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CONFERENCE ON BREAKWATERS - DESIGN & CONSTRUCTION

Institution of Civil Engineers London May 1983

THEME SPEECH

"THE WAY AHEAD"

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INTRODUCTION

1. It is true for all structures that the design and the location affect the loads. Breakwaters are extreme examples of this thesis, since even small variations in waterdepth, orientation slope etc. produce large variations in the load. As the dominant load is stochastic in nature and exhibits extreme variations, it is not surprising that we have difficulty in reaching an acceptable design procedure. However, being a little provocative, I would like to say that some of the recent failures of major breakwaters are due to gross errors, which cannot be explained away by a claim of insufficient basic knowledge. One could also say that the available knowledge did not reach the engineers on time. One way ahead is, therefore, and always has been, to ensure good communication of up-to-date knowledge.

2. In the design process we have six important areas to consider: The first is the function of the structure, which has hopefully been specified by the client. The second, the natural boundary conditions like wave climate, available materials etc. The third, the physical behaviour of the structural elements. The fourth, the constructability. The fifth, the maintenance throughout the life of the structure, and the sixth, the assessment of the structural reliability.

Function
Natural boundary conditions
Physical behaviour of the structural elements
Construction
Maintenance
Reliability

Areas to consider in the design process

I would like to give you my view on some of the ways ahead and relate them to these areas.

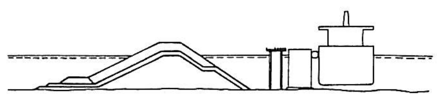
DESIGN ASPECTS

INTEGRATED DESIGN

3. A breakwater is never an individual structure which is designed for its own sake, it is always a part of a harbour or a coastal protection scheme. The function can, therefore, be set only by integrated design of breakwaters, access channels, moorings, loading systems, calling frequency, harbour operation, storm warning systems etc. An up-to-date design involves extensive use of both mathematical and physical models, which are seldom to the hand of consulting engineering companies or harbour authorities. Therefore, before the functions of the breakwater are specified, it is important to communicate with capable hydraulic laboratories or institutes, and this should be done right at the beginning of the planning stage. We still see examples where money is wasted due to lack of, what I would call, an integrated design process in which the client, the consultant, and the specialized laboratories and institutes work parallel and hand in hand. The development in stormwarning models and mooring and cargo handling systems will possibly lead to further reduction of the necessary capabilities of the breakwater, which again means shorter and lower breakwaters, maybe no breakwaters at all in some places.

SIMPLICITY IN DESIGN

4. As long as reliable data on wave climate are not available, and as long as the basic principles of rubble mound structures are not well understood, simplicity in design is essential. Generally this means the separation of functions.



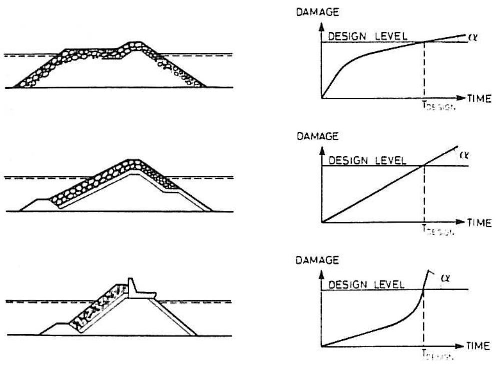
SEPARATION OF FUNCTIONS

For example it is a great advantage if the function of the breakwater is restricted to reduce transmission of wave energy. Every time we add other appendages to it, like access roads, pipeline galleries, storage areas etc., we introduce structural elements which remove it from the initially simple concept of a rubble mound. Rubble mound breakwaters are basically of the "weakest-link"-type of structure. The more elements we add, the bigger the possibility of a failure. Wave walls are especially harmful obstacles if design waves are exceeded, because they reflect the run-up and cause increase of downrush and erosion of armour.

FAILURE MODES

5. Failures are of many types. We should be aware of the fact that we ourselves, to a great extent, can decide the mode of failure. This has been the practice for many years for example in earthquake engineering.

The most ductile failure is that of a traditional rubble mound with moderate sloping armour of non-interlocking blocks. The most brittle failure is associated with steep slopes of interlocking armour in front of a wave wall. The breakwater should preferably be designed for a low rate of damage, i.e. a small angle α in the figure.



FAILURE MODES

It is essential that we design the breakwater so that the mode of failure corresponds to the quality of information constituting the boundary conditions and to the risk acceptable to the client.

WAVE CLIMATE

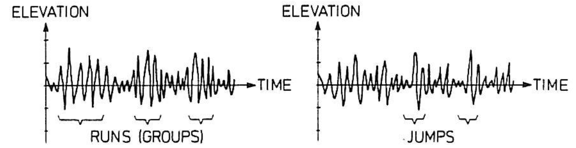
GROUPING

6. The more brittle the mode of failure of the breakwater, the more details of the wave climate we require.

Since oncoming waves are always interacting with the downrush from the preceeding wave, the actual succession of waves as produced in nature is important. For steep slopes with complex armour units and maybe a superstructure with a wave wall, we know that one single wave can initiate a fast progressive failure or even cause a failure by fluidising the armour layer, resulting in a slide of the armour as a whole.

7. High confidence estimates of the safety for such structures can only be achieved if we know the probability of occurrence of the wave patterns that are dangerous for the specific type of sea bed - breakwater profile.

8. One type of wave pattern, namely runs of big waves or wave groups, is generally accepted as an important wave pattern, not only for slow respons systems like moored ships, but also for rigid structures like breakwaters. However, there are other dangerous wave patterns, for example what I call a jump.



DANGEROUS WAVE PATTERNS

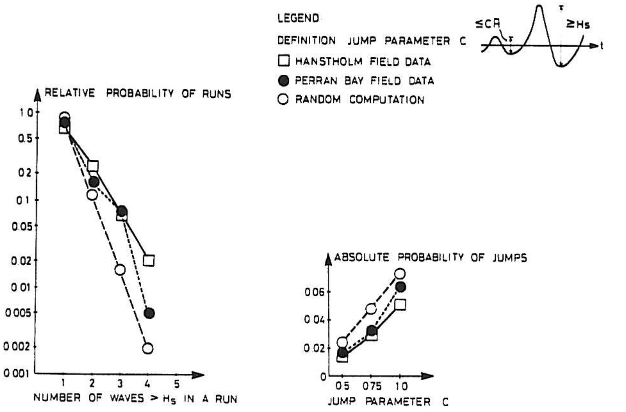
9. A jump is seen on the wave amplitude time series as a small wave followed by a big one. The small wave will stabilize the water table at a level close to MSL, which means that the large oncoming wave is not tripped by a downrush and therefore breaks higher up on the slope and creates higher run-up and overflow velocities. The run-down is also very deep and this causes big pressure gradients, which are destabilising.

10. At the Conference on Coastal and Port Engineering in Developing Countries the delegates had the opportunity to study the long swell on the west coast of Sri Lanka. The beach profile in some places was as steep as 1 in 5, so breaking took place virtually on the beach. Conditions were close to those for a flat rubber mound breakwater. Two phenomena made a very big impression on me. One was the tremendous drag-forces from a run-up, reaching 4 m/sec., which makes one wonder how breakwaters, where the process of erosion-accreation is not acceptable, can be stable. The other was that jumps caused much larger run-up that runs. This observation is in accordance with the run-up tests and armour stability tests I made at the Hydraulics Research Station, Wallingford, in 1977 (ref. 1).

11. To me there is no doubt that, dealing with rubble mound breakwaters with brittle failure modes, we must analyse the wave records also for wave jumps and other dangerous patterns that might be identified, and generate the laboratory waves accordingly. At University of Florida at Gainsville they are, at the moment, trying to identify such wave patterns.

12. It might be that the statistics of runs and jumps are complimentary in that, if a record contains many runs, then it contains few jumps and visa versa. An analysis of natural records from two storms and an analysis of corresponding random phase laboratory waves, which I made some years ago at the Hydraulics Research Sta-

tion, Wallingford showed this tendency, which is understandable (ref. 2). If there is such a general correlation, we probably only need to analyse the wave records for one wave pattern. This, however, still needs to be verified.



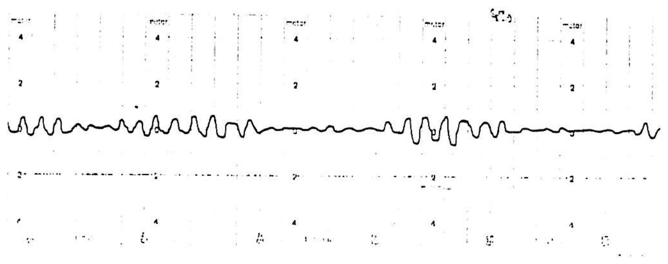
Statistics of runs and jumps (ref. 2)

13. There is some evidence that, if random phases are applied to component waves of sea, we obtain the correct statistics of wave patterns when generating the laboratory waves in accordance with the short wave spectrum (Burcharth ref. 2, Rye ref. 3, Sand ref. 4, Lundgren ref. 5). So if the hypothesis holds that groups and runs are complementary and runs are reproduced statistically correctly from the short wave spectrum with random phases, then we only need to analyse the natural wave records for the short wave spectrum and generate the laboratory waves accordingly.

It might be that the reproduction of wave sequences can be improved by restricting (linking) phases of pared first and second order harmonic component waves. This compares to the bound found in natural waves and which in physical models creates nonstationary wave asymmetry. Moreover, in models sensitive to correct positioning of the internal water table (e.g. when the wave pressure on capwall base plates or the geotechnical stability of the granular body is critical) also the second order long waves must be controlled.

14. Not all laboratories are patient enough to wait for the dangerous wave patterns in waves generated in accordance with this system. Instead they produce a deterministic amount of groupiness and hope to be on the safe side. Not all researchers believe in random phases, because natural wave records, in some cases, show more pronounced grouping than expected from random phase generation. This might be due to the limited number of waves in the available records. Twenty minutes are too short to obtain a good estimate on wave group statistic and long wave spectra.

15. Also the pronounced grouping in some records and thereby the big scatter in wave group statistics might be due to the fact that, in the analysis of wave groups, generally no distinction has been made between sea and swell. A con-



SWELL RECORD, PERRANPORTH, ATLANTIC COAST OF U.K.

siderable amount of swell will change the group statistics, because swell is characterized by very long runs.

From a physical point of view, it is also important to distinguish between sea and swell because, generally, they have different directions.

16. At present, a study of the correlation between runs, jumps, and short and long wave spectra is performed at the Danish Hydraulic Institute and at Aalborg University, partly based on long records of storm waves kindly provided by the Coastal Engineering Research Center, U.S. and the Royal Netherlands Meteorological Institute.

17. I would suggest that some of the many waverider buoys in service are set to one hour of recording, which allows us to calculate the long wave spectra, and that, when analysing wave groups, one look into the history of the storm to verify if significant swell components are present.

18. By advocating this separation in the analysis I am certainly not saying that swell is not important. In fact, I believe that for any breakwater on coastlines with long fetches, for instance ocean coasts, there is a strong possibility of the worst wave climate being a sum of swell and sea, as has been found for Sines (Mynett et al., ref. 6).

THREE-DIMENSIONAL WAVES

19. Swell and sea have, in general, different directions in deep water. It is, therefore, important that the direction of energy propagation is taken into account when dealing with deep water breakwaters. Also, there is some evidence from tests at the Technical University at Lyngby, Denmark (Broberg and Thunbo, ref. 7), that short crested seas affect rock armour differently from long crested seas, which is to be expected. Therefore, reliable arrays of wave gauges to record directional seas must be developed and brought into operation as soon as possible. It should be remembered that, for many breakwaters, overtopping is the critical factor, and that the amount of overtopping and its distribution in time and space must be greatly affected by the lengths and directions of the wave crests. There is still a great deal of research to be done in this field.

FREAK WAVES

20. For deep water breakwaters we also have the problem of freak waves, that is waves of extreme dimensions, and often with unexpected

direction of propagation. Ship records, through generations, mention these often catastrophic waves, and as a sailor I have myself met such a wave in the Skagerak. Such waves are difficult to implement in the breakwater design process, partly because they are so rare that records are very few and a statistical treatment therefore impossible, partly because laboratory generation of such waves is difficult. The Norwegian Hydrodynamic Laboratories (Kjeldsen, ref. 8) and the National Research Council of Canada (Mansard et al., ref. 9) are working on this problem.

21. If we cannot obtain wave information on the characteristics and the frequency of occurrence of such waves, we might then use a procedure where the designer decides on a certain deterministic freak wave sequence, and the breakwater is then designed such that serious damage, but not a complete failure of the breakwater occurs, when exposed to this sequence.

PHYSICAL PROCESSES IN WAVE STRUCTURE INTERACTIONS

REVISION OF ARMOUR STABILITY FORMULAE

22. The different parameters involved in wave structure interactions are known qualitatively, but not quantitatively. Consequently our formulae for calculating wave load on structural members are primitive and somewhat unreliable. The formulae are based on dimensional analysis of a few of the parameters involved and then fitted to model-test results by applying a coefficient. This is partly a "black box" approach where the coefficient must take care of many parameters and as a result is not a constant, but a function of several variables. The classical example is K_D in the Hudson equation, which is not a constant since it varies considerably with both the slope angle and the wave period for complex types of armour. For example on a horizontal bed, Dolosse and rocks of the same weight have equal hydraulic stability (Burcharth and Thompson, ref. 10). Since it was clearly shown, already in 1974, in a paper (Brorsen et al., ref. 11) presented at the Coastal Engineering Conference in Copenhagen that K_D cannot be taken as a block specific constant, I think it is time to revise the different manuals and codes of practise in this respect, and also to point out that in many cases the only tools we have, at present, are model tests.

PARAMETER ANALYSIS

23. We must, of course, proceed from this primitive stage by a more systematic investigation of the influences of the different parameters. This can only be done by restricting the number of variables, for example as done by Brebner (ref. 12) in his steady flow tests with Dolosse, by Alan Price (ref. 13) in his pull-out tests and by Burcharth and Thompson (ref. 10) in the pulsating water tunnel tests with beds of Dolosse and rocks. Such tests lead to an understanding of the relative importance of a few parameters at a time. There are many possibilities for such tests, and I will for research recommend more such deterministic tests instead of tests with complete breakwater slopes, where the relative influence of different parameters is difficult to verify, although it is possible in some cases as shown for example by Bruun and Johanneson (ref. 14), and Gravesen and Torben Sørensen (ref. 15).

MODEL EFFECTS

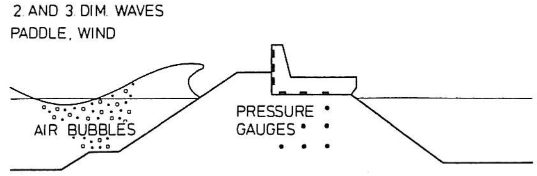
24. Since we have to rely on model tests a great deal, it is very important to consider both model effects, recording techniques and scale effects. By model effects I mean those due to deficiencies in the model, for example the use of two-dimensional waves instead of three-dimensional waves, improper reproduction of wave trains, surface roughness of armour blocks etc. Model effects can be considerable. Examples are models of shallow water breakwaters where a fairly steep bottom profile in front of the breakwater is not incorporated into the model. We know that even small variations in the bottom profile cause considerable changes in the waves attacking the structure. Where changes of profile along the breakwater are present, the model test program often has to be extended to cover a substantial part of the sea bed topography. This has to be accepted, although it will result in a considerable increase of model costs.

SCALE EFFECTS

25. Scale effects in rubble mound models are still discussed, for example by Vasco Costa (ref. 16), and for good reasons. There are two types of scale effects which especially need clarification. One is related to the pressure from breaking waves, the other to flow in the porous body.

26. Wave pressures on superstructures and armour units will in a Froude model suffer from scale effects, partly due to incorrect amount and distribution of entrapped air, partly due to wrong scaling of air compressability. Moreover, it is indicated by Joe Ploug, the chairman of the IAHR working group on wave generation and analysis, that wave models, although built and run to the same detailed specifications, show significant differencies in various respects. It is, therefore, to be expected that the amount of entrapped and saturated air will vary significantly in models of the same prototype but produced in various laboratories. Even though the part of the wave pressure, which in the model is recorded as shock pressure, can be interpreted in accordance with the compressibility law, and even though it might be argued that the model test results are on the safe side, the fact still remains that we do not know how close we are to reality.

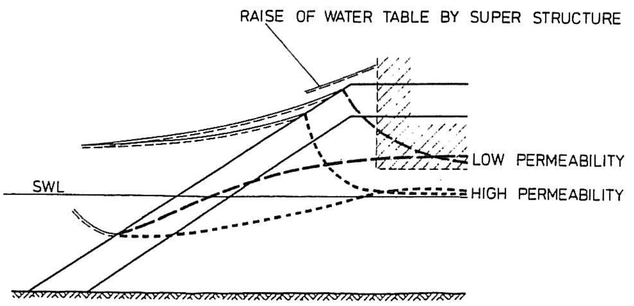
27. A research project with the participation of a number of laboratories should be started with the object of studying the pressures on



WAVE PRESSURE SENSITIVITY ANALYSIS

wave walls and capping base plates. Tests in long and short crested waves as well as tests in wind wave flumes are needed. An important part of the study should be tests to verify how sensitive pressures are to the amount of saturated and entrapped air. This might be done by adding air bubbles. As the most important part of the project, prototype recordings of pressures should be performed for the comparison and calibration of the model test results.

28. The other important scale effect is the viscous effect in porous flow. We know that there are no significant viscous scale effect in flow in armour layers even in small scale models (Burcharth and Thompson, ref. 10; Mol, Ligteringen et al., ref 17). However, this flow in the armour layer is affected by the flow in the filter layers and the core, where viscous effects might be present due to much lower permeability. This is easily visualized by the change in positions of the internal water table. Generally a viscous effect will result in too high an internal water table.

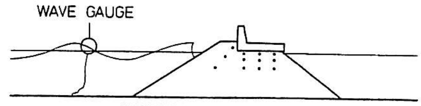


INFLUENCE OF PERMEABILITY ON INTERNAL WATER TABLE

For the armour it means larger destabilizing pressure gradients and a reduction in the valuable reservoir effect, thus giving rise to larger overflow velocities. For the superstructure too high a position of the internal water table might enlarge the wave pressures, both on the wave wall and the base, to such an extent that the results will be completely wrong.

29. A generally applied method of preventing viscous effects is to assure that the Reynold's number for the armour layer flow exceeds a certain value (Dai and Kamel, ref. 18). However, this criterion is not satisfactory, first of all because a single value of a Reynold's number cannot, in general, represent the complicated and unsteady flow field, where, even in prototype, some part of the flow will most probably be laminar or fluctuating between turbulent and laminar. This problem has recently been evaluated by Juul Jensen and Klinting (ref. 19). Another problem is that the permeability of prototype cores is very difficult to predict since the permeability is sensitive to small variations in the grading and also to separation of material when dumped. The prototype flow field is, therefore, generally poorly known. A very promising mathematical model of the porous flow has been developed by Barends et al. (ref. 20) at the Delft Soil Mechanics Laboratories, but naturally it relies on calibration against prototype data.

30. An important step in solving the problem is, therefore, to obtain prototype data on the pressure field inside various typical breakwaters for the calibration of both physical and mathematical models.



PROTOTYPE RECORDINGS OF INTERNAL PRESSURE FIELD

DEVELOPMENT (EVALUATION) OF <u>LARGE SCALE PHYSICAL MODELS</u> TO REPRODUCE PRESSURE FIELD

CALIBRATION (EVALUATION) OF <u>MATHEMATICAL MODELS</u> TO REPRODUCE PRESSURE FIELD

31. To get the mathematical models operational is very important, since such models are the easiest tools for estimating the internal pressure field and water table positions in new designs. Based on experience gained by prototype measurement it might be fairly easy to develop a technique for the construction of good physical large-scale models, but for the usual smaller scale models the solution might be, first to run a mathematical model for the determination of the internal pressure field and thereafter, by varying the core permeability, to calibrate the small-scale physical model to reproduce the calculated internal pressure field. When this is done, then other phenomena, like run-up, armour stability etc., can be studied, unbiased by viscous effects.

TESTING OF NEW DESIGNS

RUN <u>LARGE</u> SCALE <u>PHYSICAL MODELS</u>

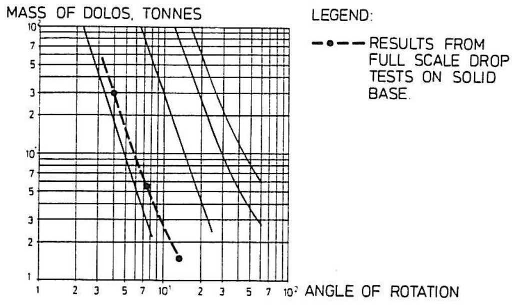
RUN <u>SMALL</u> SCALE <u>PHYSICAL MODELS</u> CALIBRATED TO REPRODUCE INTERNAL PRESSURE FIELD FOUND BY <u>COMPUTER MODELS</u>

Since measurements of pressures only require fairly simple instrumentation, it should be possible to realize such prototype recordings in new breakwaters where also wave recording takes place.

FORCES ON ARMOUR UNITS

32. Although recording techniques have developed considerably in recent years, we still for armour units require a good method of recording movements, or preferably forces or stresses. For this reason we are still missing the hydrodynamic and mechanical response functions for armour units exposed to waves. These functions are the missing link in the design procedure where both the hydraulic and the mechanical stability are considered.

33. We can find the response of individual blocks to deterministic well defined loads. For example from theoretical considerations and full scale experiments, we can obtain information on the relative variation and, to some extent, the absolute variation in strength with size of units (Burcharth, ref. 21 and 22; Silva, ref. 23).



EXAMPLES OF ISO-STRESS GRAPHS FOR ROCKING UNREINFORCED DOLOSSE OF IDENTICAL CONCRETE. WAIST RATIOS 0.30 - 0.36

The figure shows an example of such approach for Dolosse. The family of curves are iso-stress lines and the dotted curve is one found from droptests on a hard base. By relating to prototype experience, we can obtain an indirect design method (Burcharth, ref. 24).

34. However, a more direct approach for the determination of loads on armour units is wanted. The detection of armour unit movements is still an indirect method, but valuable. Photo and film technique in models can, to some extent, be used to detect the movements. However, it fails the splash zone, which, unfortunately often, is very critical and therefore most important. Moreover, the rocking stage, where no displacements take place, cannot be detected by single frame technique, and this stage is very important for some unreinforced slender concrete units.

35. The use of accelerometers or strain gauges in model armour represents a more direct method and is a big step forward. It has already been implemented as a standard procedure, for example at the Delft and De Voorst Laboratories. The instrumentation of armour units is expensive and restricted to fairly large units. As a result only a small number of units in rather few models have been instrumented so far. But to obtain reasonable confidence levels we need a whole pack in each model, say minimum 50 units, instrumented to give simultaneous readings or we need a very large number of tests with few instrumented units. To minimize scale effects, this should be done in some of the giant flumes. Also, it is very important that we include a number of instrumented armour units in new breakwaters were wave gauges are installed.

36. Another way of dealing with forces on armour units in a pack is to scale the material characteristics of the concrete in accordance with the Froudian Law of similitude and record the damage. Despite the very difficult task of scaling both tensile strength, compressive strength, modulus of elasticity and fracture toughness, Gerry Timco (refs. 25 and 26) has been very successful in his work with model concrete for small scale models. It is, of course, much easier when working at larger scales, for example length scale 1:10, which is possible in the giant flumes, and model concrete has been used in the Delta flume at De Voorst in the large scale models, for example of the redesigned Sines breakwaters. However, the modelling of the concrete can still be improved.

37. For the testing of specific breakwater designs the application of correctly scaled model concrete is the best procedure. From a research point of view, a more promising approach might be to run parallel tests with armour units made of relatively weaker materials, but with different characteristics, so that mechanical failure of armour units takes place at various stages, also before hydraulic instability is expected. In this way, we can get information on the distribution in space and time of static and pulsating loads in a pack.

38. A different approach is, of course, to use a mathematical model. Various parts of such a model can be determined and checked in physical models. For example the hydrodynamic response function of a unit in a pack might be determined by fixing the unit to the end of a flexual beam mounted with strain gauges and arranged in the pack such that the surrounding units (which must be glued to prevent movements) are not touched. The response, in terms of total force, to different flow characteristics could then be used as input in a mathematical model for the estimate of the distribution of forces on the units. Such a mathematical model must be able to construct a geometrically correct pack of armour units, to calculate the flow field from complicated boundary condition, to calculate the flow force on individual units, and to calculate the magnitude and direction of the contact forces between the units. Also, it must take into account the reallocation of contact forces when local crushing and interblock sliding take place. A promising model, able to handle geometrically simple shaped elements, is reported by Austin and Schluter (ref. 27), and no doubt models which can handle also geometrically complicated shaped armour will soon be available, but to my opinion not applicable as design tool if not calibrated against physical data. Especially information on crushing characteristics in contact points is essential.

37. All four above-mentioned approaches for studying the forces on armour units will be necessary if, in our design process, we want to be able to also design armour units as we are designing other structural elements.

INSTRUMENTATION OF ARMOUR UNITS IN LARGE SCALE MODELS

INSTRUMENTATION OF PROTOTYPE ARMOUR UNITS

STUDY OF "WEAK-MATERIAL" ARMOUR UNIT FAILURES IN MODELS

MATHEMATICAL MODELS

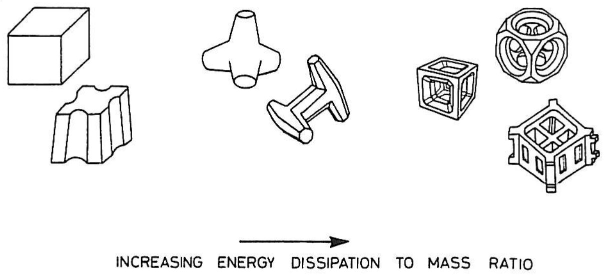
WAYS OF DETERMINING LOADS ON INDIVIDUAL ARMOUR UNITS IN A PACK

The results are needed, not only as an important brick in a consistent design method, but also because we want to be able to test breakwaters in scale models without the expensive scaling of the armour material. Because of the costs, such a project is an obvious subject for international sponsorship. A working group under PIANC should be considered.

ARMOUR UNITS

DESIGN ASPECTS

40. The development of the different types of concrete armour units was based mainly on practical experience. Only in a few cases, a more systematical design process based on both hydraulic and structural consideration has been applied. New types of units will certainly be developed parallel to progress in the understanding of hydraulic and structural behaviour of armour. At present the different types of concrete armour units might be put into the following three categories: Bulky types randomly placed (cubes, Antifer blocks), complex slender types randomly placed (Tetrapods, Dolosse), and hollowed blocks placed in patterns (Cob, Shed, Diode).



CATEGORIES OF CONCRETE ARMOUR UNITS.

41. We know that, from a hydraulic point of view, a high permeability is most advantageous (Burcharth and Thompson, ref. 10). The introduction of the ROBLOC sublayer unit by P.R.C. Harris (Groeneveld et al., ref. 28) to ensure random placement and thereby large permeability of cube armour follows this line. This means as little concrete as possible. We also know that randomly placed units involve impact forces due to movements, and also large variations in contact forces.

42. Hollowed cube types have a high permeability and exhibit practically no movements when correctly placed like a pavement, and the contact forces are uniformly distributed. This together with a structurally better shape makes the hollow types the most efficient in terms of the optimum use of concrete for dissipation of wave energy (Barber et al., ref. 29; Wilkinson et al., ref. 30). This type of unit is widely used for sea wall revetments, where construction at low tide can be done in the dry, because it demands an even underlayer, accurately placement and good toe support. However, as the technique for the construction and the control of underwater works improve, I believe we shall see the hollowed cube types of units being used also in breakwaters.

43. The slender, complex types of units, which at the moment seem to be less popular because of the recent failures, will not disappear from the scene, but the use of such units will probably be restricted to breakwaters where the wave climate can be well predicted, as for example in shallow water situations. Because of the units' good hydraulic stability relative to the mass, such units will again be used for deep water structures when a proper design method which takes into account also the mechanical properties of the units is developed. However, there is no doubt that the static and dynamic strength of the larger size slender complex types of units must be increased to allow for some movements, because such units cannot compete with more bulky types if non-movement criteria are adopted, and also because movements are unavoidable in a randomly placed pack.

44. A properly designed armour unit must have a homogeneous response to the various loads. For an analytical analysis of stresses we need not only the hydrodynamic transfer functions, as mentioned above, but also the material response. Elastic and plastic theories as well as fracture mechanics have proved to be inadequate in describing the mechanical response of concrete, for example the fatigue. Much more promising is the Continuous Damage Theory by Dusan Krajcinovic (refs. 31, 32, 33) and others. This is not very surprising, since the theory is based on the introduction of an additional kinematic variable, characterizing the density and the distribution of the microcracks and their effect on the material parameters and the stress redistribution.

CONCRETE

45. On the material side we are looking for a less brittle concrete with good long term durability. We know that the use of super-plasticizers, and thereby a low water cement ratio, and the use of puzzolan cement is beneficial in this respect. Full scale static and dynamic tests by Polytechna Harris, Milan, with 30 t Dolosse in Gioia Tauro clearly confirmed this.

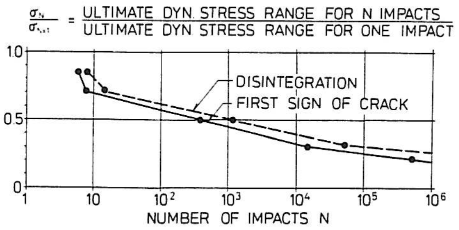
46. When dealing with brittle materials like concrete it is essential to prevent tensile stresses in the surface zones to develop during production. Even small tensile stresses in the surface regions will seriously affect the strength of the member. On the other hand, if we can assure the reverse, namely compressive stresses in the surface zone, then a considerable increase in structural strength might be obtained. Such a technique is known from production of, for instance, glass and might be possible also for concrete. For example, by ensuring more pronounced hydration and thereby expansion in the surface regions than in the centre regions.

47. Progress will be made if we can get the concrete researchers interested in finding ways of improving especially the tensile strength and the fracture toughness. For many years these properties have not attracted much interest from researchers because structural elements exposed to tensile stresses are normally reinforced.

FATIGUE

48. Armour units are exposed to repeated loads, typically several million cycles at various

stress levels during the structural lifetime. The loads are both pulsating and impacting. Recent research by Tepfers (ref. 34), Fagerlund et al. (ref. 35), Zielinsky et al. (refs. 36, 37), Tait (ref. 42), and Burcharth shows a significant fatigue effect in concrete, which clearly must be incorporated in the design procedure.



FATIGUE. IMPACT LOADED FLYASH CONCRETE DOLOSSE FLEXURAL STRESS (PRELIMINARY RESULTS, BURCHARTH, 1983)

For example the allowable flexural stress range is reduced to 40% after approximately 10,000 impacts or cycles only. Improvement of the ductility and fracture toughness, as mentioned previously, will increase the fatigue life also, but much more research in this field is needed.

REINFORCEMENT

49. Reinforcing the concrete is, of course, an obvious way of improving the strength properties. Both conventional steel bar reinforcement and fibre reinforcement are used. Results from full scale static tests and dynamic drop tests with Dolosse in the range 1.5 t - 30 t show that conventional reinforcement is superior to steel fibres of equal quantity. By using approximately 130 kg steel per m^3 concrete, spalling and not cracking seems to be the limiting factor (Burcharth, ref. 21, Polytechna Harris, Milano 1983). Fibre is beneficial in very slender and complicated structured members such as hollowed cube types of units. Chopped polypropylene fibres are, for example, used successfully in the SHED unit by Shephard Hill Ltd. Dolosse and Tetrapods are, in this respect, not slender but relatively bulky and stiff elements.

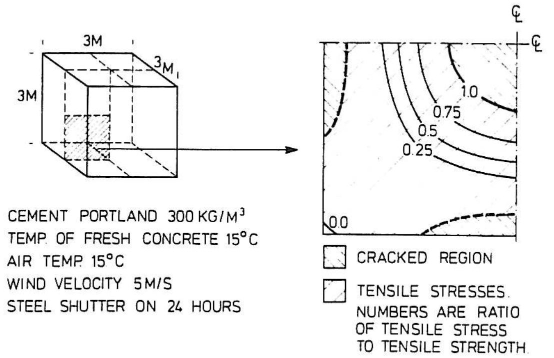
The cost of bar reinforcement will be reduced as more effective ways of using the steel and easier ways of placing the reinforcements are developed. Ongoing research in this field are full scale tests and finite element calculations by Mike Uzumeri at University of Toronto.

CORROSION

50. Corrosion has prevented many coastal engineers from using steel reinforcement. Research on corrosion is intense and promising. Results obtained so far at the Danish Corrosion Centre (F. Grønvold) show that the use of fly ash reduces corrosion, and high densified concrete with a substantial content of silica dust nearly eliminates the risk of harmful corrosion in bars of the sizes used in large armour units. The influence of crack width on corrosion is still not fully understood. The use of reinforced concrete in offshore structures will ensure further research and thereby improvements applicable also in breakwater concrete technology.

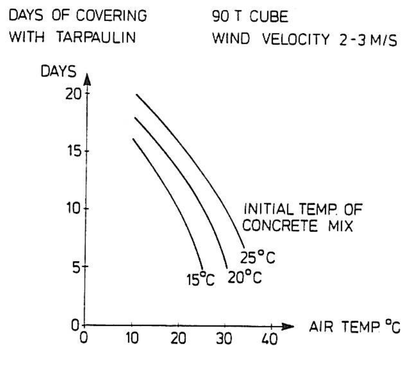
THERMAL STRESSES

51. Thermall stresses represent a serious problem for all armour units of large volume. Roughly, it can be said that micro cracking will occur if temperature differences during curing exceed 20 °C. This compares to dimensions exceeding approximately 1m x 1m x 1m, when conventional concrete and casting procedure are used. The problem, therefore, exists also for the larger complex types of units. The figure shows a calculation by the Danish Concrete Institute, BKI, of the extent of microcracking and stresses in a 70 t cube produced by conventional concrete technology. No wonder that it is found that such cubes often are very brittle.



THERMALL STRESSES IN A 70 T CUBE 100 HOURS AFTER CASTING (BKI-INSTITUTTET COPENHAGEN AND BURCHARTH, 1982)

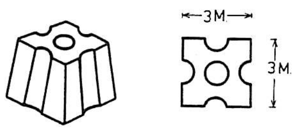
Measures to prevent thermall stresses are well known, but they all involve drawbacks. The use of low heat cement or retarder slows down production, the use of less cement reduces the surface resistance and the long term durability, the cooling of aggregates and water is expensive and impossible in some places, and the use of insulation during the curing complicates the production. This is illustrated in the figure, which shows an example of a diagram by BKI-Instituttet and Burcharth for determining the number of days where insulation must be kept on a



EXAMPLE OF DIAGRAM TO DETERMINE DURATION OF INSULATION DURING CURING. (BKI-INSTITUTTET COPENHAGEN AND BURCHARTH, 1982)

90 t cube of conventional concrete to prevent thermall cracking. It is seen that approximately 15 days are necessary, which again demands 300-1000 insulation sets, depending on the size of the job.

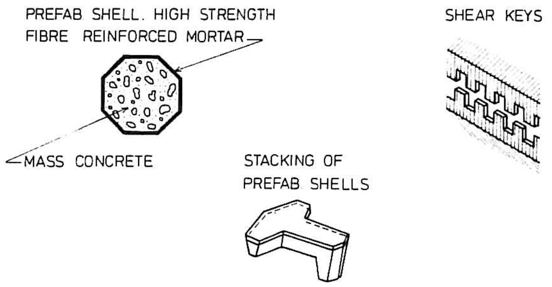
53. For some units it is easy to solve the thermall stress problem by adjusting the shape. The figure shows how this can be done for a big Antifer cube, simply by making a hole in the middle. In addition, such a modification will increase the hydraulic performance of the armour.



ANTIFER TYPE BLOCK WITH HOLE
TO REDUCE THERMALL STRESSES

COMPOSITE TYPE OF ARMOUR UNIT

54. Many of the problems related to armour units might be solved by a composite type of unit, consisting of a thin shell of very strong, ductile fibre reinforced cement and filled with mass concrete with low cement content or low heat cement.



PROPOSAL FOR COMPOSITE ARMOUR UNIT
(PRINCIPLE SHOWN FOR DOLOS-SHAPED UNIT)

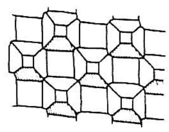
The concrete can be of an inferior quality than traditionally specified. This solves many problems in areas where good qualities of aggregates, cement and water are not available.

A good surface resistance and long term durability will be ensured by the high quality shell. The production will give few limitations to the shape because the fibre reinforced mortar shell can be manufactured by a spraying technique.

The edges of the two parts of the shell can be designed with shear keys. The feasibility of such a composite unit is not known. Besides the economy, we must study, both theoretically and experimentally, the static and dynamic strength and the fatigue life before anything can be said.

SURFACE ROUGHNESS

55. To obtain high hydraulic stability, the shape of the unit should ensure a very high permeability of the armour layer. For this reason also a very high surface roughness is desirable. This can be obtained by shaping the outer shell with fairly big roughness elements. This will also increase the stiffness of the shell so that pressure from the wet concrete does not cause too big deformations.



ROUGHNESS ELEMENTS ON ARMOUR UNITS

The positive effect of surface roughness is evident from a breakwater I saw on the Maldives and which was constructed of coral blocks. The mass of the coral blocks is approximately 20 kilos, the density only 1.1 t/m^3 and the front and back slopes nearly vertical, and still this breakwater resists long waves of more than 2 m's height. The roughness made it impossible for me to pull a block out from the pack, which I also tried in vain from a groin constructed in the same way.

MONITORING OF UNDERWATER WORKS DURING CONSTRUCTION AND STRUCTURE LIFETIME

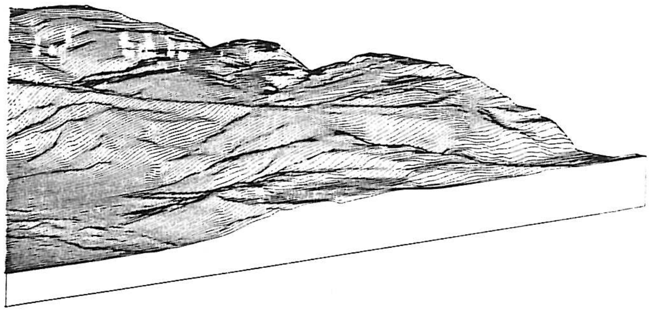
56. Many breakwater designs involve such construction difficulties that a not-as-designed structure is the inevitable result. Fracture of fragile armour units during placement and not-as-designed distribution of materials in underwater sections are the most serious problems. Although we can improve the strength of armour units and put more attention to constructability in the design, the two mentioned problems will still remain to some extent.

57. Fracture of armour units by underwater placement is due to high impact velocities. The velocity must therefore be recorded, for example by placing accelerometers on the sling (grab), and transmitted to the crane operator.

58. To secure a uniform distribution and placing density of underwater armour for some types of units, it is necessary to use a technique of visual observation. In large water depths it is not sufficient to place units to a co-ordinated grid, because even a moderate swell might displace the unit a couple of metres, which cannot be registered by the operator. Moreover, a satisfactory placement also requires a certain orientation of the unit, when placed to fit in the pack. The orientation and the rotation of the unit should therefore be visualized and controlled by the operator. Underwater low light colour video cameras mounted on a direction stabilized frame above the sling (grab) might solve the problem.

59. The inspection of the breakwater profile and the control of the coverage density (gabs

in the pack) are very important but difficult, especially in the case of big complex types of blocks. Side scan sonar has been tried with moderate success (Patterson and Pope, ref. 38). However, experts say that already available and not too expensive technique makes it possible to obtain detailed pictures, for example by presenting densely spaced profiles produced from position corrected signals from a scanning sonar mounted on a boat. The figure shows a representation for a sea bed.



EXAMPLE OF 3D. REPRESENTATION FROM MONITORED PROFILES. (EIVA LTD., DENMARK)

Devices for the monitoring of underwater works for rubble mound breakwaters can be developed further. Since the costs involved will be minor compared to the construction costs, it is obvious that the designer should stimulate such a development, simply by specifying strict procedures for the control of underwater works in the tender documents. Without development in this field it is also impossible to follow the needed programmes for underwater maintenance throughout the lifetime of the structure.

PROBABILISTIC DESIGN

61. Very often the argument is heard that a probabilistic design procedure is of little value as long as the understanding of the physics is poor. It is true that such a design process never gives you figures in which to place high confidence as long as we cannot describe the physical processes. However it is worth while to recall that the less we know about the physics, the more important it is to try to assess the reliability. The only method is the probabilistic approach, which gives you information on the risk of failure with due consideration to the uncertainty or scatter of the various parameters involved. It is no excuse not to use the method because we do not know the probability density functions of the individual parameters (or of the traditionally used black box parameters). As engineers we must estimate the distributions, just as we estimate partial coefficients or safety factors. To-day's knowledge makes it difficult to estimate the probability functions for some parameters. The advantage of the method in this respect is that you can easily vary the probability distributions and see the influence on the reliability of the structure. Also, such a sensitivity analysis is essential in providing a systematic base which the designer can use for the specification of the accuracy or confidence limits, for example for the model test results and the construction tolerances. (Kooman, ref. 39). Another advantage is that long term effects, such as for example fatigue, can be included in a meaningful way (Nielsen and Burcharth, ref. 40). It is, in fact, my opinion that we should use a level III method straight away with the estimation of unknown distribution functions. It would of course be most valuable if the various laboratories could produce and report estimates of parameter distributions.

62. The probabilistic method cannot, in an operational way, take into account the so-called gross errors, which in fact very often are responsible for unexpected failures. Gross errors are, for example, if the designer forgets or does not know about the mechanical strength problem for slender unreinforced units, or if the designer produces design sea state on wave data belonging to statistically different populations, or if the contractor puts the reinforcement bars in the wrong part of a structural member. We cannot take such gross errors into account. The evaluation of gross errors in terms of Fuzzy sets might be an operational way for the implementation in the design process (Ditlevsen, ref. 41). Another aspect of the gross error problem might be illustrated by the formula by Robin S. Colquhoun for the real probability of failure,

REAL PROB. OF FAILURE $P_{F,\,real} = 1 - \exp\left[-P_f^{P_E}\right]$

P_f is the engineer's estimate of prob. of failure.

P_E is the prob. that the engineer knows what he talks about.

(By Robin S. Colquhoun)

ACKNOWLEDGEMENT

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