2} \gamma a^2$ where $\gamma$ is the specific weight of water in lbs/ft$^3$, and $a$, the amplitude in feet, by definition = $\frac{1}{2}$ wave height (H); therefore $E = \frac{1}{8} \gamma H^2$ in ft.lbs. per square foot of water surface, i.e. ft.lbs. per foot of wave length per foot of crest length. A reduction of amplitude by 50% would reduce the energy of the waves by 75%. Although this approach did not seem to be contradicted by research, it was perhaps misleading in that it implied that the bubbles were directly responsible for the wave-attenuating effect. Investigations carried out over the past thirty years (J. Thijsse, Delft, 1936; T. Evans, Southall, 1955, et al.) showed that waves up to a certain wave length could be attenuated by an opposing surface current, set up by the bubbles, but that the bubbles themselves did not have as much effect on the waves as implied by the first theory. It can be shown
analytically (Appendix I) that waves are shortened by an opposing surface current and that, in the process of becoming shorter, they become higher until they become too steep for stability and break before reaching the protected area.

The foregoing idea was put into practice at Dover Harbour, England, in 1957. A battery of jug-shaped polyethylene units was anchored on the sea floor between the two jetties forming the harbour entrance. The units received their air from shore-based compressors and belched out large air-bubbles with a frequency which could be regulated according to the size of the waves.

Actually, the surface currents created by the air-bubbles could also be created by water jets, emerging from nozzles inclined upwards close to the water surface. A battery of nozzles would be impracticable in a harbour entrance but might have its merits in protecting a beach or an exposed construction site, provided that such a water-jet breakwater is more economical than a pneumatic breakwater.

Another device which makes use of horizontal surface currents created in the water, but which cannot be produced by water-jets for practical reasons, is the pneumatic oil barrier. It was extensively tested in Hamburg in 1957. The possibility of oil spreading out over the water surface in a harbour after a collision of ships has always been of major concern to harbour authorities, not only because of dangerous fire hazards but also because of the inevitable damage done to marine life and public property by oil pollution. Obviously, it is essential to confine the spreading oil to a small
area. Barriers made of log booms or other floating objects have proven unsatisfactory, because they can get in the way of shipping. Moreover, with a surface current of more than 0.3 feet per second, the oil tends to pass underneath the floating barriers, even when they consist of pontoons with a draught of two feet.

A pneumatic oil barrier\(^{(4)}\) consists of a perforated pipe lying on the harbour bottom and connected to an air compressor ashore or on a barge. The ascending air-bubbles decrease the specific weight of the water column directly above the bubbler with respect to the surrounding water, which together with the velocity head of the vertically induced current in the water create a local rise or hump in the water surface. This hump must be high enough to generate a horizontal current with a velocity at least as great as that with which the oil spreads out. The oil's velocity in turn depends mostly on the specific weight of the oil and very little on its viscosity.

This type of barrier can be quickly installed by a boat, equipped with a number of weighted perforated hoses and an air compressor. The boat would lay the pipe in a circle around the oil patch, thus confining it. The barrier could let ships through and immediately close again after the ship had passed. Oil would be prevented from passing underneath, such as was the case with the conventional floating barriers. However, the barrier is useless in a mass movement of water with a velocity of at least 0.7 feet per second. The bubble-screen is then dispersed so effectively that the hump in the water surface disappears and is therefore unable to maintain a
sufficiently large horizontal flow to confine the oil.

Another practical application of air-bubblers which seems to depend more directly on the screen of rising air-bubbles in water, rather than the water currents induced by the bubbles, is the use of a bubble-screen in under-water demolition. The screen of air-bubbles damps the shock waves created by the exploding charge.

In summary, practical air-bubbler applications can be classified into three main categories:

I. Bubblers for creating a local vertical water current induced by the screen of rising air-bubbles:
   a) De-icing in rivers and harbours.
   b) Reduction of salt water intrusion in estuaries.
   c) Providing surface fish with bottom nutrients in lakes.
   d) Preventing silt intrusion into river locks.

II. Bubblers for creating horizontal surface currents generated by the rising air bubbles:
   a) Pneumatic breakwaters for harbours.
   b) Pneumatic oil barriers for harbours.

III. Bubblers to produce a screen of air-bubbles without regard to the currents induced in the surrounding water:
   a) Underwater demolition around Marine Installations.

A bubbler to prevent river sedimentation would clearly belong in the first category.
Before attempting to develop an air-bubbler which would induce a sufficiently high, upward velocity in the water, able to lift suspended sediment and bed-load to the required level (region of higher velocities) in the riverflow, the mechanics of the motion of air-bubbles and their interaction with the surrounding water must be examined first in detail. The following remarks are based on previous research by others. These have been confirmed by the author's observations, wherever possible in the time available for this project.

In still water, the rising air-bubbles from a point source of air (orifice) form a cone, which subtends an angle of from zero to twelve degrees, depending on the air pressure inside the bubbler. The air emerges from the orifice with a much higher velocity than the subsequent upward velocity of the bubbles, the air-bubbles spreading out immediately after leaving the orifice. As the bubbles rise, they maintain a very slight lateral motion under the influence of the water circulation induced by the bubble screen (the upward water motion diverges from the vertical above the orifice before it changes into a strong horizontal motion near the surface). If the air pressure inside the bubbler does not exceed the hydrostatic pressure appreciably, the bubbles will rise in a straight line.

By varying the orifice diameter, the head of water above
the orifice, and in particular the flow of air through the orifice, one can create a great many sizes and shapes of air-bubbles, which, after closer examination, may be reduced to three basic types: **Spherical bubbles**, **oblate spheroids** and **spherical caps** or mushrooms. The shape depends largely on their size; in ordinary tap water, small bubbles up to a diameter of about 1.0 mm appear to be spherical. The oblate spheriod is predominant among bubbles with an equivalent radius between 1.0 mm and 10.0 mm (equivalent radius to be defined as the radius of a sphere with a volume equal to that of the bubble). The large bubbles (over 10.0 mm) take on the shape of a mushroom with a hemispherical top. A number of investigators have related the shape of bubbles to their size as well as to the viscosity of the surrounding liquid. In this case, the bubble shape can be categorized according to the magnitude of the Reynolds Number

\[ (R) = \frac{2 re U p}{\mu} \]

\( (r_e = \text{equivalent radius, } U = \text{mean, upward bubble velocity, } p = \text{liquid density, } \mu = \text{liquid dynamic viscosity} - \text{all in consistent units}) \)

- **Spherical Bubbles**, \( R < 400 \) \( (5,6) \)
- **Oblate Spheroids**, \( 400 < R < 5000 \)
- **Spherical Caps**, \( R > 5000 \)

Although it is difficult (and for our project not essential) to ascribe a particular path to each type of bubble, the small spherical bubbles seem to go straight up, while the oblate spheroids and spherical caps follow an irregular sinuous path, quite resembling a helix. The very large bubbles (spherical caps) rock to and fro
as they ascend. Haberman and Morton(7), using motion pictures, experimented extensively on the motions of bubbles in different liquids and related the motion to the prevailing Reynolds Number: they found a rectilinear motion below $R = 300$, spiraling between $R = 300$ and $R = 3000$, and rectilinear motion with rocking, above $R = 3000$.

Surface tension is largely responsible for the spherical shape of small bubbles, where the hydrodynamic forces are still relatively small. (Hydrodynamic forces are forces due to the acceleration of the air-bubbles, in particular, the shear forces acting tangentially along the bubble surfaces). Surface tension then would tend to minimize the surface area of the bubble and since the sphere has the minimum surface area for a certain volume, the small bubbles would be expected to be spherical. For very low Reynolds numbers ($R = 40$ for filtered water), it has been shown experimentally(7) that the drag coefficient of very small bubbles becomes equal to that of rigid spheres in any liquid, in accordance with Stokes' law. Their upward velocity then will depend largely on the viscosity of the liquid.

When these small spherical bubbles become larger, a circulation inside the bubble develops, which decreases the total drag relative to that of rigid spheres of the same size (Fig. 2). This circulation was observed experimentally by Garner(8). When the radii of the bubbles exceed approximately 1.0 mm, the hydrodynamic forces become significant; surface tension cannot maintain
a spherical shape any more and the bubbles become flatter. The drag force exerted on the bubble by the liquid now becomes greater than that on a sphere of equal volume.

The bubbles distinguished by large spherical caps are strictly related to hydrodynamic forces. A constant drag coefficient was determined by Rosenberg (6), Davies and Taylor for geometrically similar bubbles of this type. Considering this constant, empirically-found drag coefficient and the terminal velocity of the bubble \( W \) (when all forces on the bubble, viz. drag, buoyancy and gravity, are in equilibrium), the conventional equation for the drag coefficient \( C_D = \frac{(2)(\text{Drag})}{\rho W^2 A} \) can be transformed into an expression containing the equivalent radius:

\[
C_D = \frac{(2)(\text{Buoyancy} - \text{Weight})}{\rho W^2 (\pi r_e^2)} = \frac{(2)(\frac{4}{3}\pi r_e^3)(\gamma_{\text{water}})g}{\gamma_{\text{water}} W^2 (\pi r_e^2)} = \frac{8}{3} \frac{g r_e}{W^2},
\]

ignoring the specific weight of air. For constant \( C_D \), the terminal velocity \( W \) would then be a function of \( \sqrt{\frac{r}{r_e}} \) only. Thus, spherical-cap bubbles rise with a terminal velocity which depends on their size, not on the physical properties of the fluid.

It is interesting to note that the drag for bubbles between 0.035 cm and 0.25 cm is larger in normal tap water than in filtered or distilled water. Investigations by Gorodetskaya (9), Stuke (10), Haberman and Morton, who added various surface-active substances
to water, confirmed that the drag coefficient of medium sized, ellipsoidal (oblate spheroid) bubbles in water containing surface-active materials, is larger than that in pure water. For the large, mushroom-type (spherical cap) bubbles, these impurities did not have any effect on the drag coefficient or their rate of rise.

Tap water contains many minute particles, which tend to adhere to the surface of a bubble and travel along with it as it moves upward. These minute particles seem to impart a certain amount of rigidity to the bubble surface. The phenomenon whereby very small bubbles obey Stokes' law can then be explained, not only by the tendency of the surface tension to give them a spherical shape but also by the presence in water of minute particles, which stick to the surfaces of the bubbles and make them behave like small, rigid spheres. It is not quite clear why, in pure water, the very tiny spherical bubbles also behave like rigid spheres.

When the spherical bubbles increase in size, the minute particles are prevented from adhering to the bubble surface by the increasing shear forces. The bubble then loses its rigidity, and a circulation inside the bubble is created with a consequent decrease in drag coefficient. This drag coefficient will thence continue to decrease with increasing Reynolds numbers (obviously, the decrease will be more rapid in pure water than in tap water, see figure 3) and the upward velocity will increase with size (see
figure 4) until, at a bubble diameter of 0.7 mm, the bubble starts to flatten out with a rapid increase in drag and decrease in velocity. As figure 4 shows, this maximum velocity in the curve is less pronounced in tap water, where the circulation inside the bubble is retarded by the presence of impurities on the boundary between bubble and water, resulting in a higher drag.

Finally, for bubbles larger than about 3.0 mm, inertia becomes the dominating force in comparison to viscosity and surface tension and the velocity curves of bubbles in filtered and tap water coincide. The presence of surface-active substances has no effect, because the bubbles are now unable to hold on to them due to the high shear forces.

The velocities of the very large spherical-cap bubbles will still be a function of their sizes but much of the gain in energy by an increased buoyancy will be lost by the dissipation of energy in shedding vortices by the lenticular and semi-spherical shaped bubbles of this category. It is this shedding of vortices which actually causes the spiraling and rocking motion of the very large bubbles.

The question arises why these big bubbles take on such an unfavourable shape from a hydrodynamic point of view. A rough plot of the pressure distribution of viscous flow past a sphere indicates what would happen if this sphere has no rigid boundaries:
There will be a negative pressure perpendicular to the direction of motion and the bubble will try to adjust its shape to this pressure distribution, resulting in the formation of an approximate ellipsoid of revolution generated about its minor axis, (oblate spheroid) which is parallel to the direction of motion.

It should be emphasized that the velocity of a single rising bubble is considerably lower than the velocity of a bubble cluster. Over the past years, a number of investigators (Hoefer; Hensen; Haberman and Morton) have left no doubt that the upward velocities of the bubbles increase with decreasing vertical spacing between the bubbles; Hensen\(^{(11)}\) found an average upward velocity of 23 cm/sec for single bubbles with a diameter of 15 mm, as compared with a velocity of 35 cm/sec for a stream (cluster) of bubbles emerging from the same opening. This phenomenon might well be explained by the presence of a vortex street behind each bubble, which progressively "helps" the following bubbles in their ascent. Exner, who conducted extensive tests with bubblers in the Lake of
Luzern, found a maximum upward bubble velocity of 68 cm/sec.

Application of the Gas Law ($PV = RT$) will show that the volume of a bubble rising from a depth of about 35 feet (a representative value for the depths found near wharves such as the Fraser-Surrey Wharf), will become only twice its original volume as it reaches the surface. The radius of this presumably equivalent spherical bubble would then increase by a fact of $2^{1/3} = 1.26$. As figure 4 shows, this would hardly change the upward velocity of the bubbles, in particular the larger ones. Therefore, the water depth has very little effect upon the upward velocity of the air-bubbles in this range of depth.

It stands to reason that the size of bubbles in water, immediately after emerging from an orifice, depends on the diameter of the orifice. In unfiltered water, where the terminal velocity of the bubbles increases with their diameter the terminal velocity would thus be expected to be proportional to the orifice diameter. Tests with a number of orifice diameters in the Hydraulics Laboratory at the University of British Columbia confirmed this observation (Appendix II). The tests were carried out with different air pressures as well and the results showed that the upward terminal velocity of the bubbles is also a function of the air pressure inside the bubbler. This is conceivable since a higher pressure increases the flow of air distributed among the orifices, which again implies an increase in the number of bubbles formed per unit of time. The air-bubbles follow each other closer for higher pressures with a consequently higher upward velocity.
Stehr's (4) developed an empirical relationship between mean bubble velocity, orifice diameter and relative air pressure:

\[ w = 0.4 F_D^{0.25} (P_i/P_a)^{0.20}, \]

where \( w \) = mean upward velocity of air-bubbles in meters per second; \( F_D \) = orifice in mm; \( P_i/P_a \) = ratio between absolute pressure inside and outside the bubbler.

Stehr showed bubble terminal velocities for four different orifice diameters, each with three different values of airflow rate. Similar tests in the Hydraulics Laboratory at the University of British Columbia produced results giving an equation similar to Stehr's, except for the constant and the exponent of the pressure ratio (see App.II). The test results gave

\[ W = 6.6 A_0^{0.25} \left( \frac{P_i}{P_o} \right)^{0.40} \]

with a different constant because of different units employed (\( A_0 \) = orifice area in inch\(^2\), \( W \) = upward mean velocity of bubbles in feet per second). This equation did not hold at pressures \( P_i \) only slightly above hydrostatic pressure where the bubbles are isolated and do not follow each other closely. In the latter case the actual velocities were much lower than the velocities predicted by this equation. Stehr's equation and the author's modification of it are strictly empirical, based on a "best-fit" and not on any physical analysis. The larger orifices release a great many different sizes and shapes of bubbles, each rising with its individual terminal velocity, thus making it rather difficult to arrive at a fixed relationship between bubble velocity, orifice diameter and air pressure.

Before discussing the motion induced in the surrounding...
water by air-bubbles, some practical aspects of the behaviour of air-bubbles rising in water are reviewed:

**REVIEW**

1) As they move upward in the water and increase in size, air-bubbles take on the following successive shapes: Spherical; Ellipsoidal (oblate spheroid); Spherical Cap (Mushroom). The viscosity of the fluid determines the sizes at which the transitions in shape take place.

2) The upward terminal velocity of the bubbles is generally proportional to their size, although there is a pronounced anomaly in filtered water. In this case, changing the bubble size affects the rate of rise of small bubbles much more than that of large bubbles.

3) The rate of rise of individual bubbles, released one after another, is smaller than the rate of rise of a stream or screen of bubbles of identical size. Therefore, the upward velocity of the bubbles is proportional to the amount of air discharged by an orifice in a unit of time, and to the air pressure inside the bubbler.

4) The larger the orifice diameter, the larger the bubbles, with a consequent increase in upward velocity. Of course, the orifice diameter is limited by the capacity of the compressor and the bubbles cannot grow indefinitely. They would divide into smaller bubbles.

5) The relatively shallow depths of water of about 30 to 40 feet found near a wharf, cause the air-bubbles to expand to
a volume twice their original volume, which hardly affects their upward velocities.

6) The air emerging from a single orifice will rise to the water surface in a cone of bubbles, subtending an angle of from $0^\circ$ to $12^\circ$, this angle being a function of the air discharge, which is dependent on the air pressure inside the bubbler.

**II-2 The Upward Flow Induced in the Surrounding Water by the Rising Air-Bubbles**

The upward flow in the water induced by the screen of rising air-bubbles is basically a result of the conversion of potential energy of the air-bubbles into kinetic energy of the surrounding water.

To find an expression for the energy available in the air at the moment it leaves the orifice, iso-thermal conditions may be assumed. The expansion of the air-bubble, as it rises to the surface, takes place in an environment which is not only a virtually infinite reservoir but also has a much greater specific heat than air (specific heat of water = 1.00; $C_p$ of air = 0.24).

As the bubble rises, the work of expansion $= \int_{V_1}^{V_2} p(dV)$, where $p$ and $\Psi$ are respectively the inside pressure and volume of the bubble and $V_1$ and $V_2$ are the volumes of the bubble at two arbitrarily chosen levels 1 and 2, as it rises in the water. The work done
by the bubbles on the water, as they expand on their way from the orifice to the water surface, i.e. the total energy available in the air just before leaving the orifice, \( E = \int_{V_0}^{V_s} p(dV) = K \int_{V_0}^{V_s} \frac{dV}{V} \) (since \( pV = \) constant, \( K = K' \ln \frac{V_s}{V_0} \), where \( V_s \) = volume of bubble when it leaves the orifice. Now, if we express the volumes in the corresponding pressures and let \( H_A = \) atmospheric pressure in feet of water, \( Q_A = \) cubic feet of free air per second per linear foot length of bubbler, \( D = \) water depth at bubbler in feet, \( \gamma_w = \) specific weight of water, we have:

\[
\frac{V_s}{V_0} = \frac{D + H_A}{H_A}, \quad \text{giving:}
\]

\[
E = K \ln \frac{D + H_A}{H_A} = P_s \cdot V_s \cdot \gamma_w \ln \frac{D + H_A}{H_A}
\]

\[
\therefore E + H_A(\gamma_w Q_A) \ln \frac{D + H_A}{H_A} \quad \text{foot pounds per second per foot length of the bubbler.}
\]

From this equation, it can readily be deduced that the energy available in a bubbler will increase with a) the flow of air emerging from a unit length of bubbler; b) the depth of submergence of the bubbler. A short calculation will show that a bubble of free air (i.e. at atmospheric pressure) leaving a bubbler thirty feet deep (the depth at the Fraser-Surrey Wharf) has almost six times as much energy as a bubble of the same volume of free air leaving a bubbler four feet deep (the usual water depth in the laboratory flume).

To compare the effect of water-depth (\( D \)) and the rate of
air-discharge \((Q_A)\) on the available energy in the bubbler, let us consider again the energy equation: 

\[ E = H_A (\gamma Q_A) \ln \frac{D + H_A}{H_A} \]

and its partial derivations w.r.t. to \(Q_A\) and \(D\):

\[ \frac{3E}{3Q_A} = H_A \gamma \ln \frac{D + H_A}{H_A} \]

Therefore, at constant depth, the energy is a linear function of the air-discharge.

Furthermore,

\[ \frac{3E}{3D} = H_A (\gamma Q_A) \frac{H}{D + H_A} \]

\[ \frac{3E}{3D^2} = - H_A^2 (\gamma Q_A) \frac{1}{(D + H_A)^2} \]

Obviously, the second derivative is always negative, implying that an increase in depth will increase the energy at an ever decreasing rate. Therefore, it is evident that the available energy in the bubbler is affected more by the air discharge than by the water depth.

Searching for a relationship between the upward currents induced in the water by the screen of rising air-bubbles, and the amount of air released by the bubbler, Sir Geoffrey Taylor \(^\text{12}\) drew an analogy between this upward flow of water and the upward flow of air caused by a heat source, such as a candle flame. He reasoned that a volume \((v)\) of air-bubbles would give the same buoyancy to a unit volume of water as a rise in temperature \((\theta)\) would give to air at absolute temperature \((T)\), if

\[ v = \frac{\theta}{T} \]

(This idea is perhaps easier to see if we put \(v = \frac{\theta}{T}\); a volume of air \(v\) then gives a unit volume of water the same decrease in density as a temperature rise \(\theta\) gives to an arbitrary volume of air at

\[ v = \frac{\theta}{T} \]

The relationship between the upward current of water and the amount of air released by the bubbler is therefore

\[ F = H_A (\gamma Q_A) \ln \frac{D + H_A}{H_A} \]

where \(F\) is the upward current of water.

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\[ F = H_A (\gamma Q_A) \ln \frac{D + H_A}{H_A} \]

where \(F\) is the upward current of water.
temperature $T$).

To find an analogy between the vertical currents induced in the water by bubbles and those induced in air by heat, we release in water a volume $V$ of air from a unit length of bubbler, and we release in air $H$ calories of heat per second from a line source one centimeter long, so that $H = \frac{V}{\rho s T}$ ($\rho$ = density of air and $s$ = specific heat of air). Clearly, the same amounts of energy are involved, creating the same distribution of vertical currents. Taylor then included the vertical velocity by putting $H = \int p_{w} v_{0} (dx)$, where $v$ = vertical air velocity, $x$ is a horizontal coordinate.

$\left\lfloor (1)(dx)(w)(\rho) \right\rfloor$ is the mass of the volume of heated air passing through a horizontal face per unit time.

Considering the whole horizontal plane, bounded by a unit length heat source and $x$, we have $H = \int p_{w} v_{0} (dx)$; substituting this $H$ and $v = \frac{\theta}{T}$ into $V = H(\rho s T)$, we have $V = \int w v_{0} dx$, for $\rho$ and $s$ constant.

Taylor, referring to the work by Schmidt (1941) on currents above a horizontal line source of heat, expressed the vertical velocity as a function of $x$ and $z$: $w = W_{0} f \left( \frac{x}{z} \right)$, since, as was shown by Schmidt, the heat spreads out linearly so that $x$ is proportional
to z. \( W_0 \), being an arbitrary term, was chosen by Taylor to be the maximum vertical velocity in the centre of the rising column.

Taylor then, again referring to Schmidt's theory, put \( W_0 = K(Hg/\rho sT)^{1/3} \), an experimentally proven equation with the constant \( K \) about 1.9.

When we now substitute into this equation the previously derived expression \( H = \nu psT \), we arrive at the maximum vertical water current produced by an air-discharge \( \nu \) per second per linear foot of bubbler: \( W_0 = K(\nu g)^{1/3} \), where \( W_0 \) is the maximum upward water velocity in feet per second, \( \nu \) is the volume of air in cubic feet per second at the depth of submergence of the bubbler, and \( K \) is a constant, approximately 1.9.
SECTION III

LABORATORY TESTS

III-1 Calibration of Orifice Meter in 8-inch Pipe Leading to Flume

Most of the laboratory tests of the air-bubbler were carried out in a steel and glass flume, thirty feet long, five feet high, and one and one-half feet wide.

An 8-inch diameter water-supply pipe leading to the flume had already been fitted with an uncalibrated orifice plate (Line 8.071"; Bore 4.974"). To measure the flow into the flume during experiments with air-bubbles in moving water, a three-feet long, differential manometer was permanently installed and connected to the supply pipe on each side of the orifice plate. The orifice meter was calibrated as follows:

A vertical, sharp-crested, rectangular suppressed weir was built near the downstream end of the flume. For this type of weir, the discharge \( Q \) in c.f.s. is expressed in terms of the head \( H \) of water (in feet) above the weir: \( Q = k b H^{3/2} \), where \( b \) is the length of the weir crest in feet. According to C.W. Harris, \( k = 3.27 + 1.5 \left( \frac{H}{D} \right)^{2} + \frac{C}{H^{2}} \). \( D \) is the height of the weir crest in feet \( (H \leq 0.4D) \). The term \( 1.5 \left( \frac{H}{D} \right)^{2} \) is the correction for velocity of approach and incomplete vertical contraction. The term \( \frac{C}{H} \) is the correction for frictional drag (viscosity), where \( C = 0.020 \) for fresh water at 56°F (the temperature of the water entering the flume).

The head \( H \) on the weir was measured by a hook gauge in a plexiglass stilling well, placed outside of the flume and connected
to it and far enough upstream from the weir to avoid the down-
drop curve.

"Datum" (zero reading of the hook-gauge) was obtained by setting up
a surveyor's level outside the flume and moving the hook gauge to
a position where the tip of the gauge would be level with the brass
weir crest, which was made exactly horizontal. The elevations
were measured with a graduated levelling rod to an accuracy of
0.001 foot.
After the datum (zero reading) had been established, the main valve to the flume was opened in increments to produce corresponding increments in the manometer readings of about 0.5 inch and the weir hook gauge read. The procedure was repeated both ways (i.e. for increasing as well as decreasing flows) and the results plotted as a cartesian graph. This was followed by a double logarithmic graph merely to detect any irregularities in the readings (see figures 5 and 6). The maximum discharge available was found to be 5.79 c.f.s.

III-2 Laboratory Investigations to Determine Hydraulic Slopes as a Criterion for Incipient Motion of Bed Material

To assess the possibilities of an air-bubbler as a method to prevent shoaling of bed load, it was necessary to determine the hydraulic conditions under which incipient bed-load movement would occur in front of the wharf. It was reasonable to assume that bed load contributed in some measure to the shoaling at the Fraser-Surrey Wharf, in conjunction with the deposition of suspended sediment.

It would be very difficult to observe bed-load movement in the river due to turbidity, depth of flow and high water-velocities. Therefore, certain variables were selected which were known to affect bed-load movement and which could readily be measured in the field. By testing the interaction of these variables in the laboratory and then measuring the same variables in the field, the hydraulic conditions at which incipient bed-load transport would occur, could be predicted.
Several important variables involved in the incipient (threshold) movement of bed-load are combined in du Boy's Law: The tractive shear stress ($\tau$) exerted by a moving liquid on a boundary surface of any material is $\tau = \gamma RS$, where $\gamma =$ specific weight of the liquid, $R =$ wetted perimeter, $S =$ hydraulic slope, i.e. the sine of the angle of slope of the total energy gradient, which is the same as the slope of the hydraulic grade line (water-surface slope), and the slope of the channel bed in steady, uniform, turbulent flow. By measuring $\gamma$, $R$ and $S$ in the laboratory flume at the instant when the bed particles of a certain size commenced to move, $\tau_c$, the critical tractive shear stress for incipient motion, was obtained. If similar measurements in the prototype revealed a shear stress larger than $\tau_c$, it was reasonable to assume that bed load movement occurred in the prototype.

To measure the hydraulic slope ($S$) in the laboratory, the fixed steel and glass flume, already referred to, was employed with a plexiglass stilling well connected to each of the upstream and downstream ends. The hose connections to the stilling wells were in the floor of the flume, 18 feet apart. Each stilling well contained a plexiglass hollow cylinder, weighted with leadshot and suspended from a short brass cantilever mounted on top of the flume. (vid.fig.7). Strain gauges connected to a strain indicator were attached to the top and bottom surfaces of the cantilevers. Both cylinders were of equal dimensions and equal weights. The strain gauges and the strain indicator formed a complete Wheatstone bridge
with a balanced condition at E = 0, i.e. when the water levels in both stilling wells were at the same height. However, when the water levels were not at the same height, the differences in buoyant forces acting upon the cylinders would create different bending moments in the cantilevers, resulting in a positive or negative reading of the strain indicator 13.

To calibrate the foregoing equipment, both stilling wells were first disconnected from the flume and connected directly to each other with a plastic hose. Water was then added until the cylinders were partially submerged. After a sufficient time for the water levels in both stilling wells to reach static equilibrium, the strain indicator was set at zero. As a check, this procedure was repeated several times to make certain that the recorder pen would return to zero, regardless of the heights of the water levels in the stilling wells. (It should be emphasized that the equipment was designed to measure differences in water surface heights, not individual heights.)

To determine the relationship between the recorder units and actual differences in water-surface heights, both stilling wells were partially filled with water to the same elevation and then disconnected from each other. In one of the stilling wells, a point gauge, equipped with an electronic switch was capable of reading to an accuracy of $10^{-4}$ foot. To this stilling well, water was added and the increase in elevation read on the electronic point gauge and compared with the reading of the recorder. After a number of repetitions, it was found that two units of the recorder
diagram corresponded to a difference in water-surface heights between the two stilling wells of 0.01 feet. During this calibration the linearity of the measurements was also tested.

In the flume study, particle-sizes were selected which most commonly occurred in bed-material samples obtained at the approaches to the wharf and separate flume tests were performed on each selected fraction separated out by sieving. The bottom of the flume was covered with a thin layer of sand of a particular particle size (or fraction); flow was imposed on the sand bed and increased very slowly until incipient motion was observed. With the aid of a magnifying glass, the observer watched for the first sand particles to become agitated and "stand out" (i.e., protrude from the sand bed). When incipient motion occurred, the rate of flow was kept constant and the water depth read by a point gauge with an electronic switch (a neon light indicating contact of the point with the water surface). The recorder showed the difference in height between the water surfaces in the two stilling wells, from which the hydraulic slope could be determined.

The tests were repeated several times, mainly to acquire good judgement in determining incipient motion. The results were generally lower than the published values\textsuperscript{14} (Straub) but agreed well with the results of similar laboratory studies with Fraser River sand at the University of British Columbia in 1955. (The latter studies were carried out with varying depths of water and are extensively described in a report by
Apart from these other studies, the following values were obtained for the critical tractive shear stress \( \tau_c \):

- Diameter of sand particles 0.25 mm: \( \tau_c = 0.20 \text{ lb/ft}^2 \)
- Diameter of sand particles 0.15 mm: \( \tau_c = 0.017 \text{ lb/ft}^2 \)
- Diameter of sand particles 0.10 mm: \( \tau_c = 0.012 \text{ lb/ft}^2 \)

Normally, tests of this nature are made with tilting flumes in which the water surface is made to agree with the bedslope, thus ensuring uniform flow at constant depth. In these tests, the depth was not constant. To obtain reliable values of \( \tau_c \), the water depth had to be measured at the point where incipient motion occurred. Thus incipient motion was allowed to occur near the intake of the hose to the stilling well, where the water depth could be measured accurately with a point gauge and electronic switch.

### III-3 Determination of Settling Velocities of Fraser River Sand

The upward velocity of the water current induced by the rising air-bubbles should be at least equal to the settling velocity of the biggest sand particles in still water. If these particles are assumed to be spherical with a density of 2.65, then their settling velocities in still water vary with their diameters as follows: Particles smaller than 0.15 mm in diameter obey Stokes' Law: \( v_s = 0.545 \dfrac{d^2 \rho - 1}{\mu} \); particles larger than 2.0 mm obey Newton's Law, \( v_s = 14.38 \sqrt{\dfrac{d}{\rho - 1}} \); where \( d = \text{diameter in mm, } \rho = \text{density in grams/cm}^3 \) viz. 2.65, \( (\text{the density of quartz is} \)
regarded as being characteristic of all sediment\textsuperscript{14}; $\mu =$ dynamic viscosity in poises and $v_s =$ settling velocity in centimeters/second.

A zone of transition between the two laws covers the behaviour of spherical particles between 0.15 mm and 2.0 mm, so that, roughly speaking, the relationship between settling velocity and particle size, can be summarized as follows:

\begin{align*}
    d < 0.15 \text{ mm} & : \quad v_s \propto d^2 \\
    0.15 \text{ mm} < d < 2.0 \text{ mm} & : \quad v_s \propto d \\
    2.0 \text{ mm} < d & : \quad v_s \propto \sqrt{d}
\end{align*}

The relationship can be plotted on a log-log grid and the graph for $t = 16^\circ \text{C}$ is shown on fig.8. It has been shown by a number of investigators (Hazen, Bardwell, e.a.) that sandgrains, generally, behave like perfect spheres except that the transition zone between Stokes' Law and Newton's Law for natural sand particles is smaller than that for perfect spheres, being somewhere between 0.1 mm and 0.3 mm. The sand found in front of the Fraser-Surrey wharf has a median diameter of slightly over 0.1 mm. This being a boundary value it would call for an actual measurement of the settling velocities in the laboratory, instead of assuming values predicted by theory, or from published plots of settlement rates.

The settling velocities of the river sand were measured with a modified Puri Siltometer, after the various particle sizes had been separated by a standard sieve analysis.

The Puri Siltometer, described in detail in "Soils, their Physics and Chemistry" by A.N. Puri, in principle consists of a plexiglass tube, 200 cm long and closed at the top end,
which stands with its lower end immersed in water in an open circular metal trough. Both tube and trough are filled with water, the airtight top of the tube maintaining the water-column. A number of segmental aluminium dishes are arranged along the circumference of the bottom of the slowly rotating trough. A sample of wet sand is released instantaneously from the top of the tube by a switch-operated solenoid and distributed among the dishes according to the settling velocities of the various particle sizes. It is thus possible to carry out a sediment analysis, obtaining a grain-size distribution based on settling velocities which in turn depend on particle size, shape and density.

This apparatus was ideal for measuring settling velocities, particularly because of its quick-acting sediment releasing device, modified from the original by Puri, for the Fraser River Model Project at the University of British Columbia (1948-62). The wet sand was introduced into a conical funnel at the top of the sedimentation tube and prevented from entering the tube by a small tightly fitting, inverted conical funnel (see fig.9) having its brass stem in the centre of a solenoid. One single switch energized the solenoid, releasing the sample and simultaneously started an electrical timer. Falling particles could be observed clearly over a vertical distance of six feet; the settling rate of seven particle sizes, ranging from 0.074 mm to 2.0 mm, being measured. Care had to be taken to ensure that convection currents would not be created in the tube by sunlight or other heat sources. The
observations were repeated several times and the results (Appendix III) plotted on log-log paper, fig. 8. The slope of the best-fit straight line agrees well with the 45° slope of the transition part of the theoretical curve \((v_\text{s} \propto d)\) and there is a tendency at both ends of the range of particle sizes to follow the appropriate Stokes' Law, or Newton's Law. However, it is to be noted that nearly all experimental results consistently yield settling velocities higher than those predicted by theory, particularly for particle sizes below 1.5 \(\text{mm}\). Flocculation, which might explain the high settling rate of very fine sand, can safely be disregarded in fresh water. A reasonable explanation for the higher velocities seems to be that the boundaries of the transition region are not clearly defined. This is particularly evident in the lower part of the region. The observed velocity of 2 \(\text{cm/sec}\) for a sand particle of 0.105 \(\text{mm}\) sieve diameter would give a Reynolds number \((R)\) of 1.73; for particles of diameter 0.074 \(\text{mm}\) with an observed velocity of 1.3 \(\text{cm/sec}\), \(R = 0.82\). This smallest size tested could then still be regarded as a boundary case, since the upper limit of the laminar range for which Stokes' Law is applicable, occurs at a Reynold's number of about 0.8.

Field measurements of the dissolved solids contained in the river water in front of the wharf indicated no salt water intrusion from the ocean during the freshet (Section IV-3). Therefore, no flocculation, with a possible increase in settling rate of the very fine particles, could be expected. Hence, the settling velocities found by the Puri Siltometer Analysis were assumed to
apply to field conditions and were employed in the design of the air-bubbler.

III-4 Supply of Compressed Air to the Air-Bubbler in the Laboratory Flume

Compressed air was supplied to the air-bubbler by a compressor having a pressure regulator. To measure the mass flow of air, an orifice plate, bore diameter 1/4 inch, was installed in the existing one-inch diameter air-supply pipe and a water differential manometer was connected to the pipe, upstream and downstream of the orifice plate, with vena contracta taps in accordance with ASME specifications. To avoid abnormal turbulence in the air flow, the orifice plate was installed in a straight stretch of pipe, preceded by a smooth brass section, four feet long and 1.06 inch in diameter. A bourdon gauge, located one foot downstream from the orifice plate, measured the pressure of the air approaching the orifice, assuming negligible pressure drop across the orifice.

The mass-flow \( \dot{m} \) of air in slugs per second could be obtained from the compressible flow equation, in CYA \( \sqrt{\frac{2\rho\Delta p}{Y}} \), where the expansion factor \( Y = 1 \), \( A_o \) = cross sectional area of the orifice in ft\(^2\), \( \Delta p \) = pressure drop across the orifice in psf, obtained from the manometer reading, \( \rho \) = density of air in slugs/ft\(^3\), depending on pressure and temperature, and \( C \) = discharge coefficient. Solving for known quantities (vid. Appendix IV), we can express the mass flow in slugs per second:

\[
\dot{m} = 10.98 \times 10^{-4} \, C \sqrt{\rho \Delta h}, \, \Delta h \text{ being the manometer differential}
\]
Because of the very small quantities of air entering the bubbler, the use of published tabulated values of C in calculations of the mass flow of air was avoided and the orifice meter was calibrated directly in the following manner:

A plexiglass box was constructed as shown below, two feet high, one square foot in cross-section, open at the bottom and airtight at top and sides, except for a 1/4-inch diameter opening in the top to connect the space inside the box with a simple mercury manometer, and a similar opening near the bottom's edge to connect the box to the air-supply pipe coming from the orifice meter.
To calibrate the orifice meter, the box, (or gasometer), was completely submerged and filled with water. Air was then fed into the box after passing through the orifice meter. While the box was held down by the overhead steel frame of the flume, the increasing volume of air trapped in the box forced the water-level down inside the box. When the water-level in the box reached a point about 1.5 feet below the top of the box (measured by an attached scale), the air flow was stopped and the time observed. The air flow could thus be derived from the quantity of air flowing into the box, the air pressure and temperature:

\[ \gamma = \frac{p}{RT} \text{ lbs/ft}^3; \text{ weight flow of air } \dot{w} = \frac{\nu}{t} \text{ lbs/sec, where } \nu \text{ is the measured volume of air (cubical content of air space inside box).} \]

The coefficient \( C \) followed from \( \dot{w} = \left(32.2 \times 10.98 \times 10^{-4} \sqrt{\rho \Delta h}\right) \times C \), where \( \rho \) could be calculated and \( \Delta h \), the observed differential of the water manometer (inches of water) at the orifice meter. The calibration was carried out for different values of air flow, which again could be controlled by the pressure regulator. Figure 10 shows the calibration results, in the conventional units of weight flow. To arrive at values for the coefficient \( C \) in the previous formula, the weight flow had to be converted into mass flow.

The question might be raised whether this method is only an approximation, since the quantity of air entering the gasometer is measured at the end of a time interval, without considering possible changes in conditions affecting the air flow in the supply pipe during that time interval. However, while the setting of
the air regulator is not altered during one particular measurement, the only variable affecting the rate of flow of air emerging from the air-supply pipe would be the pressure at the discharging end of the pipe. This pressure is always equal to the hydrostatic pressure which is constant as long as the box is held in the same vertical position relative to the water-surface outside the box. Therefore, the amount of air collected inside the gasometer during a time interval $\Delta t$, corrected for pressure and temperature and divided by $\Delta t$, represented the rate of flow of air in the supply pipe per unit time at any instant and is not an average value.

The results obtained for $C$, although quite consistent, are slightly below the published (ASME) values of $C$ in the previously mentioned flow equation. This might have been caused by the construction of the orifice and vena contracta taps. At any rate, they were the observed values for this particular orifice meter as installed, consequently these values were used in the computation of air flow into the bubblers in the laboratory tests.

It should perhaps be mentioned that this gasometer was designed to fill the need for a simple and reasonably accurate method of calibrating the orifice meter in the time available. In its present form, it is difficult to handle and much too small for large air flows. A more permanent construction to hold the gasometer in
the water would be advisable, lest the unwary observer be caught unawares by the rapidly surfacing box. Finally, the gasometer could be simplified by eliminating the simple mercury manometer. The difference in elevation between water levels inside and outside of the box would automatically measure the air pressure inside the box.

III-5 Laboratory Investigations of Upward Water Velocities Induced by Air-Bubbles

To investigate the interaction between the rising air-bubbles and the surrounding water, the laboratory flume previously referred to, was employed. However, the flume being rather short and the water discharge limited, it was difficult to simulate a river flow with the correct distribution of velocities. Baffles at the upstream end and a bulkhead at the downstream end containing small gates at three different levels to regulate the flow, helped somewhat to improve the distributions. The calibration of the orifice meter in the 8-inch diameter water-supply main indicated an available maximum flow of 5.8 cubic feet per second, which gave a mean velocity of less than one foot per second in a depth of five feet of water (flume width 1-1/2 feet). Installing side constrictions in the flume to increase the velocity was not practical since the already limited width was needed to accommodate at least a one-foot-length of perforated air hose laid on the bed of the flume, perpendicular to the flow. Decreasing the depth of water would have increased the flow velocity; however, this flume
was selected because it was the only one in the laboratory capable of accommodating a water depth of five feet. The latter feature was considered important in this study involving the vertical distribution of velocities.

A horizontal flow in the flume was less important than the vertical flow induced in the water by the screen of rising air-bubbles and which had to overcome the settling velocities of the sediment to prevent deposition. In a horizontal flow, the screen of air-bubbles naturally becomes inclined in the downstream direction and loses its intensity. The bubbles spread out and do not follow each other closely; consequently, they are less effective in entraining the surrounding water than they would be in still water. The familiar hummock or hump in the water surface, seen above a column of bubbles in still water, disappears very rapidly when there is a horizontal flow. This is an indication that the kinetic energy given to the water by the bubbles (and converted again into potential energy at the surface in the form of this hump) has either been dissipated in turbulent friction or spread out over a large area. Both causes can result in smaller maximum vertically-upward water velocities. Therefore, although turbulence in the river flow would help considerably to keep the sediment in suspension, the upward water velocities induced by the bubbles are quite likely adversely affected. The complex interaction of rising air-bubbles in a superimposed horizontal flow of water, and its effect upon sediment moving either in suspension or as bed load, could be investigated to advantage in a very carefully
prepared and controlled environment such as existed in the Fraser River Model at the University of B.C. (1948-1962), or perhaps, at greater cost, in the prototype. However, with the laboratory facilities available for the air-bubbler project, the observations of the circulation induced in the water by the screen of rising air-bubbles were carried out in still water. If the maximum, upward, water velocities induced by the bubbles in still water would be less than the settling velocities of the sediment grain sizes most commonly found near the wharf, it was felt that little would be gained by investigating this relationship in a superimposed horizontal flow in a laboratory flume having a limited depth and flow.

The laboratory air-bubblers were made of one-inch (I.D.) diameter polyethylene hose 16 inches long. Five different test sections were prepared with orifices (air holes) varying in diameter from 0.020 to 1/16 of an inch, with spacings varying from 1/4 to six inches. They were successively placed on the bottom of the flume, mid-way along its length and perpendicular to the sides. They were readily interchangeable; one end being connected to the laboratory compressed-air supply via a 1/4-inch diameter polyflo tube, leading to the top of the flume and down inside; the other end was similarly connected to a Bourdon pressure gauge installed outside the flume, at the same height as the air-bubbler hose.

With the above arrangement, the bubbles created a relatively free, two-dimensional water circulation parallel to the
glass side of the flume. This is a marked advantage of having a flume instead of a square or round tank; particularly when a line source of air and not a point source, is under investigation.

A reference grid graduated in feet was painted on the glass front side of the flume as well as on the opposite or rear steel side. The steel side had previously been painted white, which facilitated visual observation of flow patterns, traced by dyes.

Some preliminary research on air-bubblers was carried out before the prototype bubblers were placed in the Fraser River in May 1966. The available technical literature on air-bubbler applications was concerned mainly on the effect of rising bubbles on the upper layers of water, such as occurred in pneumatic breakwaters, de-icers, pneumatic oil barriers, etc. The orifice spacing and diameter, which seemed effective for pneumatic breakwaters and de-icers, would not necessarily prevent sedimentation.

For these preliminary investigations a 16-inch length of hose with three 1/16-inch diameter orifices, spaced six inches apart, was placed on the bottom of the laboratory flume at right angles to the flow. To trace the water circulation, meriam, neutral oil, drops (specific gravity = 1.00), coloured red, were injected into the water at different depths. The injector was the tip of a finely drawn glass tube attached to a cylindrical brass messenger, one-inch long and 3/8 inch diameter, which could slide along a taut, vertical nylon fish-line between the water surface
Meriam oil was fed to the glass tip through a nylon intracath, 1 mm in diameter, connected to a syringe outside the flume. The nylon fish-line passing through the messenger could be moved parallel to itself and by pulling on a second nylon fish-line, attached to the messenger, the observer could place the messenger at any desired position in the water.

Although the meriam drops were not employed to measure water velocities accurately, they mapped a continuous 2-dimensional flow pattern, a distinct advantage over point, velocity measurements with a current meter. The general flow pattern agreed closely with that observed by many previous investigators. It showed a very rapid upward water movement in the immediate vicinity of the bubbles; an equally rapid horizontal movement at and near the water surface away from the bubble screen; a weak and rather confused circulation downward a few feet away from the bubbles; and a slowly increasing horizontal motion towards the bubbler along the bottom. Near the
Two-dimensional water circulation induced by rising air bubbles from a line source as obtained from the observation of Meriam drops bottom, a slight inward motion away from the walls of the flume towards the centre was also observed, obviously the wall effect on the circulatory pattern.

It was not possible to observe properly the path of the meriam drops injected into the centre of the screen of air-bubbles. The water velocities at various points in the flow pattern increased with an increase of air pressure inside the bubbler hose, which depended on the rate of air flow. As the injector was moved horizontally away from the bubble screen, the upward velocities of the injected meriam drops decreased rapidly. The most important observation, however, was the absence of any upward water velocity near the air-bubbler hose. Meriam drops injected between the air holes and within approximately four inches above the bubbler,
remained nearly motionless. They started to move when injected within an inch horizontally (measured along the hose) of the air holes. However, there was obviously a region between the orifices which would present no barrier to the moving bed sediment. Without examining the upward velocities further, in other regions above this particular test bubbler, the six-inch spacing had to be rejected as possible design for the prototype bubbler. After a number of trials with other spacings and orifice diameters, (still using the meriam drops), a spacing of 1/4 inch and a diameter of 0.0135 inch were selected for the orifices of the prototype. This selection was based mainly on attempts to minimize the dead-water region close to the bubbler. An orifice spacing smaller than 1/4 inch was not considered practical for the prototype bubbler because it was felt that too many holes per unit length of hose would weaken it. The hose would probably be subjected to severe tensile and bending stresses in the field. The small diameter for the air holes was favoured over larger diameters because of the limited capacity of the air-compressors available. Another consideration was that, for the same volume of air escaping, a great many small air-bubbles would have a larger total cross-sectional area than a few large air-bubbles and would therefore entrain more water.

The urgency of preparing the prototype bubblers before the onset of the freshet in the spring of 1966 temporarily halted the laboratory studies. After a three-months' period of field investigations, the laboratory studies were renewed and a
miniature current meter was installed in the flume to replace the Meriam drop injector.

This "Miniflow" current meter, manufactured by Armstrong Whitworth Aircraft Limited, England, is primarily designed to measure very low velocities of flow in a laboratory. It consists of a stainless steel probe, 18 inches long and 1/16 inch in diameter, with a measuring head 0.6 inch in diameter containing a five-bladed Cobex plastic rotor. The revolutions of this rotor are counted electronically and displayed by three Dekatron counters (Plate 3). A detailed description of the electronic arrangement is beyond the scope of this thesis but has been published by the manufacturers in one of their brochures.16

Only the components of flow velocity perpendicular to the plane of the rotor could be measured. To study the upward water velocities, therefore, the meter probe had to be horizontal. The connection between the probe and the coaxial cable leading to the Dekatron counters had to remain dry and it was necessary to be able to move the rotor to any desirable position near the bubble screen. To satisfy the foregoing three requirements, the end of the probe which was connected to the coaxial cable was sealed in the horizontal leg of an L-shaped, one-inch diameter steel pipe. The vertical leg of this pipe could slide in an adjustable clamp fastened to the top of the flume. Water was prevented from entering the pipe by a rubber plug, which also held the probe firmly in position, keeping the plane of rotation of the rotor horizontal.
Possible errors in the measurement of water velocities, caused by the presence of the steel pipe near the region of flow, were ignored.

Approximately 600 velocity determinations were obtained, each requiring at least ten readings. At each point of velocity measurement, five different rates of air flow were introduced; five different test bubblers were studied, each with a different orifice diameter and spacing; the depth of water was varied from five to two feet; the position of the current meter was varied vertically between half-a-foot above the bottom of the flume and one foot below the surface; in addition, a number of readings were taken in the lowest layers, between 0.2 foot and 1.0 foot above the bottom of the flume. The position of the current meter was varied horizontally between a point directly above the bubbler and a point three feet from the centre of the bubble screen.

The probe was usually kept on the longitudinal centre line of the
flume; however, a few measurements were made to investigate wall effects, by moving the rotor close to one wall of the flume.

As was mentioned earlier, the air flow was measured by a calibrated orifice meter. Tap water was employed in the flume tests. Since the miniflow current meter could only register in conductive liquids, sodium silicate (waterglass) had to be added to the tap water, at a concentration of 1:1000 by volume.

It should be noted that the miniflow current meter registers both direct and reverse flow. The rotor had to be watched carefully at very low flows when turbulence at times might reverse the flow, resulting in abnormal readings. Very fine, almost invisible hairs in the water, sometimes fouled the delicate rotor spindle; this could easily be detected by an unexpected sudden drop in the pulsing rate displayed by the Dekatron counters. However, it would be good practice to check the rotor frequently, even if fouling is not noticeable.

Results of Velocity Measurements

The tables in Appendix V show some of the results of velocity measurements made and figures 12 to 15 were selected to illustrate the discussion.

For ease in orientation, a cartesian coordinate system is introduced, the x-y plane coinciding with the bottom of the flume, the y-axis along the bubbler, the x-axis along the longitudinal centre line of the flume. The z-axis, measured from the origin upward, denotes the direction of the upward velocities, induced
in the water by the rising air-bubbles.

Figure 12 represents the upward velocity distributions at levels 0.5, 1, 2 and 3 feet above the bottom of the flume, for an orifice diameter of 0.020 inch and a spacing of 0.250 inch. The depth of water was four feet. Although measurements of water velocity were taken for five different rates of flow of air, only two sets are plotted, viz. for a maximum of $3.2 \times 10^{-3}$ lbs/sec/ft length of air hose and a minimum of $0.9 \times 10^{-3}$ lbs/sec/ft. The two curves for each level may be regarded as an envelope of all five rates of air flow investigated. For the sake of clarity of the graph, the bubble screen is not shown.

The graph shows clearly how the volume of water moving vertically upward increases with vertical distance above the bubbler. However, the upward velocity does not change appreciably with vertical distance. There is a very pronounced peak velocity in the centre of the bubble screen, with a maximum upward velocity of 1.60 feet per second, occurring two feet directly above the air-bubbler.

The measurements closely confirmed Taylor's prediction that the maximum upward water velocities induced by the rising air-bubbles are proportional to the third root of the volume.
rate of air flow. The constant $k$ in his equation $W_o = k(Vg)^{1/3}$ was calculated from the average of ten different values of volume rate of air flow, each of the ten values being the average of twenty measured repetitions. It was found to be 1.49, giving $W_o$ (the maximum upward water velocity in f.p.s.) = 1.49 $(Vg)^{1/3}$, where $V$ is the volume rate of air flow in c.f.s. (see appendix VI). Bulson, who carried out large-scale experiments with pneumatic breakwaters at Southampton, England, found a maximum horizontal velocity at the water surface, induced in the water by the bubbles, $U_o = 1.46 (gV)^{1/3}$ feet per second, and a maximum upward water-velocity $W_o = 0.79 U_o$. (Bulson had at his disposal a large graving dock and a water depth of 34 feet.)

Of particular interest in this project were the maximum, upward water velocities very close to the river bed, induced by the rising bubbles. Detailed velocity measurements were made in still water at levels 0.2, 0.4, 0.6, 0.8 and 1.0 feet above the bottom of the flume, directly above the bubbler. Figures 13 to 15 represent the distributions of the upward water velocities at 0.2 feet depth intervals between 0.2 and 1.0 feet, directly above the bubbler. It should be recalled that in all of these tests the bubbler hose was perpendicular to the longitudinal axis of the flume (the x-axis in figures 12 to 15).

Each of the three figures 13 to 15 refers to one particular orifice diameter and spacing (both orifice diameter and spacing were changed for each test bubbler to keep the volume rate of air flow approximately the same for a given air pressure in the bubbler).
Conclusions drawn from observations with the meriam drops were confirmed by the miniflow current meter: there was hardly any upward velocity of the water between the orifices when spaced six inches apart. Even at a vertical distance of one foot above the bubbler, the upward velocity was very small. Figures 12 to 15 all show a one-foot length of bubbler hose, but with orifice spacings reduced by factors two and four. The upward velocities of the water between the orifices are clearly greater with smaller orifice spacings, above the 0.4 foot level. Above a level of 0.5 feet, measured above the bubbler hose, there is hardly any variation in the upward water velocities along the Y-axis, for a bubbler with an orifice spacing of one and one-half inches. It is noteworthy that the upward water velocity above the 1/16-inch diameter orifices, increases much more rapidly with height than that above the smaller diameter orifices. This is probably related in some way to the size of the bubbles and the quantity rate of air flow per orifice. The above remark does not contradict Taylor's analogy, which relates the quantity rate of air flow per foot of bubbler to the maximum upward velocity of the water. The maximum upward water velocity was never attained below a level of two feet above the test bubbler, in any of the tests performed in the laboratory in connection with this project.

The results of the laboratory experiments with varying depths of water were inconclusive. Altering the water depth in the flume from two to five feet, was apparently too narrow a
range to obtain a reliable indication of the effect of depth upon upward velocities at a given point, other variables remaining constant. There seemed to be a slight increase in upward velocities with depth at points outside the bubbler screen, although there was no such trend in the bubble screen itself. Since the results were inconsistent and inconclusive, they were rejected.

To test the effect of the sides of the flume on the observed water velocities, the upward velocities at a distance of one inch from a side of the flume were compared at regular intervals with those normally observed in the centre of the flume, all other conditions remaining the same. The velocities near a side of the flume were found to be 10% to 20% lower than those at the centre.
SECTION IV
FIELD INVESTIGATIONS

IV-1 Determination of Hydraulic Slopes as a Criterion for Incipient Motion of Bed Material

Inspection of Federal Public Works sounding records showed a tendency for the river bottom to form bedwaves and dunes in front of the Fraser-Surrey wharf during a freshet. This tendency was also clearly observed and recorded in the field in 1955 by the staff of the Fraser River Model Project. It was also observed in the Fraser River Model tests. It was therefore assumed that bed movement contributed to the shoaling during freshet periods.

To ascertain that the hydraulic conditions in the approaches to the wharf supported this assumption, the critical tractive shear stress necessary for impending movement of the various fractions of Fraser River sand found on the river bed in front of the wharf, was investigated in the laboratory (III-2). From these laboratory results, the minimum slope of the water surface which would satisfy the hydraulic conditions for impending movement of bed material near the wharf was calculated. It was realized that this method had some debatable features: regardless of how accurately the slope of the water surface could be measured between two points in the river near the wharf, it was quite unlikely that the slope at all intermediate points would be exactly the same. However, the distance over which the slope would be measured was relatively small (the length of the wharf). Consequently, the difference between the upstream and downstream
water-surface elevations at the wharf would be small and probably very sensitive to fluctuations in water levels caused by passing ships, wind and tide. Finally, the computation of the critical tractive shear stress from the observed water-surface slope would be a debatable approximation. In calculating the critical tractive shear stress \( (\tau_c) \), the boundaries of the body of water responsible for this shear stress are determined by the hydraulic slope, depth and the hydraulic radius. In wide and relatively shallow alluvial rivers, the hydraulic radius is often closely approximated by the river mean depth. Since the analysis was only concerned with the hydraulic conditions in the immediate vicinity of the wharf; replacing the hydraulic radius by the average local depth and arbitrarily assuming boundaries within which only the water-surface slope and average depth were known, was an approximation which could lead to discrepancies. However, to gain at least some indication of the behaviour of the river bed near the wharf, it was decided to measure the difference in water-surface elevations between a point near the upstream end of the wharf and one near the downstream end, and assume a uniform, straight-line water-surface profile between these two points, which were 1030 feet apart.

The method for measuring water-surface slopes in the laboratory flume (III-2) could not be employed in the field because the wharf did not provide the necessary rigid base on which to mount the small brass cantilevers. Furthermore, the lengths of the wires for similar instrumentation at the wharf
would lead to inaccuracies and the tidal oscillations varying from four to six feet would have to be eliminated. An entirely different electronic and mechanical device was developed in the Civil Engineering Instrument shop by Messrs. H.W. Schmitt, electronics technician and E.M. White, machinist. Since the instrumentation employed in the laboratory flume test and in the field is fully described in an unpublished manuscript prepared jointly by Mr. Schmitt and Professor E.S. Pretious, a short description of the field method and its principles may suffice here.

A plexiglass float, eight inches O.D. and 4 inches long, was contained in the upstream and downstream stilling wells. Each float was suspended by a very fine wire, guided over a two and one-half inch diameter plexiglass drum and partially counter balanced by a small weight. The drum was mounted on a small wooden platform above each stilling well and attached to the shaft of a potentiometer, which converted the vertical movement of the float into a small D.C. voltage. Any difference in the water levels between the upstream and downstream stilling wells was transferred to the two sliders A and B (see figure 17) of the two potentiometers, resulting in a potential difference, which was measured by a voltmeter and recorded continuously on a graph. The bottom of each stilling well was closed, except for a small hole (1/4 inch diameter) lined with a short length of pipe, in the exact centre, which effectively damped transient surges caused by
passing ships, waves, etc.

The design of the circuit eliminated the effect of tidal oscillations on the voltmeter readings, although a small correction would have to be applied to the readings in order to correct for the time travelled by the tide wave between the two ends of the wharf. Assuming no river flow and the tidal disturbance reaching the two stilling wells at exactly the same instant, both floats would rise or fall an equal amount during a small time interval. Sliders A and B on the potentiometers would then move an equal amount and (see figure 17), the potential differences between A and P, Q and A, would remain equal to those between B and R, S and B respectively, resulting in zero voltmeter reading.

With a tide wave having a velocity of propagation of about 28 feet per second in 25 feet of water \( V = \sqrt{gd} \) for a shallow water wave, a tidal disturbance would not reach the two stilling wells at exactly the same instant. A rising tide with a local range of three feet in five hours would, at a given instant, cause a difference in elevation between the water levels in the upstream and downstream stilling wells of \( \frac{1030}{28} \times \frac{3 \times 12}{5 \times 3600} = 0.07 \) inch. Since the difference between the upstream and downstream water-surface elevations, caused by the river flow alone, was expected to be anywhere between 0.1 and 1.0 inches, there would be a measurable contribution to the slope, by the tide wave.
The calibration of the equipment was first carried out in the Hydraulics Laboratory of the University of British Columbia by setting up the measuring instruments and placing the two floats in buckets filled with water. One potentiometer slider was set at its centre and the slider on the other potentiometer moved to a position giving a zero reading on the voltmeter. One of the buckets was then raised 0.125 inches (by aluminium strips), resulting in a graph recording, which depended on the voltmeter reading. This procedure was repeated several times, then reversed and finally extended to larger differentials in height to check the linearity of the voltmeter readings.

One of the major difficulties encountered in the design of the field instrumentation was the selection of the stilling well locations, particularly the upstream one. The most suitable location for the upstream well would have been the outer corner of the wharf; however, this corner was subject to considerable vibration caused by the large air compressor supplying the air-bubblers. The corner was also exposed to wind, waves and floating debris and the instruments might be damaged by mooring lines and the movements of ships. To avoid these disturbing influences, the stilling well was built at the blunt upstream end of the main wharf near the point where the catwalk commences (see figure 16). This location proved to be well protected, although the piling and fenderboom tended to retard the river velocity somewhat, possibly resulting in a slightly higher water level locally. The other stilling well was built close to the downstream corner of the wharf.
Both stilling wells were fastened securely to the piling with heavy steel brackets and timbers.

An initial attempt to measure the water-surface slopes was made during the last week of June 1966. A zero reading on the voltmeter was established at local high water, when the incoming tide and river flow produced slack-water conditions at the wharf. Unfortunately, the wharf at this time was occupied by ships. Considerable activity with heavy equipment near the upstream part of the wharf disturbed the very sensitive instrumentation so much that the slope measurements had to be deferred to a more favourable period. Also there was evidence that continuous heavy rain during this period had swelled the timbers supporting the platforms, introducing an error in the common reference datum.

On July 29, 1966, the instruments were again set up and this time the slope of the water-surface was measured successfully during a full tidal cycle under ideal conditions. The maximum difference in water-surface elevation between the two stilling wells was recorded at 0.69 inches, yielding a slope of $0.56 \times 10^{-4}$. The tidal contribution to this difference in elevation was calculated to be only 0.02 inches (vid. APP.VII). The maximum water-surface slope on that day occurred about two hours before local lower low water at New Westminster and virtually coincided with a maximum surface velocity in the river, 50 feet off the wharf, of six feet per second in the seaward direction.

For bed-sand particles of 0.25 mm diameter, the water-surface slope required for impending bed-load movement would be in
the order of $0.1 \times 10^{-4}$, corresponding to a difference in elevation of 0.13 inches between the water levels in the upstream and downstream stilling wells. The graphs of July 29 and July 30 indicate a difference in elevation greater than 0.13 inches for a period equal to 2/3 of a complete tidal cycle. Since the observations were carried out near the end of the freshet (Fraser River Discharge at Hope, B.C. was 170,000 cfs on July 30), we could postulate that the water-surface slope necessary for impending bed-load movement exists for at least two-thirds of the time during the freshet and probably longer at higher river flows. Field investigations and model tests carried out by the Fraser River Model Project indicated that active bed-load movement accompanied by bed waves and dunes should be expected in this area when the Fraser River Discharge at Hope exceeds 225,000 cfs.

**REMARKS.** Although this method of measuring the slope of the water surface has merit in regard to accuracy and sensitivity, the field tests brought to light certain deficiencies which should be corrected before future measurements are made.

For establishing a reference datum for the recorder, a zero slope has to be accurately established between the two measuring points under consideration (vid. the water levels in the two stilling wells). This is possible if the river flow can momentarily be halted by a high flood tide, assuming no wind effects. Theoretically, it should be possible to determine the difference in water-surface elevation between these two points at any instant by optical precise levelling, comparing the result with the.
simultaneous reading of the recorder (voltmeter) and thus obtain a reference datum. However, no matter how well the surges in the stilling wells are damped, there is always an oscillation which makes an instantaneous comparison by precise levelling a most frustrating, if not impossible task. Before employing this instrumentation on a river where there is no reversal of flows (no tidal effect), one would have to introduce some refinements and modifications in the present design.

As for the locations of the stilling wells, it would be advisable to build them away from the wharf, on a more solid foundation and connect them to submerged pipes ending somewhere near the wharf face. By moving these pipes at will, one could measure the water-surface slope between any two points in the approaches to the wharf, without having to move the stilling wells. The stilling wells in the present study were rigidly attached to the piling of the wharf. They were each ten feet long, well beyond the local range of tide and river stages which could occur during the freshet and summer. However, the observations were made near the end of the freshet, when the water-surface at low tide approached the bottom of the stilling well. At this point, surface waves had a disturbing effect on the readings (vid. Appendix VIII). The stilling wells should therefore be made manually adjustable, i.e. able to move vertically up or down so that they can be adjusted periodically to keep the bottom always submerged sufficiently to eliminate oscillations due to surface waves.
Finally, the question might arise whether this rather sensitive method of measuring very small water-surface differences in elevation over a short distance could not have been avoided by comparing the records of existing automatic tide gauges located upstream and downstream from the wharf. Unfortunately, the nearest installed automatic recording tide gauges were at the Rice Mills' wharf, 7 miles downstream and at the Federal Public Works Wharf in New Westminster, 1 mile upstream (see figure 1). Over such a large distance, the curved water-surface profiles, due to the unsteady flow caused by the tides cannot be ignored and it would be quite wrong to use the difference in water-surface elevations between these two gauge stations as a base for calculating the differences in water-surface elevations between intermediate points by linear proportion. It was therefore necessary to install the instruments in locations close enough apart to be able to ignore the curvature of the water-surface profile and at the same time far enough apart to provide measurable differences in water-surface elevations.

IV-2 Survey of River Currents in the Vicinity of the Fraser-Surrey Wharf

As was mentioned in the Introduction, the location of the Fraser-Surrey Wharf on the convex side (Plate I) of a bend in Annieville Channel makes its approaches naturally vulnerable to shoaling. However, not only the location of the wharf but also the alignment of the wharf face relative to the main river current.
inevitably contributes to shoaling. Earlier float studies of flow patterns in Annieville Channel, in the prototype and later on the Fraser River Model at U.B.C., demonstrated quite clearly that the wharf face diverges in the downstream direction from the river current, tending to create a dead-water area and back-eddy along the downstream face of the wharf. Moreover, the blunt upstream end of the wharf forces the flow away from the wharf, further aggravating the foregoing conditions. Freshet peak flows suppress this dead-water area considerably, but even so, there is still a noticeable retardation in current velocity in the area bounded by the wharf face and a line from the upstream end of the wharf to a point about 100 feet off the downstream end.

To map the flow pattern in this area more accurately, the surface currents were measured a number of times, generally just before and after local low water at New Westminster. The wharf had to be free of ships, of course, a necessity which limited the number of observations that could be taken.

The flow pattern was obtained as follows: The position (F) of a passing float in the river could be fixed from the wharf by its bearing relative to the wharf face (an angle of 90° for all measurements); its vertical angle and the height of eye (h) above the water level, giving the distance: \( d = h \tan \text{BOF} \). The wharf face was marked off at intervals of 100 feet and the vertical distance between eye and water level measured by tape. Small wooden floats, painted orange, to be easily distinguishable
from floating debris, were thrown into the river at various distances from the wharf face and from various points along the wharf. They were subsequently carried downstream by the current. Walking along the wharf, ahead of the float, the observer would wait for the float to pass a 100 or 200 foot mark, etc., take the vertical angle by sextant, and record the time the float travelled between marks. To define the instant of passing, the float was lined up with some object on the opposite river bank, which had previously been identified by turning off a horizontal angle of 90° at each point of observation. Average surface velocities over distances of 100 or 200 feet, as well as positions, could thus be mapped for a number of floats.

This simple, one-man method could not be carried out too far from the wharf, because the tangent of an angle close to 90° changes rapidly and a slight inaccuracy in the sextant vertical angle could therefore lead to a considerable error in distance \( d \). At large distances, only a theodolite would give accurate results. However, the area of interest was well within the limits of accuracy
of a sextant and after some practice, a sufficient number of observations could be obtained within a reasonable time to obtain a good flow pattern. A time limit of one-half hour for a complete set of observations minimized changes in current velocities due to tidal effects.

The results (figures 18 and 19 are two sample sketches of the float survey) show a considerable retardation of surface velocity in a triangular area enclosed by the wharf face and a line connecting the upstream corner of the wharf with a point about 80 feet out from the 2 + 00; mark. A dead-water area near the wharf extends approximately from 8 + 00 to 4 +00; downstream form 4 + 00, the flow near the wharf appears to slowly accelerate again.

Judging by the surface flow patterns the most adverse flow conditions near the wharf exist in the area downstream of 8 = 00. It is in this area that shoal deposits more rapidly than elsewhere in the approaches to the wharf.

In addition to the float survey, measurements of the sub-surface currents were taken during the 1966 freshet. The Water Resources Branch of the Department of Mines and Technical Surveys provided a current meter and operators at regular intervals. The current meter (Price, Gurley 622 A) measured the river velocities at 1/10 depth intervals, from the surface down to one foot above the river bed. The velocity traverses were taken at four different locations (see figure 16) near the wharf, at river discharges (metered at Hope B.C.) varying from a freshet maximum of 281,000 cfs...
(June 15) to a low of 150,000 cfs (July 5) and at different stages of the tide.

A velocity profile at the buoy (vid. figure 20) on June 15 (peak discharge at Hope, see hydrograph, figure 25), 120 feet off the upstream end of the wharf, shows a maximum velocity of 7.5 feet per second at a depth of 0.2 of the total depth, which is in close agreement with the normal, vertical, velocity profiles of river flow. The velocity traverse for intervals of one-foot depth was taken one-half hour after Lower Low Water at New Westminster on that particular date, and was well outside of the influence of the wharf. Under these conditions, it can reasonably be assumed that, in the approaches to the Fraser-Surrey Wharf, the river velocity, at the surface or just below the surface, rarely, if ever, exceeded 7.5 feet per second in 1966.

The conventional current meters employed did not measure directions of flow. This was rather unfortunate because no conclusive proof that there was no reversal of flow in the sub-surface water layers during local high tide could be obtained. The surface flow at no time reversed during the peak of the freshet period. However, if there was a sub-surface reversal of flow, we might expect a zero velocity, or at least a velocity close to zero, somewhere in the vertical, velocity profiles. Close inspection of the graphs does not reveal such a reversal at any point, not even in the last velocity traverses taken on the 5th of July 1966, when the freshet was nearly over and when the surface currents near the wharf at local high tide were one foot per second in the seaward
direction.

From these observations, it seems reasonable to assume that there was no reversal of flow in the sub-surface layers during the 1966 freshet period, which extended approximately from the beginning of May to the end of July (the duration of the freshet period will be examined more closely in section IV, paragraph 3). Even if a reversal of flow occurred during a very high flood tide, it would probably be of such short duration that the general direction of sediment transport would hardly have been affected. Furthermore, flow reversals when they do occur, take place in the surface layers, particularly upstream of the limit of intrusion of the salt-water wedge.

Figures 21 to 24 represent a group of velocity traverses taken in one day. (June 29, 1966) at four different locations along the wharf face. Profiles obtained at the same locations on other days during the freshet have similar characteristics: they all show vertical, velocity distributions near the wharf which are much more irregular than those at the buoy, 120 feet off the upstream end of the wharf. This is particularly noticeable at the mid-point of the wharf. Considering all observations collectively, the average flow velocity at mid-depth along the wharf might roughly be estimated to be about one half of the velocity at mid-depth near the buoy. However, definite conclusions should not be drawn from the limited amount of data available. The measurements could only be taken when the wharf was free of ships and this made planning and organizing difficult; especially since the crowded harbour facilities
at New Westminster forced ships to shift berths without much advance notice. It was rather a frustrating but not uncommon experience to watch a ship approach the wharf a few hours after arrangements had been made with the Water Resources Branch for a current meter survey.

IV-3 A Study of the Sedimentation at the Approaches to the Fraser-Surrey Wharf

To properly introduce a study of the sedimentation occurring in front of the wharf, a brief review of the terminology employed in the subject would, perhaps, be in order.

Sediment is transported by a river as **suspended sediment**, if it is in suspension in the turbulently moving water, or as **bed load** if it rolls and slides along the bed. In addition, there is **wash load**, consisting of very fine particles of silt and clay which travel with about the same velocity as the river, and the **dissolved solids** (salts and chemicals). If some of the bed load bounces along the bed, it is called **saltation load**. When the sediment settles down and becomes, at least temporarily, part of the river bed, it is called **bed material**. Wash load, although normally comprising the largest part of the total sediment load transported, does not settle readily as long as there is appreciable flow and therefore, hardly affects river sedimentation. On the other hand, it could affect very much the siltation of a reservoir.

Shoaling features in a tidal river reflect the sediment characteristics and hydraulic behaviour. The variables to be
considered in a study of shoaling which occurs in front of the Fraser-Surrey Wharf are: suspended sediment; bed load; river discharge; tide; river geometry. Relatively few detailed field investigations of the sedimentation in this part of the river had been carried out in the past and we again had to reply heavily on the cooperation of the Water Resources Branch of the Department of Mines and Technical Surveys (the present Department of Energy, Mines and Resources), who made available their sedimentation personnel, field equipment and laboratory. As with the survey of the local river currents, the sediment sampling program was hampered by ships and barges berthed at the wharf. Therefore the only way to arrive at reasonable conclusions concerning the sedimentation trends at the wharf, was to compare the scattered sediment data gathered there with data obtained regularly at Port Mann, B.C. The Water Resources Branch maintain their nearest sediment field station at Port Mann, which is about three miles upstream from the wharf and about two miles upstream of New Westminster. Unfortunately the suspended-sediment sampler (P61, weight 80 lbs) had to be transferred from Port Mann to the Fraser-Surrey Wharf, which meant that no similar measurements at Port Mann were available for comparison on that day. However, by interpolation between data taken on the previous and following days at Port Mann, a fairly reliable relationship between sediment conditions at the wharf and Port Mann could be established.
At Port Mann, the suspended sediment concentration* is measured daily by depth integrating and point integrating samplers. The samples are analysed in the Water Resources Sedimentation Laboratory at New Westminster, resulting in daily total concentrations, corrected for the amounts of dissolved solids. The total concentration in grams per liter for the Fraser River at Port Mann varies from 0.01 in the winter to about 0.5 during the freshet, at times going as high as 0.9 during a rapid rise of the freshet, which normally occurs in the middle of May (vid. hydrograph, Fig. 25). The maximum seasonal concentrations at Port Mann virtually coincide with the maxima in the river hydrographs for Hope, B.C. A maximum daily total concentration of 0.9 grams per liter occurred at Port Mann two days before the Hope discharge of 261,000 cfs on May 13, 1966; another maximum daily total concentration of 0.5 was recorded three days after the freshet peak discharge of 281,000 cfs on June 15, at Hope. Both of these daily maxima were obtained from observations taken at Local Lower Low Tide during the freshet season.

The tides have a marked effect on the sediment concentrations because the river flow, which transports the sediment, has a reduced

* "Concentration", i.e. the weight of the sediment found in a unit weight of the water-sediment mixture, is a misnomer. It does not recognise the obvious fact that water moves faster than suspended sediment (except wash load) and that therefore a sampler measures concentrations which are too low. A more appropriate term would be sediment "charge", the ratio of the weight of the sediment to the weight of water per unit time. However, to avoid confusion the term "concentration" will be maintained. "Total" concentration implies the sum of suspended load, wash load and dissolved solids, as an average over a vertical.
velocity at high tide in the New Westminster area and an increased velocity at local low tide. Therefore, in comparing sediment concentrations based on river discharge in this area from day to day, the phase of the tide at which these concentrations were measured should be specified. Even then, such a comparison can only be approximate, since the tide curve keeps varying from day to day, with different maximum and minimum tide heights. Moreover, the daily discharges of the Fraser River at New Westminster depend mainly on the upland fresh-water discharges (metered at Hope), modified by the tidal effects superimposed on them. However, heavy rains in the water sheds downstream of Hope, (e.g. Harrison Lake, Pitt Lake, etc., see Fig. 1), can also markedly affect the tributary inflow. The sediment concentration is an extremely complex function of these varying tides and river discharges and it may take several years of data collecting in the New Westminster area before this function can be derived and before it will be possible to predict at which river discharges and tide phases shoaling will commence and stop. This knowledge would be of great practical value to harbour authorities because it would enable them to operate their anti-shoaling devices (such as air-bubblers, or perhaps underwater jets) in an economically feasible manner.

To illustrate the dependence of the river gauge heights at New Westminster on the tides in the Strait of Georgia, the New Westminster tide curve between June 22nd and June 25, 1966 was
traced from the automatic gauge record and the hourly heights of tide at Point Atkinson (entrance to Vancouver Harbour, see Fig. 1) plotted on the same graph (Fig. 26).

When examining the observed tide curve at New Westminster, B.C. for the freshet of 1966, one observes that the concentration of suspended sediment at Port Mann at local low water is, roughly speaking, twice as large as that at local high water. A good example of the foregoing occurred on June 19th, 1966, when the suspended-sediment concentration was 0.176 at higher high water, and 0.349 at lower low water. On March 28th, 1966, the concentrations were 0.013 and 0.026 respectively. There are exceptions to the above trend, depending on the river conditions upstream, (viz. slides, a collapsing river bank, sudden rainfall over a tributary water shed, etc.).

The suspended-sediment concentration at Port Mann, averaging about 0.02 grams per liter during the winter, increased markedly at the beginning of May 1966; an increase which coincided with a sharp rise in the river-discharge at Hope. In the first week of that month, the concentration during local low tides averaged about 0.1 grams per liter, rising to 0.9 grams per liter on May 11th. The river discharge at Hope rose above 150,000 cfs on May 8th and from then on the records at Port Mann show a concentration during local low tides remaining well above 0.1 gram per liter until the end of July when the river discharge at Hope decreased rapidly to slightly above 150,000 cfs.

It has not been established that this discharge of 150,000 cfs
at Hope indicates a definite transition between normal and above-normal suspended-sediment concentrations at Port Mann. The Water Resources Branch has only been collecting sediment data in the New Westminster area (Port Mann) since the early summer of 1965 and it would take a number of years of record to prove this figure right or wrong. However, the available data so far, indicate a marked tendency for the river to increase its suspended-sediment carrying capacity in the New Westminster area when the river discharge at Hope is above 150,000 cfs. Therefore, in studying causes of sedimentation, it seems reasonable to use the discharge of 150,000 cfs at Hope as a reference for the duration of freshets in the lower Fraser River, until further information is available.

Figures 27 to 30 are typical particle-size distribution curves of the suspended sediment in the Fraser River at Port Mann, B.C. They represent the particle-size distributions at 5, 15, 25 and 35 feet depths on June 30th, 1966, taken at local higher high water. The curves show an increase in the median diameter of the suspended sediment from 0.0092 mm at five feet depth, to 0.012 mm at 35 feet depth, with no particles larger than 1.0 mm at depths less than 15 feet below the surface.

On that same date, during local lower low water, a total concentration of 0.312 grams per liter was measured at the Fraser-Surrey Wharf. It was not possible to compare this value with the concentration at Port Mann because the sampler was the only one
available in the area at that time, thereby preventing simultaneous measurements at the two places. However, on the previous day at Port Mann, the concentration at exactly the same local tide height was 0.351; the concentration at Port Mann on the following day (July 1) was not measured directly at the same tide height, but after comparison with other measurements on that day and on following days, could quite safely be assumed to lie between 0.30 and 0.35. Other comparisons indicated a further close correspondence between sediment concentrations at Port Mann and the Fraser-Surrey Wharf. It should be remembered that the entire flow of the Fraser River passes Port Mann, whereas in passing the Fraser-Surrey Wharf it has been diminished by the off-takes into the North Arm and Annacis Channel which amount to a total reduction of about 20% in the flow entering the trifurcation.

The point sampler at our disposal did not collect sufficient amounts of suspended sediment to plot a particle-size distribution curve. The station at Port Mann periodically obtains such distributions by the pumping method (i.e. collecting samples at ten-second intervals from river water, which is pumped up continuously by a pump on a catamaran while she crosses the river). However, it seems unlikely that the suspended sediment at Port Mann would change its particle-size distribution as it travels downstream without reflecting a change in the total concentration. Since field observations did not reveal any appreciable change in total concentration of the suspended sediment between the Port Mann and the Fraser-Surrey Wharf, this can be taken as evidence
that the distribution curves of the suspended sediment at these
two places are similar. This conclusion may seem bold, lacking
sufficient statistical evidence. The margin of error, however,
may be negligible compared to the daily changes in particle-size
distribution curves at one particular location. The curves
shown in figures 27 to 30 are distributions obtained at local
higher high water. The Water Resources Branch files in New
Westminster contain a similar set of curves for the same date,
obtained at local lower low water. At each depth, the curve
representing particle size distribution at low water shows a
marked shift to the right, compared to the one at high water.
This shift to the right is illustrated by a comparison of the
median diameters:

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Median Diameter (mm)</th>
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<tr>
<td></td>
<td>Local Higher High Water</td>
</tr>
<tr>
<td>35</td>
<td>0.012</td>
</tr>
<tr>
<td>25</td>
<td>0.012</td>
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<tr>
<td>15</td>
<td>0.010</td>
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<tr>
<td>5</td>
<td>0.009</td>
</tr>
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Within less than six hours, as the tides changed from low to
high, the median diameters decreased by factors varying from two
to eight! This striking daily fluctuation in particle-size
distribution at Port Mann, which most likely will follow a similar
variation near the wharf, makes it practically impossible to
determine or predict exactly the amount of suspended sediment
involved in the shoaling at the wharf. There can hardly be any
doubt that suspended sediment contributes largely to this
shoaling. The concentration of suspended sediment during the
freshet is at least ten times, frequently twenty to thirty times,
greater than it is at normal flows. Furthermore, the river
decelerates considerably in the immediate vicinity of the wharf
and simply cannot support this heavy sediment load in suspension.

While measuring the concentrations of the suspended-sediment
samples collected at the wharf, the concentrations of dissolved
solids were also obtained (by evaporation and weighing). In
this connection, values were found, ranging between 53 and 74
milligrams per liter. The salinity of the Strait of Georgia,
around the mouth of the Fraser River, is about 25.7 (eight-years
average for June at East Point). The samples were taken as low
as one-half foot above the river bed and at relatively low river
discharges (about 200,000 cfs at Hope), when the incoming flood
tide would have been able to penetrate the river further upstream
than it would at much higher discharges. However, the laboratory
analysis of the dissolved solids established beyond reasonable
doubt that there is no salt water intrusion in the river area off
the Fraser-Surrey Wharf during a freshet, a fact which had already
been indicated by the sub-surface current measurements (IV-2).

In contrast with suspended sediment, the particle-size dis-
tributions of bed load and bed material appear to be much less sensitive to tidal effects. Figures 31 and 32 are distribution curves of bed material and bed load at Port Mann, compiled from the Water Resources Branch records for arbitrarily chosen dates, June 24th and 25th, 1966. The curves show an over-all increase in particle sizes for both bed load and bed material with decreasing heights of tide (see also the tide curves for New Westminster and Point Atkinson for these two particular dates, figure 26). The median diameter of bed load in mid-stream increases from 0.37 to 0.43 mm; near the shore, from 0.20 to 0.22 mm as the local tide height drops. The particles are larger in mid-stream (where it is deeper), a common phenomenon. As for bed material, its median diameter increases from 0.39 to 0.41 mm in mid-stream and from 0.21 to 0.29 mm near the shore. Bed material on the average is slightly coarser than bed load, which is to be expected. It is difficult to distinguish sharply between bed load and bed material; bed material becomes bed load as soon as the critical tractive shear stress (III-2) has been exceeded. For a given hydraulic slope, bed material would always be slightly coarser than bed load. Therefore, to examine the composition of sediment which an air-bubbler would have to prevent from settling, a study of the bed material found at the Fraser-Surrey Wharf would eliminate the need for any further information regarding bed load.

At the Fraser-Surrey Wharf, bed-material samples gathered at various points throughout the freshet season in 1966 (as well as
a few samples taken in 1965, show an almost constant median
diameter of 0.14 mm, and a great similarity in the shape of their
distribution curves. This median diameter of 0.14 mm is, roughly
speaking, more than one-half the median diameter of the bed
material sampled near the shore at Port Mann.

As the Water Resources Branch collects more information con­
cerning sedimentation behaviour in the New Westminster area, some
of the assumptions made in this discussion may be refuted. How­
ever, these assumptions, although not supported by strong statist­
ic evidence based on continuous long-term records, are based on
accurate data gathered and analyzed so far by experienced and well
trained observers employing up-to-date equipment.

IV-4 Installation and Operation of the Air­
Bubblers at the Fraser-Surrey Wharf

The design of the air-bubblers and their installation on the
river bed at the Fraser-Surrey Wharf, were based on preliminary
laboratory tests and limited information concerning the hydraulic
conditions in the river in front of the wharf. By the time these
preliminary tests had been completed and arrangements had been made
for the field surveys and financing of the bubbler equipment, the
freshet was impending.

The operation of the prototype bubblers was essential to obtain
a clear understanding of their behaviour and possibilities.
Problems most certainly would arise during the field tests, which
perhaps would never be exposed in the laboratory. The time avail­
able for the field tests, however, was limited by the duration of the freshet, while the laboratory tests could be carried out indefinitely. It was therefore decided to proceed with the design of the prototype bubblers and planning their lay-out on the river bed as quickly as possible, even though future research and field investigations might call for a different approach. Another important consideration was the rapidly increasing river flow during the initial stages of the freshet, which would make it more difficult to place the bubblers in the planned positions.

The experiments performed with drops of Meriam neutral oil in the laboratory flume (III-5) had clearly shown the importance of a very close spacing of the orifices in the bubble hose. During these preliminary experiments, air holes with a diameter of 0.0135 inches, spaced 1/4 inch apart, and an estimated rate of flow of free (atmospheric) air in the order of one cubic foot per minute per foot length of bubbler hose, created a fairly good upward motion in the water near the bed of the flume, although accurate measurements had to be deferred to a later date. This arrangement appeared to be a reasonable basis for the design of the bubblers in the field. Larger orifices would admittedly create larger bubbles with higher upward velocities. However, not much was known about the flow of compressed air through small orifices in a polyethylene pipe. A very conservative orifice diameter of 0.0135 inches (corresponding to the smallest drill locally available) was selected to allow for a certain margin of error in the design of the total length of the bubblers. Once
the bubblers were anchored on the river bed, it would be a simple matter to add another bubble hose if the air compressor was not operating at its full capacity. However, it would be almost impossible to alter orifice diameters and spacings if the total cross-sectional area of the orifices proved to be too large for the capacity of the compressor, because part of the bubbler hose (outer end) would then remain full of water.

The bubblers were to be placed in approximately 30 feet of water at the wharf. To supply one cubic foot of air per foot length of bubble hose at a depth of 30 feet in water, a compressor would have to deliver nearly two cubic feet of free air per foot of bubble hose. It was intended to make the bubble hoses each 120 feet long and place them in a direction perpendicular to the face of the wharf. For each bubble hose, 120 feet long, the compressor would have to deliver about 240 cubic feet of free air. A compressor having a capacity of about 500 cfm could then be expected to supply sufficient air to operate two bubblers, each of the above length.

Two, portable, rotary-screw compressors were rented from the Atlas Copco Organization; one a large compressor (Cummins) delivering 620 cfm of free air at 100 psi and a smaller one (Deutz), delivering 365 cfm of free air at 100 psi. Initially, two polyethylene hoses, one inch in diameter and 150 feet long, were prepared for the large compressor and one similar hose for the small compressor. However, when the large compressor was obviously operating well below its capacity, a third hose of the same design as the other two
was added. All bubbler hoses were prepared in the Civil Engineering Laboratory at the University of B.C., shortly before the freshet. More than 5000 air holes were drilled in each 120-foot length of the perforated part of the hose. The remaining, unperforated length of 30 feet led from the river bed up to the water surface, where it was connected to a rubber hose fifty feet long and two inches in diameter, leading to the compressor on the wharf. The river ward end of the perforated length was closed with an aluminium plug.

To overcome the buoyancy of the air hoses, steel cables, 5/8 inches in diameter, were fastened to the hoses. One-foot length of polyethylene hose of one-inch inside diameter with a density of 0.97 would be subject to a buoyant force of
\[ \frac{\pi}{4} \frac{1}{144} \times 62.4 = 0.34 \text{ lbs.} \] (ignoring the wall of the hose, which had a density almost equal to the water). One foot length of steel cable 5/8 inches in diameter would have a submerged weight of
\[ \frac{\pi}{4} \frac{(5/8)^2}{144} \times (487 - 62.4) = 0.90 \text{ lbs.} \] which was well above the buoyant force on the hose. As a further precaution against the drag forces of the current, both ends and centre of each hose were weighted with short lengths of boom chain, weighing 80 lbs each, which were attached to the cable, two chains at each end and one in the centre.

The bubbler hoses were placed on the river bed during the last week of May, 1966, in the positions indicated on figure 16. At the upstream end of the wharf, three parallel bubblers, 75 feet apart, were connected to the 620 cfm compressor. They were
pointing in a direction perpendicular to the wharf face to produce three, parallel, barriers or screens of rising air-bubbles each 100 feet long. (The perforated portion of each hose was actually made 120 feet long to allow for irregularities in the river bed and bends, etc.). The smaller compressor, when not used as a mobile air-supply unit in attempts to remove localized shoals near the wharf, or as a replacement of the large compressor in case of a break-down, was kept at mark 6 + 00 (about half-way along the wharf) and attached to the fourth bubbler. The rather isolated position of the fourth bubbler was chosen, not so much to prevent shoaling as to observe the effect of a single bubbler on the adjacent river bed. The river bed at the fourth bubbler being flat and sandy, any change in the configuration of the bed here due to the currents induced in the water by the screen of rising air-bubbles, would immediately be apparent on an echo-sounder graph. Similar observations were not possible in the area near the upstream end of the wharf, where the other three bubblers had been placed. The river bottom there was irregular and very uneven due to sunken logs and other debris.

The 100-ton snagboat "Samson", a sternwheeler, was made available by the Department of Public Works, Canada, to place the bubblers on the river bed. The previously mentioned steel cables, 5/8 inches in diameter, were fastened to the bubble hoses with stainless steel clamps at six-foot intervals and with marline at intermediate two-foot intervals (vid. plate 4). Care was necessary to keep the steel cables from blocking the air holes
since there was evidence obtained from field trials in 1965 that
the air jets entraining fine sand, were capable of eroding holes
in the steel cable.

The bubblers were sunk at local high tide, when the river
currents were relatively slack. After the "Samson" had anchored
about 100 feet off the wharf, the end of a bubbler cable was
connected to a steel ring which was hauled across and above the
water by sliding it along a tight, 1/2-inch diameter wire con-
necting the wharf to the "Samson" (plate 4). The end of the
first bubbler, paid out from the upstream end of the wharf, was
fastened to a 1-1/2 ton concrete block. This block was also the
anchor for a large, conical, river-buoy, painted red and marking
the up-stream, river ward boundary corner of the air-bubbler lay-
out (see plate 5). The river ward ends of the three other
bubblers were each weighted with two boom chains already described,
after being pulled on board the "Samson". They were then dropped
in the river approximately fifty feet upstream from their planned
positions, to allow for the set of the current. The middle of
each bubbler was anchored with one boom chain, the land-ward end
with two boom chains as already described.

Bubbling operations commenced on May 24th, 1966, and continued
until the 12th of July. The PR 620 compressor operated for a
total of 868 hours and the PR 365 operated 950 hours.

The installation of the bubblers was executed almost without
a flaw, thanks to the powerful aid of the large, well equipped and
manned "Samson". Altogether different, however, was the operation
of the bubblers themselves. Many unexpected difficulties were encountered which seemed to be absent in the laboratory but which haunted the full-scale field trials. It soon became clear that the Fraser-Surrey Wharf was not as ideally suited for field tests of the bubblers as had originally been anticipated. Although it would have been impossible to estimate beforehand the exact number of ships berthing at the wharf during the 1966 freshet period, relatively few ships were expected to venture near the wharf during this period, in view of the severe shoaling which occurred there during the previous freshets. Nevertheless, it was only one day after the installation of the first bubblers, that a large ship, the "Atlantic Breeze" (see plate 5), berthed at the wharf; its hull partly covering the upstream bubblers. This ship along with other ocean-going vessels or cargo barges occupied the upstream half of the wharf for a total of 27 days out of the 49 that
the bubblers were in operation. A ship with a draught of 26 feet in 30 feet of water would only leave four feet of water under her keel. Sediment which was carried upward along with the flow induced in the water by the air-bubbles, would almost immediately be stopped by the ship's bottom and deflected downwards. Travelling along the hull after being arrested in their ascent, the air-bubbles would entrain some of the surrounding water, but it is unlikely that this induced water flow, after part of its energy had been dissipated in friction, would be capable of carrying much sediment to higher levels in the river-flow alongside a ship.

On the other hand, a ship's hull would create a local constriction in the river-flow, resulting in a higher water velocity between hull and river-bed and a possible removal of bed material from underneath the ship. This bed material might gradually accumulate immediately downstream of the stern of a ship due to back eddies, but part of it would later be agitated by the ship's propeller during her departure and be carried further downstream. A ship, occupying a berth which normally shoals due to low river velocities, appears to help prevent shoaling. For example, in figures 18 and 19, a ship berthed between 8+00 and 4+00 would clearly be effective as a shoal inhibiter by giving to the plan outline of the wharf a configuration which almost coincides with the streamlines of the river. However, the frequent use of the wharf by ships defeated the efficient operation of the bubblers and became a disturbing factor in the field tests.

A further serious setback was the erratic behaviour of the
large air compressor at the upstream end of the wharf. This unit suffered its first breakdown one day after it started to operate and had to be recalled for a three-day overhaul. It went out of order again within an hour after its return to the wharf and went back to the repair shop for a further five-day period. From then on, this compressor had numerous, minor breakdowns and needed frequent attention. Its speed varied from 1500 to 1700 rpm, corresponding to a flow rate between 510 and 575 cfm of free air, based on a calibration table provided by Atlas Copco. This was about 80 cfm below its designated capacity. The smaller compressor performed well, operating slightly below capacity and needing very little attention. Its usual position on the wharf was at mark 6 + 00, as mentioned previously.

Both compressors were recalled for overhauls on July 12th, 1966 and were replaced by two small compressors, a Deutz Diesel of 315 cfm and a Ford Diesel of 160 cfm. These units were used partly for the bubblers at the upstream end of the wharf and partly in attempts to remove a localized shoal hump at mark 8 + 00. Unfortunately, their performance was unsatisfactory. The larger compressor went up in flames in the early morning of July 24th and the other one, after frequent minor failures, suffered a major breakdown on July 28th. The contributions made by these last two compressors to the overall effort may be virtually disregarded; the last day of effective bubbling operation thus being July 12th, 1966. The buoy together with some fragments of the bubblers were picked up by the "Samson" on July 29th, 1966.
One prerequisite for the successful outcome of the entire project was the river velocity at the wharf downstream from the bubblers.

Sediment raised by the air-bubbles to a higher level in the river-flow had to be carried downstream past the wharf by a sufficiently strong river current. Except during the peak of the freshet, the mean river velocities in an area downstream from 8 + 00, and extending about 100 feet out from the wharf face, rarely appeared to exceed two feet per second. Appendix IX is a reproduction of a chart for determining the largest grain sizes to reach a deposit area, appearing in "The Hopper Dredge" (U.S. Army Engineers) as fig. 188. This chart is of significance to agitation dredging and was quite useful in the design of the bubblers (which in essence are a form of agitation dredging).

Reference to the chart and a short calculation showed that particles larger than 0.16 mm in diameter would not clear the wharf when carried downstream from a point near the river surface at mark 8 + 30, by a current with a mean velocity of two feet per second and assuming the depth at the point of deposit to be 30 feet. This means (vid. fig. 33), that at best, not more than 40% of the sediment agitated by the air-bubbles and carried all the way up to the water surface, would be transported past the wharf. The foregoing percentage is based on theory and may actually be much lower. At any rate, the absence of a sufficiently strong horizontal flow velocity at the downstream portion of the wharf, was without a doubt one of the most adverse con-
ditions surrounding the project, from the viewpoint of alleviating shoal.

Some practical observations and features of the bubblers may be worth mentioning here.

There was no evidence of clogging of the air holes throughout the field operation, nor of any gradual filling of the hoses with fine sand. The compressors were stopped frequently (due to breakdowns and also to permit echosounding), allowing the river water to fill the hoses. It was suspected that sand would be left behind in the hoses after the air had forced the water out again. However, there was no trace of sand in any of the hose sections (including one end section) which were salvaged at the end of the operation.

Accumulation of sand (shoal) on top of the bubblers did not affect their performance in the field. This had already been clearly demonstrated in the laboratory flume when, with the air supply turned off, two feet of sand were dumped on top of a bubbler under water. When the air was turned on again, the bubbles forced their way upward through the sand in a matter of seconds.

The air holes drilled in the polyethylene hose did not retain their original diameter but had become smaller after a few hours, apparently due to creep in the polyethylene. Unfortunately, this shrinkage was not consistent and might well be affected by the temperature at which the holes were drilled. The holes might shrink even more under water. This factor of uncertainty could
introduce appreciable errors in the design of bubblers, particularly where thousands of air holes are involved.

The polyethylene hoses should be kept well away from the compressors. The compressed air leaving the compressors is quite hot and can eventually melt the polyethylene material. The best way to avoid a rupture was to keep the polyethylene hoses under water and join them to the compressors by the heavy rubber hoses that came with the compressors.

The precaution of anchoring a bubbler to the 1-1/2 ton concrete block proved unnecessary. Those bubblers which were each anchored to two 80-pound lengths of boom chain also remained in position throughout the operation.

The polyethylene bubblers should be handled very carefully when placed on the river-bed. The main advantage of using polyethylene is that it is cheap and readily available; however, the hoses kink and break easily. It might be advisable to test a different material in future experiments of this nature.

In deciding the length of bubbler hose to use, it was assumed that the air delivered by the compressor to the bubblers was under sufficient pressure at the furthermost orifice to overcome the hydrostatic pressure there. This requirement enlisted the results of Stehr's elaborate investigations of the pressure drop in the bubbler hoses of exactly the same material (polyethylene) but of a slightly smaller diameter\(^4\). Stehr derived an equation (pages 307 to 312 of his paper\(^4\)) for the pressure drop in an unperforated polyethylene hose:
\[ P_A = P_0 \sqrt{1 - 0.0684 \frac{LZ \cdot W^2}{dZ \cdot T_0}} \]

where \( P_A \) and \( P_0 \) are the air pressures at a downstream point (away from the Compressor) and an upstream point in the hose, respectively, in kg/cm²; \( LZ \) is the distance between these two points in meters; \( W_0 \) is the air velocity in meters per second; \( d_Z \) is the diameter of the hose in mm; \( T_0 \) the temperature, in degrees kelvin, of the air in the hose and \( \beta \) a coefficient of friction, which Stehr determined empirically and expressed as a function of the mass flow of air through the hose: viz. \( \beta = 2.48 G^{-0.148} \), where \( G \) is the mass flow in kg/hr. Converted into foot-pounds-seconds units and degrees Fahrenheit and with slight modification, this equation becomes (see appendix X):

\[ P_A = P_0 \sqrt{1 - 1.56 \frac{q_o^2 \cdot LZ \cdot \beta}{FZ^2(5/9 T_o + 255) \cdot d_Z}} \]

where:

- \( P_A \) = pressure of air in psia at the far end of the supply hose (i.e. at the beginning of the bubbler hose);
- \( P_0 \) = pressure of air in psia in the beginning of the supply hose (see following sketch);
- \( q_o \) = flow of air (at pressure \( P_0 \)) through the supply hose in cu.ft./sec.;
- \( LZ \) = length of supply hose (unperforated) in feet;
- \( FZ \) = cross sectional area of supply hose in inch²;
- \( T_o \) = temperature in °F of the air in the supply hose;
- \( d_Z \) = diameter of supply hose in inches.

The friction coefficient becomes \( \beta = 0.82 G^{-0.148} \), where \( G \) = weight...
flow of air in lbs/sec obtained from \( G = (q_o)(\gamma_o) \); where \( \gamma_o \) is the specific weight of the air, \( \gamma_o = \frac{144P_o}{R(460 + t)} \), \( t \) in degrees Fahrenheit; \( R \) the gas constant, 53.3; \( P_o \) in psia.

For a rough, but very conservative check of the pressure drop in the supply hose at the wharf, let us assume a length of unperforated polyethylene supply hose, \( l_2 \), of 100 feet (more than three times the length actually used); the rate of flow of free air (at 14.7 psia) to the supply hose = 200 cfm (assuming that the large compressor is operating at nearly full capacity and that the air is equally distributed over three hoses); and finally a temperature \( T_o \) of air in the supply hose of 50°F (assuming that the temperature of the compressed air inside the hose is about equal to that of the surrounding water; both the polyethylene supply hose and bubbler hose are submerged).

A short calculation (see appendix X), using Stehr's modified equation, will show that the pressure drop in the supply hose will be slightly more than \( \frac{1}{10} \) of the air pressure in the beginning of the supply hose.

The air pressure gauges on both compressors on the wharf generally registered a pressure of approximately 90 psig while
operating the bubblers, corresponding to 104 psia. After passing through the rubber hose into the polyethylene supply hose, the air would be at a pressure $P_0$, less than 104 psia (due to friction losses in the short rubber hose and the connection); the pressure drop in the polyethylene supply hose would then be no more than 11 psi. This low value for the pressure drop is not surprising in view of the very smooth inside wall of these polyethylene hoses.

Stehr also investigated the pressure drop in the bubble hose, i.e., the perforated section. He measured the air pressures at the beginning and the end of the bubble hose (points A and E in the sketch) and at two intermediate points. His tabulated results show that the pressure drop between the beginning and the end of the perforated hose is, for the same rate of air flow, almost exactly one-third of the pressure drop in an unperforated hose of the same dimensions and material. This result is the same as that derived theoretically for incompressible flow in pipes having perforations of constant diameter and uniform spacing throughout, the theory given in some texts on applied hydraulics. At a pressure drop of less than 11 psi in 100 feet of unperforated polyethylene hose, the pressure in 120 feet of perforated polyethylene hose would therefore be approximately 4 psi, giving a total pressure drop in the polyethylene hose (perforated and unperforated) of no more than 15 psi. Additional friction losses would be expected in the rubber hose and at fittings and bends.
However, the inside diameter of the rubber hose was twice that of the polyethylene hoses and since only two fittings were involved and no sharp bends, it may be safely assumed that, under the above conditions, the air pressure in the riverward end of the bubble hoses, after deduction of pressure losses, was well above the hydrostatic pressure of 13 psi. (i.e. 30 feet of water).

IV-5 Sounding

To examine the efficacy of the bubblers in reducing sedimentation in front of the Fraser-Surrey Wharf, the hydrographic launch "Sounder" of the Federal Public Works Department carried out weekly sounding surveys at the immediate approaches to the wharf during the freshet period. The launch "Port Fraser" of the Fraser River Harbour Commission took daily soundings in the same area, both operations being carried out whenever the presence of ships did not prevent them. The sounding lines of the Public Works Department were perpendicular to the wharf, 25 feet apart and extending approximately 120 feet out from the wharf face. The sounding lines of Fraser River Harbour Commission were parallel to the wharf, 20 feet apart and as far out as 100 feet from the wharf face. The "Sounder's" survey results were compiled on field sheets. Two blue prints of these field sheets are included as appendices XI and XII. The two prints are records of the hydrographic surveys made on May 31st, 1966, shortly before the compressors were operating more or less continuously, and on July 19th, 1966, shortly after the air-bubblers had effectively ceased operation.
The fieldsheets illustrate a gradual decrease in water depths over the entire project area during the freshet; however, the downstream part of the area clearly shows a much more pronounced trend to shoal than the upstream part.

Three series of longitudinal profiles, parallel to the wharf (alongside the wharf-face and at distances of 50 and 100 feet out from the wharf-face) show a similar trend (appendix XIII). The necessary data for these profiles were provided by the fieldsheets of the weekly soundings, taken by the "Sounder" during the freshet of 1966.

To compare the river-bed configurations after freshets in previous years, similar longitudinal profiles were plotted for the post-freshet, pre-dredging hydrographic surveys of the approaches to the wharf in the years 1957-1965. They show almost consistently a shoal area at the down-stream part of the wharf and a relatively deep area near the upstream end of the wharf, probably caused by scour as a result of a crowding of the streamlines near this upstream end.

A convincing example of how much the bubblers as an anti-shoaling device depend on a strong horizontal flow, can be found at mark 6 + 00 of the wharf, where the fourth bubbler was placed. Along the line of the bubbler hose, there was no sign of a trench scoured out of the river-bed, or any increase in depth which might have suggested some local deepening due to the bubbler. This particular bubbler, although occasionally covered by a grain ship, had been operating quite strongly and the location had been
specially selected to observe the effect of an individual bubbler on the surrounding sand bed, and also to keep in suspension sediment agitated at the upstream end of the wharf. However, the flow conditions at this part of the wharf were very poor (low velocities and back eddies) and the soundings indicated quite clearly the importance of having a strong horizontal flow, without which the bubblers would fail as shoal-inhibitors.

No definite conclusions were drawn from the soundings taken by the "Port Fraser". Her sounding lines, although run with great diligence and skill, did not follow control survey lines, whereas the parallel sounding lines perpendicular to the wharf run by the "Sounder" were controlled by ranges established on the wharf by survey measurements. Furthermore, distances out from the wharf were accurately measured by stretch-line and sextant angles. However, the sounding graphs from the "Port Fraser's" echosounder were very useful in following the progress of the shoaling and were retained for possible future reference.

IV-6 Shipping at the Fraser-Surrey Wharf During Freshets

On the base of records provided by the B.C. Pilotage Authorities at New Westminster, a statistical study was made of ship's activities at the wharf during freshets from 1957 to 1966. The results are presented as a histogram (fig.34), which shows on the horizontal axis the freshet durations and on the vertical axis the number of days per freshet (in percent) when the wharf was partly
or wholly occupied by ships.

As a reference for the duration of a freshet, the Fraser River discharge at Hope of at least 150,000 cfs had been chosen (see IV-3). Vessels with draughts of less than ten feet were not considered; their draughts generally varied between 15 and 25 feet.

The histogram clearly illustrates that the freshet period of 1966 has been more favourable to shipping at the wharf than any of the previous freshet periods. The wharf was occupied by one or two vessels 41% of the duration of the freshet, a slightly lower percentage than in 1963; however, the freshet in 1966 lasted much longer than in 1963. Of the freshets considered, these two distinguished themselves by the lowest maximum daily discharges at Hope; the peak discharge in 1963 was 272,000 cfs. (June 16); the peak discharge in 1966 was 281,000 cfs. (June 15).

This remarkable agreement not only in peak discharges, but also in the dates that they were registered, would tempt one to infer that low freshets create favourable conditions at the wharf for shipping, perhaps even a decrease in shoaling. Unfortunately, the 1964 freshet with a peak discharge at Hope of 408,000 cfs (June 21), the highest in ten years, showed a decrease in shipping at the wharf of only 7%, compared to 1963. Of course, the number of days that a wharf is occupied by ships is not necessarily the result of its accessibility. There are also economical and political factors.
SECTION V

Discussion of Results

This project covered such widely varied topics in the laboratory and field that it was not always possible to go into each particular aspect as thoroughly as desired, with the time and equipment available. However, in spite of limitations, and set-backs encountered during the research, the information gathered and experience gained should be of value to future research of a similar nature. The most important observations will be briefly discussed in the following summary.

Velocity measurements made with a miniature current meter in the laboratory flume filled with four feet of still water, showed that a screen of rising bubbles emerging from a perforated, one-inch diameter bubble hose, placed on the bed of the flume, was capable of inducing a vertical flow in the surrounding water which was sufficiently strong to lift fine sand from a point about 1.0 inch above the top of the bubble hose to the water surface, provided that:

a) sufficient air was supplied to the bubbler (a minimum of 0.0025 pounds of air per second per foot of bubbler hose);

b) the spacing between the orifices was sufficiently small (a spacing of 1/4 inch appeared to be the most satisfactory one);

c) the diameter of the air holes was small enough to enable the compressor to provide all air holes with a strong flow of air without exceeding its capacity.

The term "fine sand" applies to a grain size smaller than
0.2 mm, according to the U.S. Bureau of Standard Sieve Series.

It was shown analytically (II-2) that the potential energy of the bubbles, which is converted into kinetic energy of the surrounding water, increases with the water depth above the bubbler and with the volume of air supplied to the bubbler.

A sieve analysis of the bed material found in front of the Fraser-Surrey Wharf indicated that 90% of this bed material consists of fine sand (IV-3).

Therefore, an air-bubbler, similar in design to the one employed in the laboratory tests should be able to lift to the river surface both bed load and suspended sediment (being lighter than bed material, see section IV-3), found in front of the Fraser-Surrey Wharf. In conjunction with a strong river current, sufficiently fast to carry this agitated sediment past the wharf, several bubble hoses placed in the river-bed perpendicular to the river-flow and of sufficient length, should a priori be able to prevent sedimentation in the approaches to the wharf.

However, although being a necessary condition to the success of a bubbler as a shoal-inhibiter, the superposition of a horizontal flow has also an adverse effect upon the bubbler's operation.

The laboratory tests (see fig. 12) showed that the maximum upward velocities induced by bubbles in still water were confined to a very narrow zone, vertically above the bubbler. Outside of this zone, there was a sharp drop in the upward velocities induced in the water, particularly on a horizontal plane about one inch
above the bubbler hose, where a strong upward flow in the water is most needed to contend with bed-and saltating load. Once a relatively large particle escapes from this zone of maximum upward velocities, it will almost immediately enter a region where the upward velocities in the water are unable to support it any longer, and the particle will sink back to the bed. Turbulence, which is always present in river-flow, would force a large number of sand particles out of the narrow vertical zone. An entirely different, but more obvious effect of the horizontal flow in a river would be the dispersion of the bubbles with a consequent loss in energy and hence decrease in upward water velocities (II-1). Therefore, the laboratory experiments performed in still water, although very encouraging, are bound to exaggerate the effectiveness of an air-bubbler in the prevention of shoaling. Consequently the design of a prototype bubbler, if based on laboratory tests in still water, should include an ample safety factor in the air supply to the prototype bubbler.

In the laboratory flume available for the bubbler tests, it was not possible to superimpose upon the bubble screen a horizontal flow large enough to simulate river conditions (III-1). Consequently, the imposition of a fairly deep flow on a screen of rising bubbles would be a worthwhile project for future research on bubblers for preventing shoal. To observe the water circulation, induced by rising air-bubbles, a large flume would be almost essential. Bulson used a graving dock with a width of 106 feet and a depth of 36 feet above the keel blocks. The cross sectional area of this dock would
require an exorbitantly high discharge to simulate river-flow (Bulson, who concentrated on pneumatic breakwaters, was not interested in introducing a horizontal flow). However, a reasonable site for a large flume, where, at the same time, sufficiently high horizontal velocities could be created, might be found near a river in British Columbia. The existing Robertson Creek test flume on Vancouver Island (15 feet deep), operated by the Federal Department of Fisheries, appears quite suitable for such tests.

Although the field tests at the wharf suffered some unexpected set-backs, they provided valuable experience. For example, the apparently simple procedure of placing the bubble hoses on the river-bed in a strong river current, required careful planning. The success of the operation showed how important it was to have the assistance of a powerful, well equipped vessel with skilled personnel. Much less satisfactory than the installation of the bubblers, was the operation of the compressors to supply air to the bubblers. It would be advisable in future operations to have the presence of a technician continuously, or preferably, a standby compressor. Instead of relying entirely on an rpm indicator on the compressor (from which the rate of flow of air could be estimated from performance curves and calibration tables), an air flow meter would be more accurate and positive.

The presence of ships at the wharf (IV-4) was detrimental to the operation of the bubblers; on the other hand, the function of the bubblers was to keep the wharf open for shipping. The only
solution is to install the bubblers upstream of the wharf and in locations just outside of the normal berth of a ship, if possible. A bubbler location downstream of the stern of a ship should be avoided in view of the dead-water area caused by the ship's hull (assuming that the ship's bow faces into the river current).

Inspection of the longitudinal profiles of the river bed parallel to the wharf, at distances of 100, 50 and 0 feet out from the wharf-face (Appendices XIII and XIV), plotted from the 1966 weekly freshet soundings as well as from a 10-year record of post-freshet, pre-dredging soundings, shows that the downstream half of the Fraser-Surrey Wharf is more susceptible to shoaling than the upstream half. There is a consistent depression in the river-bed near the upstream end of the wharf, probably caused by scour as a result of the contraction of the river-flow just outside the upstream end of the wharf. Furthermore, the current measurements during the freshet of 1966 show a marked decrease in river velocities (surface as well as sub-surface) in the downstream part of the immediate approaches to the wharf. This retardation of flow (due to causes mentioned in Section IV-2) would naturally contribute to shoaling. However, the installation of bubblers near the downstream part of the wharf would not be advisable due to the lack of horizontal flow to carry the agitated sediment past the wharf and out of the project area. This was demonstrated by placing the bubbler on the river-bed at mark 6 + 00 and also by blowing large quantities of air into the sand-bed at mark 8 + 00 with a vertical pipe connected to the small compressor, in attempts
to remove a local hump. This local shoal hump was reduced in height but a new shoal was created in the immediate vicinity because of the absence of a horizontal flow to carry the agitated sediment downstream. The three bubblers at the upstream end, however, did have the support of a strong horizontal flow and there is reason to believe that their presence diverted part of the sediment, which otherwise would have been deposited near the downstream end of the wharf.

It should be pointed out that the Federal Department of Public Works dredged a "sediment trap" (see fig. 16) upstream from the wharf following the 1965 freshet, which trapped about 100,000 cubic yards of sediment during the 1966 freshet. This "sediment trap", in conjunction with the bubblers and a rather small freshet may all have contributed to the relatively favourable shoaling conditions at the wharf during the summer of 1966 with the consequent increase in shipping over similar periods in preceding years.

Although it is very doubtful that air-bubblers of any design could alleviate shoaling, at the downstream part of the wharf, in view of the low river velocities there, it would have been worth while to continue tests with bubblers at the upstream part of the wharf in future by varying the design of the bubblers and studying the effect of freshets upon the shoaling. Unfortunately, however, this will not be possible. In the fall of 1966, plans were approved to build a river-control structure in front of the Fraser-Surrey Wharf, consisting of a pile dike 950 feet long and a pile dike 330 feet long connecting it with the upstream end of the
Fraser-Surrey Wharf. This structure, which was completed in the winter of 1966-67, is one of the control structures proposed by the Fraser River Model Project at the University of British Columbia in 195919, to minimize dredging maintenance in the Trifurcation Area and at the same time increase the navigation channel widths and depths. The Fraser River Model had showed that this structure was needed to keep in motion the bed material moved downstream by the action of the training structures further upstream. The structure creates a sheltered basin at the wharf, which cannot be entered by bed material brought downstream because of closed piling below water, which enclosed the wharf approaches on the river-ward and upstream sides. However, the upper portion near the surface consists of open piling to produce flushing action and it is possible that the installation of one or more air-bubblers in this basin would be instrumental in preventing suspended sediment from settling. At any rate, further large-scale field-tests with air-bubblers on the Fraser River estuary should be conducted elsewhere in the future.
SECTION VII
CONCLUSION

General. Without a strong horizontal river current to carry the agitated sediment outside of the project area, the air-bubbler cannot prevent shoaling.

Preliminary Studies. Before deciding if the installation of a system of air-bubblers in a shoal area is warranted as a shoal-preventative measure, field studies should be carried out during the period when shoaling occurs. These studies would include soundings (to follow the pattern of shoaling); current velocity measurements; float studies (to determine flow patterns) and sediment sampling and analysis. The current velocity measurements should cover the entire project area, but should concentrate particularly on that part where shoaling appears to be most severe and where the flow conditions are most adverse (see page 67). If adequate equipment for current measurements is not available, a simple float survey as outlined in IV-2 would give a good approximation to velocities. However, sub-surface velocity measurements (preferably with a directional current meter) would be desirable in tidal estuaries if a reversal of flow is suspected at flood tides. If no up-to-date samplers are available for measuring suspended sediment, or bed load, a sieve analysis of the bed material which is easily obtained with a drag-bucket type of bed-material sampler, would be quite in order since bed-load and bed-material have almost identical grain-size distribution curves (see figures 31 and 32)
and suspended sediment is generally finer than bed load and more easily kept in suspension by agitation.

If the results of these field-surveys (i.e. soundings, velocity measurements, flow patterns, and sediment analysis) together with an examination of the "Chart for determining largest grain sizes to reach deposit area" (Appendix IX) indicated that the installation of air-bubblers is justified, then the proposed design of the bubblers is as follows.

**Suggestions for design and operation of air-bubblers.**

**DIMENSIONS AND MATERIAL:** One-inch I.D. polyethylene hose, which is fairly cheap, non-corrosive, light and easy to drill. The length of each air-bubbler should be equal to the width of the project area (since the bubblers will be placed in a direction perpendicular to the river flow).

**ORIFICE DIAMETER AND SPACING.** The diameter of the air holes should be small, preferably 0.0135 inches; with a close spacing of 1/4 inch. This produces a dense screen of rising air-bubbles for a given compressor capacity. For details, see page 81. This diameter and spacing may seem very small compared to other air-bubblers described in the literature. However, it should be emphasized that most of these other bubbler designs were aimed at creating a strong horizontal flow near the surface, or a strong vertical flow, without the particular need for a strong upward flow in the bottom layers of the surrounding water, close to the bed and near a bubbler hose. In the case of a shoal inhibiter, the effect of the air-bubbles on the bottom layers of the water in producing an upward flow is very
important and the laboratory experiments have shown quite clearly that a close spacing of the air holes with a correspondingly small diameter (in view of the limited capacity of an air compressor) are preferable to a large spacing and large orifice diameters (see also figures 13-15).

AIR PRESSURE: The air pressure at the river ward end of the bubbler hose should be well above hydrostatic pressure (twice the hydrostatic pressure is suggested). Air pressure and rate of air flow are interdependent but a pressure well above hydrostatic pressure would prevent partial filling of the hose with water due to possible pressure fluctuations. For calculations of pressure drop in a polyethylene hose, Stehr's equation on page 92 could be used (determining $\beta$ empirically); the pressure drop in the perforated part of the hose (i.e. the part which serves as the actual air-bubbler) should be taken as 1/3 of the pressure drop in an equal length of unperforated hose under similar conditions. The results, allowing also for pressure losses in connections, etc. (say 10%) would determine the minimum required pressure at the compressor. This could be checked by installing a pressure gauge at the outer end of the hose, with the compressor operating and before the bubbler was lowered into the water.

RATE OF FLOW OF AIR: It was found that one cubic foot of air per minute, per foot length of bubbler hose, at the prevailing hydrostatic pressure was adequate. When the bubbler is placed in 30 feet of water, this would mean 2 cfm of free (atm. pressure) air.

COMPRESSOR CAPACITY would be defined by the rate of flow of air required
and the total length of perforated (bubbler) hose, e.g. 200 feet of bubbler hose in 30 feet of water would require a compressor capacity of approximately 400 cfm.

INSTALLATION OF BUBBLER HOSES. A recommended procedure for installation is described on page 84, see also plate 4. Installation from a boat pulling away from the wharf is not recommended, since the movement of the boat cannot be controlled accurately enough in a river current to prevent breaking or kinking the hose.

OPERATION. The presence of a standby compressor would be desirable, to guarantee continuous operation. A technician should be at hand at all times and an air flow meter should be available.

For some additional practical details regarding the bubbler hoses, vid. page 90.

NUMBER OF HOSES. It was shown (ref. Section V) that the maximum upward water velocities induced by the bubbles, are confined in a very narrow vertical zone above the bubbler hose, particularly near the river-bed, providing a relatively weak barrier against bed load and saltation load. A much more effective barrier or screen against these sediment fractions would be obtained by three parallel air-bubbler hoses, spaced about one foot apart.

Comparison with other applications of air-bubblers.

Of all the possible practical applications (see also Section I) of the property of rising air-bubbles to create a circulation in the surrounding water, the shoal inhibitor appears to be the least efficient, because:

A) The upward flow induced in the water by the rising bubbles
is weakest where it is most needed: viz. near the river bed.

B) The horizontal flow in the river, without which the bubblers would not operate as shoal-inhibitors, has also an adverse effect upon the magnitude of the vertical velocities in the water induced by the air-bubbles. Although a strong horizontal flow would be most beneficial to the bubbler's operation in carrying the agitated sediment out of the project area, it would require large quantities of air to overcome the reduction in upward water velocities due to the scattering and dispersal of the air bubbles.

The Fraser-Surrey Wharf.

Although much valuable experience was gained during the field tests at the Fraser-Surrey Wharf, it must be recognized that a clear appraisal of an air-bubbler as a shoal-inhibiter was severely hampered by a number of factors, including the presence of ships, the flow conditions in the approaches to the wharf, and the unsatisfactory performance of the compressors. However, due to the unfavourable river flow conditions near the downstream end of the wharf, it is very doubtful that air-bubblers would have alleviated shoaling at this wharf to such an extent as to warrant their permanent installation there.

Suggestions for Further Research

Detailed studies are necessary to determine how a horizontal flow can effect the efficiency of a screen of rising air bubbles in preventing shoal. A large experimental flume with a width of 5 feet or more and a depth of about 10 feet would permit detailed observations of vertical distributions of velocity and would eliminate wall effects.
Such a flume should also be capable of producing horizontal velocities comparable to those occurring in the prototype when shoaling occurs. The large outdoor test flume built by the Department of Fisheries at Robertson Creek on Vancouver Island, might conceivably serve this purpose.


4. E. Stehr, "Berechnungs grundlagen für Preszluft". Ölsperren, Franzius Institut, Hannover (in German).


11. W. Hensen, "Model Versuche mit pneumatischen Wellenbrechern", Franzius Institut, Hannover (German).


14. H. Rouse, "Engineering Hydraulics".


APPENDIX I

PNEUMATIC BREAKWATERS

Velocity of Surface Waves: \( C^2 = \frac{g\lambda}{2\pi} \tanh \frac{2\pi h}{\lambda} \), where \( c \) = celerity of waves; \( h \) = still water depth and \( \lambda \) = wave length.

For \( h \gtrsim \frac{\lambda}{2} \) (deep water wave): \( \tanh \frac{2\pi h}{\lambda} \to 1 \).

\( \therefore \ C^2 = \frac{g\lambda}{2\pi} \), or \( c = \sqrt{\frac{g\lambda}{2\pi}} \).

If waves are met by an opposing current with velocity \( v \), \( c_1 = \frac{g\lambda}{2\pi} \), where \( c_1 \) is the absolute celerity, \( c_1 = c - v \).

Therefore, since \( c > c_1, \lambda > \lambda_1 \).

When a surface wave in deep water meets an opposing current, both its celerity and wave length decrease.

Rate of Transmission of Wave Energy =

\[
\frac{4\pi h}{4\pi h} = \frac{4\pi h}{4\pi h} \cdot \frac{\frac{4\pi h}{\lambda}}{\frac{\frac{4\pi h}{\lambda}}{\lambda}},
\]

where \( a = \) amplitude.

\( \lambda \) is shorter in an opposing current than in still water.

\[
\begin{align*}
\text{Consider the term } & \left( 1 + \frac{\frac{4\pi h}{\lambda}}{\frac{4\pi h}{\lambda}} \text{ cosech} \frac{\frac{4\pi h}{\lambda}}{\lambda} \right), \\
\text{or } y &= 1 + x \text{ cosech}(x) \left( x \text{ always positive} \right) .
\end{align*}
\]

\[
\begin{align*}
\frac{dy}{dx} &= -x \text{ cosech}(x) \text{ coth}(x) + \text{ cosech}(x) = \\
&= \left( 1 - x \text{ coth}(x) \right) \text{ cosech}(x),
\end{align*}
\]

where \( \text{cosech}(x) \) always positive for \( x > 0 \).

Consider the term \( 1 - x \text{ coth}(x) \):

\( x \text{ coth}(x) \) is always \( > 1 \) for all positive \( x \) values.

\[
\begin{align*}
\therefore \ & 1 - x \text{ coth}x \text{ is negative for all positive } x \text{ values}; \quad \frac{dy}{dx} < 0. \\
\text{The term } & \left( 1 + \frac{\frac{4\pi h}{\lambda}}{\frac{4\pi h}{\lambda}} \text{ cosech} \frac{\frac{4\pi h}{\lambda}}{\lambda} \right) \text{ decreases with decreasing } \lambda.
\end{align*}
\]
Therefore, assuming deep water waves and no dissipation of energy in the transition from still water to opposing current:

\[ \lambda \text{ decreases } \]
\[ c \text{ decreases } \]
\[ a \text{ must increase.} \]
APPENDIX II

Average velocities of bubbles emerging from 1' bubbler in flume, under varying pressures.

Empirical Equation: \[ W = 6.6 \frac{A_Q}{(P_i/P_o)^{0.4}} \]

where \( A_Q \) = cross area orifice in square inches;

\( P_i \) = absolute pressure inside bubbles

\( P_o \) = absolute pressure outside bubbles

(both \( P_i \) and \( P_o \) in psia)

\( W \) = upward velocity of bubbles in ft./sec.

Water depth in flume = 4 feet.

(Orifice diameter = 0.036", \( A_Q = 10.2 \times 10^{-4} \))

\[ P_o = 16.4 \]

<table>
<thead>
<tr>
<th>( P_i )</th>
<th>( (P_i/P_o)^{0.4} )</th>
<th>( W_{calc} )</th>
<th>( W_{obs} )</th>
<th>error (%)</th>
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</thead>
<tbody>
<tr>
<td>49</td>
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<td>1.22</td>
<td>1.28</td>
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</table>

(Orifice diameter = 0.026", \( A_Q = 5.3 \times 10^{-4} \)).

<table>
<thead>
<tr>
<th>( P_i )</th>
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<th>( W_{calc} )</th>
<th>( W_{obs} )</th>
<th>error (%)</th>
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<tr>
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<td>1.12</td>
<td>1.12</td>
<td>1.14</td>
<td>2</td>
</tr>
</tbody>
</table>

(Orifice diameter = 0.020", \( A_Q = 3.1 \times 10^{-4} \)).

<table>
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<th>( W_{calc} )</th>
<th>( W_{obs} )</th>
<th>error (%)</th>
</tr>
</thead>
<tbody>
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<tr>
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<td>1.20</td>
<td>1.05</td>
<td>1.18</td>
<td>12</td>
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</table>
APPENDIX III

Settling velocities of Fraser River sand deposited in front of Fraser-Surrey Wharf.

<table>
<thead>
<tr>
<th>Retained by Sieve</th>
<th>Mean setting time in seconds, distance = 183 cm.</th>
</tr>
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<tbody>
<tr>
<td>mm</td>
<td>No.</td>
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<tr>
<td>2.00</td>
<td>10</td>
</tr>
<tr>
<td>0.84</td>
<td>20</td>
</tr>
<tr>
<td>0.42</td>
<td>40</td>
</tr>
<tr>
<td>0.25</td>
<td>60</td>
</tr>
<tr>
<td>0.147</td>
<td>100</td>
</tr>
<tr>
<td>0.105</td>
<td>140</td>
</tr>
<tr>
<td>0.074</td>
<td>200</td>
</tr>
<tr>
<td>PAN</td>
<td></td>
</tr>
</tbody>
</table>

Arrows denote the maximum size found in the following sieve; i.e. the size of particles which have the maximum v_s for that particular sieve.

<table>
<thead>
<tr>
<th>Theoretical * settling velocities (cm/sec.)</th>
<th>Settling velocities found with PURI (cm/sec)</th>
<th>Dia (mm) particles</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
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</tr>
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<td>5.6</td>
<td>5.5</td>
<td>0.42</td>
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<td>3.2</td>
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<td>0.25</td>
</tr>
<tr>
<td>1.5</td>
<td>2.3</td>
<td>0.147</td>
</tr>
<tr>
<td>0.8</td>
<td>1.9</td>
<td>0.105</td>
</tr>
<tr>
<td>0.4</td>
<td>1.3</td>
<td>0.074</td>
</tr>
</tbody>
</table>

* Based on U.S. Bureau of Reclamation data.
Compressible flow, mass flow $\dot{m} = CYA_0 \sqrt{2 \Delta p}$

$D_o = 0.25'' \quad A_o = 0.057 \quad A_o = \frac{0.0491}{144} \text{ ft}^2.$

Assume $p_1 \sim p_2 \quad \text{Expansion Factor } Y = 1.$

\[
\Delta p = \frac{1}{12} \Delta h \left( \frac{14.7}{34} \right) = 0.036 \Delta h \quad \text{(inches)}
\]

\[
\dot{m} = \frac{0.0491}{144} \sqrt{144} (0.072) p_i \Delta h \quad C = \quad \text{slugs/sec.}
\]

\[
= 10.98 \times 10^{-4} \ C \sqrt{p_i \Delta h} \quad \text{slugs/sec.}
\]

(Where $p_i$ in slugs/ft$^2$, $\Delta h$ in inches)

\[
p_i = \frac{144 (P_a + 14.7)}{(1715) (460+t)} \quad \text{slugs/ft}^3.
\]
## CALIBRATION DATA

<table>
<thead>
<tr>
<th>Airpressure at orifice meter (psi) $P_a$</th>
<th>Manometer at orifice m. (inch water) $\Delta h$</th>
<th>Manometer at Gasom. (inch Mg) $h_m$</th>
<th>Airvol. in Gasom. (ft.$^3$) $\sqrt[3]{V}$</th>
<th>$\Delta t$ secs</th>
<th>$t ^\circ F$</th>
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<td>80</td>
<td>50</td>
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<td>35</td>
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<td>1.59</td>
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<td>19$^5$</td>
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<td>1.59</td>
<td>15</td>
<td>47</td>
</tr>
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<td>1.25</td>
<td>1.66</td>
<td>14$^5$</td>
<td>47</td>
</tr>
</tbody>
</table>

Air pressure in gasometer: $p_a = (13.6)(62.4)\left(\frac{h_m}{12}\right) + 2116 = 70.72\ h_m + 2116\ \text{psf.}$

Specific weight of air $\gamma = \frac{p}{R_o T} = \frac{p_a}{(53.4)(508)} = (0.37)(10^{-4})p_a$

At orifice: $\rho = \frac{144\ (P_a + 14.7)}{(1715)(^\circ F+460)} = (0.166)(10^{-3})\ P_a + 14.7$ slugs/ft.$^3$.

$W = \frac{\sqrt[3]{V}}{\Delta t}$ lbs/sec.

Calculations of $C$ in $\dot{m} = (10.98)\ (C)\ \sqrt[3]{\rho \ h_w}\ (10^{-4})$,

where $\dot{m}$ in slugs/sec.

$C = \frac{\dot{m} \ (10^4)}{(10.98) \sqrt[3]{\rho \ h_w}}$ (if $\dot{m}$ in slugs/sec.)

or $C = \frac{\dot{W} \ (10^4)}{10.98 \sqrt[3]{\rho \ h_w}} \times \frac{1}{32.2}$ (if $\dot{W}$ in lbs/sec).
\[ P_a + 14.7 \]

<table>
<thead>
<tr>
<th>( \rho )</th>
<th>( h_w )</th>
<th>( \sqrt{\rho h_w} )</th>
<th>( \gamma )</th>
<th>( \omega )</th>
<th>( W )</th>
<th>( C )</th>
<th>( R )</th>
</tr>
</thead>
<tbody>
<tr>
<td>psi</td>
<td>Slugs/ft(^3)</td>
<td>inch</td>
<td>x ( 10^{-1} )</td>
<td>lbs/ft(^3)</td>
<td>lbs/sec</td>
<td>x ( 10^{-3} )</td>
<td></td>
</tr>
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<td>4.96</td>
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<td>0.71</td>
<td>0.081</td>
<td>1.52</td>
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<td>1.90</td>
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<td>0.56</td>
<td>11.76</td>
</tr>
</tbody>
</table>

Reynolds Number: \( R = \frac{VD \rho}{\mu} = (4.03) \times 10^7 \times m. \)

\[ \mu = (3.8) \times 10^{-7}; \ D = \frac{1}{12} \text{ ft.} \]
**APPENDIX V**

Sample Measurements with mini-flow current meter (Armstrong-Whitworth) of Upward Water Velocities induced by Bubbles in Laboratory Flume.

- **D** = Depth of Water, in Feet.
- **T** = Temp. of Water, in Centigrades.
- **H** = Height of Probe above Bed, in Feet.
- **P** = Air pressure in Bubbler, in Psig.
- **P_g** = Air pressure at orifice meter in psig.
- **h_g** = Reading Water manometer at Orifice meter in inches.
- **t_g** = Air temperature in Centigrades.
- **W** = Weight flow of Air in lbs/sec.
- **V** = Upward Water velocity in Inch/second.

**TEST 1.** Spacing Orifices 1\(\frac{1}{4}\) inch.

Diameter Orifices 0.020 inch.

Probe directly over bubbler.

<table>
<thead>
<tr>
<th>D</th>
<th>T</th>
<th>H</th>
<th>P</th>
<th>h_g</th>
<th>p_g</th>
<th>t_g</th>
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**TEST 16.** Spacing 6 inch Dia. 1/16" Probe directly above central orifice of bubbler.

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**TEST 17.** Spacing 6 inch Dia. 1/16" Probe directly over bubbler, but midway between orifices.

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APPENDIX VI

TAYLOR'S ANALOGY

\[ W_o = K (Vg)^{1/3} \]

\[ W_o = \text{max. velocity of water in fps}. \]

\[ V = \text{volume of air/second/foot length of bubbler (cfs/ft)} \]

\[ g = \text{acc. of gravity in ft/sec}^2. \]

Calculate \( V \) (i.e. volume of air at the depth of the bubbler)

\[ \gamma = \frac{P(4')}{RT} = \frac{2116.3 + 4 \times 62.4}{53.4 \times 510} = 0.087 \text{ lbs/ft}^3, \]

in 4 ft of water.

\[ \therefore V = \frac{o}{0.087}, \text{ where } o = \text{weight flow of air in lbs/sec}. \]

49 orifices per foot of bubbler, dia orifices 0.020", spacing 1/4".

<table>
<thead>
<tr>
<th>( W ) lbs/sec ( \times 10^3 )</th>
<th>( V ) ft(^3)/sec ( \times 10^2 )</th>
<th>( (Vg)^{1/3} )</th>
<th>( W_o (\text{exp.}) ) ft/sec</th>
<th>( K )</th>
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Assume first reading wild \( W = 1.0 \times 10^{-3} \)

Coverage of all remaining nine values of \( K \) : 1.49

\[ W_o = 1.49 \ (Vg)^{1/3}. \quad g^{1/3} = 3.18 \]

\[ \therefore \text{Max. upw. velocity of water } W_o = 4.74 \ V^{1/3} \text{ ft/sec}. \]
Graph recorded by Tide Gauge at New Westminster

Vertical Scale: 1" = 1 Ft of Water

\[ v = \sqrt{g \cdot d} = \sqrt{32.2 \times 2.5} = 28 \text{ ft/sec.} \]

\[ \therefore \text{Tidal Comp.} = \frac{1030}{12} \times \frac{2.25 \times 12}{15 \times 3600} = 0.02" \]

Appendix VII. Field Calculations of Tidal Contribution to Elevation.
APPENDIX VIII - GRAPH OF WATER-SURFACE SLOPE MEASUREMENTS, FRASER-SURREY WHARF

(See Figure 17)
EXPLANATION OF CHART

TO FIND MAXIMUM DIAMETER OF PARTICLE WHICH WILL REACH BOTTOM UNDER GIVEN CONDITIONS, USE THE FOLLOWING FORMULA:

\[ T = \frac{0.5475}{V} \]

WHERE
- \( T \) = TIME IN MINUTES FOR PARTICLES HAVING A SPECIFIC GRAVITY OF 2.65 TO FALL 100 CENTIMETERS (3.28 FT.)
- \( D \) = DISTANCE IN FEET FROM DREDGE TO POINT OF DEPOSIT
- \( V \) = VELOCITY OF CURRENT IN FEET PER SECOND
- \( d \) = DEPTH OF WATER IN FEET AT POINT OF DEPOSIT

EXAMPLE

\[ D = 2400 \text{ FT.} \]
\[ V = 5 \text{ FPS} \]
\[ d = 20 \text{ FT.} \]
\[ T = \frac{0.5475 \times 2400}{5 \times 20} = 1.31 \]

ENTER CHART WITH 1.31 MIN., AND FIND MAXIMUM DIAMETER OF PARTICLE = .15 MM.
Flow of air through Pipeline (After Gibson)

Pipe of constant diameter $d$. Flow decreases with length at constant rate due to lateral off-takes, each off-take taking same flow; equal spacing.

Rate of decrease of flow = $q$ cfs/foot.

At $P$: flow = $Q - qx$. Loss of head up to $P = (h_f) x$

At $P$: $(h_f) dx = \frac{fV^2}{2gd} dx$, but since diameter is constant $\frac{V}{V} = \frac{Q - qx}{Q}$

\[ \therefore (h_f) dx = \frac{fV^2}{2gd} \left( \frac{Q - qx}{Q} \right)^2 ft, \]

\[ (h_f) x = \frac{fV^2}{2gdQ^2} \int_0^x (Q - qx)^2 dx \]

\[ = \frac{fV^2}{2gdQ^2} \left[ Q^2 x - Qqx^2 + \frac{Q^2}{3} x^3 \right] \]

Flow at downstream end of $x$ is $Q_x$. Then $Q = Q_x + qx$, from continuity.

Substitution and simplification gives:

\[ (h_f) x = \frac{fV^2}{2gdQ^2} \left[ Q_x^2 x + Q_x qx^2 + \frac{Q^2}{3} x^3 \right] \]

At $x = l$, $Q_x = 0$; $Q = qx$.

\[ \therefore (h_f)_{x=l} = \frac{1}{3} \frac{l}{d} \cdot \frac{V^2}{2g} \] (with lateral offtakes).

Note: loss of head only $\frac{1}{3}$ of loss of head for steady flow without any decrease in flow along pipe due to lateral offtakes.
Flow of air through Pipeline (After Gibson)

Pipe of constant diameter $d$. Flow decreases with length at constant rate due to lateral off-takes, each off-take taking same flow; equal spacing.

Rate of decrease of flow = $q$ cfs/foot.

At P: flow = $Q - qx$. Loss of head up to $P = (h_f)_x$

At P: $(h_f)_x = \frac{fV^2}{2gd} dx$, but since diameter is constant $\frac{V}{V} = \frac{Q - qx}{Q}$

\[
(h_f)_x = \frac{fV^2}{2gdQ^2} \int_0^x (Q - qx)^2 dx
\]

\[
= \frac{fV^2}{2gdQ^2} [Q^2x - Qqx^2 + \frac{1}{3}q^2x^3]
\]

Flow at downstream end of $x$ is $Q_x$. Then $Q = Q_x + qx$, from continuity.

Substitution and simplification gives:

\[
(h_f)_x = \frac{fV^2}{2gdQ^2} [Q_x^2 + Q_xqx^2 + \frac{1}{3}q^2x^3]
\]

At $x = l$, $Q_x = 0$; $Q = qx$.

\[
(h_f)_{x=l} = \frac{1}{3} \frac{q}{d} \cdot \frac{V^2}{2g} \text{ (with lateral offtakes).}
\]

Note: loss of head only $\frac{1}{3}$ of loss of head for steady flow without any decrease in flow along pipe due to lateral offtakes.
FRASER-SURREY DOCK

FRASER-SURREY DOCK SOUNDINGS
 SCALE: 1" = 50'

NOTES:

Soundings are in feet referred to Local Low Water Level, which is 6" feet above Soundhead Datum.


FRASER RIVER, B.C.
AT NEW WESTMINSTER
FRASER SURREY DOCK
SOUNDINGS.

APPROVED:

CHIEF ENGINEER
OTTAWA 196

EXAMINED BY:

PUBLIC WORKS OF CANADA

DISTRICT ENGINEER

CERTIFIED CORRECT

SURVEYED BY

DESIGNED BY

DRAWN BY

CERTIFIED Cop

PLAN No.
APPENDIX XV
EQUIPMENT AND MATERIALS USED IN FIELD

**Bubbler.**

**Compressor:** PR 620 Cummins Diesel

620 cfm normal operating press. 100 psi

Speed max/min 1900/1400.

**Compressor:** PR 365 Deutz Diesel

365 cfm normal operating press. 100 psi

Speed max/min 1800/1400.

**Hose:** Carlon plastic, 1 inch I.D.

1-1/4 inch O.D.

Polyethylene series, standard type 11.

80 psi. Available at Fleck Bros. $13.93/100 ft.

**Clamps:** Tridon HAS-24 (3/4" - 1-1/2" I.D. and 1-1/16" to 2" O.D. ALL STAINLESS.

Available at Fleck Bros. $42.10/100.

**Vessels.**

**Launch "Sounder".** Length 40 feet O.A.


**Launch "Port Fraser".** Length 37 feet O.A.


**Snag boat "Samson".** Length 90 feet O.A.

100 Tons. Triple Exp. engine. Stern Wheeler.

Crew: 14.
PLATE 1. NEW WESTMINSTER TRIFURCATION AREA, FRASER RIVER, B.C. (MAY 1959). COURTESY GEORGE ALLEN.
2a. Steel and glass flume in the Civil Engineering Hydraulics Laboratory at the University of B.C.; showing strain indicator (on table) for measuring hydraulic slopes, and plexiglass stilling well (right, foreground).

2b. Injector of meriam drops (releasing a meriam drop) and 16" long polyethylene hose with orifices (dia. 0.026 inch; spacing 0.75 inch) and emerging screen of air-bubbles. Air pressure inside bubbler 10 psig. The heavy black horizontal line above the injector belongs to the grid painted on the glass of the flume.
Plate 3. Measurements with the Miniflow Meter

3a. Probe with rotor of "Miniflow" current meter, measuring upward water velocities induced by rising air-bubbles.

3b. Dekatron counter unit of Miniflow meter with probe in foreground.
Plate 4. **Installation of air-bubblers on river-bed at Fraser-Surrey Wharf. May 24th, 1966.**

4a. Part of the one-inch diameter bubble hose shortly before installation. Note 5/8 inch steel cable; stainless steel clamp; boomchain; and small balloon to keep hose suspended above river-bed while being payed out from wharf to "Samson".

4b. Bubble hose being payed out from wharf to "Samson". "Port Fraser" assisting.
Plate 5. Fraser-Surrey Wharf (facing downstream), May 25, 1966, showing location of PR 620 Compressor and buoy. "Atlantic Breeze" has just arrived to load lumber. Courtesy George Allen.
Plate 6.  **Sounding at the Fraser-Surrey Wharf, June 1966.**

6a. View from top of Grain Elevator. "Port Fraser" running sounding lines parallel to upstream part of the wharf. Note bubbles at mark 6 + 00.

6b. "Sounder" surveying with stretch line and range poles (one shown).
Plate 7. Sampling of Suspended Sediment at Fraser-Surrey Wharf, June 1966.

7a. P-61. Point Sampler being lowered from the stern of the "Fort Fraser".

7b. Recovering bottle containing sample.
INTERNAL CIRCULATION IN RISING AIR BUBBLES AND ITS EFFECT UPON DRAG-COEFFICIENT

NO CIRCULATION DUE TO IMPURITIES AT INTERFACE. STREAMLINES LEAVE INTERFACE OVER PART OF REAR HEMISPHERE OF BUBBLE (SEPARATION), INTRODUCING FORM DRAG; LARGE DRAG-COEFFICIENT

CIRCULATION DELAYS SEPARATION; SMALL DRAG-COEFFICIENT

FIG. 2
DRAG COEFFICIENT AS A FUNCTION OF REYNOLDS NUMBER
FOR BUBBLES RISING AT THEIR TERMINAL VELOCITY IN WATER
CONTAINING VARIOUS SURFACE-ACTIVE MATERIALS

(AFTER HABERMAN & MORTON)

FIG. 3
TERMINAL VELOCITY OF AIR BUBBLES IN TAP WATER (21°C)
AS A FUNCTION OF BUBBLE SIZE

(AFTER HABERMAN & MORTON)
CALIBRATION OF ORIFICE METER
IN 8" PIPE LEADING TO STEEL & GLASS FLUME
1 1/2 FT WIDE AND 5 FT HIGH

FIG. 5

MAX FLOW: 5.79 CFS

DISCHARGE (Q) IN CUBIC FEET PER SECOND

MANOMETER DIFFERENTIAL (Z) IN INCHES
CALIBRATION OF ORIFICE METER IN 8" PIPE LEADING TO STEEL & GLASS FLUME, 1 1/2 FT WIDE AND 5 FT HIGH.

FIG. 6
**SLOPE MEASUREMENTS IN HYDRAULICS LABORATORY**

**UNIVERSITY OF B.C.**

**BALANCED CONDITION:**

\[
\frac{R_1}{R_1'} = \frac{R_2}{R_2'}, \quad E = 0
\]

**LOAD:**

- \( R_{1_L} > R_1 \) or \( R_{2_L} > R_2 \)
- \( R_{1_L} < R_1' \) and \( R_{2_L} < R_2' \)

**BUOYANCY CHAMBER WITH BALLAST**

**STILLING WELL**

**TO DOWNSTREAM END OF FLUME**

**STILLING WELL**

**TO UPSTREAM END OF FLUME**

**EXCITATION VOLTAGE**

**FIG. 7**

**ABA 1966**
SETTLING RATE OF SAND PARTICLES IN WATER

(RESULTS OF PURI SILTOMETER ANALYSIS WITH FRASER RIVER SAND TAKEN FROM FRASER-SURREY WHARF)

FIG. 8
PUKI SILTOMETER,*
DROPPING DEVICE AND ITS CIRCUIT DIAGRAM

SCALE (SILTOMETER): 1/4 IN = 1 IN.

* FRASER RIVER MODEL VERSION

FIG. 9
Fig. 10

TO AIR BUBBLER IN FLUME

ONE-INCH DIAMETER SAFETY PIPE LEADING

CALIBRATION OF 1/4-INCH DIAMETER ORIFICE IN
UPWARD VELOCITIES INDUCED IN WATER BY RISING AIR BUBBLES

DEPTH OF WATER: 5 FEET
ORIFICE DIAMETER: 0.020 INCH
ORIFICE SPACING: 0.250 INCH

SCALES:
HORIZONTAL (X): 1' = 1'
VERTICAL (Z): 1' = 0.5'
VELOCITY (W): 1' = 1 FT/SEC

WARNING FLOW OF AIR = 3.2 x 10^-3 LBS/SEC/FT
WARNING FLOW OF AIR = 0.9 x 10^-3 LBS/SEC/FT

MAX VELOCITY = 10 FT/SEC
AT 3.2 x 10^-3 LBS/SEC/FT

VERTICAL DISTANCE IN FEET
HORIZONTAL DISTANCE IN FEET
AIR BUBBLER (1', 0')

FIG. 12
UPWARD VELOCITIES INDUCED IN WATER BY RISING AIR BUBBLES

DEPTH OF WATER: 4 FEET
ORIFICE DIAMETER: 0.0625 INCH
ORIFICE SPACING: 6.00 INCH
WEIGHT FLOW OF AIR: 2.3 x 10^{-3} LBS/SEC/FT
AIR PRESSURE IN BUBBLER: 30 PSIG

SCALES:
HORIZONTAL: 1" = 2'
VERTICAL: 1" = 0.2'
VELOCITY: 1" = 1 FT/SEC

FIG. 13
UPWARD VELOCITIES INDUCED IN WATER BY RISING AIR BUBBLES

DEPTH OF WATER: 4 FEET
ORIFICE DIAMETER: 0.045 INCH
ORIFICE SPACING: 3.00 INCH
WEIGHT FLOW OF AIR: $2.3 \times 10^{-5}$ LBS/SEC/FT
AIR PRESSURE IN BUBBLER: 20 PSIG

SCALES:
HORIZONTAL: 1" = 2"
VERTICAL: 1" = 0.2'
VELOCITY: 1" = 1 FT/SEC

ORIFICE
BUBBLER

HORIZONTAL DISTANCE IN INCHES
0 3 6 9 12

FIG 14
UPWARD VELOCITIES INDUCED IN WATER BY RISING AIR BUBBLES

DEPTH OF WATER: 4 FEET
ORIFICE DIAMETER: 0.036 INCH
ORIFICE SPACING: 1.50 INCH
WEIGHT FLOW OF AIR: 2.3 x 10^-3 LBS/SEC/FT
AIR PRESSURE IN BUBBLER: 20 PSIG

SCALES
HORIZONTAL: 1" = 2"
VERTICAL: 1" = 0.2"
VELOCITY: 1" = 1 FT/SEC

0-2
0.4
0.6
0.8
1.0

BUBBLER
FIG 15
FIELD INVESTIGATIONS AT FRASER-SURREY WHARF DURING FRESHET PERIOD, MAY-JULY 1966

EBB FLOW

FRASER-SURREY WHARF

SCALE: ONE INCH = 125 FEET

(See also Figures 18 and 19 for observations of Surface Currents)

FIG. 16
WATER-SURFACE SLOPE MEASUREMENTS, FRASER-SURREY WHARF, NEW WESTMINSTER

BALANCED CONDITION

VOlT METER

SUSPENSION WIRE

FLOAT

STILLING WELL

RIVER LEVEL

DOWNSTREAM

UPSTREAM

FIG. 17

1030'
SURFACE CURRENTS, 
FRASER-SURREY WHARF

JULY 19, 1966, 17:00 HR PST  
(TWO HRS AFTER LLW AT NW.)

HOPE DISCHARGE 213,000 CFS

TIDE CURVE RECORDED AT  
FRASER RIVER AT NEW WESTMINSTER, B.C.

JULY 19, 1966

FRASER - SURREY WHARF
SCALE: ONE INCH = 125 FEET
CURRENTS IN FEET / SECOND

FIG. 18
SURFACE CURRENTS, FRASER-SURREY WHARF
JULY 24, 1966, 17:00 HR PST
(ONE HR BEFORE LLW AT NW)
HOPE DISCHARGE 205,000 CFS

EBB FLOW

FRASER-SURREY WHARF
SCALE: ONE INCH = 125 FEET.
CURRENTS IN FEET/SECOND

FIG. 19
Fig. 30

DEPARTMENT OF NORTHERN AFFAIRS AND NATIONAL RESOURCES
WATER RESOURCES BRANCH

FRASER RIVER

PORT MANN

Date 30th. 1955 Time 19:00
G. Ht. Discharge PPM
Sample 6223 Depth 35'
Sta. W.H. Analysis Bottom withdrawal

DIAMETER IN MILLIMETERS

PER CENT FINER
FRASER RIVER AT PORT MANN, B.C.

DISTRIBUTION CURVES OF BED MATERIAL

JUNE 24, 1966

% FINEER THAN

LOCATIONS

SAMPLE NO 6096 - Near Shore, Lower High Water
- 6098 - Mid Stream
- 6108 - Near Shore, Lower Low Water
- 6110 - Mid Stream

Numbers refer to Water Resources records.
Tides are low.

MEDIAN DIAMETERS

SAMPLE NO 6096 - 0.21 mm
- 6098 - 0.386 mm
- 6108 - 0.240 mm
- 6110 - 0.408 mm

FIG. 31

GRAN SIZE IN MM

SAMPLER: SM 54
FRASER RIVER AT PORT MANN, B.C.
DISTRIBUTION CURVES OF BED LOAD
JUNE 25, 1966

% FINER THAN

% FINE

LOCATIONS
SAMPLE NO 6118 Near Shore, Lower High Water
6122 Mid Stream
6128 Near Shore, Lower Low Water
6133 Mid Stream

Numbers refer to Water Resources records.
Tides are local.

MEDIAN DIAMETERS
SAMPLE NO 6118 0.203 mm
6122 0.372 mm
6128 0.216 mm
6133 0.428 mm

FIG. 32

SAMPLER: ARNHEM

GRAN SIZE IN MM
DEEP-SEA VESSELS BERTHED AT FRASER-SURREY WHARF DURING FRESHETS, 1957-1966

Vertical scale indicates percentage of freshet days when deep-sea vessels occupied wharf (one inch = 10%)
Horizontal scale shows duration of freshets (one inch = 100 days), i.e. when Hope Discharge > 150,000 cfs

FIG. 34