

2. CONSTRUCTION

Because of difficulties in locating plant in the surf zone, it was decided to construct the outfall out of precast reinforced concrete box sections (Figure 2) using a "leopfrogging" method of construction whereby all construction operations were carried out by a crane/pile driver located on top of previously placed units.

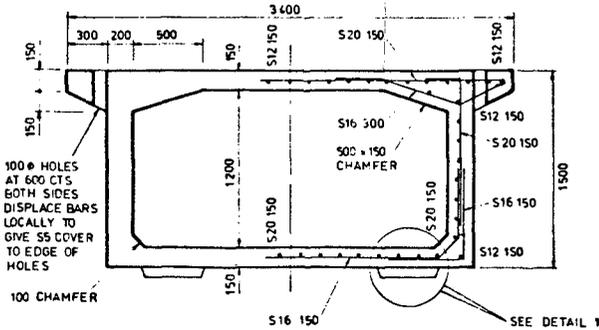


FIGURE 2 CROSS-SECTION OF BOX UNIT

The first stage of construction involved building a temporary support platform to enable a crane to get from the barge to the waterline. This ramp was built level with the top of the concrete box units. A crane travelled down this ramp and proceeded to drive the two pile bents (Figures 3 and 4). Each pile bent consisted of two steel H piles at 1 metre centres. Precast unreinforced concrete protective jackets were then jettied around the H piles to provide scour protection (Figure 5).

The second stage involved placing precast reinforced concrete headstocks (Figures 6 and 7) on top of the pile bents, levelling them off by means of a special level adjustment device, locking them into place to prevent their displacement by waves, and then grouting them up to form a fixed connection with no pile steel exposed (Figure 7).

The third stage in the construction process necessitated the crane lifting up a box unit which had been wheeled down from the casting yard to a position behind the crane, swinging it around and placing it on the headstocks (Figure 9). The stainless steel bolts in the headstocks were fed through slotted holes in the base of the box units and stainless steel pods clamped the box units to the headstocks once in place (Figure 8). The slotted holes accounted for any longitudinal misalignment of the piles.

STAGE 1—DRIVE PILES FROM PREVIOUSLY
ERECTED BOX UNIT AFTER SETTING TO
REQUIRED LEVEL. DROP SHORBRACE UNIT
INTO HOLES IN STIFFENERS AND FIT
RETAINING NUTS (SEE CONNECTION
DETAILS ON DWG. N° 1956/104.)

SUGGESTED CRANE
UNIT TO BE A
N C K 605 WITH
915 WIDE TRACKS

PILING GUIDE
CONSTRUCTED
TO SUIT.

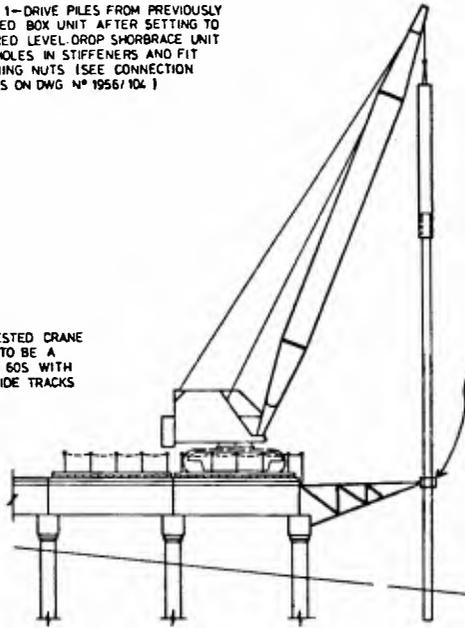


FIGURE 3 STAGE 1 CONSTRUCTION PROCEDURE



FIGURE 4 PILE DRIVER BEING LIFTED INTO POSITION



FIGURE 5 JACKETS BEING JETTED IN AROUND PILES

STAGE 2—DROP CONCRETE PIPE OVER PILE AND
JET TO REQUIRED LEVEL. DROP R C CAGE
DOWN PIPE. LIFT PRECAST HEADSTOCK
UNIT INTO PLACE AND FIX DOWN WITH
CLEAT SEAL BETWEEN HEADSTOCK AND
CONCRETE PIPE AND PLACE GROUT UP TO
TOP OF HEADSTOCK.

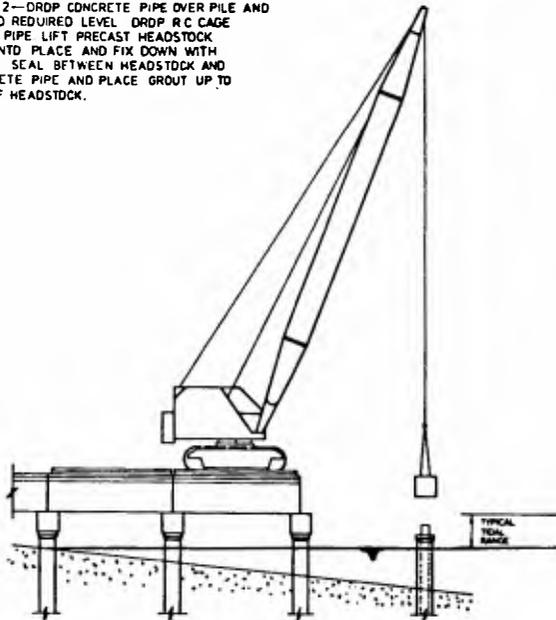


FIGURE 6 STAGE 2 CONSTRUCTION PROCEDURE

This procedure was repeated for a total of nine box units (each approximately 4.9 metres long) and a special precast outlet unit was then placed on four piles at the seaward end (Figure 10). This outlet unit was designed to prevent blockage of the outlet should the shoreline extend out to its previous level. At the shoreward end, a transition piece connected the box units to the existing circular 1.83 metre diameter reinforced concrete stormwater pipes.

STAGE 3--WHEN IN-SITU CONCRETE IN HEADSTOCK HAS ATTAINED A STRENGTH OF AT LEAST 30 MPa, LIFT PRECAST BOX UNIT INTO PLACE AND BOLT DOWN. ADVANCE CRANE AND REPEAT PROCESS.

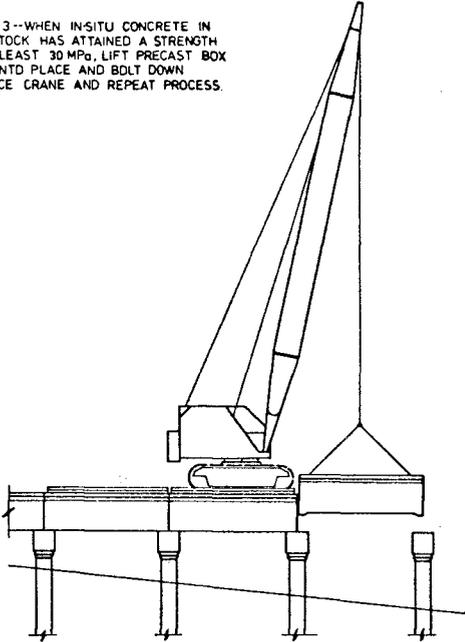


FIGURE 9 STAGE 3 CONSTRUCTION PROCEDURE

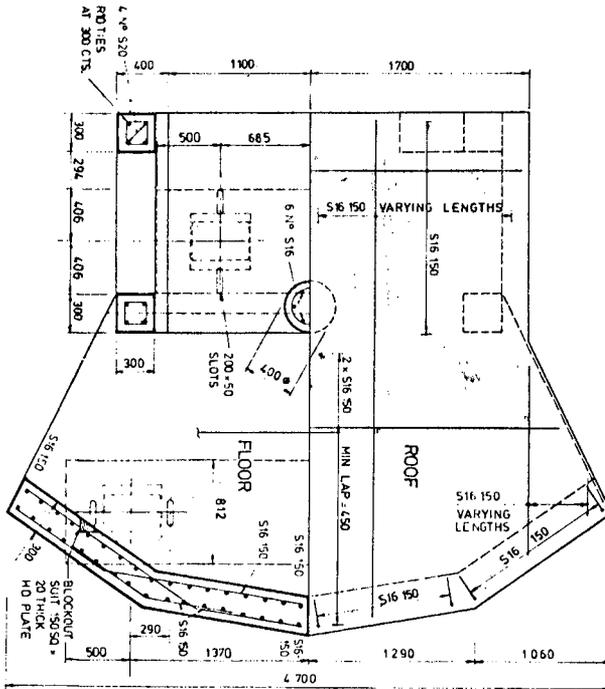


FIGURE 10 SPLIT PLAN OF OUTLET UNIT

In retrospect, the construction procedure worked very well. However a short time after the sixth box unit had been placed, reasonably heavy seeps were experienced. This particular unit broke up in the region of the holding down bolts. Evidence of site stiff present at the time indicated that the nuts of the holding down bolts had worked loose and the unit had been pounding up and down under the action of the waves. It was at this stage that the possibility of high shock pressures on the slab immediately adjacent to the headstocks occurred to us, and a rough model test was carried out to indicate the magnitude of this effect. The tests indicated that the wave pressures on the underside of the slab were sufficiently high (of the order of 60 KPa prototype) to warrant a full scale model investigation. Details of this investigation are presented in Section 3.

At the same time as the work was being carried out to determine what wave loading to use in checking the box units, further investigation and analysis was carried out to determine the inherent strength of the box units. It had been assumed in the design stage that the

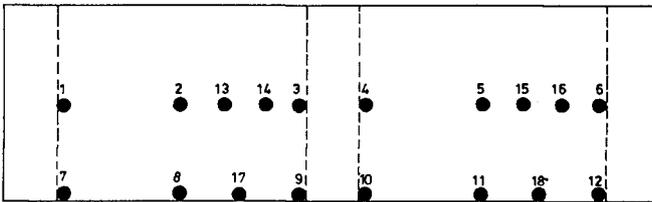
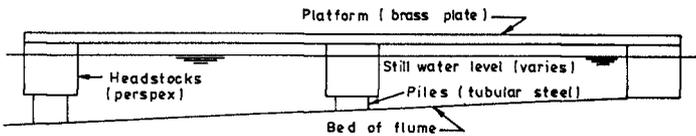
outlet unit would shield the box units from direct wave attack and no allowance had been made for any vertical wave loading. The box units had been designed for a static water pressure of 12 kPa and were only nominally reinforced in the base slab. A finite element analysis was carried out for a wave loading of around 360 kPa (based on an increase in wave energy as water depth increased) and for the crane loading during construction. The analysis revealed a shear weakness in the construction case and indicated that for the assumed wave loading, the box unit would stay intact but with severe cracking occurring. Both these problems were exacerbated by the Contractor's omission of reinforcement in critical areas. The problem of the shear weakness during construction was easily remedied by the insertion of packing plates and further holding down bolts were put in to enable the bottom of the box units to act more as a two way slab and thus have greater flexural strength under wave attack.

The outfall was completed on this basis and it was decided to re-evaluate the question of further strengthening when the detailed model tests were completed.

3. HYDRAULIC MODEL INVESTIGATION

3.1 Model and Instrumentation

The nature of the breaking wave uplift pressures on the underside, or soffit, of the stormwater outfall was investigated in a laboratory wave flume 0.9 m wide using a model of two bays of the stormwater outfall constructed at a scale of approximately 1 in 12 (Figure 11).



NOTE

1. Location for transducer and number shown thus 

FIGURE 11 MODEL OF OUTFALL

To enable measurement of the uplift pressure a total of 18 holes were drilled through the horizontal platform of the model, comprising 9 through the centre-line and 9 along the edge, in which could be positioned a pressure transducer flush with the platform soffit. Only one single pressure transducer was used throughout the investigation and therefore simultaneous measurement of uplift pressures over the platform could not be achieved.

The pressure transducer was a Stothom differential pressure transducer Model PM1311C. This transducer is a flush diaphragm construction with a working range of ± 175 cm H₂O and natural frequency 3,500 hertz. The response of the transducer to wave pressures was amplified and displayed on a Tectronix 912 storage cathode oscilloscope via a low pass filter. The low pass filter attenuated the effects of component frequencies above 1,000 hertz which it was thought could excite vibration of the transducer at its natural frequency.

3.2 Experimental Conditions and Procedure

The model was placed centrally in the wave flume on a beach slope of 1 in 20, approximately 17.5 m from a periodic progressive wave generator. Throughout the testing the model wave period was maintained constant at 2.3 seconds (8 seconds prototype). Incident waves were always made to break seaward of the model by variation of the stroke of the wave paddle. The wave motion impinging on the model was therefore the surge formed after breaking.

The work of previous researchers had shown that wave pressures exerted on horizontal platforms (and vertical walls) were characterised by an initial impact pressure of high magnitude and short duration followed by a pressure of longer duration and lesser magnitude. The nature of the initial short duration impact pressure was the prime concern of this investigation and was studied in two stages. The first stage was aimed at establishing temporal characteristics of the impact pressure and involved simultaneous measurement of both impact pressure rise time and magnitude. Readings were taken for only the centre-line transducer locations and for still water levels 10 mm and 20 mm below the soffit of the platform. The elevation of the soffit above the bed of the flume was such that the most seaward headstock rested on the bed (Figure 1).

The second stage of testing was concerned with determining the distribution of the magnitude of the impact pressure over the platform as well as a study of the slowly varying pressure. In this case readings were taken at each of the 18 locations for the transducer. The difference between the soffit and still water level was varied, in steps of 10 mm, between a lower water level value at which no contact occurred between the incident wave and the soffit and an upper water level value at which impact pressures ceased to occur (see Section 1.3.4). In addition, two elevations of the

platform soffit above the bed of the flume were considered. Initially the elevation of the soffit was set as described for the first stage of testing. A second series of tests was then undertaken with the soffit raised 25 mm by insertion of spacers into the support piles.

Both the magnitude and rise time of the initial impact pressure were subject to considerable variation from impact to impact for a given test condition. In order that meaningful average values were obtained 20 readings of impact pressure magnitude and rise time were taken for each combination of transducer location, difference between soffit and still water level and soffit elevation above the bed. The magnitude of the slowly varying pressure was much more regular and a representative value was taken from the display of 3 to 4 waveforms only.

Over the range of conditions studied in the model investigation the variation in the breaking wave height in the model was from 7 cm to 19 cm (0.85 m to 2.3 m prototype) and in the water depth along the structure was from 2 cm to 15 cm (0.25 m to 1.8 m prototype).

3.3 Results and Discussion

3.3.1 General

Although the experimental equipment comprised essentially only a single pressure transducer and storage cathode ray oscilloscope, several informative and interesting conclusions could be drawn from the hydraulic model investigation.

As expected, when impact occurred, and irrespective of the location of the transducer on the platform, the recorded pressure in general comprised an initial impact pressure of potentially high magnitude and short duration (of the order of milliseconds) followed by a slowly varying pressure of longer duration, which was typically first positive then negative.

The nature of the slowly varying pressure is fairly well understood and can be related to the hydrostatic effects with due allowance made for the influence of vertical fluid acceleration. It is therefore proposed only to discuss here some of the important aspects of the impact pressure. Accurate measurement of the impact pressure is difficult because of its extremely short rise time and the finite size of the pressure transducer's sensitive diaphragm. Together these factors may lead to a so called "transducer area defect", related to the transducer's spatial resolution (French, 1969).

French proposed that a transducer would not accurately record a pressure distribution unless the characteristic half-length of the pressure pulse (the product of the rise time and wave celerity) was greater than about three times the radius of the pressure transducer's sensitive diaphragm. If this was not the case a record of

time-dependent pressure would be produced in which the recorded pulse was of longer duration and of lesser amplitude than a record produced by a transducer of vanishingly small area or simply of somewhat smaller area. A correction procedure developed by French increased the recorded impact pressures in his study by a few percent to as much as 50% although it is stated that the procedure cannot be applied with certainty.

The Statham PM131TC transducer used in this investigation did not satisfy the above criterion for "accurate measurement" proposed by French. As the purpose of the investigation was to establish the nature of the breaking wave impact pressures no attempt has been made to correct the results for the above factors. It is therefore likely that the results presented in the following sections may tend to underestimate the magnitude of the impact pressure and overestimate its rise time. Use of the low pass filter could be expected to contribute to this tendency.

3.3.2 Rise Time of Impact Pressure

The rise time of the impact pressure was defined as the duration of time between the initial sharp rise of the pressure above static level and the peak of the pressure response.

For the centre-line gauges and the two water levels tested the average rise time of the impact pressure varied between 1.8 milliseconds and 8.5 milliseconds. Although subject to considerable variation from impact to impact there did appear to be a relation between the rise time and magnitude of the impact pressure, such that the shorter the rise time the higher was the magnitude of the impact pressure. This relationship has in fact been reported by previous researchers, notably Bagnald (1939) and Rass (1954), who found that the area enclosed by the pressure-time curves tended to approach, but never exceed, a definite value. This area of the pressure-time curves up to the peak of the impact pressure was found to be a fraction of the wave momentum and is thought to be related to the destruction of the original momentum of the kinetic mass involved in the impact.

3.3.3 Longitudinal Variation of Impact Pressure Head

Figure 12 shows the variation in magnitude of the impact pressure head along the centre-line of the structure for the two elevations of the soffit relative to the bed of the flume. Impact pressure head is measured in centimetres of water and each plotted point represents the average of twenty readings of the impact pressure for a given difference between the soffit and still water level.

It should be noted that the plotted points do not represent a simultaneous envelope of impact pressure on the structure. The impact pressure is in fact considered to be a phenomenon travelling with the line of initial contact between the wave and the soffit and at any one time to be confined to this immediate vicinity. (This was

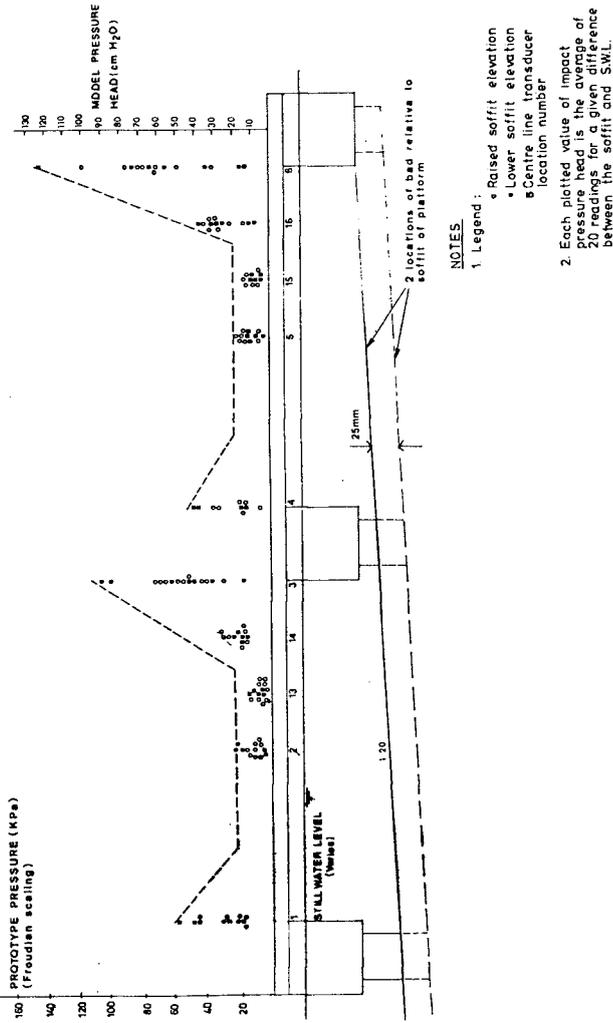


FIGURE 12 LONGITUDINAL VARIATION OF IMPACT PRESSURE HEAD

first proposed by French and confirmed by observation during this investigation.) An estimate of the spatial extent over which the impact pressure acts can be obtained from the product of the wave celerity and the rise time. From the results of this investigation the extent is of the order of only millimetres (model). This means that only very localised regions of the structure, each of wave length or short, are subject to the impact pressure at any one instant. Quite a considerable length, however, could be subject to the slowly varying pressure at any one time.

The effect of the headstocks on the magnitude of the impact pressure head is immediately evident. Impact pressures are higher in the regions immediately seaward and shoreward of the headstocks with the effects most pronounced immediately seaward of the headstocks. (Note that the plotted points represent pressures caused by incoming waves and not backwash. The effect of backwash for the conditions tested was found to be negligible.) The influence of the headstocks appears to extend over a region either side of the headstock, equal to about the projection of the headstock below the soffit.

The highest average magnitude of the impact pressure head recorded in the model was 123 cm H_2O (or about 12 kPa) at transducer location 6 immediately seaward of the most shoreward headstock (Figure 12). If scaled according to Froude's Law, which is generally considered to overestimate pressures of this type (Richert, 1974), a prototype pressure of 150 kPa would be predicted. These pressures are clearly extremely high in comparison to normal design loads and their magnitude alone suggests that they would be of concern in the design of prototype structures. Before their effect on a structure could be evaluated, however, the structure's response to the spatial and temporal characteristics of the pressures would need to be established.

In regions of the platform remote from the effect of headstocks the magnitude of the impact pressures are much lower. The results of this investigation are in agreement with the work of previous researchers who studied impact pressures on horizontal platforms (without vertical obstructions) and found that the magnitude of the impact pressure head rarely exceeded 5 times the height of the incident wave above still water level (French, 1969; Wong, 1970; Massel et al, 1978).

3.3.4 Variation of Impact Pressure Head with Difference Between Soffit and Still Water Level

Figure 13 shows the manner of the variation of average impact pressure head with difference between the soffit and still water level for transducer locations 6 and 3. The manner of the variation is typical for each of the transducer locations but is best illustrated by the results of locations 6 and 3 due to the wide range in the magnitude of the impact pressure head recorded at these locations.

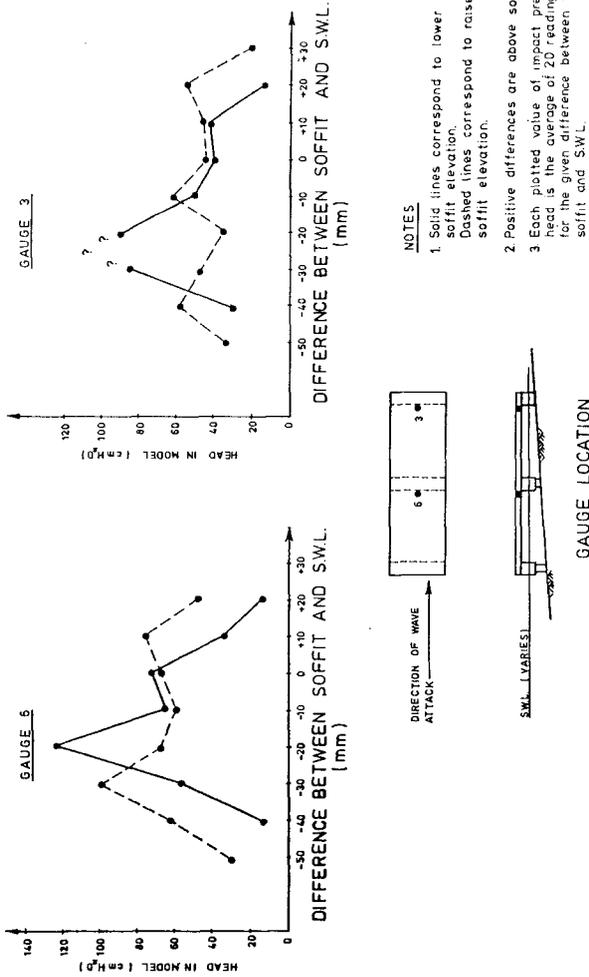


FIGURE 13 VARIATION OF IMPACT PRESSURE HEAD WITH DIFFERENCE BETWEEN SOFFIT AND S.W.L.

The highest single impact pressure recorded in the model was 300 cm H_2O , also at transducer location 6, and if scaled in the same way corresponds to about 360 kPa in the prototype.

Most evident from Figure 13 is that impact pressures at a location on the platform occur only over a certain range of differences between soffit and still water level (or alternatively, for a certain range of water depths). The lower limit of the range is the trivial case of when the water level is so low that no contact is made between the incident wave crest and soffit of the platform. The upper limit of the range appears to be for the case of when there is no air gap between the body of water in front of the incident wave and the soffit of the platform. In this case the impact pressure appears to be "drowned" and the record of wave pressure comprises the slowly varying pressure only. This observation lends support to French's conclusions that impact pressures on a horizontal platform are associated with a wave propagating into an empty air filled space (French, 1969).

An important inference from the above results is that providing wave activity is present and the water level is within the "critical range" wave impact pressures will always occur and furthermore will continue to occur, at the wave frequency, until the water level varies outside the limits of the "critical range" or the waves themselves cease to propagate. This is in direct contrast to breaking wave impact pressures on vertical obstructions which are extremely sensitive to the form of the breaking wave at the point of contact with the obstruction and are therefore by nature infrequent.

4. CONCLUSIONS

The main conclusions of the hydraulic model investigation are:

- (1) Short duration impact pressures occurring on the soffit of the precast units vary in magnitude along the unit. Prototype pressures predicted by Froude's law are of the order (maximum) of 150 kPa seaward of headstocks, 60 kPa shoreward of headstocks and 20 kPa in the central region between headstocks. (It should be remembered that the equivalent prototype height of incident breaking waves was of the order of only 1 to 2 metres.) The region of influence of the headstocks appears to extend from the face of the headstock for a distance about equal to the projection of the headstock below the soffit of the units.
- (2) The rise time of the impact pressure is extremely short, of the order of milliseconds and observation during testing confirmed that the impact pressure is a phenomenon confined to the immediate vicinity of the wave front and travelling with the wave velocity. These two aspects imply that only a narrow lateral strip of the box units is subject to wave impact loading at any one instant.

- (3) Impact pressures on the soffit at the precast box units occur for only a certain range of water levels and can be related to the difference in elevation between the soffit of the box units and the still water level. Within the range of water levels there is a "critical" water level for which maximum impact pressure occurs. Far lower or higher water levels the impact pressure is reduced even though for the case of the higher water level the breaking wave height is increased. For the particular conditions studied in this investigation the water levels within which impact pressures occurred extended over a vertical range of about 65 mm in the model, corresponding to approximately 0.75 m in the prototype.

The above conclusions have the following implications with respect to the prototype:

1. The wave loading predicted by the model tests is much higher than that assumed in design. The finite element analysis showed that the structure can safely withstand an overall pressure of around 10 kPa on the underside of the units combined with an impact pressure peaking from 10 kPa at a distance equivalent to the headstack depth away from the headstack to a maximum of around 60 kPa in the region of the headstack. This is significantly less than the maximum pressure envelope predicted by the model tests.



FIGURE 14 WAVE IMPACTING ON OUTLET UNIT

2. Because of the location of the soffit of the units at mean sea level combined with a tidal range of 1.33 metres it is extremely likely that the units have in the two years since completion been frequently subjected to wave forms which the model tests indicate produce high impact loading (Figure 14). For example 8 second waves over a tidal period of 6 hours represent 2700 waves and a significant proportion of these would strike the headstock when the water level was at or around the critical level.

Despite the above the structure has remained intact and a recent inspection revealed no obvious signs of distress. One can only assume from this fact that:

- (i) Non-idealised conditions in the prototype and/or the use of Froude's law result in the model tests overestimating actual prototype wave impact pressures.
- (ii) The response of the structure to the temporal and spatial disruption of actual wave pressures is such that significant redistribution and damping of internal stresses result.

It is likely that in practice a combination of the above factors probably occurs. The relative importance of these two factors is expected to become clearer when a prototype wave pressure study of the outfall (planned to be undertaken shortly by the Department of Public Works, NSW) is completed.

The aim of the paper has been to highlight the manner in which site-specific conditions govern design and construction in the surf zone, and in particular the need for wave loading to be fully evaluated in addition to normal functional loads. The results of the model tests have given the authors a valuable insight into the nature of wave loading on horizontal platforms in the surf zone and, whilst several questions still remain unanswered, it is hoped that the paper will be of assistance to designers and provide the basis for a better understanding of the performance of structures located in the surf zone.

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