The overtopping of seawalls

A comparison between prototype and physical model data

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Summary

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Significant sections of the United Kingdom coastline are protected from flooding by sea walls. These sea walls, which are commonly fronted by sand or shingle beaches, have a wide range of cross-sections ranging from vertical faces to relatively shallow sloping structures with gradients approaching 1:5. Whatever the sea wall cross-section, the selection of the crest elevation is of primary importance in determining the overtopping discharge performance of the structure and hence the susceptibility of the hinterland to flooding.

Traditionally the overtopping performance of simple sea wall cross-sections has been determined from empirical equations whilst complicated cross-sections have been assessed using site specific physical models. The empirical equations employed to estimate overtopping have generally been derived from physical model data obtained during wave flume tests at scales ranging from 1:15 - 1:30.

A concern with using empirical equations derived from wave flume tests is that the physical model does not reproduce all the physical effects present at prototype sea walls. The most obvious deficiency of physical models is the omission of onshore winds which generally accompany storm events. This omission has two major influences. Firstly the onshore wind raises the still water level at the structure (called wind set-up) and secondly often causes water thrown into the air to be blown over the sea wall. This latter influence may be particularly important for vertical or near vertical walls and slopes topped with a recurve where water reflected from the structure is commonly thrown up into the air.

A research project was therefore undertaken to measure overtopping at prototype sites. The aim of the study was to compare prototype discharges with those obtained using physical model techniques. Two sites were subsequently selected on the North Wales coast to complete the fieldwork exercise. The first was a vertical wall whilst the second was a 1:4 simply sloping sea wall.

This report discusses the selection of the sites, the measurements made and how they compare with existing prediction methods. The study forms part of a continuing programme of research into the behaviour of sea walls being carried out at HR Wallingford with support from the Ministry of Agriculture, Fisheries and Food under Commission FD0201, Marine Flood Protection, Sea Defence Structures.

For further information about this study, please contact Dr D M Herbert of the Coastal Group at HR Wallingford.

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1 Introduction

1.1 General

HR Wallingford have been contracted by the Ministry of Agriculture, Fisheries and Food (MAFF), under Commission FD0201, to investigate the overtopping discharge performance of prototype sea walls and compare their performance with existing prediction methods derived from physical model test results. This document outlines the methodology behind the fieldwork, the fieldwork deployment techniques employed in order to fulfil the aims of the study, the test results and the conclusions drawn.

1.2 Background

Over the last twenty years HR Wallingford has been involved in a continuous research programme into the overtopping performance of sea walls. This programme has resulted in the publication of design guidelines concerning the overtopping of plain sloping and bermed sea walls (Reference 1), sloping sea walls topped with a return wall (Reference 2) and vertical walls (References 3 and 4). All of the empirical equations used in the design guidelines were derived from two and three-dimensional random wave physical model studies generally carried out at scales ranging from 1:15 - 1:30.

The physical models used in obtaining the overtopping data were designed according to the Froude scaling law. This law states that all linear dimensions are reproduced to a geometric scale, λ, whilst time is scaled to $\sqrt{\lambda}$. Use of the Froude law, in combination with the range of scales employed in this research, ensured that the quantity of green water discharging over the sea wall was adequately reproduced in the model. The models did not, however, reproduce certain other effects present at prototype sites.

The principal omission from the physical models was the effect of onshore winds. Onshore winds have two major effects causing an increase in still water level at the structure (called wind set-up) as well as causing water thrown into the air to be blown over the sea wall. The effect of onshore winds was deliberately omitted from the models because of two major practical difficulties. The first difficulty includes the reproduction of identical wave conditions with and without wind, since the addition of wind modifies the wave conditions generated in the model, whilst the second problem involves the scaling of water droplets, which are an almost identical size in the model as in the prototype. A further minor consideration regarding the effect of wind is that when it is included waves will tend to break earlier, and hence further away from the sea wall, than if wind is omitted.

Although anecdotal evidence suggests that overtopping due to spray is small in comparison to the total discharge overtopping a sea wall, little research has been completed to confirm this. A research project was therefore initiated to measure discharges at prototype sea wall sites. This presented several difficulties, not least amongst them selecting sites where significant overtopping was likely to occur. It was the aim of the project to compare the prototype measurements with existing prediction methods and thus ultimately quantify the accuracy of physical modelling techniques.

1.3 Report outline

Following this brief introductory section, Chapter 2 of this report describes the site selection criteria for the fieldwork. Chapter 3 details the fieldwork deployment methodology and measurements whilst Chapter 4 outlines the results of and inferences drawn from the test measurements. The conclusions of the study are given in Chapter 5.

2 Site selection

2.1 Selection criteria

Although the United Kingdom (UK) has many hundreds of miles of sea walls the selection of potential sites for a fieldwork deployment exercise is in fact severely limited. The most stringent criteria is to find a site where significant overtopping occurs on a regular basis. Fieldwork exercises are expensive to undertake and any deployment of equipment must therefore be accompanied by a relatively good chance of obtaining useful information within the period of the project. Ideally two or more sea walls were required in the same vicinity that fitted the following criteria:-

i) the sea walls were regularly overtopped
ii) the sea walls had significantly different cross-sections.

Meeting these criteria would ensure that the most cost effective approach to any fieldwork deployment was taken.

Site selection was further complicated by the need to obtain permission from the owner of the sea wall for any deployment. Many of the sites considered for use had public access immediately in front of and behind the sea wall. This meant that, unless a non-intrusive means of measuring overtopping could be derived, any equipment would either limit public access or be liable to damage from vandals. Furthermore it was preferable that any sites selected should be relatively close to HR Wallingford for ease of deployment and equipment maintenance.

A vertical wall site at Colwyn Bay in North Wales was identified as commonly suffering significant overtopping when onshore winds coincide with spring tides. The structure, which affords protection to the Old Colwyn area, has a promenade and roadway immediately behind its crest. However the roadway at the eastern end of the sea wall only provides access to the promenade and is not a major traffic artery. The roadway is closed to vehicular traffic during storm events, which occur about a dozen times a year, because of the high overtopping discharges and large quantity of shingle thrown over the sea wall crest.

During the winter of 1993/94 a new sewer pipe was being installed behind the sea wall and hence the promenade and roadway were closed to the public in order to allow the contractor access. Confirmation from Colwyn Borough Council and the contractor that any fieldwork deployment would not interfere with the sewer pipe works provided the ideal opportunity to measure overtopping in the field and Old Colwyn was thus selected as one of the deployment sites.

It was hoped that a further deployment site could be found close to the Old Colwyn wall so that discharges could be recorded at a second structure without committing significant extra resources. A search for an alternative sea wall cross-section in North Wales which commonly suffered overtopping was unsuccessful. However a 1:4 sea wall topped with a small wave recurve was identified at Prestatyn approximately 12 miles to the east of Old Colwyn. This sea wall had only relatively recently been constructed and subsequently offered a high degree of wave protection against overtopping. However, the 1:4 slope incorporated a 3.5m wide berm midway down its length. This berm is positioned at an elevation over 1m above the level of mean high water spring tides. The 1:4 sea wall at Prestatyn was therefore selected as the alternative site with the intention of deploying equipment on the berm and measuring overtopping until the berm became inundated with water.

The locations of the Old Colwyn and Prestatyn sea walls are illustrated in Figure 1.

2.2 Description of sites

2.2.1 Old Colwyn

The Old Colwyn shoreline faces northwards and is situated approximately 8 miles to the south east of Great Ormes Head. The frontage is exposed to the north and northeast but is partially sheltered from the northwest by Rhos Point. The coastline is characterised by a mainly sandy lower beach with some patches of cobbles. The upper beach, the width of which varies along the frontage, is formed of shingle.

A vertical stone faced sea wall, approximately 3m high and originally constructed in about 1900, is sited at the rear of the beach. A typical cross-section of the sea wall, which is backed by a promenade and roadway, is illustrated in Figure 2. A combination of lower beach levels and a reduced sea wall crest elevation means that overtopping is significantly greater at the eastern, rather than the western, end of the frontage.

2.2.2 Prestatyn

Prestatyn is situated on the north east coast of Wales, immediately to the west of the Dee Estuary, and is exposed to significant wave action from the north and northwest. The whole of the Prestatyn frontage comprises a wide sandy beach backed by a series of sea walls. Over the last decade long stretches of the old mass concrete sea walls have now been replaced by a new revetment. A cross-section through this revetment is shown in Figure 3.

The revetment was constructed at a slope angle of 1:4 and was topped with a relatively small wave recurve. The upper half of the revetment was comprised of open stone asphalt whilst the lower half was constructed from asphaltic concrete. A 3.5m wide berm is sited at the top of the asphaltic concrete slope. The installation of the revetment has been accompanied by the construction of rock groynes. The rock groynes, combined with the shallower slope of the revetment when compared to the old mass concrete sea walls, has resulted in a build up of the beach and the revetment toe is now buried.

3 Fieldwork deployment

3.1 Methodology

Significant thought was given to the means of measuring prototype overtopping discharges. The first difficulty to be considered was whether to undertake a long term or short term deployment. A long term deployment could take place over the winter months in an effort to capture all the storm events. Alternatively a short term deployment of a few days could be undertaken when storm events and high tidal levels coincided and hence overtopping was likely. It was subsequently considered that it might be necessary to undertake several short term deployments in order to obtain a data set of sufficient size.

By far the majority of sea walls around the UK coastline are open to the public. Any long term deployment of equipment, unless completely non-intrusive, would therefore be susceptible to vandalism. A non-intrusive system involving placing flow meters in storm water drainage pipes was considered but was discounted mainly because of the difficulties of finding a suitable site.

It was therefore concluded that an intrusive system should be used and with it the acceptance that the relevant measurements would be undertaken in a series of short term deployments during storm events. Storm events were predicted by identifying spring tidal dates and monitoring the Meteorological Office Weathercall forecasting system during these dates for periods of storm activity.

The severity of the environment in which measurements would have to be made, combined with the limited number of measurement opportunities, meant that a reliable means of measuring overtopping was required. It was therefore decided to deploy a wave/tide recorder in order to measure the inshore wave conditions and water levels whilst using a large tank to capture overtopping water. The deployment of this equipment is outlined in more detail in Section 3.2.

The vertical wall at Old Colwyn did not include an upstand and hence the front face of the overtopping tank, which was lower than the other three sides, protruded above the crest of the structure. This was considered to be acceptable as it meant that the effective crest height of the wall had been increased.

A similar problem to that described above also existed at Prestatyn. In this instance an artificial 1:4 wooden slope was devised to provide a extension to the existing slope up to the lip of the overtopping tank sited at the rear of the berm.

3.2 Fieldwork measurements

3.2.1 General

The first fieldwork deployment exercise was completed in late January 1994. This deployment lasted five days including the time taken to assemble and dismantle the equipment. During observations of the wave activity on the berm of the Prestatyn sea wall it quickly became apparent that conditions were significantly more severe than during an earlier site inspection visit. Deploying equipment and personnel on the berm was considered dangerous and so the measurements at this site were abandoned.

At Old Colwyn the quantity of overtopping varied considerably along the length of the sea wall. This was due not only to a difference in crest elevations along the structure but also the variation in beach levels at the toe of the sea wall. A site was subsequently selected for the equipment deployment which was considered representative of the mean discharge along the length of the frontage.

Each series of overtopping measurements was completed over about three hours during periods of high tides. The overtopping measurements were not necessarily continuous as the large quantity of green water and shingle overtopping forced the work to be suspended. The overtopping was such that it was not only a danger to the personnel involved but it also completely inundated the measuring tank. In one period at the top of the tide overtopping was so great that spray was observed passing over the top of the lamp posts along the promenade and landing half way up the railway embankment on the landward side of the roadway. In light of the success of the measurements at Old Colwyn no further deployments were undertaken.

All of the equipment deployed was levelled into position prior to any measurements being recorded. In particular the relative elevations of the equipment to the sea wall toe and crest were obtained.

3.2.2 Overtopping

A large tank was positioned immediately behind the crest of the sea wall in order to collect water overtopping the structure. The tank, which was constructed on site, was 2.44m long, 1.22m high and 1.22m deep. The seaward side of the tank, which was lower than the other three sides, was 0.91m high.

The level of water in the tank was measured every minute using a float gauge positioned on the rear external wall. A lid was placed over the tank when it was filled to capacity and drain valves opened. Water pumps were also used to increase the rate of draining the tank. When the tank was empty the lid was removed and overtopping measurements restarted.

3.2.3 Wave conditions

Wave conditions close to the toe of the sea wall were measured using a DNW5 wave and tide recorder. This self-contained instrument incorporates a micro-processor controlled pressure transducer to measure the variation in water pressure above the unit. A sampling frequency of 2 Hertz was employed and the data obtained recorded on a removable solid state memory. Using this sampling frequency the DNW5 had the capacity to record 72 hours worth of data which was slightly less than the length of time of the deployment. The recorder was therefore only switched on during periods of high water levels and was switched off as the tide receded.

The DNW5 recorder was deployed on the beach fronting the sea wall and was sited about 40 metres from the toe of the structure. The cylindrical instrument, approximately 0.16m in diameter, was clamped in a steel frame which in turn was firmly anchored into the beach. The DNW5 was deployed with the full knowledge that the recorded wave conditions would include a contribution from wave reflections off the structure whilst the water levels would include the wind set-up component.

Analysis of the pressure record enabled both the wave and water level conditions at the site to be derived. The data was filtered using high and low pass filters to respectively separate the wave component (high frequency) from the water level record (low frequency). The wave record was analysed using spectral techniques in segments just over 17 minutes long (equivalent to 2048 data points). The resulting spectra were corrected for depth attenuation of the pressure signal to give the inshore significant wave height, H_s, and the mean, T_m, and peak, T_p, wave periods.

3.2.4 Water levels

Water levels at the site were recorded using the DNW5 wave and tide recorder as described above. The low frequency component obtained from the instrument was analysed in segments marginally over 4.25 minutes in length (equivalent to 512 data points). The data obtained during each segment was averaged to give a mean water level which subsequently used in the analysis of the test results.

3.2.5 Observations

The overtopping performance of sea walls is influenced by the angle at which the incident wave impinges upon the structure. Research work using long crested waves (Reference 1) has suggested maximum overtopping occurs for wave angles of 15° off normal. Recent research, however, using short crested seas (Reference 5) suggests that maximum overtopping occurs for normally incident waves.

A careful watch was therefore kept in order to ascertain the obliquity of the wave attack. The wave action appeared to be approaching the site from the north of west but diffraction around Rhos Point combined with refraction effects as the waves passed through shallower water caused a significant change in the angle of wave attack. At the shoreline the direction of wave energy was nearly perpendicular to the axis of the sea wall and at no time was more than 10° off normal.

4 Test results

4.1 Previous work

The majority of research into the overtopping of sea walls has been completed using physical models. The number of studies undertaken at prototype scales and/or designed to assess the effects of physical processes not reproduced in physical models is severely limited.

The Shore Protection Manual (Reference 6) quotes an equation applicable only to regular waves in order to quantify the effect of onshore winds on overtopping. Although the equation is unverified the Shore Protection Manual states that the formula is believed to give a reasonable estimate of the effects of onshore winds. This equation allows the derivation of a wind correction factor, K', which may then be multiplied by the calculated overtopping rate to give the expected prototype discharge. The equation states:-

$$K' = 1.0 + W_f \left((R_c / R) + 0.1 \right) \sin\alpha \tag{1}$$

where W_f is a coefficient dependant on the onshore component of the windspeed,
α is the structure slope angle to the horizontal,
R_c is the sea wall freeboard (the distance of the crest of the structure above still water level)
and R is the run-up distance.

The value of R_c/R ranges from $0 < R_c/R < 1$ and hence:-

$$1.0 + 1.1\ W_f \sin\alpha > K' > 1.0 + 0.1\ W_f \sin\alpha \tag{2}$$

The following values of W_f are proposed for use in equation (1):-

Wind speed (m/s)	W_f
0	0
13	0.5
26	2.0

Equation (1) illustrates that the effect of onshore wind increases as the steepness of the sea wall slope increases. This agrees with anecdotal evidence which suggests that more spray is thrown into the air for steeply sloping structures and especially for vertical walls (Reference 4). A comparison between the values of K' obtained for a vertical and 1:4 simply sloping sea wall under a 26m/s onshore wind is given below:-

	Vertical	1:4 slope
Minimum	1.2	1.05
Maximum	3.2	1.53

Further interrogation of equation (1) shows that for a given onshore wind speed and wave conditions, the wind correction factor, K', for a particular sea wall slope will be greatest when $R_c/R \rightarrow 1$ (ie R_c is large) and will be smallest when $R_c/R \rightarrow 0$ (ie R_c is small). However when R_c is large less overtopping will occur than when R_c is small. Hence the maximum onshore wind effect occurs when green water overtopping is small but as the quantity of overtopping increases the effect of onshore winds becomes insignificant.

Recently de Waal (Reference 7) has investigated the influence of wind on wave overtopping in a series of wave flume model tests on a vertical wall. Spray being thrown into the air was mechanically collected and the quantity compared with green water overtopping discharges. The influence of spray overtopping was discovered to be related to the relative crest height, R_c/H_s, (where H_s is the significant wave height) and the water depth at the toe of the wall, d_s. A spray transport factor, W_s, was defined as:-

$$W_s = \frac{\text{Total overtopping rate (green water plus spray)}}{\text{Green water overtopping rate only}} \tag{3}$$

The maximum value of W_s was found to be W_s=3.0 for structures in shallow water with a large relative crest height, R_c/H_s. This finding gave good agreement with the advice contained in Shore Protection Manual and outlined above. Despite the maximum value of W_s=3.0, de Waal showed that the effect of spray reduced dramatically for increasing water depths. In many cases values of W_s approaching 1.0 were derived implying that the influence of spray transport is negligible.

Although the number of studies into wind effects on overtopping is severely limited, more data is available to quantify the magnitude of wind set-up. Recently the CIRIA/CUR manual (Reference 8) has suggested the following method to calculate wind set-up, η_w, for constant water depths and wind fields:-

$$\eta_w = 0.5\ F\ C_w(\rho_{air}/\rho)\ U_w^2 / (gh) \tag{4}$$

where F is the fetch length,
C_w is the air/water friction coefficient which varies between 0.0008 - 0.003 depending on the wind speed,
U_w is the wind speed,
h is the water depth,
ρ, ρ_{air} are the density of seawater and air respectively
and g is acceleration due to gravity.

The CIRIA/CUR manual recommends that the above method should only be employed if local water level measurements are available for comparison as wind set-up is strongly affected by the nearshore bathymetry and coastal alignment.

4.2 Comparison of physical model and prototype data

In comparing the prototype measurements from Old Colwyn with empirical equations derived from physical model tests consideration had to be given to the following points:-

i) the method of analysing the fieldwork data in order to ensure that the method was broadly similar to that employed on the physical model data,

ii) the empirical equations with which the fieldwork measurements were to be compared.

A considerable number of authors have proposed methods to enable designers to estimate the overtopping discharge performance of vertical walls. Goda (Reference 9) initially completed research into vertical walls and proposed a graphical method, later extended by Herbert (Reference 3), in order to estimate mean overtopping discharge rates. Recently work by Franco et al (Reference 10) and Allsop et al (Reference 4) has resulted in the derivation of empirical equations. All of the physical model data upon which the above methods are based was obtained over recording intervals of several hundred waves.

In order to achieve the best possible comparison between the prototype and model results it was considered important to ensure that the analysis procedures were broadly similar. This aim, however, presented a particular difficulty in selecting the interval over which the overtopping discharge should be averaged.

The changing tidal conditions at the site means that selecting a long averaging interval, equivalent to say 500 waves, would result in significant different water levels at the beginning and end of the averaging interval. Alternatively the scatter of results would be very large if an overly short averaging interval is selected. An averaging interval equivalent to about 100 waves was finally decided upon. Analysis of the wave conditions at the site indicated a mean wave period of approximately 5-6 seconds and an averaging interval of 10 minutes was subsequently adopted. Some recording intervals were effectively less than the 10 minutes quoted above, especially those for the higher discharges, as they included the time taken to drain the tank.

The next decision to be made was which of the alternative prediction methods should be used for the comparison. It was considered that an accurate comparison using the graphical prediction method employed by Goda and Herbert would be difficult to achieve due to the high degree of interpolation that would be required. The work of Franco et al and Allsop et al both resulted in empirical equations of the form:-

$$Q_* = A \exp(-B\, R_c / H_s) \tag{5}$$

where $Q_* = Q/(g\, H_s^3)^{0.5}$
A and B are empirical coefficients
and Q is the mean overtopping discharge rate.

The work of Franco was applicable to vertical caissons in deep water and resulted in values of A=0.2 and B=4.3. Allsop, however, used data obtained in both deep and shallow water and quoted values of A=0.03 and B=2.05. Given that the Old Colwyn wall is in shallow water it was considered appropriate to use the work of Allsop for the comparison with the prototype data.

The test results from the deployment are presented in dimensionless form in Figure 4 along with the empirical prediction lines of Franco and Allsop. The first comment to make is the considerable scatter in the data set. This is not surprising as the nominal 10 minute recording interval (roughly equivalent to 100 waves) for the fieldwork data is less than would be used to obtain mean overtopping discharges from a physical model. The extra averaging that subsequently takes place in the physical model data set thus results in a reduced quantity of scatter.

Prototype data was obtained in the range $1 < R_c/H_s < 7$ which is significantly greater than the range of Allsop ($1 < R_c/H_s < 3.25$). Using the method of least squares a line of best fit, with the same format as equation (5), was calculated using all the data points and the following equation derived:-

$$Q_* = 0.00138 \exp(-0.654\, R_c / H_s) \tag{6}$$

For values of $R_c/H_s > 6$ the best fit line for the prototype data gave values of Q_* up to 3 orders of magnitude larger than the prediction line of Allsop. Conversely the line of Allsop predicted larger values of Q_* than the prototype data for $1 < R_c/H_s < 2$.

However the prediction line for the prototype data is somewhat misleading. The level of the minimum measurable discharge, coupled with the range of wave conditions that resulted in measurable overtopping, meant that the minimum value of Q_* was approximately 1×10^{-5}. Hence for larger values of R_c/H_s only the large values of Q_* were capable of being measured and the smaller values of Q_* were ignored. This tended to cause the line of best fit to be overly shallow.

The prototype data was therefore reanalysed using only data points in the range $1 < R_c/H_s < 4$. The value of $R_c/H_s = 4$ was selected as the nominal cut off point as beyond this it was not considered that a satisfactory range of Q_* was obtained. The revised line of best fit, also illustrated in Figure 4, produced the following equation:-

$$Q_* = 0.00475 \exp(-1.15\ R_c / H_s) \qquad (7)$$

As with the previous best fit line, the revised prediction line gave lower values of Q_* than Allsop for $R_c/H_s < 2$. Conversely for $R_c/H_s > 2$ the prototype data suggests larger values of Q_* than Allsop with the divergence increasing for increasing values of R_c/H_s. For a value of $R_c/H_s = 3$ the revised prototype prediction line gave values of Q_* 2-3 times greater then Allsop whilst at $R_c/H_s = 4$ this increase in Q_* was in the order of 5-6.

The results broadly agreed with the advice in Shore Protection Manual (equation (1)) and the work of de Waal which both predicted maximum increases of a factor of 3 in overtopping for large values of R_c/H_s. However both references did not predict any reduction in overtopping for smaller values of R_c/H_s. This may in part be explained by the overtopping tank being unable to cope with the very largest discharges and hence there was a tendency to slightly distort the data set at low values of R_c/H_s.

The fieldwork illustrated that under the most extreme conditions empirical equations derived from model studies gave an acceptable estimate of the likely discharge at the prototype site. However, as the severity of the conditions decreased, the level of discharge at the prototype structure was several times greater than that suggested by empirical equations.

4.3 Allowable overtopping discharges

Although not an objective of the fieldwork deployment some consideration was given to the likely dangers posed, as perceived by the author of this report, by the level of discharges overtopping the sea wall. The prototype overtopping measurements could not be compared directly to the perceived dangers as the front lip of the overtopping tank effectively increased the crest height of the sea wall. The measured discharges therefore had to be corrected so that they represented overtopping at the crest of the sea wall rather that at the lip of the tank. This correction was achieved in the following manner:-

i) the measured wave height and water level data was input into the revised empirical equation derived from the prototype data (A=0.00475, B=1.15) to give a predicted overtopping rate at the lip of the tank;

ii) a predicted overtopping rate was then obtained for the sea wall crest in the same manner as described above but using the reduced crest elevation;

iii) a correction factor, defined as the ratio of the predicted discharge at the sea wall crest to the predicted discharge at the overtopping tank, was then derived. This correction factor was subsequently applied to the measured overtopping value to give an equivalent discharge rate at the crest of the wall.

Presently accepted international guidelines (Reference 8) suggest the following admissible overtopping discharges for vehicles and pedestrians:-

<u>Vehicles</u>

Safe at all speeds	< 0.001 l/s/m
Unsafe at high speed	0.001 - 0.02 l/s/m
Unsafe at any speed	> 0.02 l/s/m

Pedestrians

Wet, but not uncomfortable	< 0.004 l/s/m
Uncomfortable but not dangerous	0.004 - 0.03 l/s/m
Dangerous	> 0.03 l/s/m

Recently it has been suggested by Franco et al (Reference 10) that the admissible discharges outlined above are overly conservative and may be increased by a factor of ten.

The maximum overtopping discharge measured during the deployment was 8 litres per second per metre length of sea wall (l/s/m) which gave an equivalent discharge at the wall crest of approximately 16 l/s/m. From observations made during the deployment it was considered that discharges in excess of 0.2 l/s/m might result in the loss of control of a vehicle driven at slow speed. This is one order of magnitude higher than the value suggested in Reference 8.

Work at the crest of the sea wall was able to proceed safely at discharges up to 0.1 l/s/m. For discharges in excess of this personnel could not safely be permanently positioned at the crest of the structure. It should be noted however that the critical discharge of 0.1 l/s/m was applicable to adults who were expecting to get wet and were dressed in protective clothing. A more stringent criteria approaching that of the presently recognised value of 0.03 l/s/m would apply to children.

5 Conclusions

This report describes a fieldwork deployment exercise undertaken to measure overtopping discharges at prototype sea wall sites. The aim of these measurements was to allow a comparison to be made between prototype data and data obtained from model tests, upon which present design guidelines are based.

The conclusions of the study are:-

1) Under the most severe conditions ($R_c/H_s < 2$) prototype discharges and discharges obtained from physical model studies were in good agreement. However, as the severity of the conditions decreased the level of discharge at the prototype structure was several times that suggested by physical model data. The divergence between prototype and model data increased as the ratio of R_c/H_s increased so that at $R_c/H_s = 4$ the prototype sea wall suffered 5-6 times more overtopping than predicted by empirical equations.

2) The measurements completed in the present fieldwork deployment exercise have been insufficient to quantify the effect of wind set-up. It is therefore recommended that the designer, when assessing the overtopping performance of sea walls using empirical equations, takes into account wind set-up by incorporating an allowance in the still water term in the relevant equation. The most appropriate means of completing this is to derive water levels from measured conditions which automatically include a wind set-up component. An alternative, but by no means as accurate a method, is to assess the wind set-up component of the water level using theoretical or empirical equations.

3) From the authors own personal experience of the fieldwork deployment it is suggested that the present internationally accepted admissible overtopping discharges for vehicular travel are too stringent. A limit of 0.1 l/s/m is suggested for vehicles travelling at slow speed. The authors perception of discharges that would pose a danger to pedestrians are similar to present accepted criteria.

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Figures

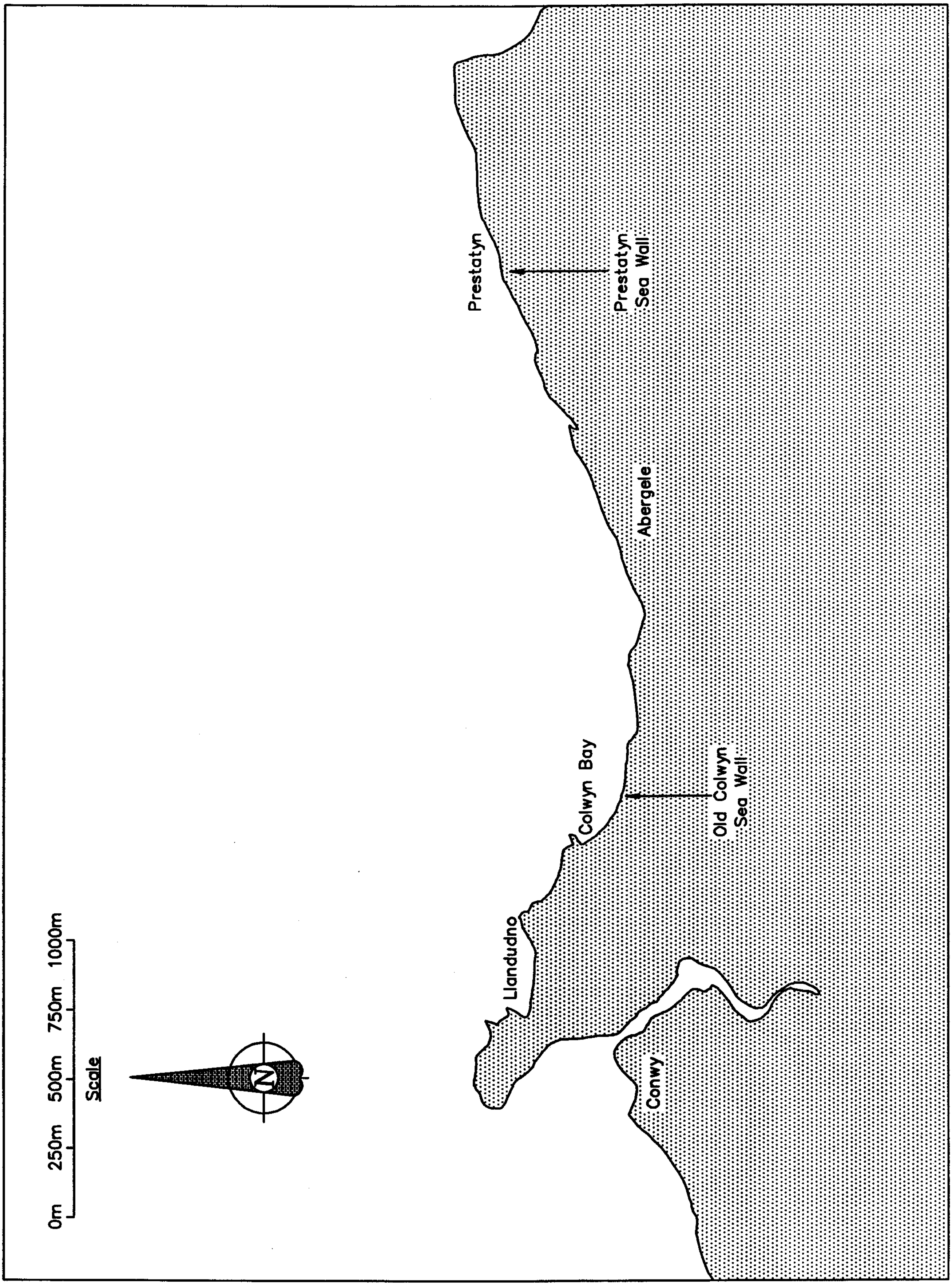

Figure 1 Location map

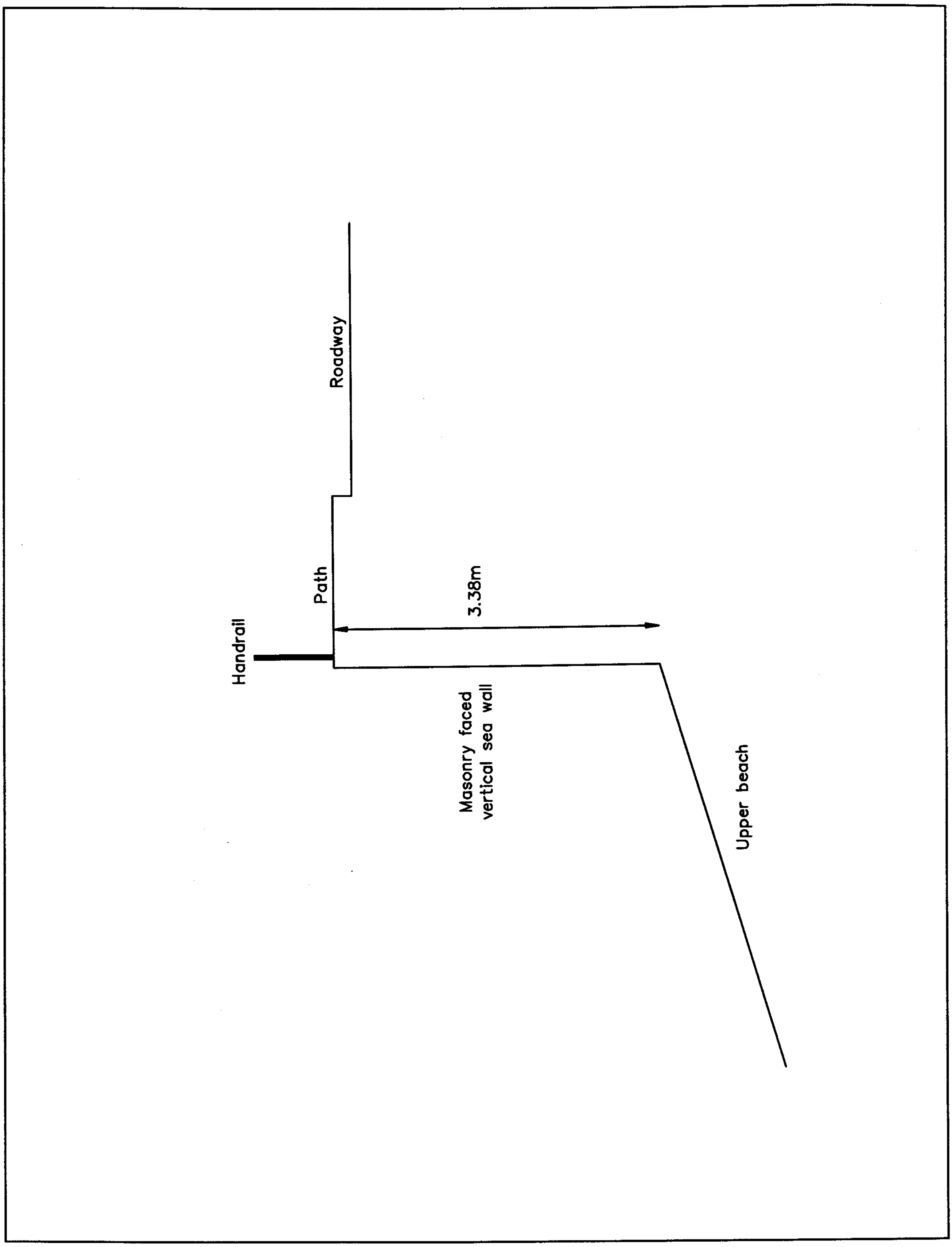

Figure 2 The Old Colwyn sea wall

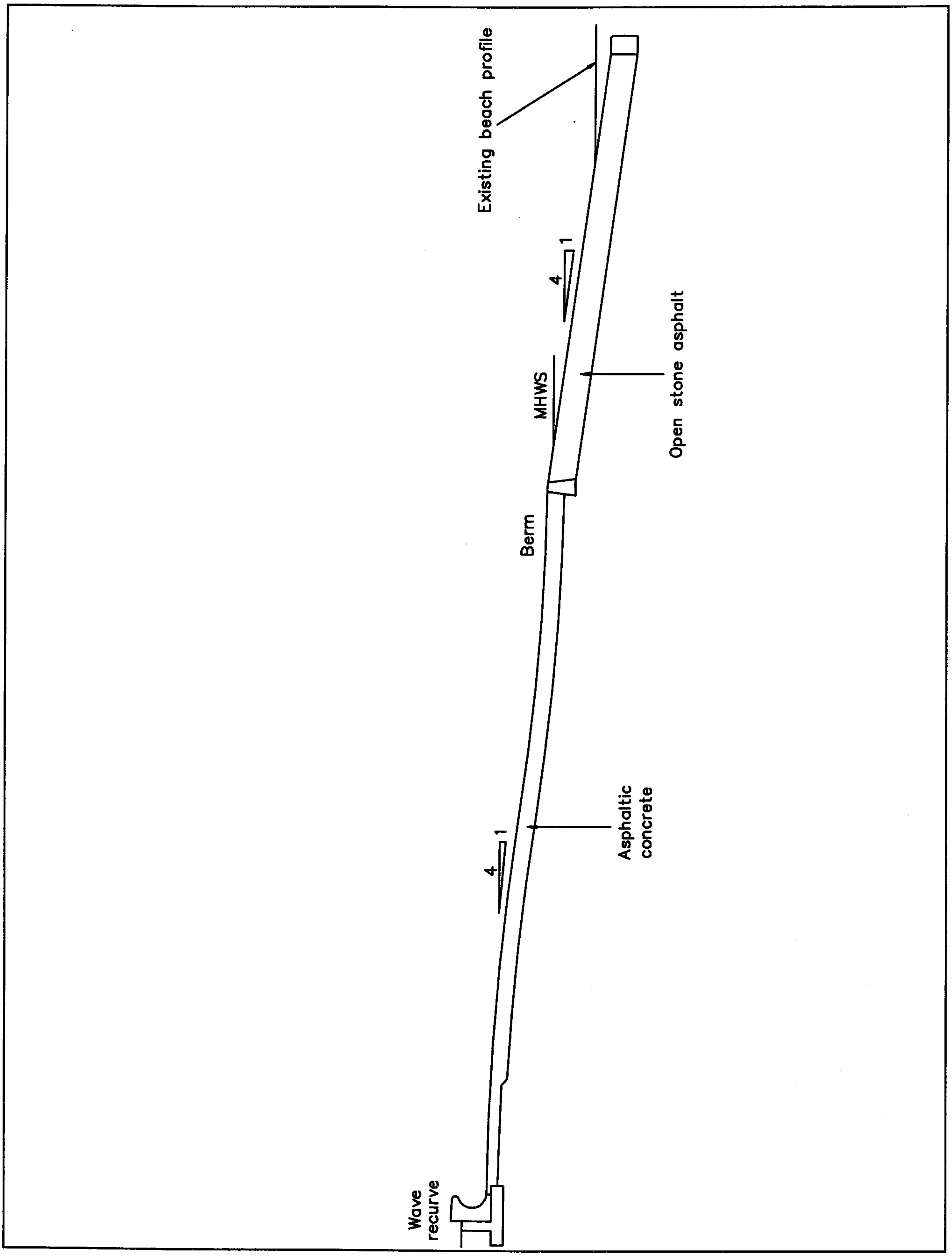

Figure 3 The Prestatyn revetment

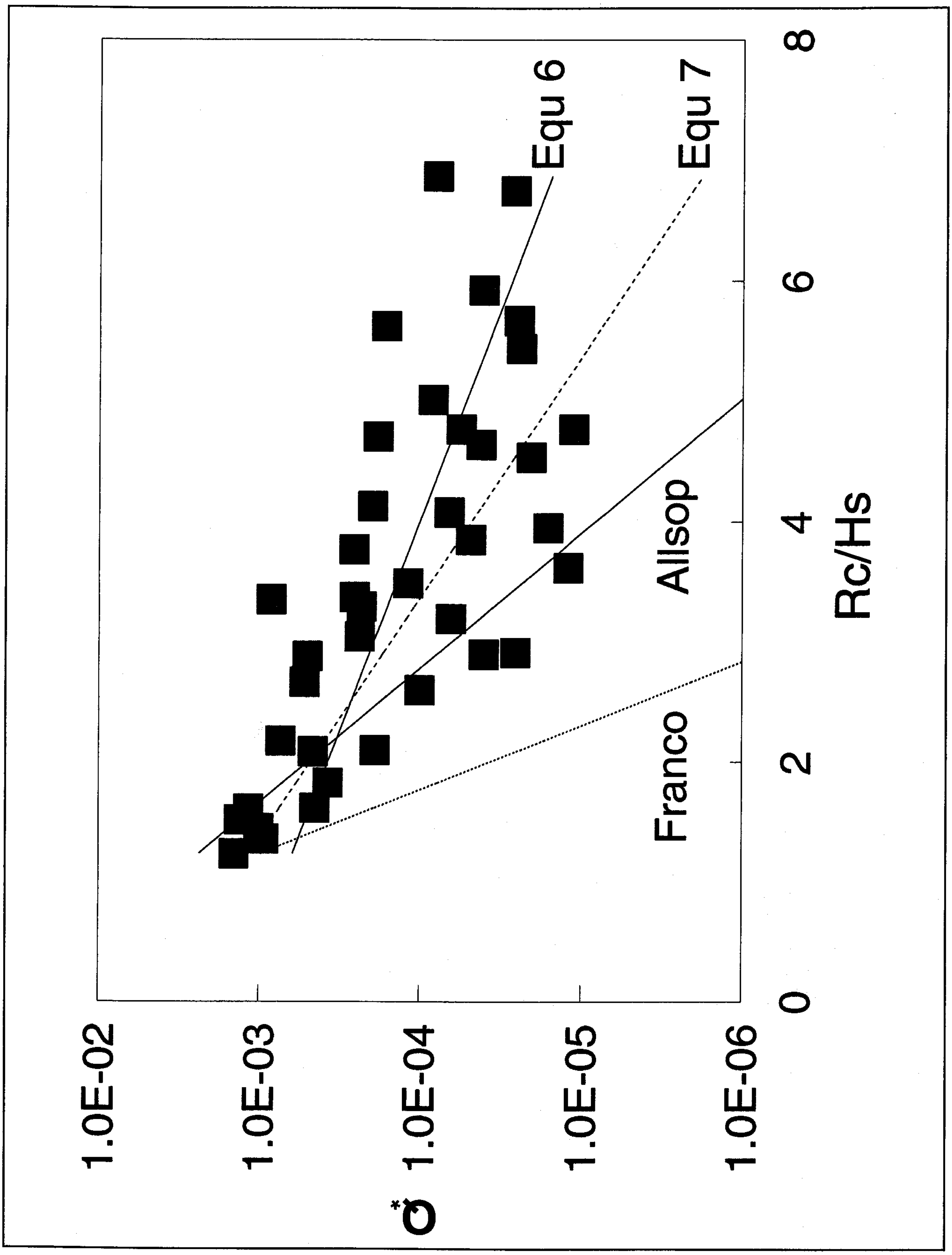

Figure 4 Overtopping discharge results