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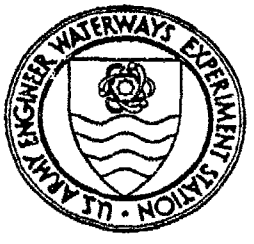
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PROCEEDINGS

Workshop on
MEASUREMENT AND ANALYSIS OF STRUCTURAL RESPONSE IN CONCRETE ARMOR UNITS

23-24 JANUARY 1985

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COASTAL ENGINEERING RESEARCH CENTER

WATERWAYS EXPERIMENT STATION

Vicksburg, Mississippi U. S. A.

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This Proceedings is the record of a workshop hosted by the Coastal Engineering Research Center, Waterways Experiment Station, of the U.S. Army Corps of Engineers, January 23-24, 1985. The objective of the workshop was to review the current state of knowledge of the structural strength of breakwater concrete armor units and to discuss past and proposed measurements of the structural forcing and response. The invited participants represented a purposeful mix of coastal engineers, structural engineers, concrete specialists, and laboratory and field experimenters. Both researchers as well as engineers involved in the design and construction of rubble-mound breakwaters participated.

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PREFACE

This Proceedings is the record of a workshop hosted by the Coastal Engineering Research Center, Waterways Experiment Station, of the U.S. Army Corps of Engineers, January 23-24, 1985. The objective of the workshop was to review the current state of knowledge of the structural strength of breakwater concrete armor units and to discuss past and proposed measurements of the structural forcing and response. The invited participants represented a purposeful mix of coastal engineers, structural engineers, concrete specialists, and laboratory and field experimenters. Both researchers as well as engineers involved in the design and construction of rubble mound breakwaters participated.

The motivation and sponsorship of the workshop was provided by the Crescent City Prototype Dolos Study. This program was begun by the Office, Chief of Engineers to take advantage of the repair of the dolosse section at Crescent City, California. Because of the unique nature of the study and the unlikelihood of another opportunity to acquire prototype scale data from dolos units, the participants in the workshop were invited to advise the Corps in the planning of the proposed measurements.

The workshop was conducted in an informal manner, with a considerable amount of time devoted to discussion. The Proceedings reflect that informality, and to preserve the character of the workshop, little editing has been done. In some cases formal papers with supporting figures were not available and transcripts have been provided.

Those of us involved in the study wish to thank, again, the participants. In all cases the papers and discussions were interesting and informative. Students and investigators who desire to become more familiar with the problem of breakage of concrete armor units will benefit from the results reported here as well as the wide range of hypotheses and opinion.

PROTOTYPE STUDIES OF

CONCRETE ARMOR UNITS

Crescent City Prototype Dolosse Study

Gary Howell

Abstract

The U.S. Army Corps of Engineers, Coastal Engineering Research Center, has begun a program to acquire measurements of the structural forces and responses of 42 tonne dolos armor units in a prototype breakwater. Twenty new dolosse will be instrumented as part of the rehabilitation of the breakwater at Crescent City, California. Dynamic measurements of two bending moments and the torque about one shank-fluke interface will be obtained from strain gages mounted on rebar within the dolos. Six of the dolosse will be instrumented to measure the velocity of motion with six degrees of freedom. Data will be digitized at the point of measurement by an on-board microcomputer system. Measurements of wave height, period, and direction will be made simultaneously with the strain measurements. Hydrodynamic pressure measurements will be made in the core material of the breakwater, as well as in the dolos matrix. Data will be used to provide boundary conditions for a finite element structural analysis model of the dolos.

1 Introduction

Past studies of the structural strength of dolosse have generally been based on dryland tests or numerical model studies. Both of these techniques require better description of prototype forcing functions and boundary conditions. Due to the complex hydrodynamics of the turbulent regime in and around a breakwater, analytic determination of the boundary conditions is

*U.S. Army Corps of Engineers, Waterways Experiment Station, Coastal Engineering Research Center

difficult. Uncertainties in applying scaling laws to reproduce armor unit structural characteristics make the use of small-scale physical models of dynamic structural loading subject to doubt when results are applied to the prototype.

The presently planned rehabilitation of the breakwater at Crescent City, California, has provided a unique opportunity to study the structural response of large dolosse in the prototype. New 42-tonne dolosse will be cast and placed on top of existing dolosse. The wave climate at Crescent City is sufficiently severe, each year, to guarantee waves which will cause measurable motion and impact loading of the dolosse. Figure 1 shows the layout of the breakwater. The main leg is approximately 3760 ft long, and the easterly dogleg is approximately 1000 ft long. The dolos section is comprised of 246 unreinforced dolosse distributed along the last 230 ft of the main stem. Since their placement as part of the breakwater rehabilitation in 1974, up to 70 units have been reported to be broken (Edmisten, 1982). At least 22 of these breakages can be attributed to the placement operations (Markle and Davidson, 1984). Since these reports, there is evidence that deterioration of the structure is accelerating with additional broken units and subsidence in the center of the section.

2 Measurement Plan

2.1 Objectives

The primary objective of the Crescent City measurement program is to provide high quality field measurements which may be used to calibrate and verify models of structural stress of dolos. Because of the many proposed mechanisms and contributing factors to dolos breakage, the study should also provide intensive monitoring of all phases of the dolos casting, placement, and subsequent exposure to storm waves. Evaluation of field measurements of strain and dolos motion will be used to provide boundary conditions for Finite Element Method (FEM) model of dolos sufficient to provide detailed descriptions of stress distributions within the dolos. Likewise, analytical models of dolos motion due to wave forcing will be evaluated with prototype data from both offshore wave measurements and dolos

velocity measurements. The results of these analyses, when validated by prototype data, could lead to a simplified structural design procedure based on design wave conditions similar to that available for breakwater stability. Although many aspects of the study are fixed by budget and scheduling constraints, it is anticipated that the results of this workshop will provide guidance towards the implementation of the field measurements.

2.2 Measurement Approach

Based on structural considerations and field experience, it is generally accepted that dolos fractures tend to occur along lines of high stress concentration in and around the shank-fluke interface. The primary parameter of the field measurement program is the measurement of stresses at this interface. Twenty dolosse with strain measurement instrumentation cast internally will be placed on the breakwater along with the other dolosse to be used in the rehabilitation.

Since impact of the dolos is believed to be the primary cause of fracture, six of the strain instrumented dolosse will also be instrumented with accelerometers which will provide measurement of three translational and three rotational accelerations. These will be integrated once to provide a velocity vector which may be used to completely define the motion of any point on the dolos.

Measurement of the hydrodynamic forces acting on individual dolos in a prototype breakwater is difficult with existing techniques. A more feasible approach is to use pressure transducers distributed within the breakwater to measure related hydrodynamic phenomena. Additional transducers will be placed along the seaward face of the cap to measure pressures associated with the wave wall formed by the cap. Data from the transducers can be used individually to determine pore pressures, uplift forces and breaker height. The data will also be analyzed as gage arrays to estimate breaker angles and characteristics of reflected wave energy.

Offshore directional wave spectra measurements will be made simultaneously with the breakwater measurements. These measurements will provide input to the wave force prediction models and be correlated with the dolos motion measurements.

2.3 Supporting Measurements

In order to characterize the concrete material properties, the breakwater structural characteristics, local bathymetry, and the environment of the prototype dolosse, a number of supporting measurements are planned. The casting of 42 tonne dolosse has recently been monitored at Humboldt Bay to determine the extent of curing temperature induced stresses in the concrete. The initial results of those tests will be reported in these proceedings (Norman, 1985).

During casting of the instrumented dolosse at Crescent City, samples of concrete will be taken and returned to WES for material properties testing. Cast and cured dolosse will be subjected to structural modal testing and analysis. These tests will be performed on both instrumented and non-instrumented dolosse to allow for comparison. Additionally, a smaller prototype dolos will be subjected to extensive modal analysis at the Structures Laboratory at WES. These tests will be performed dry and at various submergence depths and in various orientations. The results will be used to compare with the dry land modal analysis of the 42-ton prototype dolos as well as the FEM model.

The instrumented dolosse will be monitored during transport and placement on the breakwater. These data will be analyzed with the aid of a video tape log of the transport and placement procedure.

Once the instrumented dolosse are placed on the breakwater, an extensive monitoring program will be performed during the 2 to 3 year test. Instrumented dolosse will be marked with concrete dye to distinguish them from other dolosse. Both aerial photogrammetry and underwater sonar surveys will be performed regularly to track the position of the dolosse during the nesting and settlement phases.

3 Dolos Instrumentation System

The severe environment of a prototype breakwater requires specially designed instruments and data acquisition systems. The high data sampling rates required to capture dynamic strain events requires that the number of strain gages be minimized. Some traditional concrete strain measurement techniques, such as surface-mounted strain gages, were rejected due to gage

and wiring protection problems. The selected method is to measure gross structural parameters which can be used together with FEM calculations to provide detailed descriptions of the stress distribution within the dolos. Two bending moments and the torque at the shank fluke interface (Figure 2) will be the primary measurements. This method allows the use of strain gages cast internal to the dolos. While it would be desirable to insrument both ends of the dolos, only one will be instrumented due to cost considerations. The moments and torque will be determined by the measurement of strains on four rosettes constructed of reinforcing steel bars as shown in Figure 3. Two moments and two redundant torque measurements can be obtained from algebraic combinations of the strain measurements on each bar.

A system of metal conduit and steel reinforced hose is used to provide support for the instruments and protect the wiring during casting. All strain gages and wiring are tested for waterproofing under pressure cycling. The protecting conduit and hoses are also pressure tested and then filled with oil to provide additional protection from water intrusion. The conduit hose extends to a steel cylinder sleeve cast into the dolos at the lower center of the shank. The wiring continues into an oil-filled flexible tubing, terminating in a waterproof electrical connector inside the sleeve.

After the dolosse are cast and the integrity of the instrumentation is verified, a waterproof cylinder containing a specially designed dolos processor is mated to the underwater connector and installed in the internal steel sleeve. Dolosse instrumented for acceleration will have processor cylinders which also contain the required accelerometers. The dolos processor is an intelligent system which will digitize the signals from the strain gages and acclerometers and provide an interface to the dolos signal cable. All channels of data are sent over the dolos signal cable in a single, high speed serial bit stream. A second serial interface provides commands to the dolos processor from shore.

4 Data Acquisition

Retrieval and recording of data from prototype dolos during storm conditions constitute the most difficult parts of the study. The extremely severe

environment in and around the breakwater during storm conditions requires that the data acquisition system be designed in such a way as to facilitate protection. Also, after placement in the breakwater, access to the dolosse is impossible. This requires that the internal dolos instrumentation and the connecting cable be designed for no maintenance over the lifetime of the system.

Because dynamic strain data from the response to impacts are desired, the sampling rates are quite high. Current plans call for a strain bandwidth of 125 Hz with a sample rate of 500 Hz for each strain channel. It is estimated that this bandwidth should be adequate to distinguish the first five structural response modes of a 42-ton dolos.

Figure 4 shows a block diagram of the data acquisition system. The strain and velocity signals are digitized on board the dolos by the dolos processor. These data are sent by a cable for each dolos to the top of the breakwater. The cable provides power to the dolos processor and instrumentation as well as a two-way digital link. Each dolos signal cable will connect to one of two data concentrators mounted in waterproof cylinders. The two concentrators will be mounted in holes in the breakwater cap on the protected side of the breakwater. The breakwater pressure gages will also connect to the concentrators. Each concentrator will combine data from all of the dolosse onto one cable. The two cables from the concentrators will be connected to a computer trailer parked at a protected site on land. The trailer will contain a high performance minicomputer system which will be programmed to save time histories for impact induced strains and velocities while rejecting data during static conditions. The data from the offshore directional gage will also be acquired directly by the minicomputer system. All data will be saved on disk and then archived on magnetic tape for analysis at WES. A remote telephone link will allow control of the system from WES and rapid recovery of small samples of data.

5 Dolos Cable Protection

During the fall of 1984, a test of proposed cable protection systems for the instrumented dolosse was conducted at Crescent City. The primary system tested was modified anchor chain. Chain sizes up to 3 in., weighing from

70 to 80 lb ft were used. The primary purpose of the chain assemblies was to form an anchor system for the dolos signal cable coming up the breakwater to the cap. Various methods of attaching the cable to the chain were implemented, including holes, clamps, and bindings. Additionally, an armored cable was deployed without chain.

After subsequent inspections, it was determined that the chain assemblies did not move during mild to moderate wave conditions. Attempts to inspect chain motion during severe wave conditions were unsuccessful, however indications from other inspections indicate little, if any, movement due to wave action. The armored cable did show cyclic motion during moderate conditions, indicating that cable fatigue failures of unanchored armored cable would occur. None of the methods employed to attach the cables to the chains proved to be entirely satisfactory. The methods using holes in the chains caused too much stress to the cable due to multiple sharp bends. The section using a straight run of cable along the chain was the most satisfactory, except that the attachment methods either failed or were too time consuming to install.

An additional problem, learned from the test, was the possibility of damage to the cable during installation due to the ability of the chain to bend at a sharp angle, thus forcing the cable below its minimum bend radius.

Based on the results of the test, it has been decided to investigate other methods of attaching the cable to the chain, as well as restricting its minimum bend radius.

6 Conclusions

The dolos armor unit has proven to be an economical and effective tool for breakwater construction by the Corps of Engineers. However due to several recent catastrophic failures of dolos breakwaters in other countries, and experience with breakage of dolos units on Corps breakwaters, there has been increased concern about the structural strength of the dolos unit. Previous assumptions that hydraulic stability was sufficient to guarantee structural strength have been called into question. Progress towards a structural design procedure for dolos units has been slow due to the complete lack of

prototype data. Although field measurements in such an environment are clearly high risk, prototype scale data will help establish or confirm the breakage mechanisms and permit more rapid progress on the various models currently under development.

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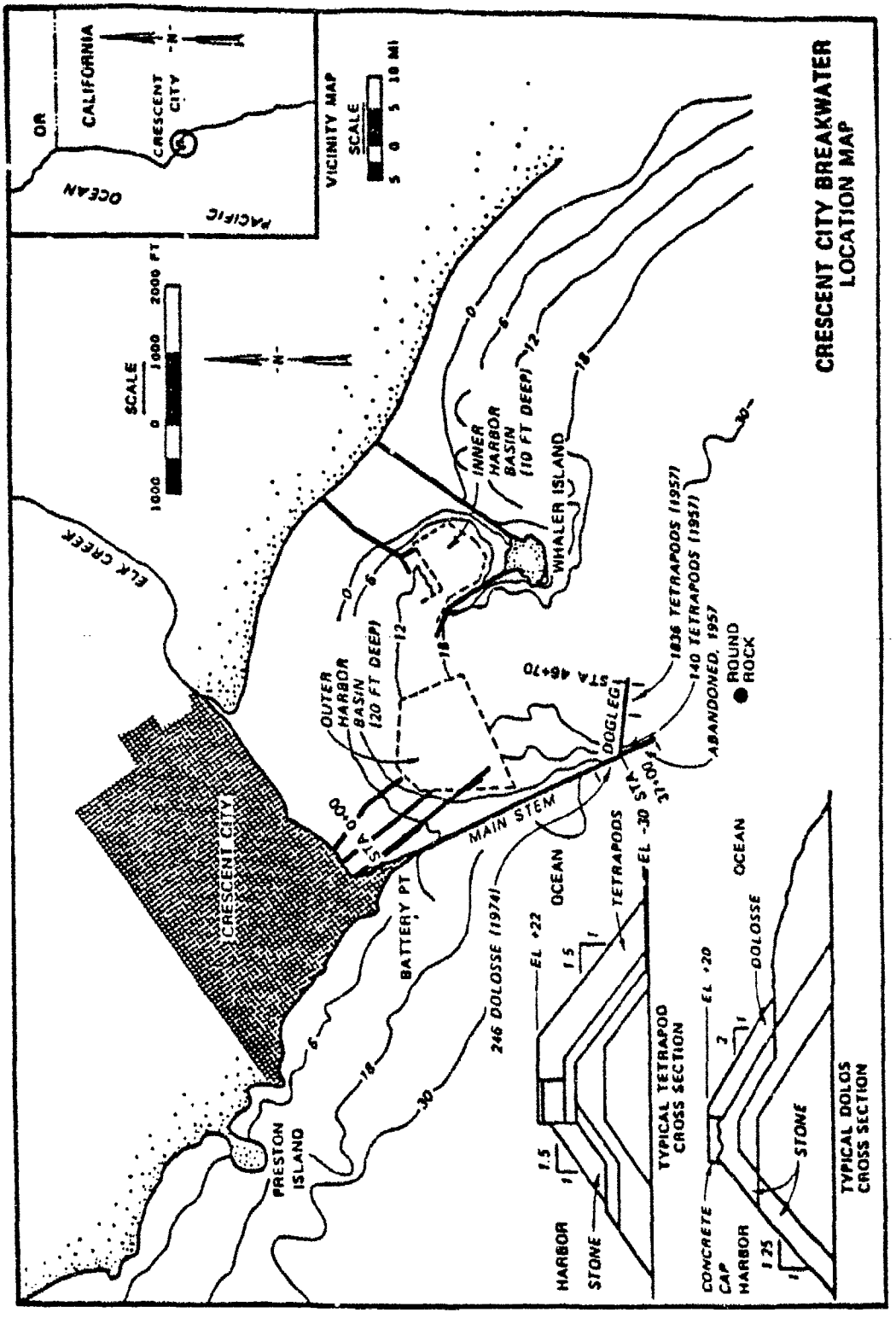
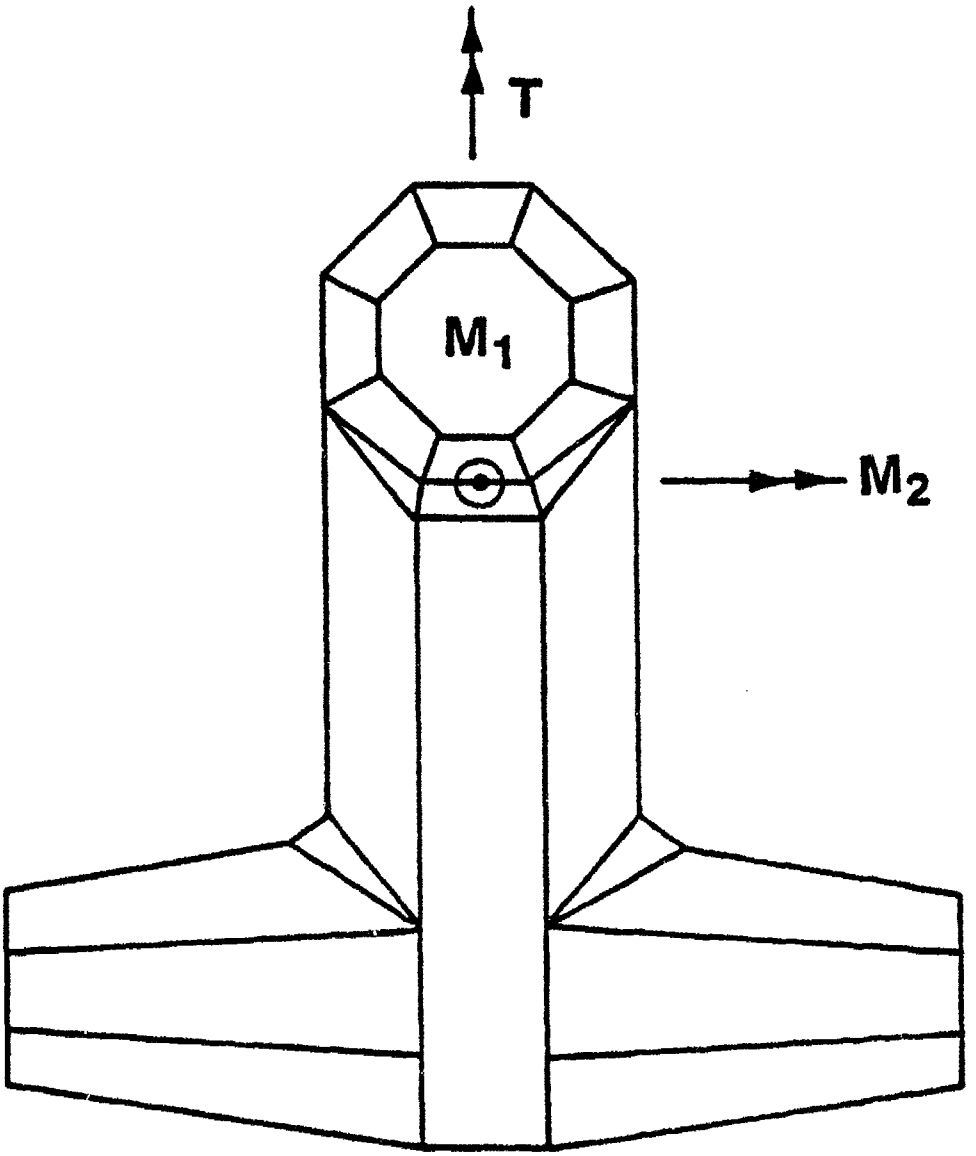
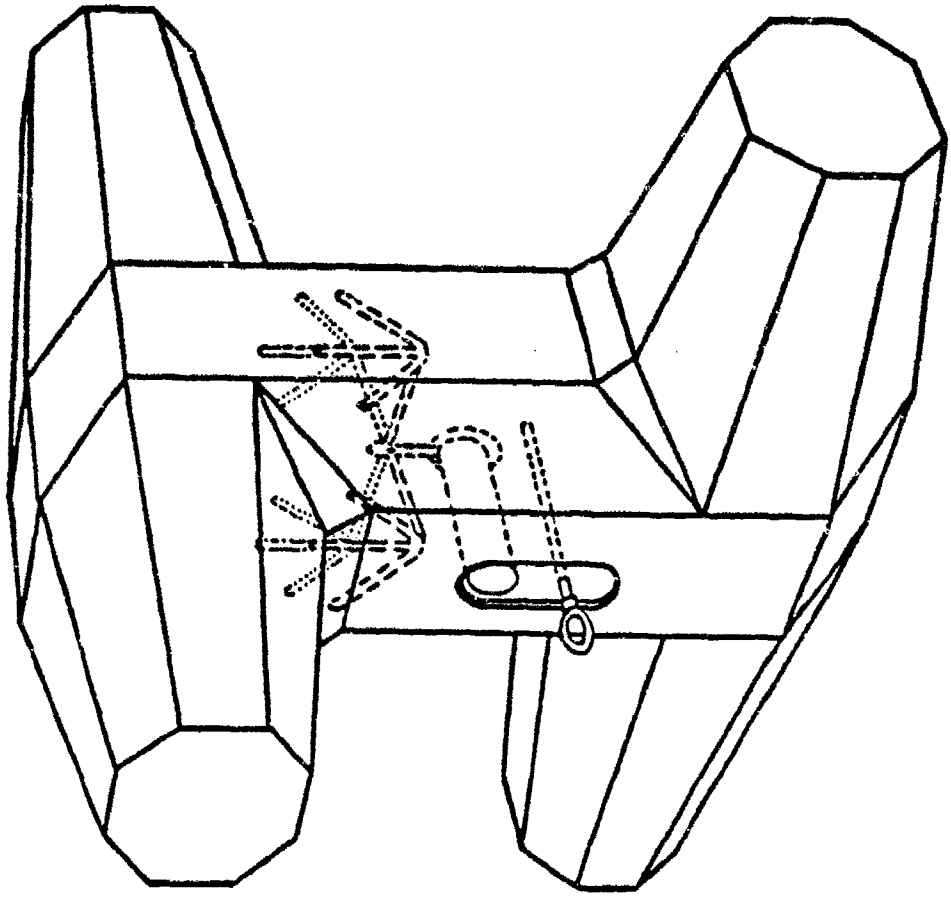


FIGURE 1. CRESCENT CITY BREAKWATER
(FROM MARKLE & DAVIDSON, 1984)



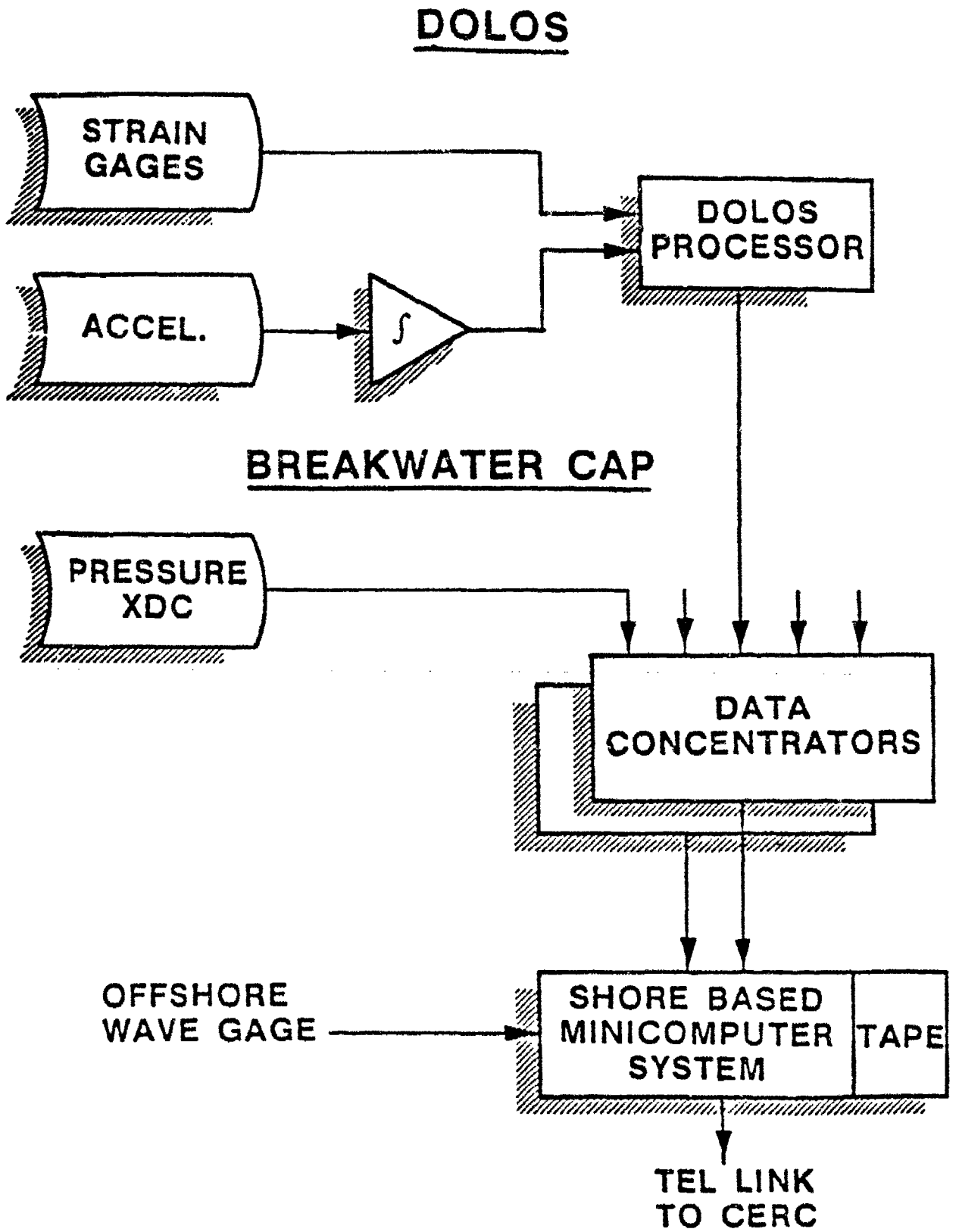
DOLOS MOMENTS AND TORQUE DEFINITION

FIGURE 2



DOLOS INSTRUMENTATION

FIGURE 3



DATA ACQUISITION BLOCK DIAGRAM

FIGURE 4

DISCUSSION OF "CRESCENT CITY PROTOTYPE DOLOSSE STUDY"

MR. GARY HOWELL

DR. TEDESCO:

How do you propose to correlate these measurements you've selected in the field to measure stress evaluations with the environmental condition causing it?

MR. HOWELL:

The wave data will be recorded simultaneously and the pressure data within the breakwater will be recorded simultaneously, so we will essentially have time series data. We're going to do simple statistical correlations, and want to check some of the proposed models' results. In addition, we hope to come up with ideas of our own. It's a difficult problem.

Our structures people think once we have the motion of the units, then we can go to stresses. That's probably a problem that traditional engineering techniques can be applied to, but the problem of going from the environmental wave problem to motion is difficult. We don't have the answer to that question.

Do you have some suggestions, Rod?

DR. SOBEY:

I appreciate your problem with most of the data that you've got and the need to concentrate the data. You talked about reducing it to static averages, wave induced averages, and impact signals.

Do you have any intention of keeping any complete traces? By reducing the data in this way, you're, in a sense, predetermining the analysis to be used on the data.

MR. HOWELL:

We'll have the capability of doing that in short segments. What we would hope to do is take samples for several minutes, at the minimum, of complete time series every once in a while, certainly during a storm situation. As far as getting one hour of records, as you might do for waves, of the actual stresses at 400 hertz, we just can't do it.

MR. MAGOON:

Did you gather any data on those cylinders you put out? Was there anything in those cylinders?

MR. HOWELL:

No. The cable was looped back on itself. The conducters were paired and connected, so we could test to see if there was a break.

MR. MAGOON:

I guess the main question or comment I would like to make is that if the purpose of the work that you are doing is to improve design, then it would seem like you should consider a number of design procedures' scenarios to fit these measurements into the necessary calibration. The other point is that you may be in fact verifying hydraulic model. If in fact that is a goal, then you could perhaps take a little different an approach. If you assume that accelerations, for example, are correct in the hydraulic model, then perhaps you would look at a different aspect of what you're measuring here. So I think it would be important early on to hear what the assumptions are in the models, what the model's characteristics you're trying to prove are, and what you're assuming you don't have to verify.

Obviously, hydraulic models aren't going to go away. Hydraulic models are the most important tool. You mentioned that was in the future. I would think that you could make several basic assumptions of what was needed for those verifications or the actual design test.

MR. HOWELL:

I guess what we're trying to do is assume as little as possible. I'm trying to separate myself from any of the basic theories which have been proposed and to take data which I can use to test them. So most of our assumptions have been not based on a particular model or use of data but on survivability and technical limits.

In other words, if I could figure out a way to do more accelerations less expensively, then we'd do it.

MR. MAGOON:

If I could just respond. I didn't mean that you would bias the measurement to any given hypothesis, but whatever the hypothesis, the data base that you generate would satisfy all of those types of procedures, even though we don't know what those procedures are at this time. You may well come up with a new one based on the prototype information. I think you have to decide what are the known scenarios and then, in addition, other data that you could gather also.

MR. HOWELL:

We have divided the problem conceptually in three parts. The first part of the problem is the wave part, the forcing function which causes drag forces on the unit or motions. Once the unit has a drag force and a motion, there's either a pulsating stress, as Burcharth would define it, or impact stress. Going from that motion to that impact and the resulting stresses of the unit is the second part of the problem. The third part of the problem is once we can predict stresses within the unit, can we predict whether or not it will break. We feel that the difficulty in the knowledge about these problems is probably in reverse order.

I think we're pretty confident that once we're given stresses within the units, we can tell whether it will break. We're fairly confident that once we know motions of the unit and see when it impacts, that we can determine stresses, although that's still a fairly difficult problem. Then of course the most difficult part of the problem is the question of the environmental load on the motions of the unit. It's not clear to me that the same type of analysis as used for hydraulic stability, where we're basically looking at rocking or various subjective determinations of motion, is sufficient for this, but I think it's certainly a difficult question.

Those are, I guess, the assumptions that we're making. What we have tried to do is design the measurements such that we can characterize those, the interfaces of those black boxes, if you will, for the problem.

DR. ZWAMBORN:

As I see it, the most important correlation is your correlation with the environmental conditions you mentioned. Could you give us a little bit more on how you are going to do this? Just looking at the model density and the area, I understand it's quite difficult. There are areas where you may not be able to define the conditions easily. Could you give us a little more detail about how and where you are going to measure the input wave conditions?

MR. HOWELL:

This is something that we don't see as cast in concrete, so we can certainly modify it, at least on the recommendations of the workshop.

To answer your question, what we see as one possibility at this time would be an off-shore directional wave gage in some area where the wave climate could be assumed to be fairly homogeneous and then, we hope, a single point wave measurement directly in front of the section which has the instrumented units.

Now, admittedly Crescent City is basically a shallow breakwater, I think, as compared to what you're doing in Europe, and the bathymetry is not as regular as we would like; however, it is more regular than say, for example, Humboldt or some other sites. So, it's not all good and it's not all bad.

There will be some difficulty in characterizing exactly the wave climate on a particular section. We feel that by having a good measurement of the directional waves, at a point where we can at least calibrate a standard wave model, as well as having a scaler measurement, if possible--we're not quite sure that we can do that, but if possible--directly in front of the breakwater, we will be able to do the best we can.

If anyone has some ideas about how to do it better, we'd certainly like to hear them.

DR. ZWAMBORN:

One more comment on this. A lot of people here can answer this question. Would a single point recording take into account the reflection? Would a single point recorder right in front of the breakwater provide you sufficient information to get the incident wave condition on the breakwater? Would you need, for instance, two?

MR. HOWELL:

You're referring to the problem of reflection off the breakwater?

DR. ZWAMBORN:

Yes. You get 30 percent reflection.

MR. HOWELL:

We hope we won't have that much reflection.

DR. ZWAMBORN:

Maybe we can get back to this.

DR. WOOD:

Yes, let's come back to that question. I'd like to allow the next presenter. Bring that up in discussion.

MR. MAGOON:

I have a point about the wave gage. I think you'll have in the concrete armor units themselves the best wave gage that was ever built, particularly when the waves are small, because all these units will elongate if you have a five foot wave. You'll have, right in the section itself, the world's greatest wave array that you can ever imagine. So I would think that you're actually going to see the waves on the units. Right at that point, the units will be elongating and doing whatever they're doing. There will be enough of them. You'll probably find that the units themselves, once they're calibrated, will be extraordinary wave gages.

MEASUREMENT OF TEMPERATURES AND ASSOCIATED STRAINS DURING DOLOSSE CONSTRUCTION

by

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and

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Structures Laboratory

Measurement of Temperatures and Associated Strains During Dolosse Construction

Background

For dolosse armor units to be effective in dissipating incident wave energy over a practical time period, the unit must maintain its structural integrity. Quite often structural failures of such units are associated with cracks which have grown or propagated to an extent such that the unit is incapable of resisting its design loads. In order to accurately assess a particular armor unit design concept, the extent of cracking at the time the unit is first put into service must be known. Therefore, the probability of construction-related cracking should be evaluated. The work reported here was directed toward experimentally evaluating the probability of significant construction-related cracking for dolosse structures.

Objective

The objective of this study was to measure temperatures and strains during the dolosse construction and curing process. Based on the magnitudes of these strains the potential for construction-related cracks to form at critical regions in the structure is evaluated.

Test plan

Two dolosse were used for measurement verification. The dolosse forms at the Eureka, California, construction site were made available for use in the thermal strain study. These forms were set up at the north end of the construction site with one form on the ocean side (west dolosse) and one on the east side (east dolosse). Care was taken to simulate normal construction procedures for the two dolosse being tested. The test plan was to continuously monitor temperatures and strains at critical points in the dolosse from the time of concrete placement until strains associated with the construction phase were felt to have reached a steady-state condition.

Concrete placement

Concrete used in the dolosse construction consisted of deformed steel fiber-reinforced concrete with 0.42 water-cement ratio. Placement of the concrete was accomplished by a truck equipped with an 8-cu yd hopper and self-contained conveyor system for discharge into the dolosse forms. The concrete

was dropped through form openings at the tops of the fluke and the shank and then vibrated. The impact of falling concrete along with vibrating effects made placement of thermal strain instrumentation very difficult. Concrete was placed in the test dolo's forms on 18 June 1984 from 1530 hr to 1745 hr or approximately 1-hr 45-min placement time for each dolosse. Forms were removed after 1 day and jack stands, which were used to keep the axis of the shank horizontal, removed after 7 days.

Instrumentation

Twenty Carlson strain meters were placed in each dolosse to record temperatures and strains during the construction and curing process. These meters are 4 in. long and measure both temperature and strain with resolutions of 0.1° F and 3 μin./in., respectively. The sensing device in the Carlson strain meter operates on the principle that a metal wire will change resistance due to a strain and also will change resistance due to a temperature change. Precision four-terminal resistance measurements were made to the nearest 0.0001 ohm to determine strain and temperature. Relative gage locations, as shown in Figure 1, were the same for each dolosse; detail locations of the gages are presented in Figures 2 through 5. The metal frame shown in these figures was designed so that its ends would fit tight against the inside surface of the dolosse forms. This was required to withstand the effects of the concrete placement.

Data were recorded on five magnetic disks in a Hewlett Packard (HP) 3597A data acquisition system and HP 85 computer. A portable generator was used to power the system. In addition to the instrumentation discussed above, four test cylinders were cast and monitored during the test program.

Presentation and discussion of test results

Measured temperatures and strains are presented in Figures 6 through 29. Figures are labeled east or west dolosse, section, and central or outside portion of cross section. A legend identifying actual gage numbers is presented in the lower right-hand corner of each figure.

Measured temperatures were normal with maximum temperatures occurring near the axes of the shank and fluke. The highest temperature rise occurred at the center of section CC and was approximately 85° F based on an initial temperature of 60° F. This maximum temperature occurred approximately 20 hr after placement of the concrete. Temperatures at points nearer the outside surface were more sensitive to the daily changes in ambient temperature as shown in Figure 11.

During the first few hours of the curing process, concrete slips at the concrete-strain gage interface until bond is developed. This bond is essentially completed at time of initial set or about 6 to 8 hr. Prior to this time gage-indicated strains are not accurate. For example, the four strain records of Figure 13 would indicate that gage 4502 recorded the largest strains of the four gages with a fairly constant value of approximately -6 μin./in. However, when the strain records prior to set are neglected for each gage and they are plotted from a common preset strain point, the actual strains are as shown in Figure 14. Also, strains associated with free thermal expansion were determined for this concrete and subtracted out of all strain records. Therefore, the strains measured after initial set represent strains associated with stresses. With this in mind, an analysis of all the measured strain data indicates very little, if any, construction-related stresses were developed for the two dolos tested. The maximum relative strains over a cross section appear to occur at section CC as shown in Figure 25. However, these strains are only on the order of 5 μin./in. which is negligible compared to expected ultimate strain capacity of the concrete.

In all strain records there is a slight constant shift in strain beginning at approximately 25 hr after placement and ending approximately 25 hr later. Because it occurred in both structures and all strains were of one polarity, it appears to be an electrical shift rather than mechanical. In fact, until all the length changes due to the temperature coefficient were removed from the data, the shift was not recognized. So no effort was made at test time to check for electrical ground problems. Because the amount of shift is in the range of the minimum resolution of the meters (3 με) and does not approach the ultimate strain capacity of typical concrete (100-150 με), it was not considered significant.

Conclusions

Based on the results of these tests, there appears to be no real potential for significant construction-related cracks.

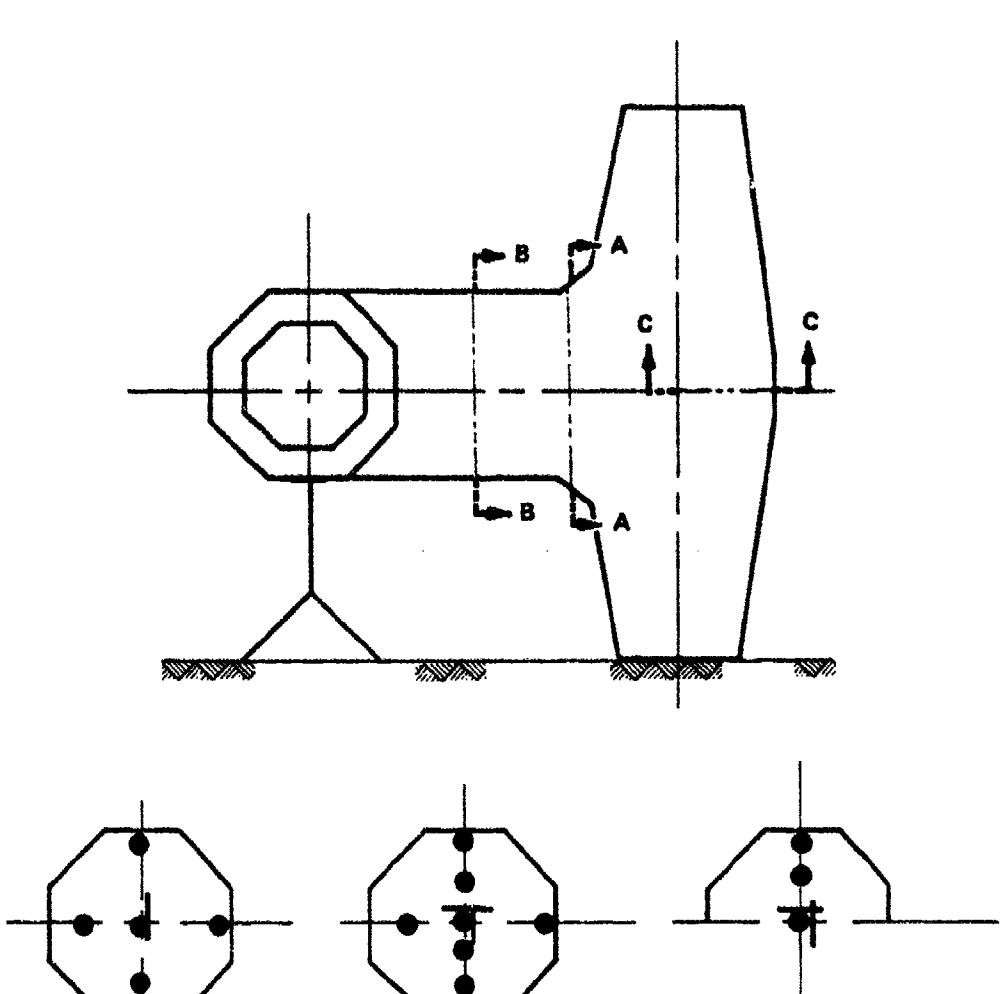


Figure 1. Location of temperature and thermal strain gages.

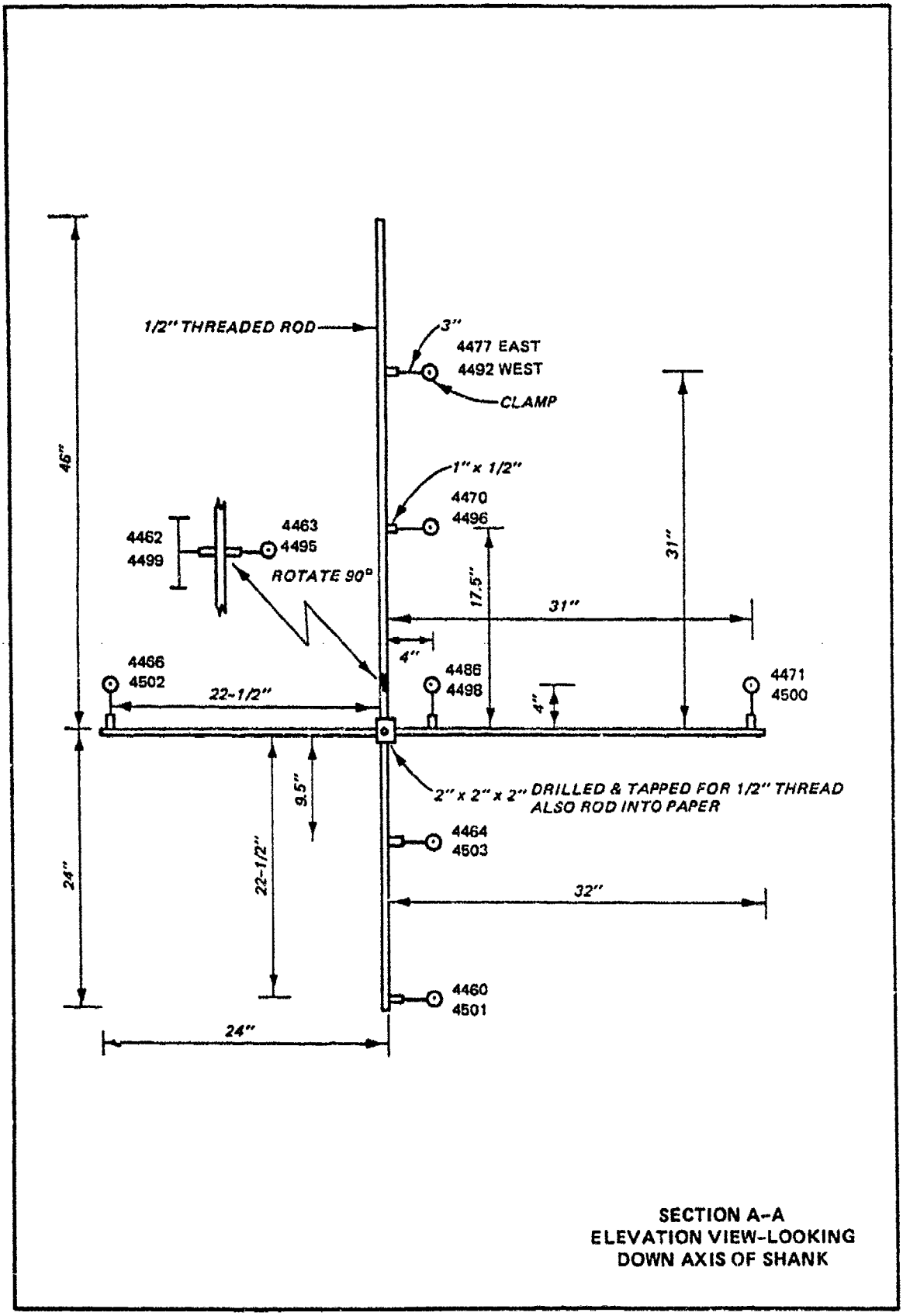


Figure 2. Detail gage locations.

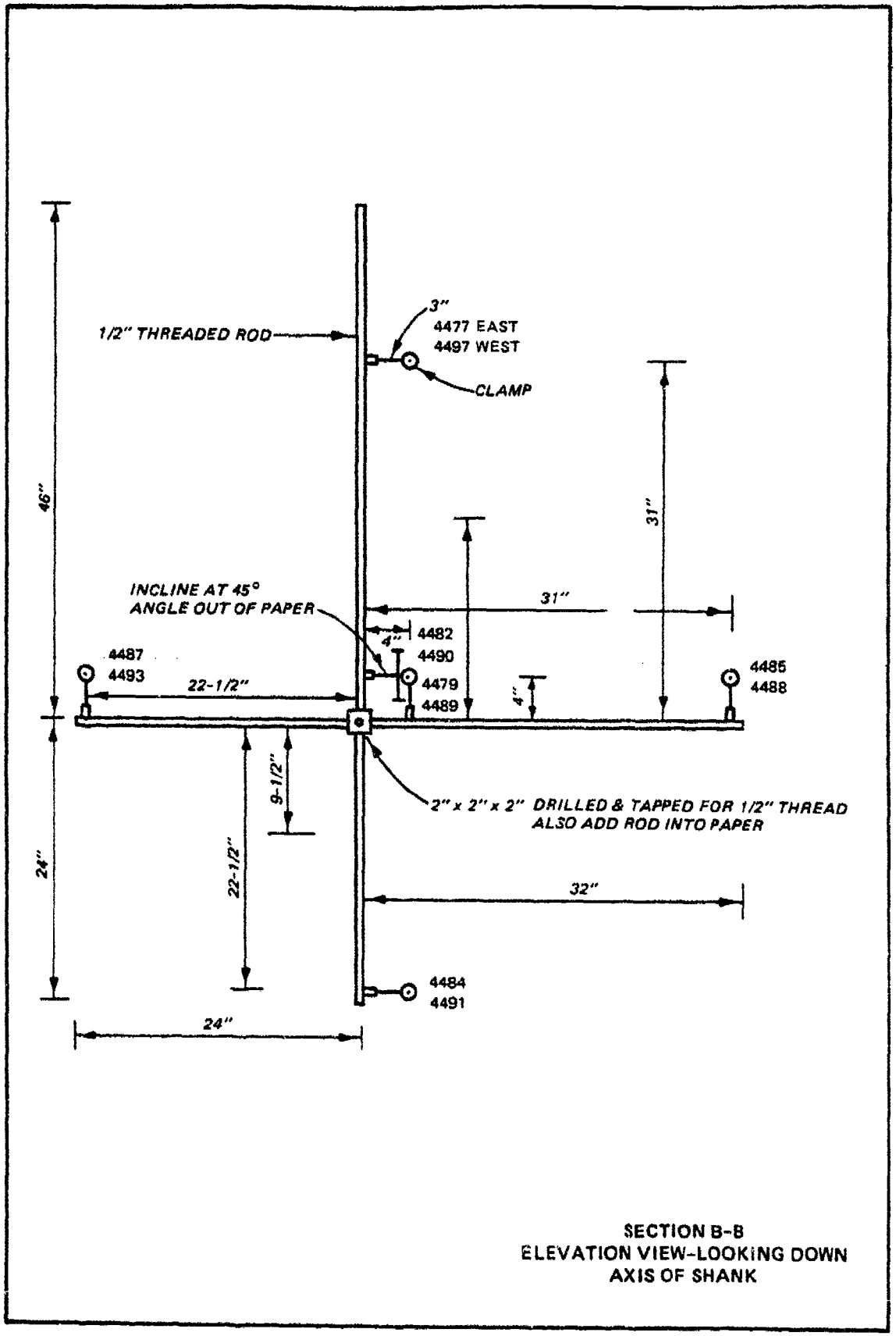


Figure 3. Detail gage locations.

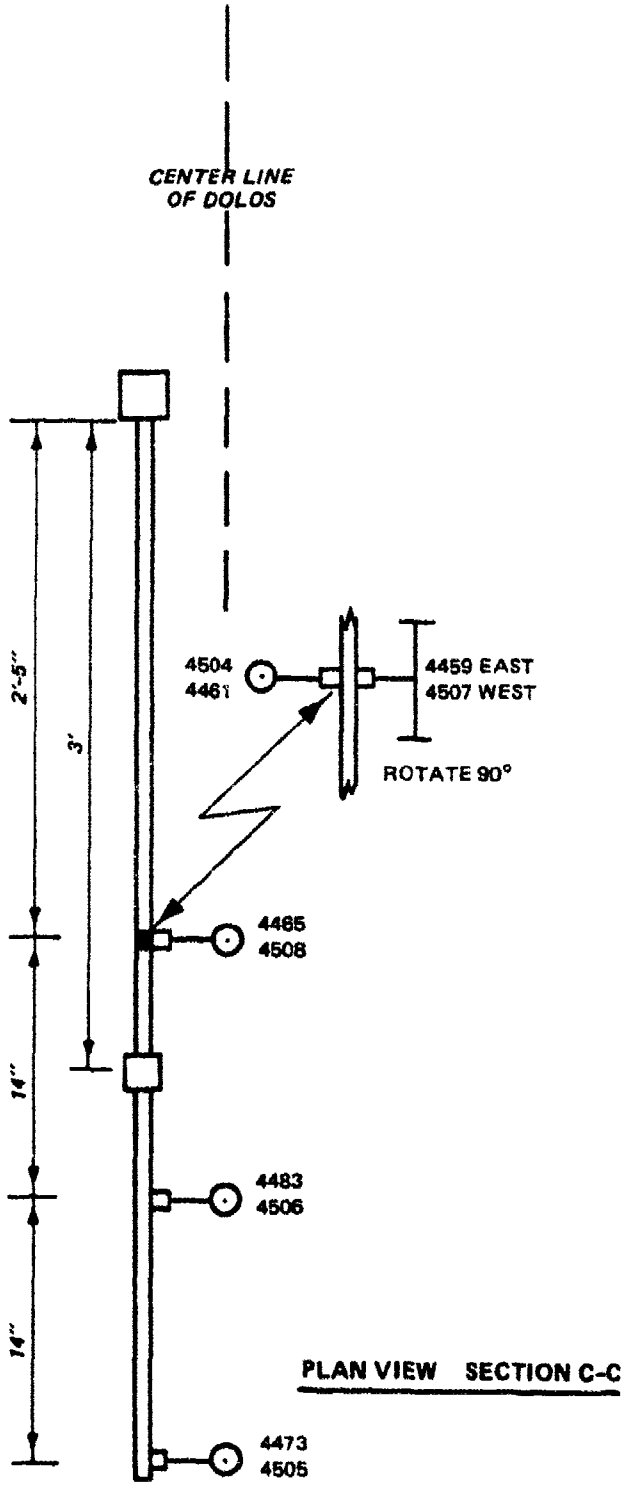


Figure 4. Detail gage locations.

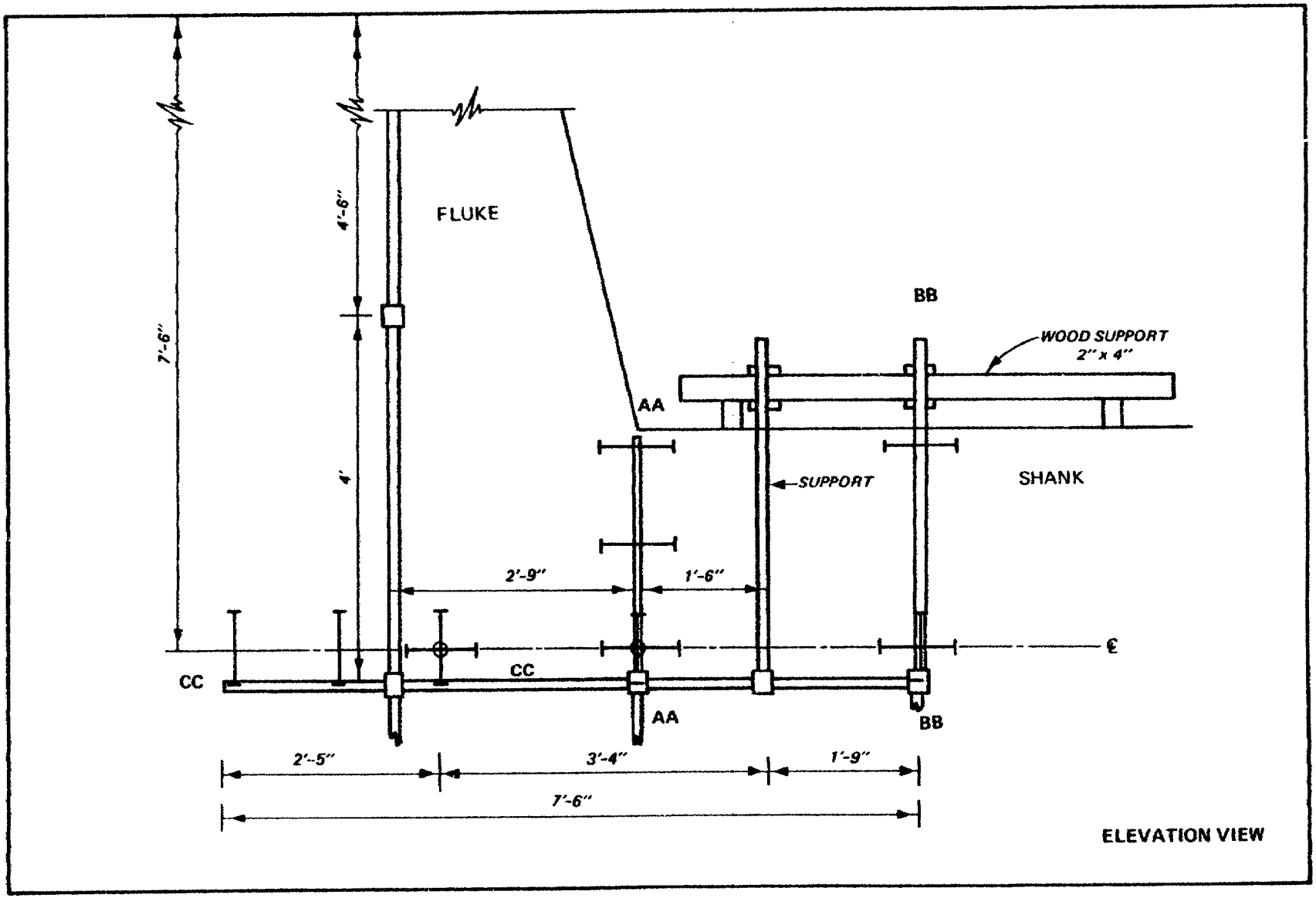


Figure 5. Detail gage locations.

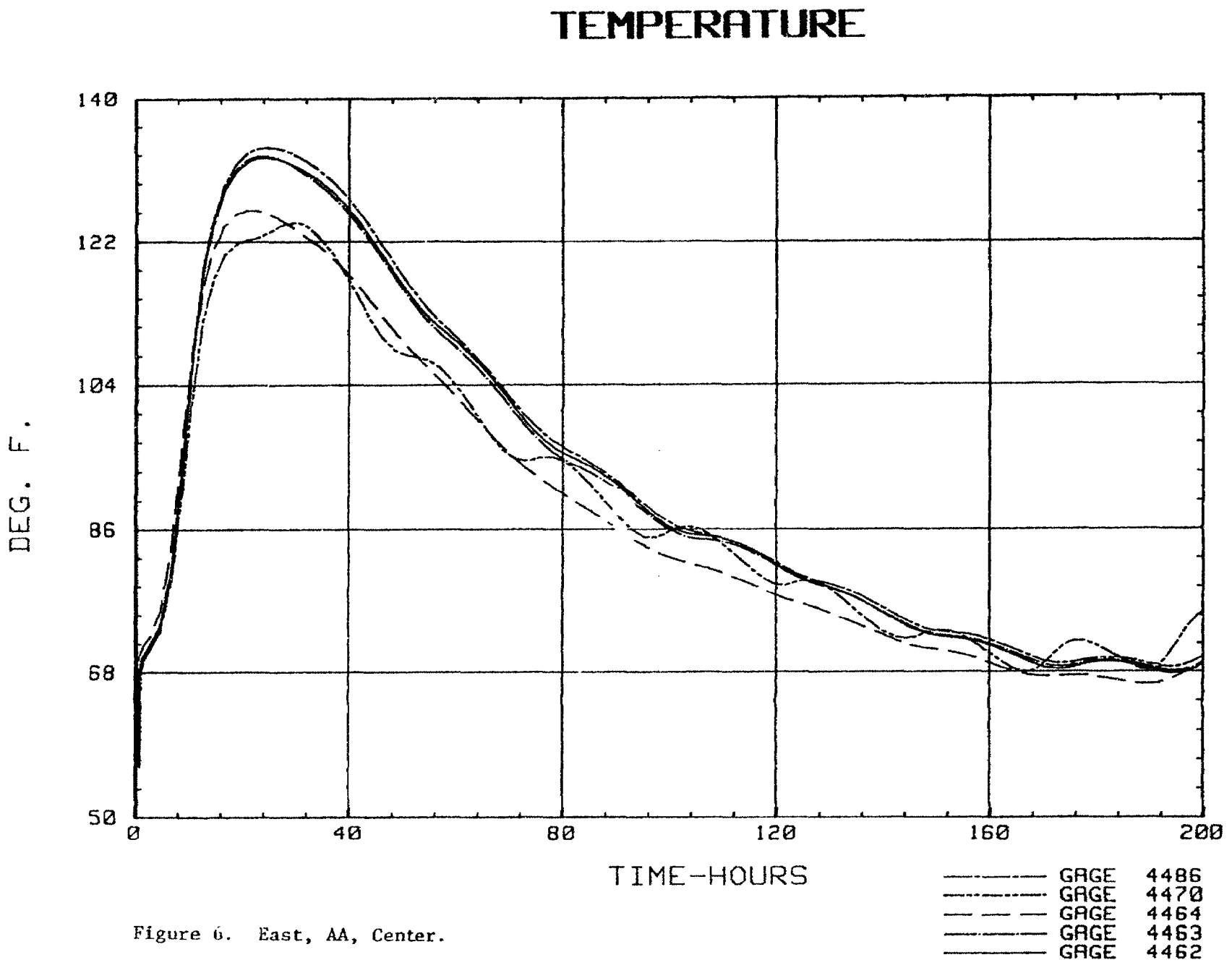


Figure 6. East, AA, Center.

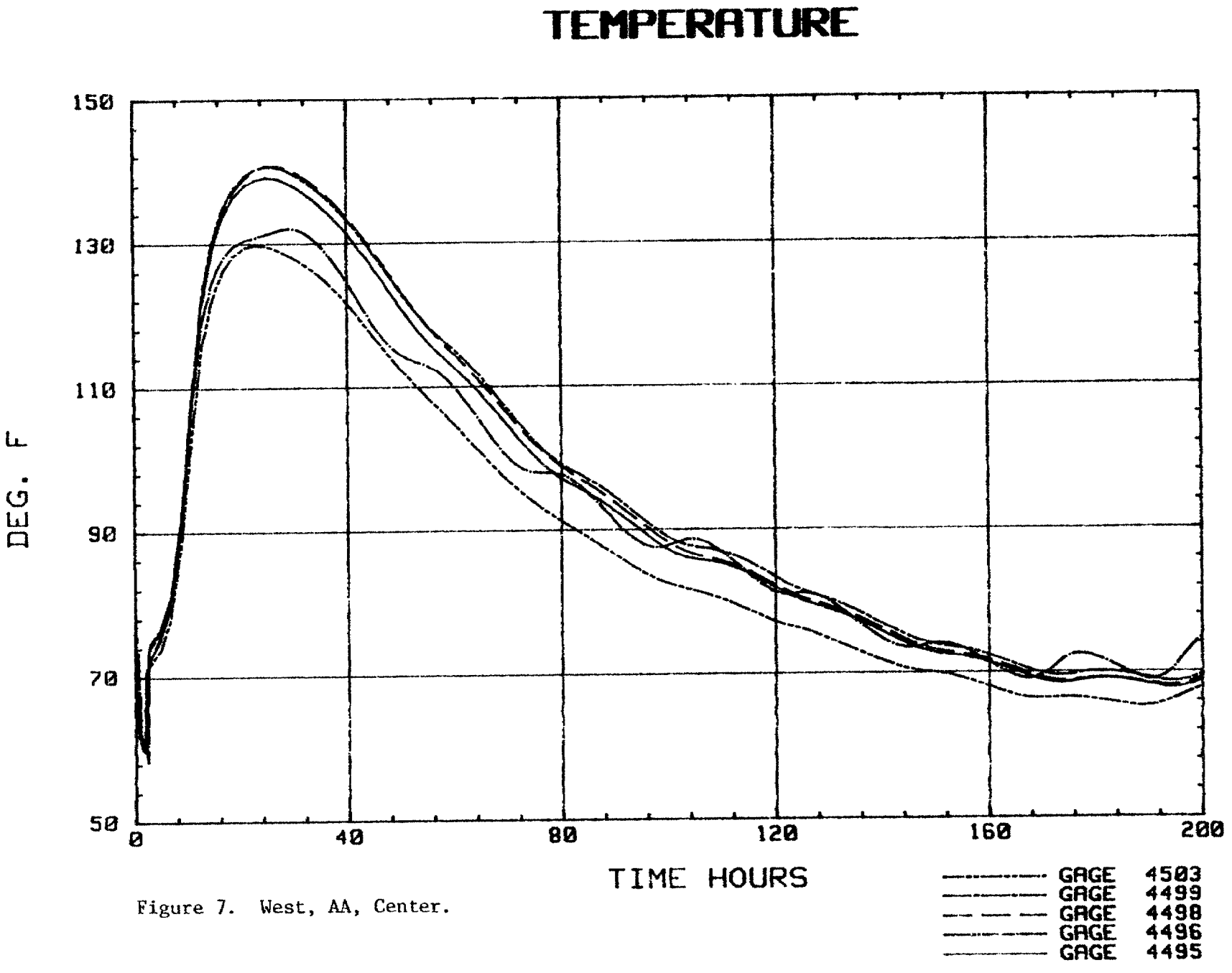


Figure 7. West, AA, Center.

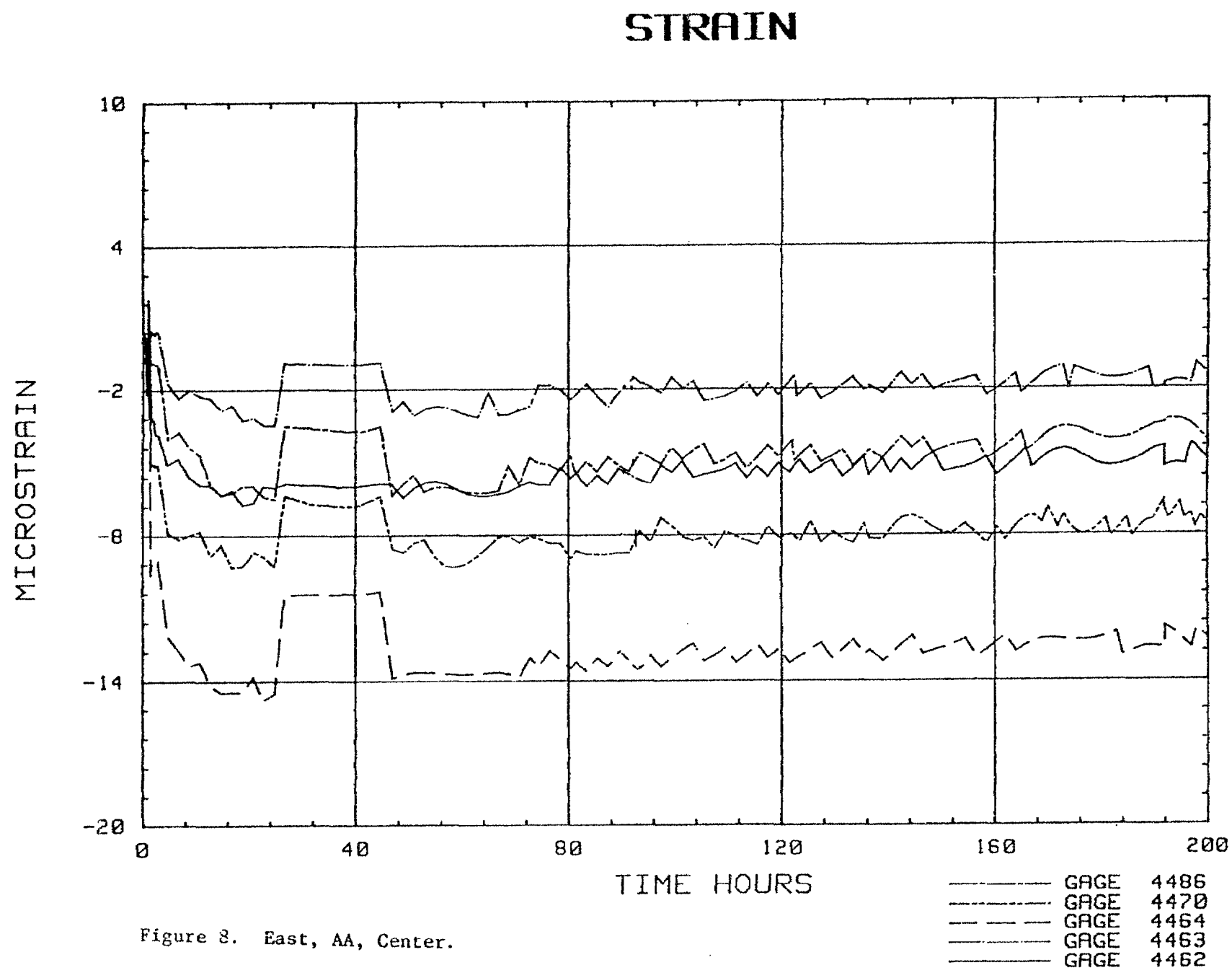


Figure 8. East, AA, Center.

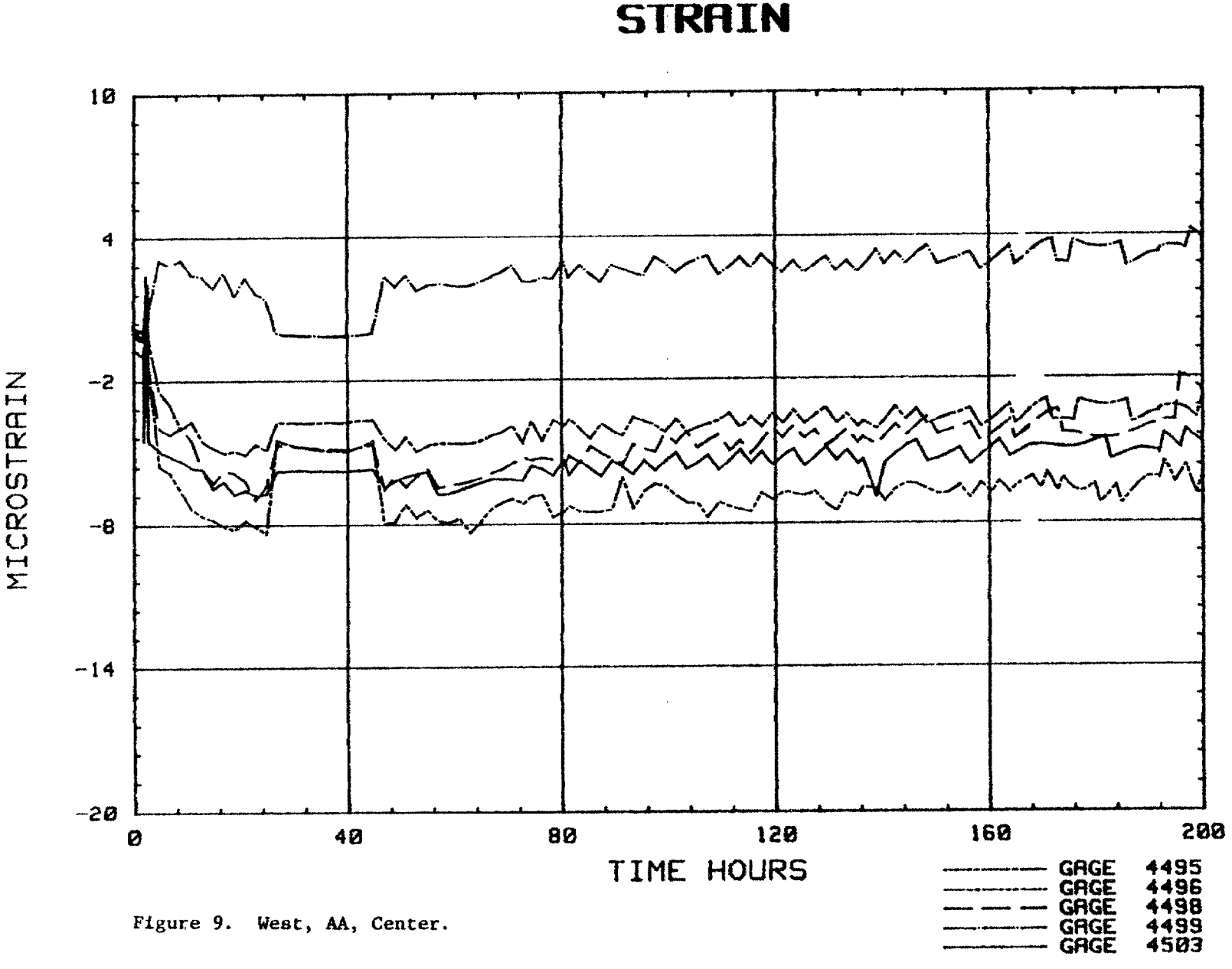


Figure 9. West, AA, Center.

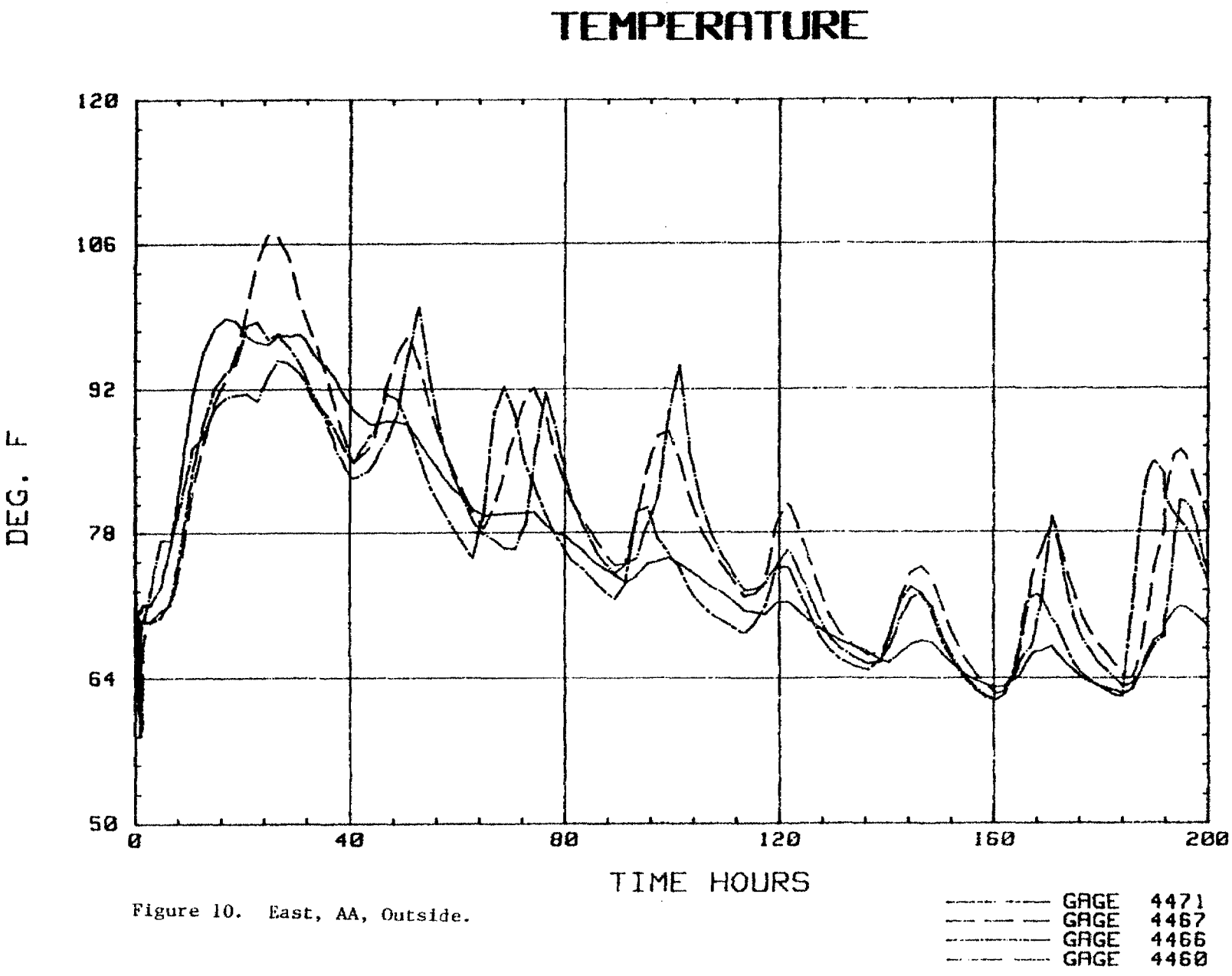


Figure 10. East, AA, Outside.

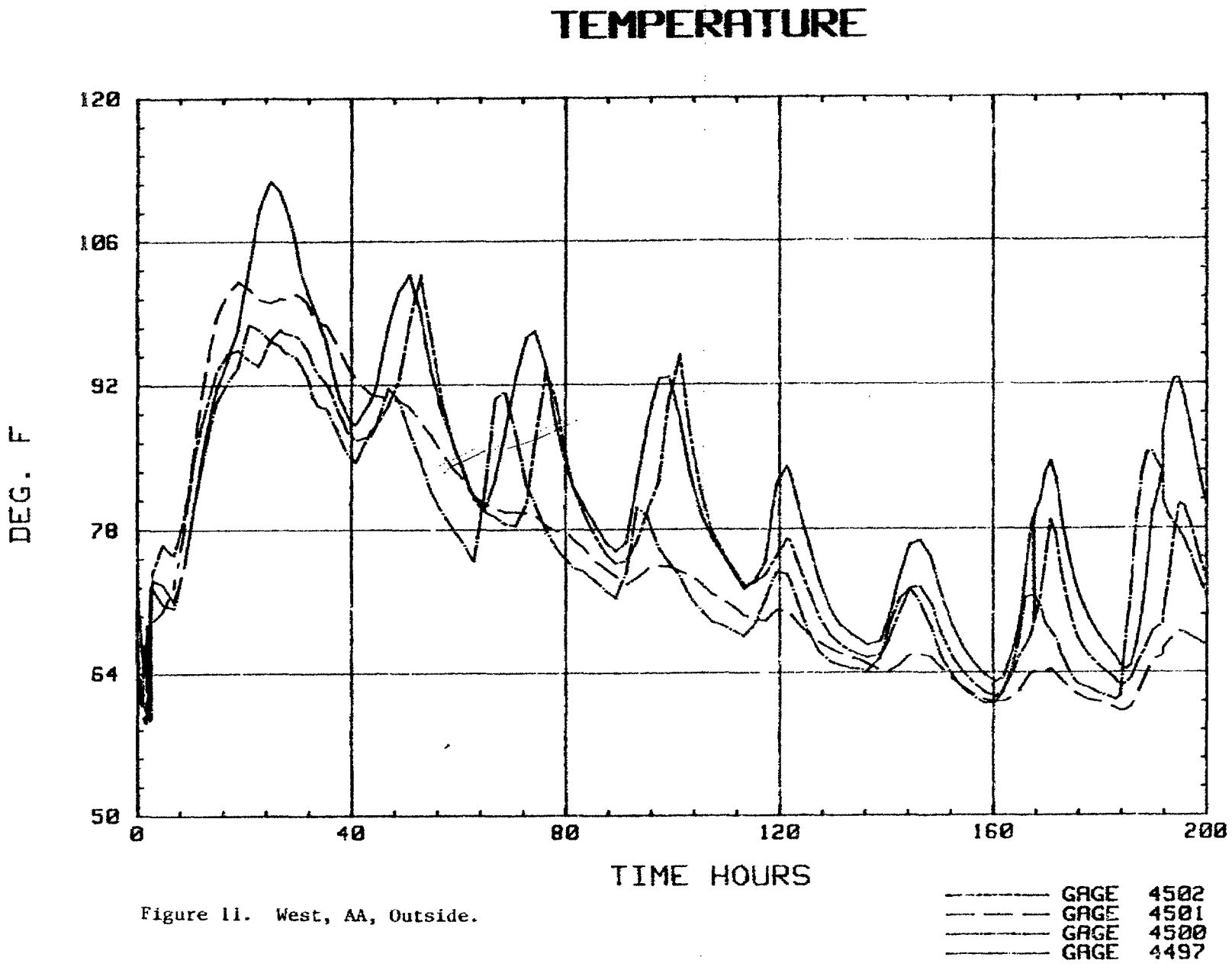


Figure 11. West, AA, Outside.

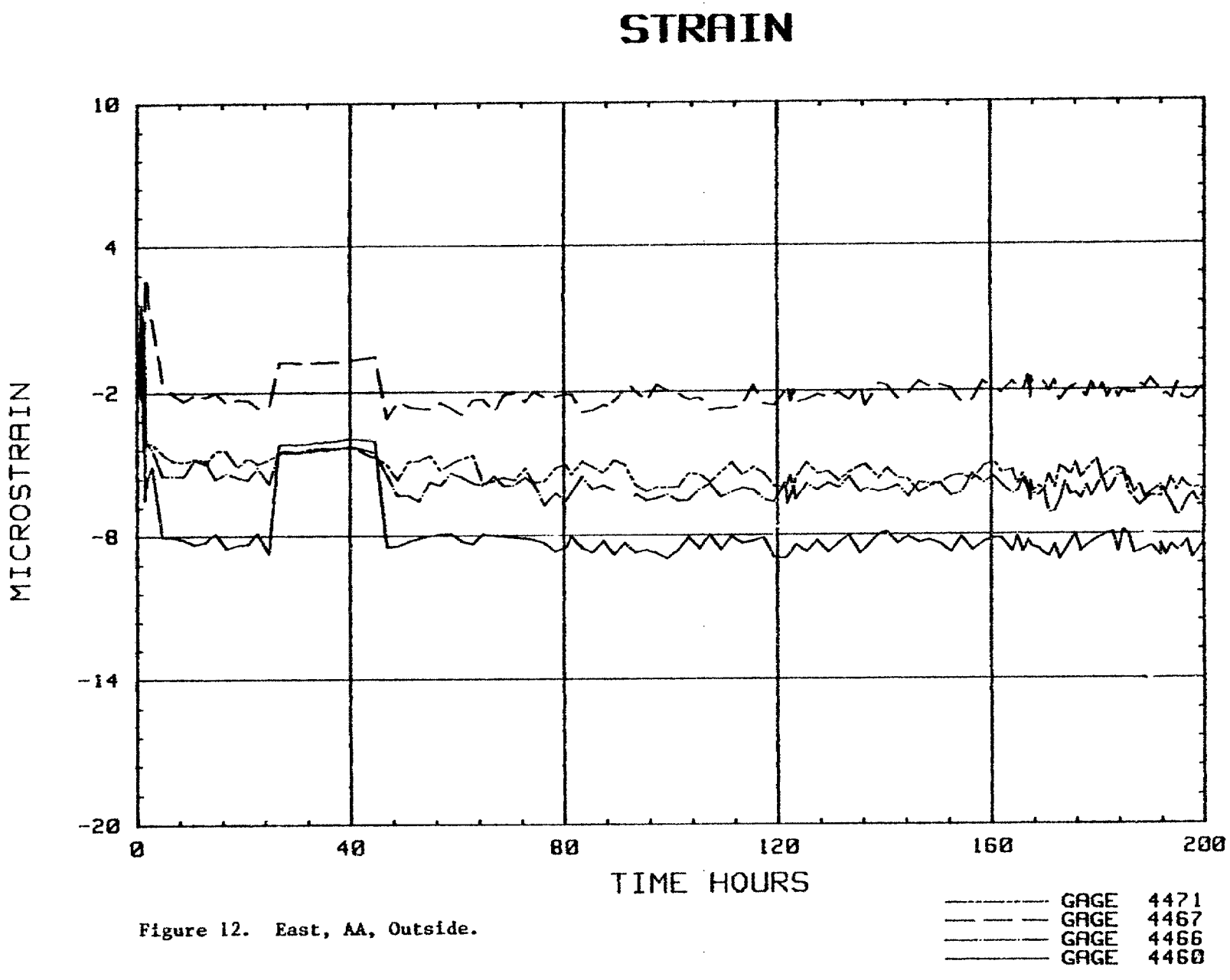


Figure 12. East, AA, Outside.

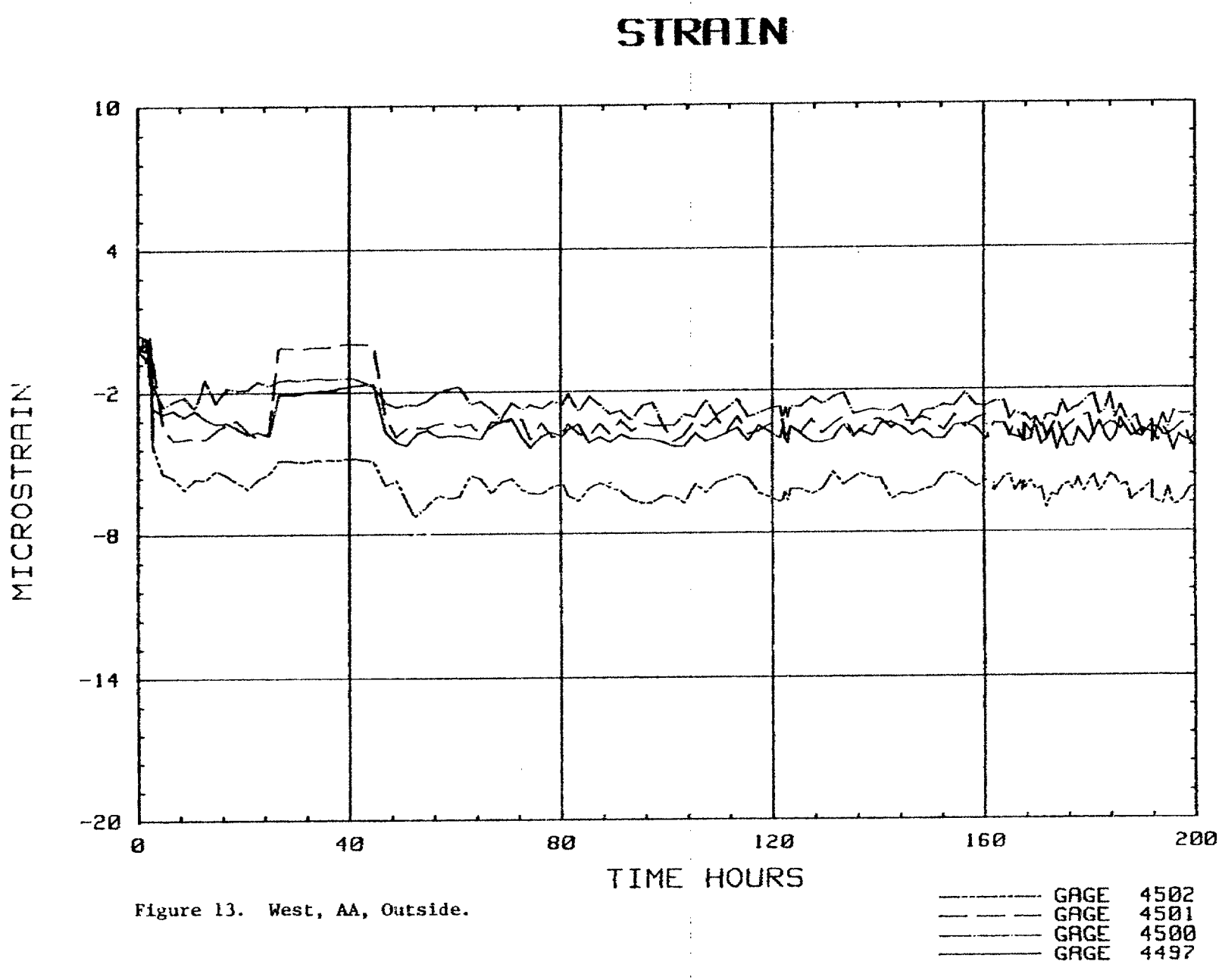


Figure 13. West, AA, Outside.

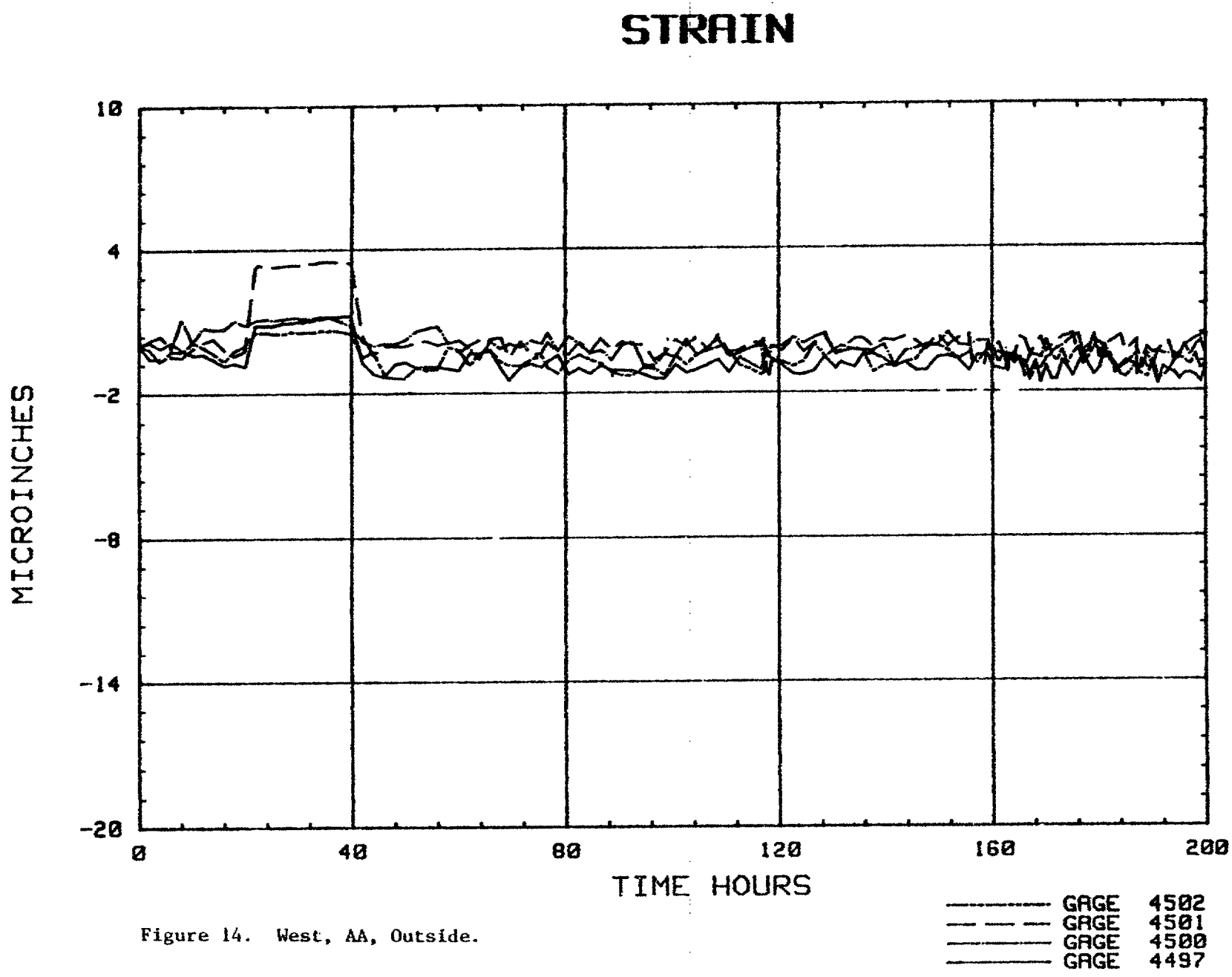


Figure 14. West, AA, Outside.

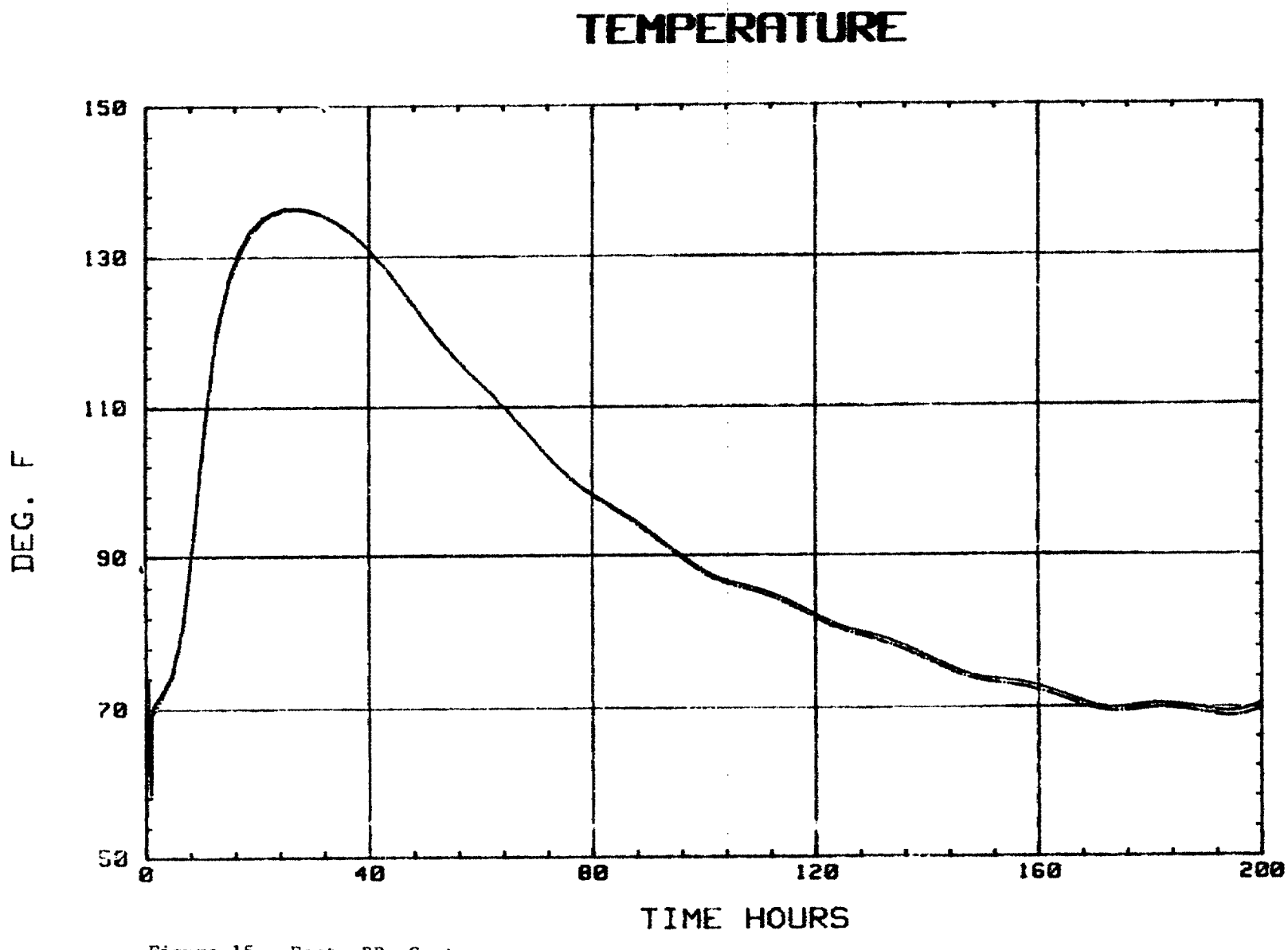


Figure 15. East, BB, Center.

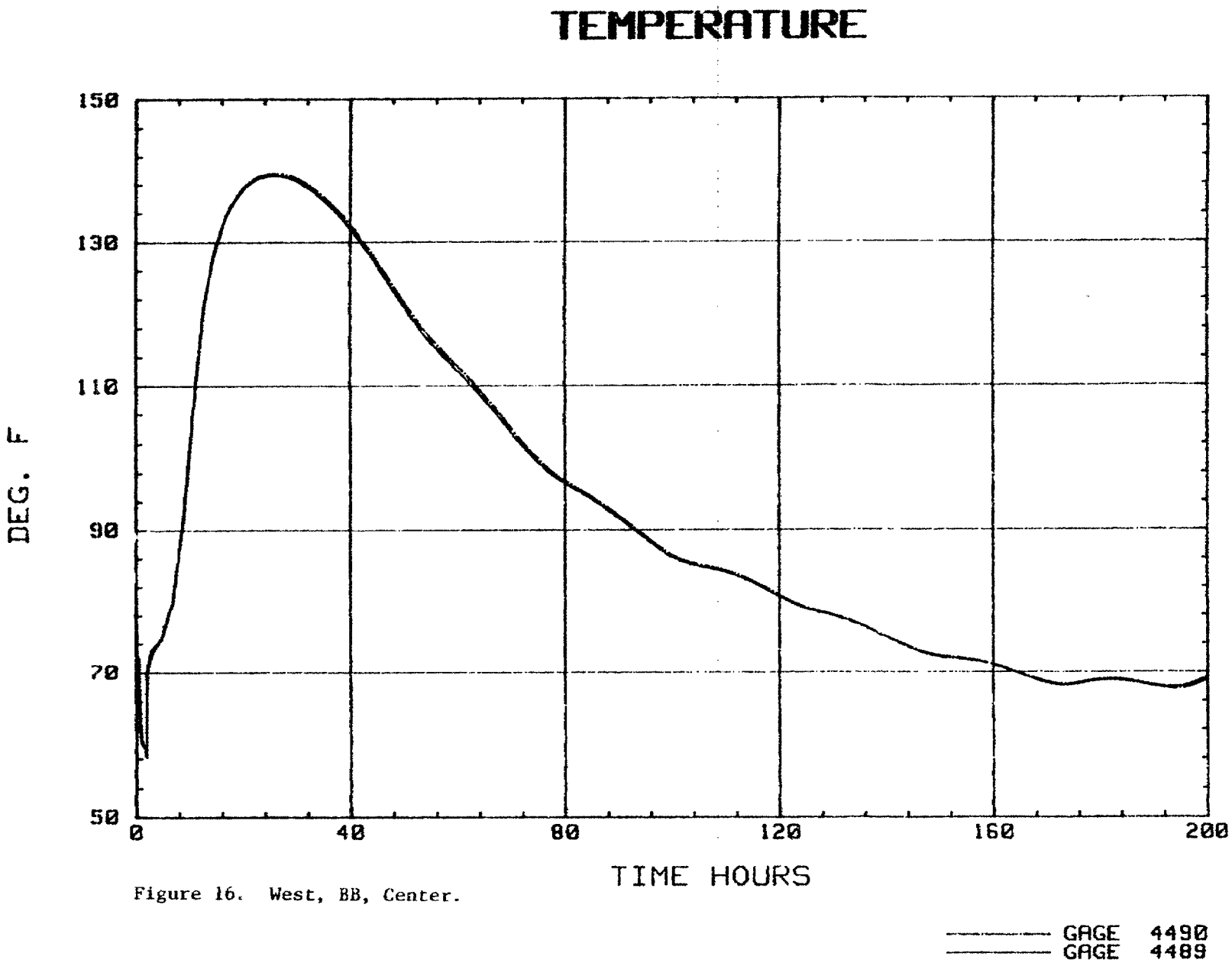


Figure 16. West, BB, Center.

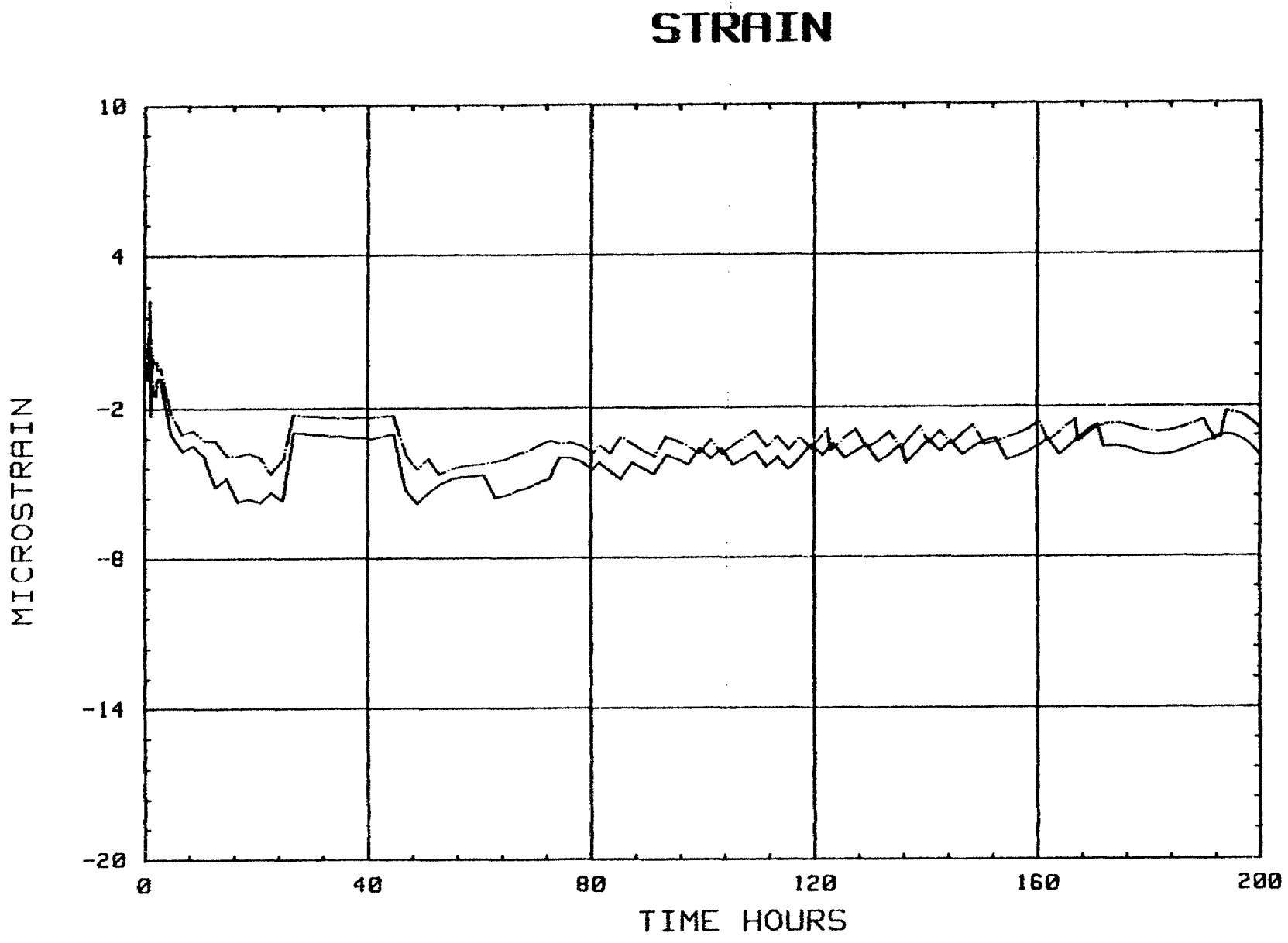


Figure 17. East, BB, Center.

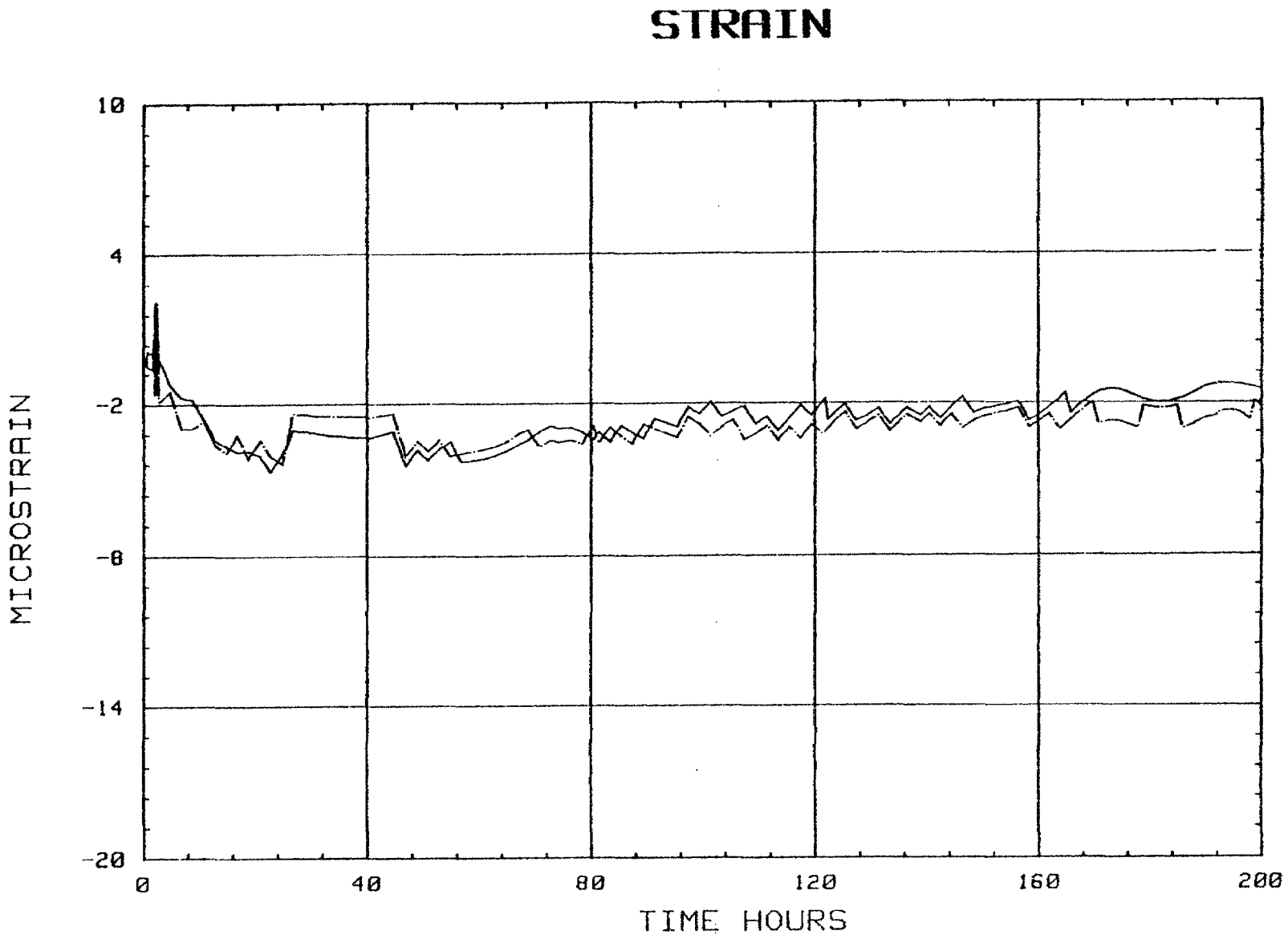


Figure 13. West, BB, Center.

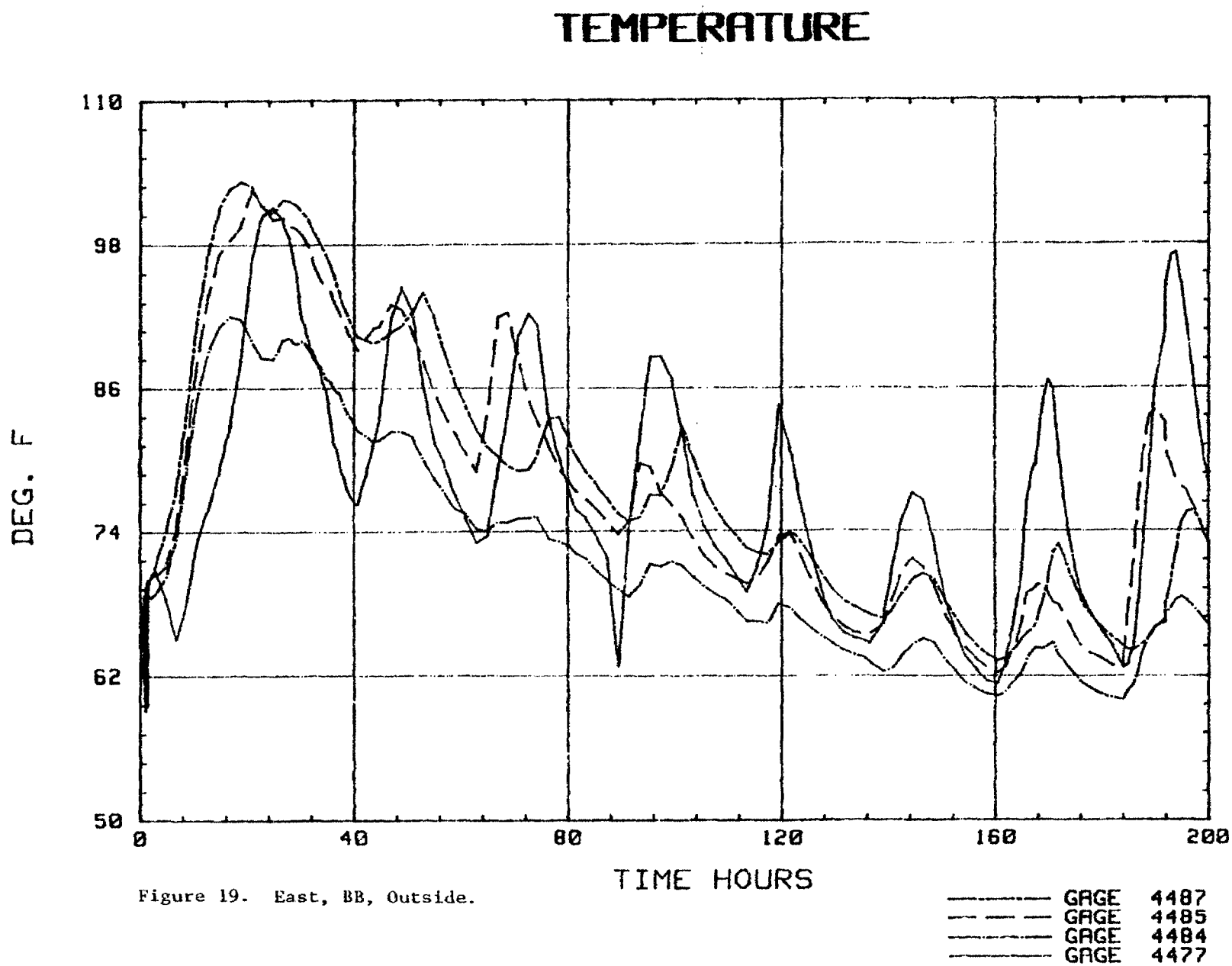


Figure 19. East, BB, Outside.

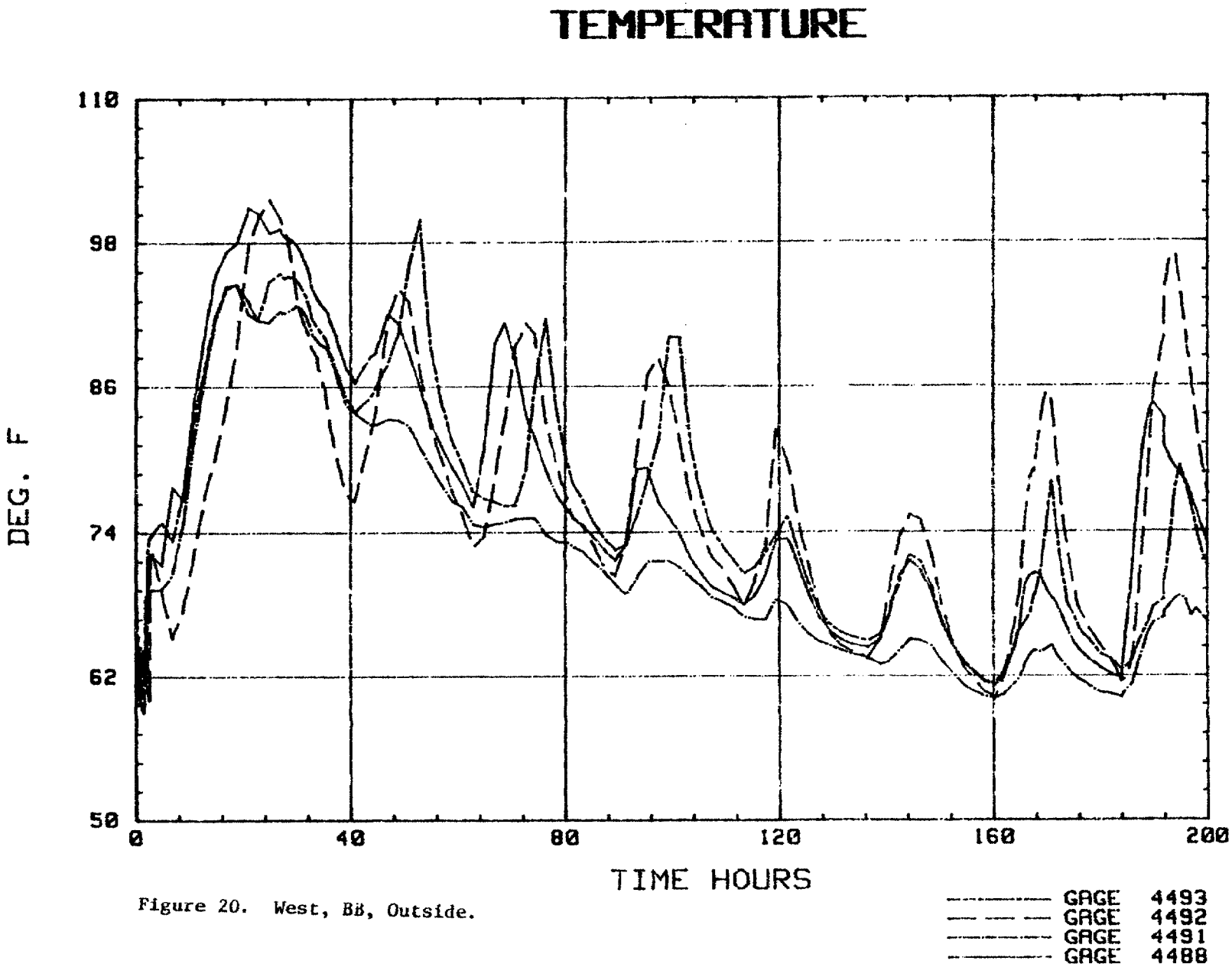


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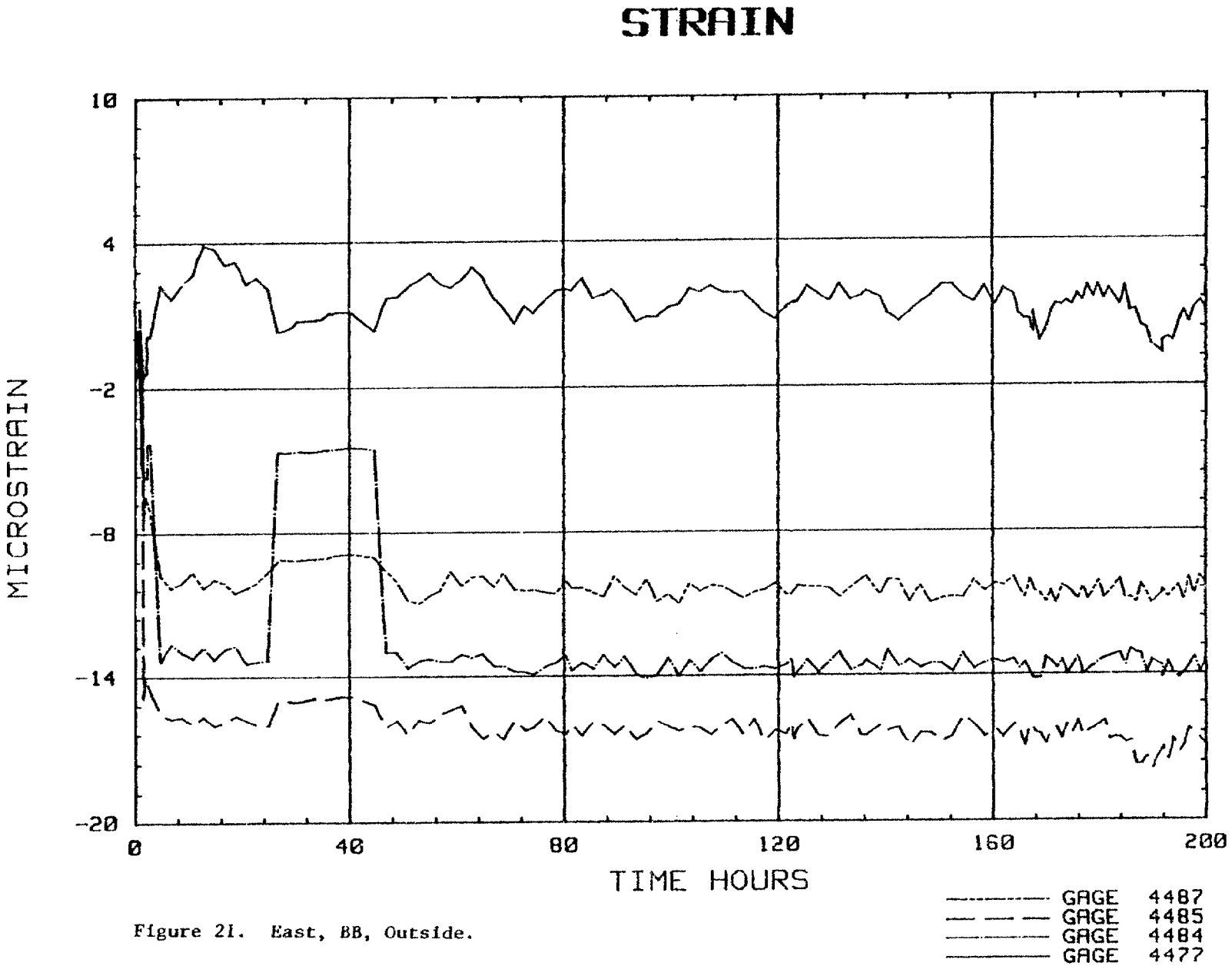


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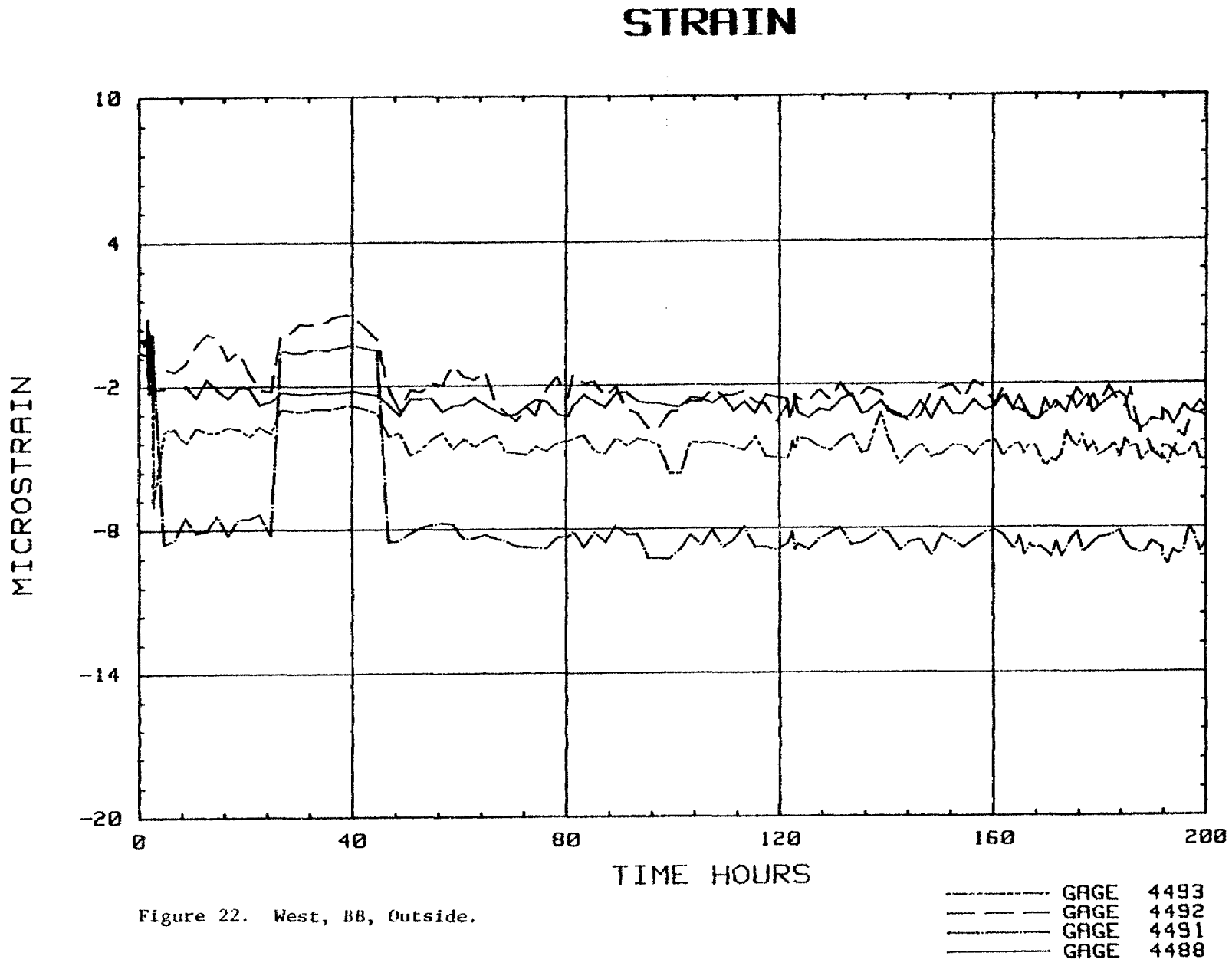


Figure 22. West, BB, Outside.

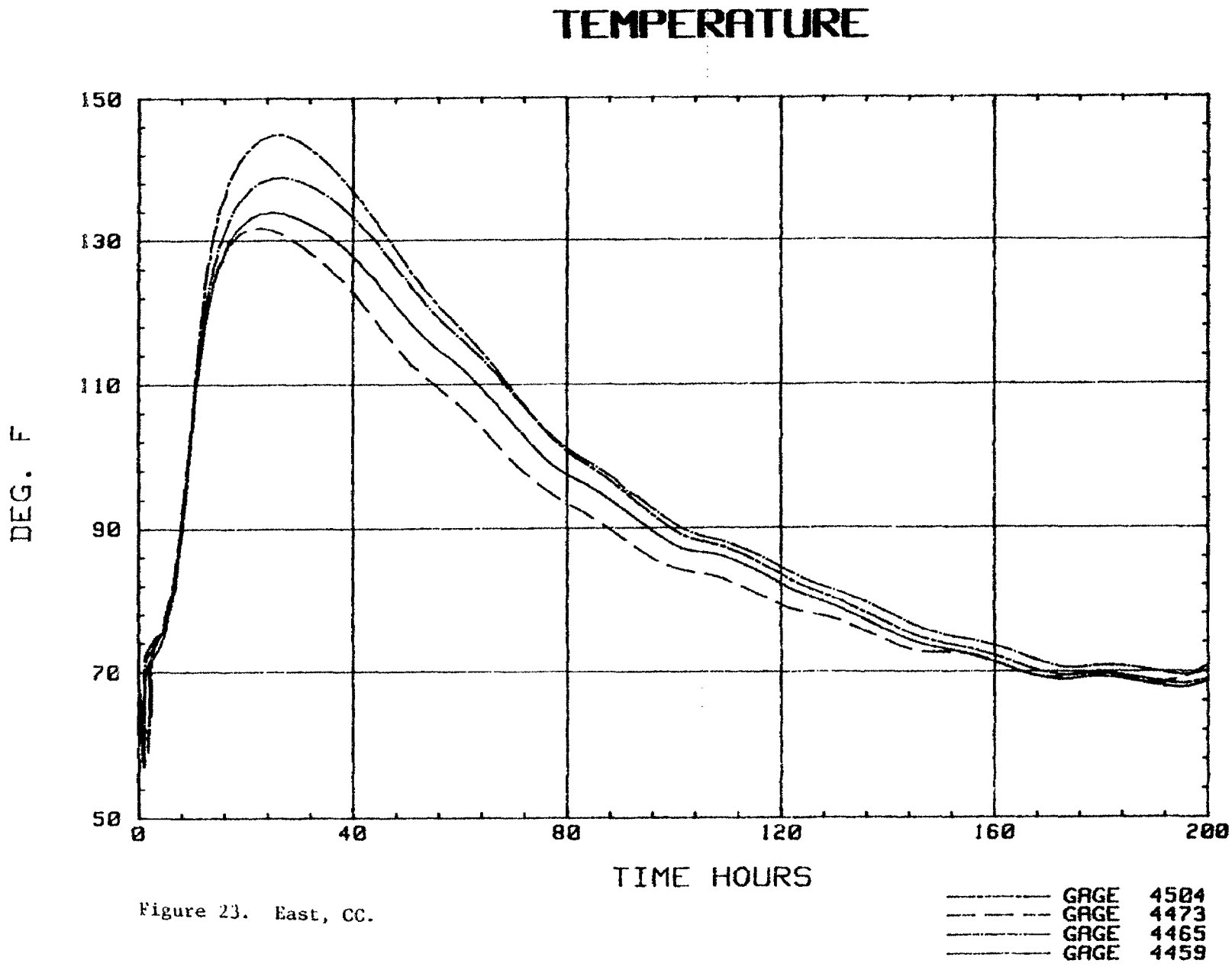


Figure 23. East, CC.

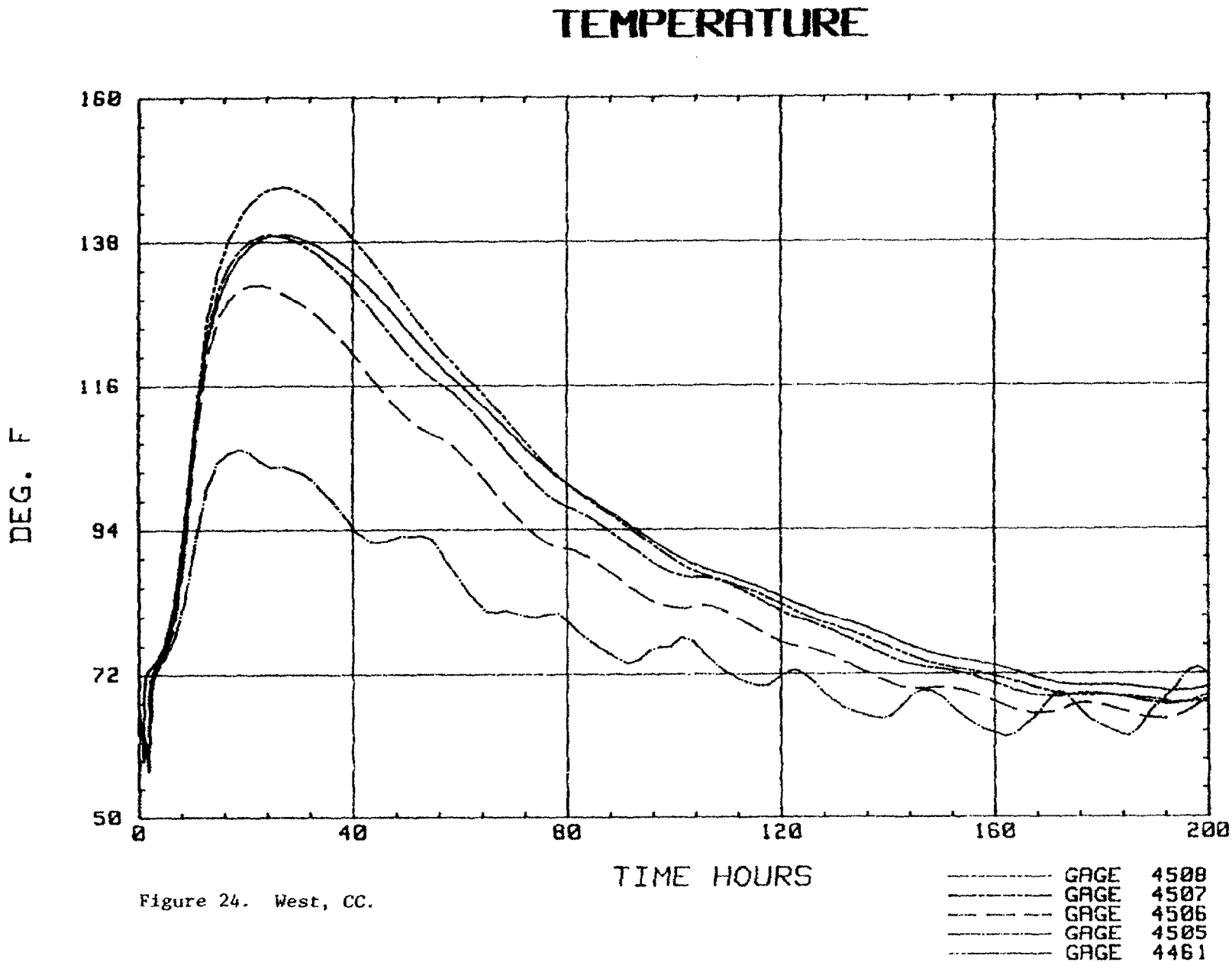


Figure 24. West, CC.

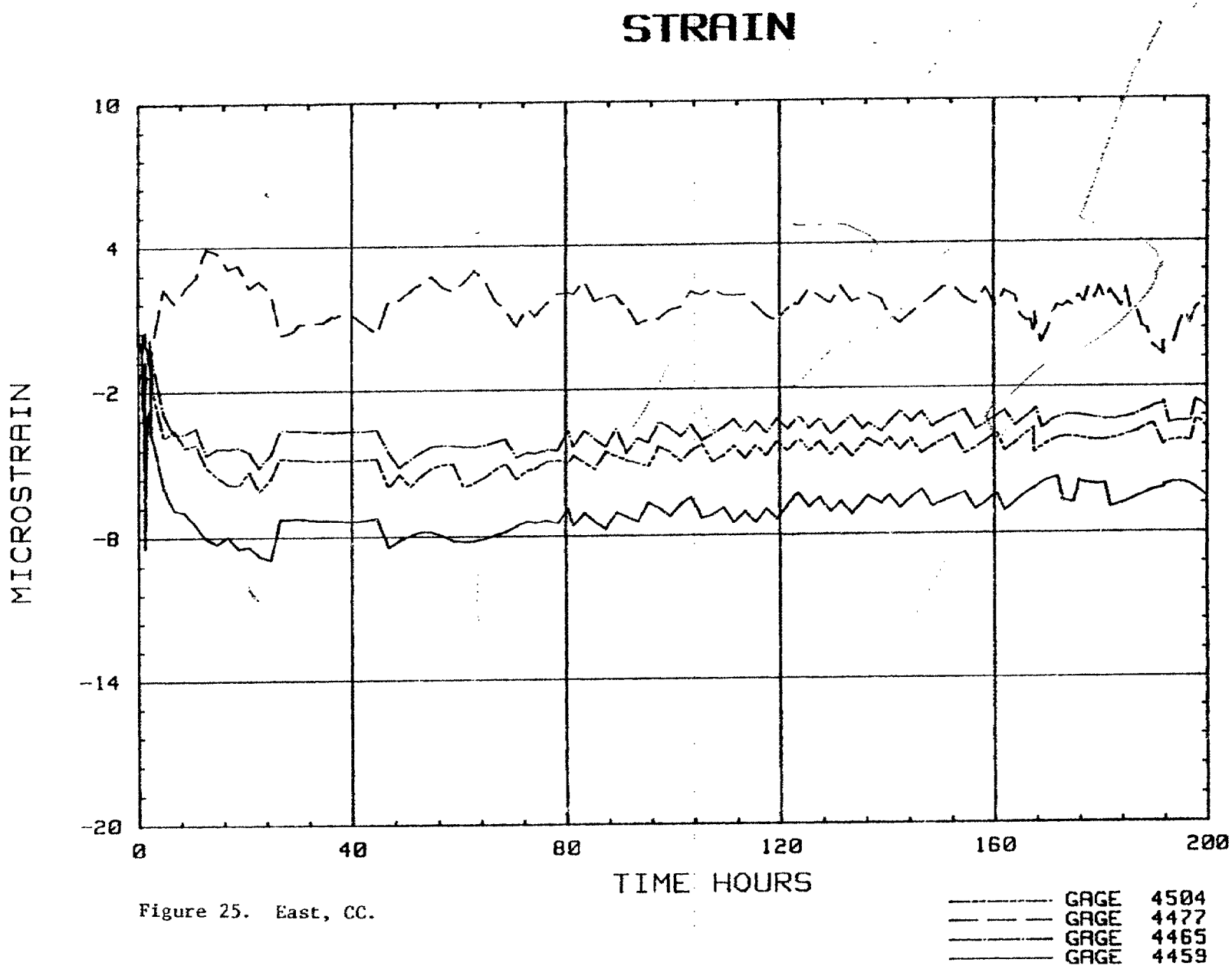


Figure 25. East, CC.

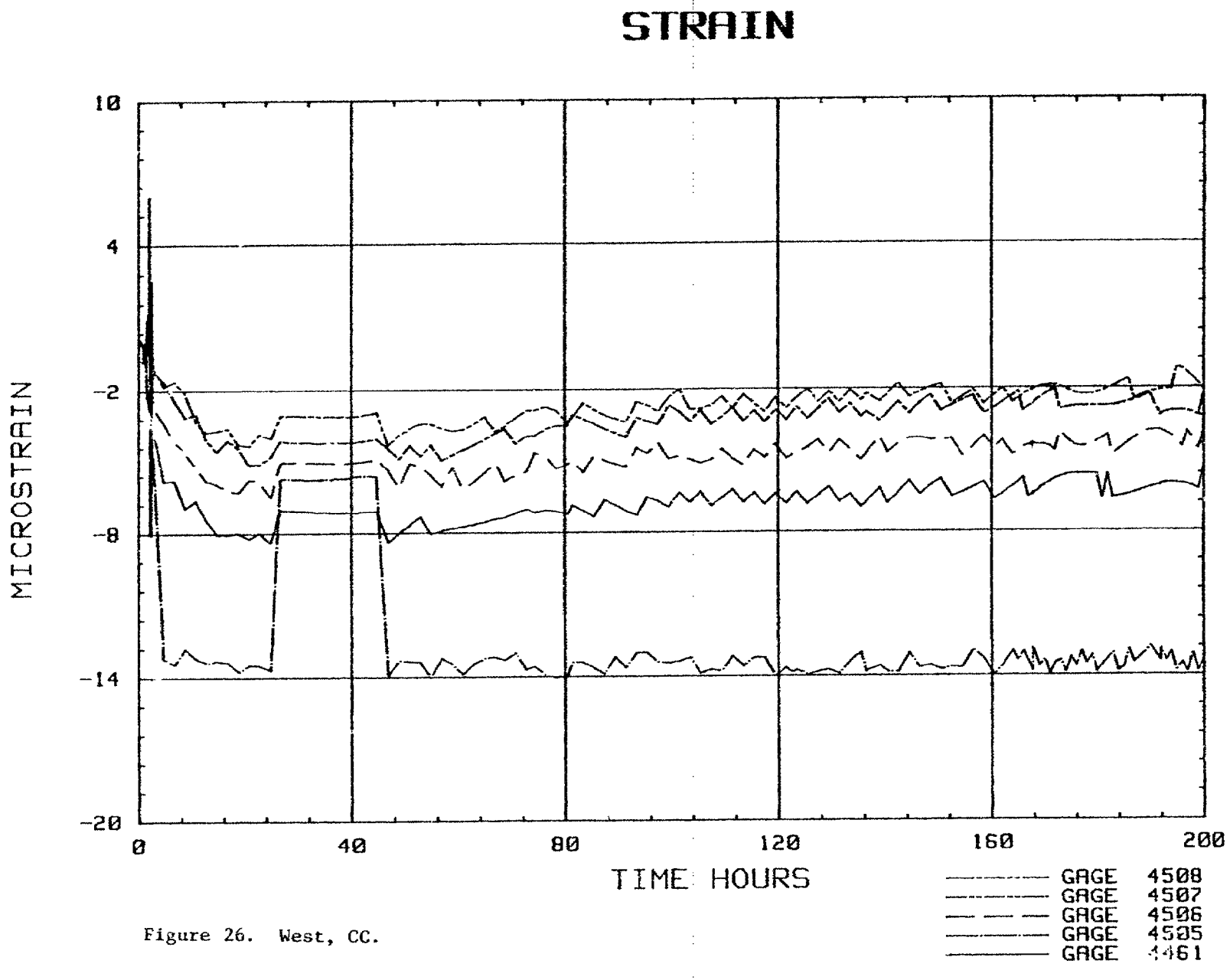


Figure 26. West, CC.

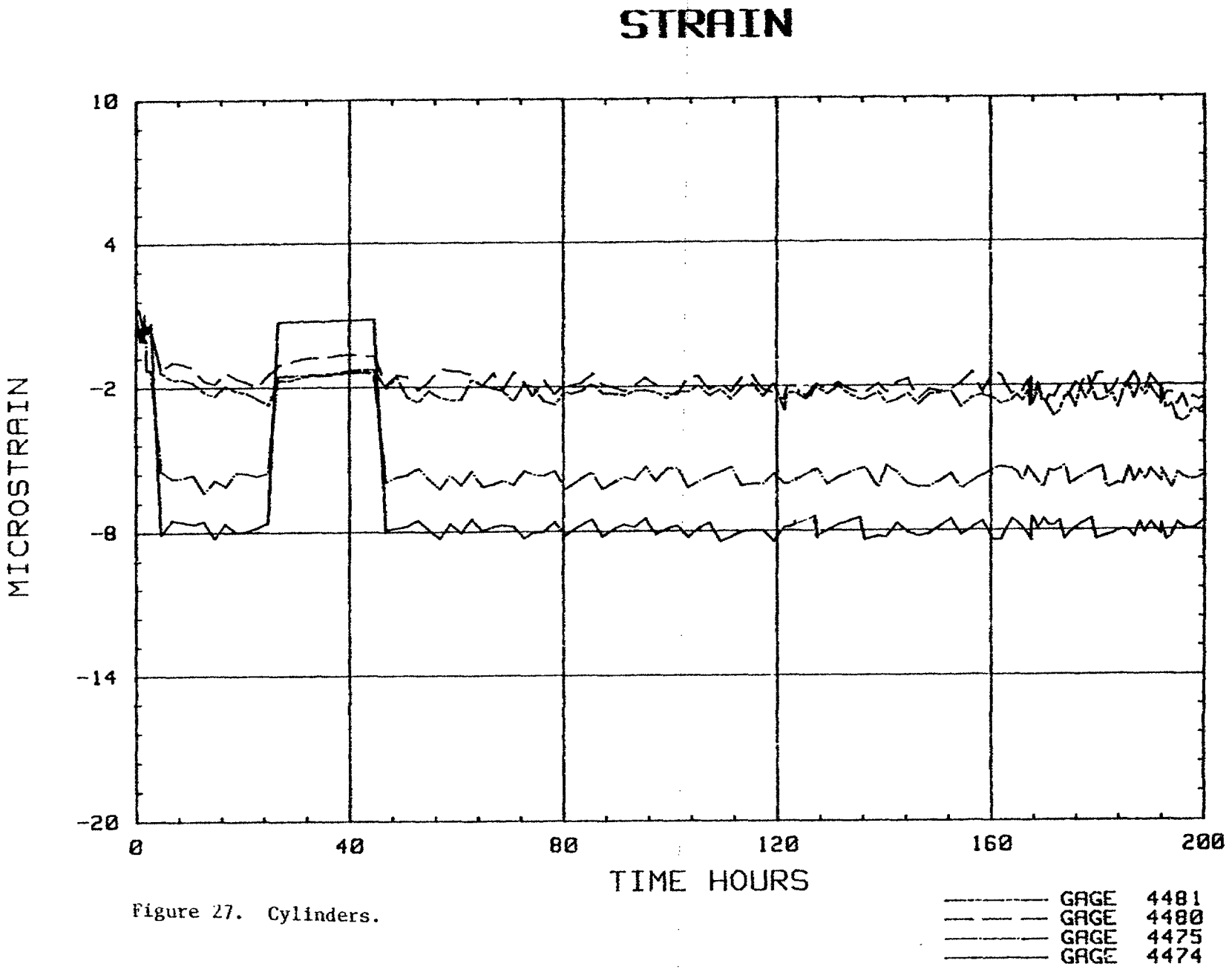


Figure 27. Cylinders.

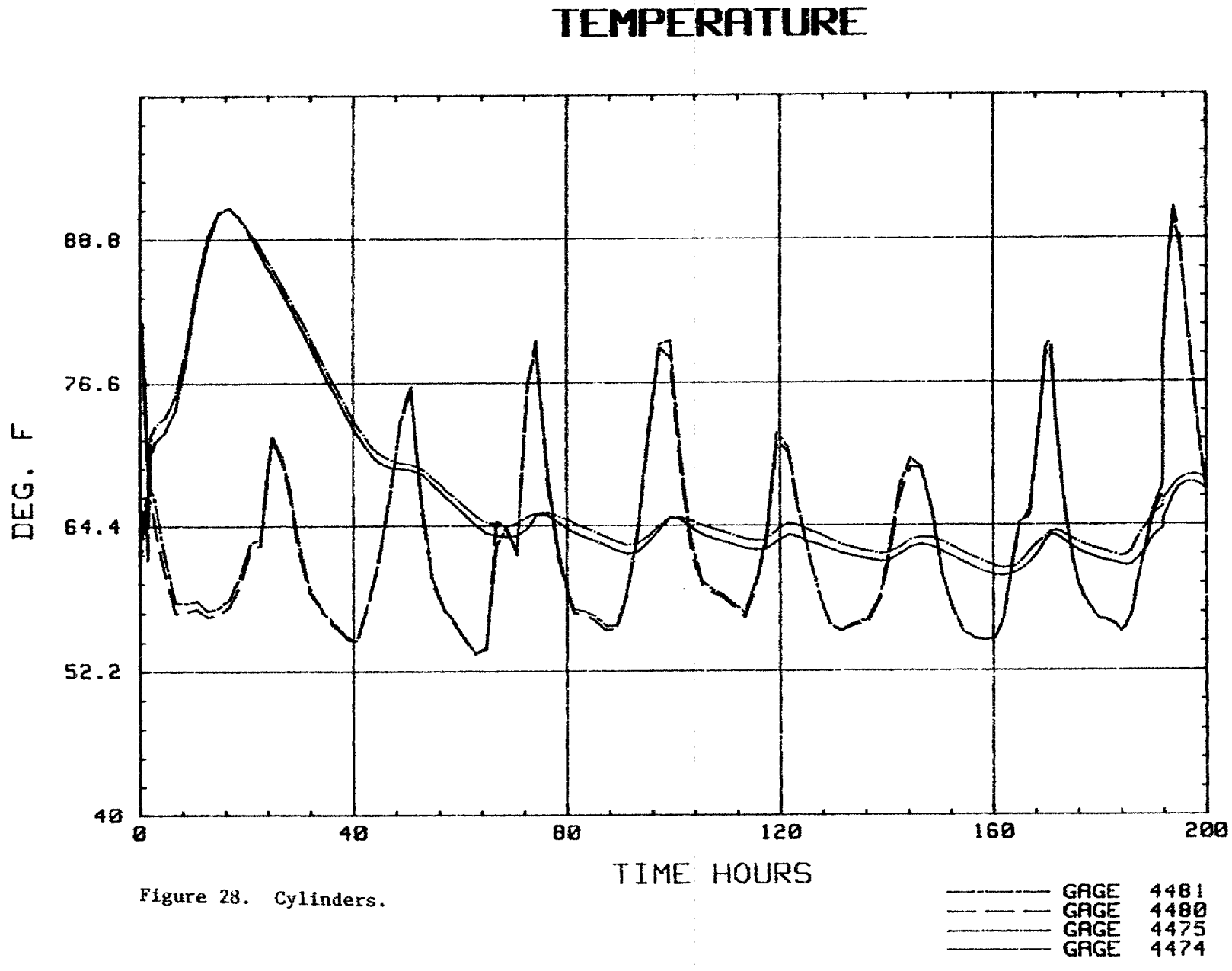


Figure 28. Cylinders.

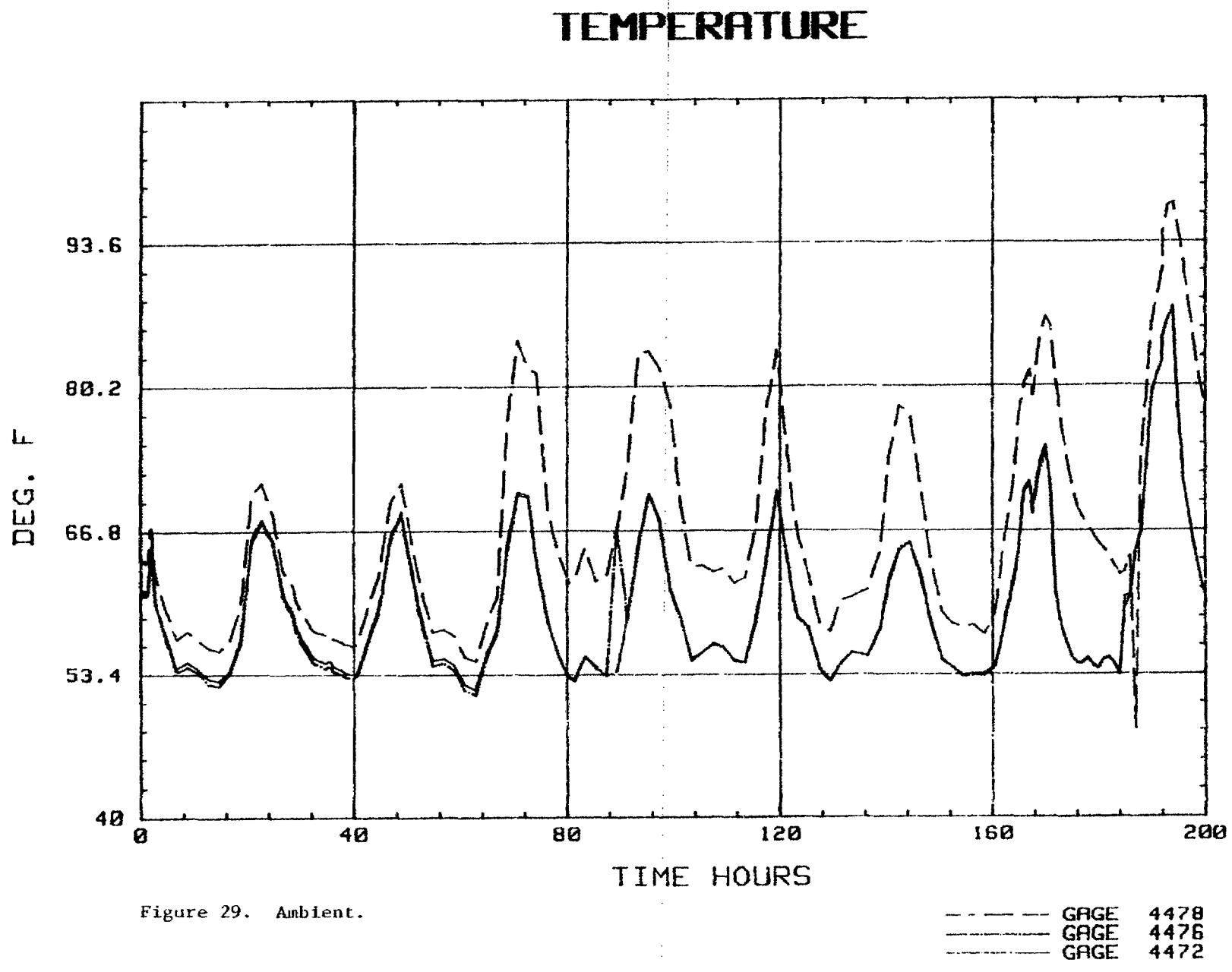


Figure 29. Ambient.

DISCUSSION OF "PROTOTYPE STUDY OF THERMAL STRAIN IN DOLOS"

MR. DEAN NORMAN

MR. MAGOON:

Could you comment on that large strain at about 35 hours that you had there? What was that?

MR. NORMAN:

That shift?

MR. MAGOON:

Was that forms removed or something?

MR. NORMAN:

No, that was just associated with some type of electrical current at the site. The strains were so small that any type of fluctuation like that in the local field messed up the signals. Mitch, is that correct?

MR. ALEXANDER:

We were operating almost at the noise level. Using the generators, we were subject to getting some ground shift. We never did know what happened at the time. The amount of data was so small that it wasn't obvious that we even had a ground shift until we took the thermal coefficient and took out the free expansion, which brought the strains down so small that they were insignificant.

MR. MAGOON:

Then how do you know which is signal and which is noise?

MR. ALEXANDER:

We noticed that all of the shifts were in the same direction and were able to note that it was a mechanical thing that took place out there. Obviously, if such a mechanical thing had taken place, we would have some tension, some compression. All of these are in the same direction.

MR. NORMAN:

It occured on the 40 gages, too, at the same time.

MR. MAGOON:

There was no shock, no small earthquake, nothing like that ever occured?

MR. NORMAN:

No. They were all constant in amplitude. That little shift was constant. If we were measuring a dynamic motion, it would have had some kind of variation in it. It was just a shift, flat, and heard on 40 gages.

MR. ALEXANDER:

Stayed there several hours.

MR. NORMAN:

Whenever I see that, I think it's not real strange.

MR. LILLEVANG:

Does that mean you don't have any data for those hours that the shift persisted?

MR. ALEXANDER:

I think the data were still right on top of the DC shift, and yes, I think we still would say we have them.

MR. LILLEVANG:

So you have to make a judgement as to what the shift is in order to find out what the strains are.

MR. ALEXANDER:

We should make the comment that even if we considered it was a real mechanical shift, it would still be insignificant. The strains were so small that we would have had to get up at the ultimate strain capacity of the material to produce any problems. We felt it's still insignificant strain, even if we consider it to be a real mechanical strain.

MR. NORMAN:

You're still not going to have more than 10 microinches per inch.

MR. LILLEVANG:

I have another question then. When you were describing the sensing of temperature as it grew and subsided, I didn't see whether or not you had data there with which you could take a temperature transept and show us what the variation of temperature was with depth from surface to intercenter of shape.

MR. NORMAN:

We have about a maximum of one cross section, I believe, of nine points.

MR. NORMAN:

In terms of plotting--we can plot a thermal contour plot on the cross section.

MR. LILLEVANG:

Have you done that?

MR. NORMAN:

No, sir, we have not yet.

MR. LILLEVANG:

What would you expect to see if you examined the data?

MR. NORMAN:

We expect to see a larger temperature near the center of the axis of the section and then, going off, the smaller temperatures near the outside. The internal restraints that that provides are apparently causing no thermal strains or related strains of any real consequence.

MR. ALEXANDER:

We were going to plot the gradients, but we didn't have enough strain to make it worthwhile plotting. We would have to have got some strains to be significant in order to link it to the gradients, and we didn't have any so we didn't plot the gradients.

MR. LILLEVANG:

Applying the best estimate you could make of what the effective strain is--effective on the whole length of the shank--and taking your unit values and extending them to the length of that shank, what do you end up with as the actual of changing length?

MR. NORMAN:

I don't know. Ten microinches per inch times--how tall is the--

UNIDENTIFIED SPEAKER:

One hundred eighty inches long.

MR. NORMAN:

Well, ten times ten to minus six times one hundred eighty. I don't have that number, but, again, I think that's an extremely small--

MR. ALEXANDER:

One mil. Approximately one mil.

MR. NORMAN:

One to two. We didn't see anything that looked suspicious or of concern in terms of magnitude of these strains and the temperature. They were as we expected also.

DR. LIGTERINGEN:

I would like to ask if you can provide some more details about the cement mix and ratio in that.

MR. NORMAN:

I don't have that. We can get a detailed mix design, I'm sure. Mitch, can you say any more about that right now?

MR. ALEXANDER:

You're asking for the mix design?

MR. NORMAN:

Any more detail concerning the mix design? It was a 0.42 water to cement ratio.

MR. ALEXANDER:

I don't know what it was.

MR. NORMAN:

I will get you some details of the mix design.

MR. MAGOON:

Could I add one point? It seems to me it would be important to find out what caused the ground shift at the experiment you did here and whether it takes some kind of isolation or whatever. It seems it would be a very important point not to have it in the prototype or the ones that you install later.

MR. NORMAN:

I think that is an obvious thing we're always looking for, and I think it's easier said than done. I'm not an experimentalist from that standpoint, but I know we've always had various problems with shifts in signals that are not real data.

MR. ALEXANDER:

It's a common problem in instrumentation. We have had to watch shields and watch connections, and a lot of times we have to spend a good bit of time trying to get our instrumentation correct before we get rid of these ground problems. It turned out that the numbers we were measuring were large enough so this shift didn't occur. What we were measuring was actually the thermal expansion. These numbers were large compared to this ground shift that occured, so we were printing out the data at the

compared to this ground shift that occured, so we were printing out the data at the same time, all the time. We were getting the data printed out on a paper tape. The thermal expansion is so much larger than this ground shift that we didn't even notice that anything unusual had occured. It wasn't until we came back to the lab, took out the thermal expansion and got these small strains that the ground shifts showed up as being significant data.

MR. MAGOON:

We found that with instrumentation we had in San Francisco Bay and also, I believe, at Crescent City, that the effect of radar in close ships seemed to affect our instrumentation. It would get what you're calling a ground shift. It just went up at some point and then produced some sort of voltage on the system that it saw. We could see when it was turned on. The radar seemed to have quite an effect.

MR. NORMAN:

We had a problem like that in a project in the Panama Canal. We were measuring some strains in the locks. Ships would have communications coming through the locks that would foul up our instrumentation.

ON THE RELIABILITY OF RUBBLE MOUND BREAKWATER DESIGN PARAMETERS

Hans F. Burcharth
Department of Civil Engineering
University of Aalborg, Denmark

1. INTRODUCTION

The recent failures of major rubble mound breakwaters has taken many professional engineers by surprise. Is it really possible that we do not have a reliable design method — after hundred years of breakwater design and construction and also intensive research for the last 20 years? The answer is yes. The state of the art and the design tools are not satisfactory compared to those available in other branches of civil engineering such as for example structural engineering.

I shall try to explain the difficulties we are facing in breakwater engineering, especially for rubble mound breakwaters, by summarizing some of the uncertainties we have to deal with in the design process. A good overview of the uncertainties and the related consequences is of paramount importance to the designer. Without such knowledge it is impossible to evaluate the safety of a structure — a situation that is unacceptable for a professional engineer.

It is important to point out that the damage to a breakwater never depends on one single parameter such as for example the wave height. Moreover, the time history (duration) of the impact is of importance. This means that a discussion of uncertainties in breakwater design really should be based on the joint probability density functions of the involved parameters supplied with statistical information on the related persistance.

The following presentation is not in accordance with this since each parameter is treated separately. This is done for the sake of simplicity and also because it will still serve the main object of the presentation.

2. BASIC NEEDS IN DESIGN

For most civil engineering structures (buildings, bridges etc.) it is possible to design and check the structural performance by means of theory. This is because many years of research and experience have established the prerequisites which are

- *Information on size of all major types of loads*, often stated in standards as characteristic maximum and minimum values, which again are based on information of the statistical properties such as mean, standard deviation and frequency distribution.
- *Information on the structural response to the loads*, implemented in formulae which are in most cases the outcome of theories based on basic physics, but are in some cases more or less empirical.

Both loads and the response to those loads are known quantitatively to such an extent that meaningful safety factors can be specified in the various standards.

Although this is well known to all professional civil engineers, it is deliberately mentioned here as a reference for the following discussion on rubble mound breakwaters, for which the situation is completely different.

3. ENVIRONMENTAL LOADS

3.1 Waves

The ideal situation, depicted in Figure 1, where both short term and long term wave statistics can be established from on-the-site records almost never exists.

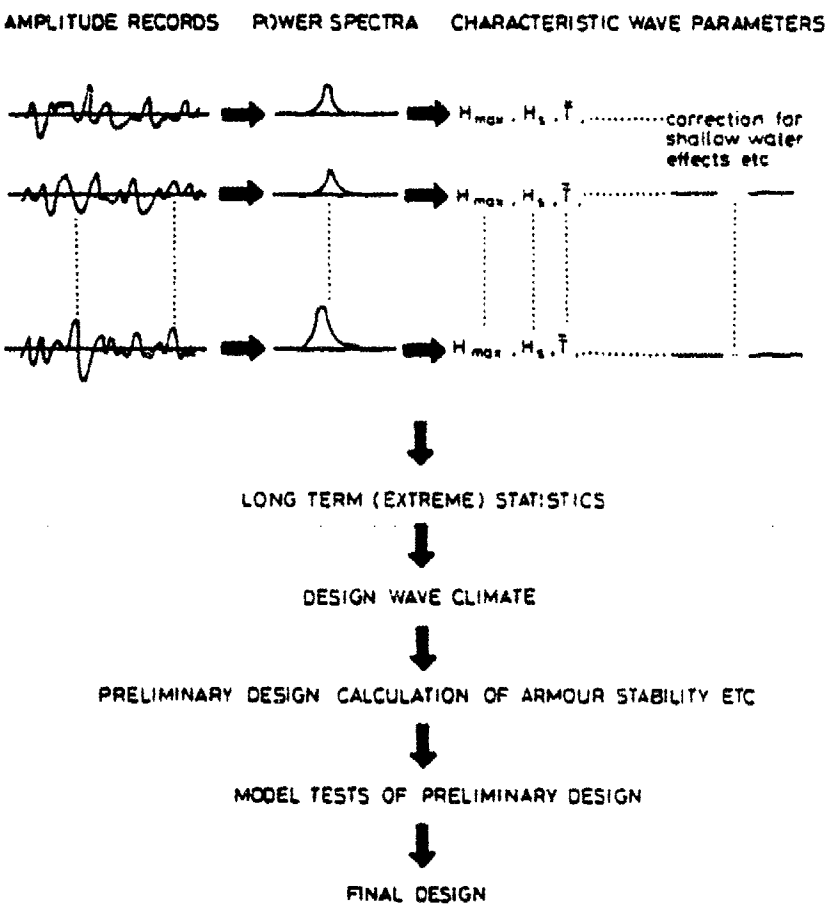


Figure 1. Ideal procedure for the establishment of design wave climate.

Usually one has to establish design wave conditions by hindcasting from meteorological observations and/or some wave records covering relatively short periods. In some areas visual wave observations from ships are available too.

It is clear from this that it is not possible to get reliable statistical values for all the wave parameters of importance. These are wave heights H, periods T, spectral shape, groupiness, direction of propagation and duration of storms.

Let us examine the wave height. This is generally the most important parameter since cover layer stability in terms of block weight is more or less proportional to the wave height

in the third power. The uncertainties in the determination of extreme wave heights may result from the following sources:

A. Errors in measurements, visual observations or hindcasting of the wave data on which the extreme statistics are based.

B. Errors related to extrapolation from short samples to events of high return periods, i.e. low probability of occurrence.
Errors due to the choise of exceedence level.
Errors due to the method of fitting data to a chosen distribution.

C. Lack of knowledge about the underlying distribution for the extreme events.

D. Errors due to plotting positions.

E. Climatological variations.

ad A. Errors in wave data

Le Mehaute et al., 1984 discussed the uncertainties and systematic errors or bias related to the wave data under the assumption of errors being normally distributed. They reported the following "typical" normalized standard deviation σ' defined as the absolute standard variation divided by the expectation ("mean") value of H_s:

Direct wave measurement	$\sigma'_M = 0.05$	bias 0.00
Visual observations from ships	$\sigma'_M = 0.20$	bias 0.05
Hindcast (excluding hurricanes and other tropical storms)	$\sigma'_M = 0.15$	bias 0.05

It should be noted that the two last set of figures are applicable only when the sample populations are ranked statistically. A direct comparison in the time domain, i.e. comparison of individual sea states, generally shows larger discrepancies. Moreover the figures are average figures. For instance it is believed that wave data based on to-day's most advanced hindcast models applied to relatively restricted areas, such as the North Sea, where high quality weather maps are available, will show a smaller uncertainty.

ad B. Errors due to short samples.

Estimates on events of low probabilities are often performed in the following two different ways:

1) The extrapolation of data from frequent measurements or observations. The data are often compiled at intervals $\Delta t = 3, 6$ or 24 hours, which gives a large sample, N events, even in the case of a short time of observation or record length Y in years. The order of magnitude of N is often 1000 - 10,000.

2) The extrapolation of relatively few data sets representing the max significant wave height H_s for a number of storms exceeding a certain level, H'_s. The data are often determined from hindcasts and the sample size N is typically within the range 10 - 30.

Wang et al., 1983, examined the uncertainties related to the first method. They considered the long term distribution of H_s to be of the exponential type which also includes the often used Weibull distribution,

$$P(H_s) \equiv P[H \leq H_s] = 1 - \exp\left(-\left(\frac{H_s - A}{B}\right)^{\gamma}\right) \qquad (1)$$

where A is signifying the background noise level or lower-bound, B is the scale parameter and γ is the shape parameter. All three characteristic variables are normally determined by best fitting to the observed data.

Assuming the data asymptotically normally distributed about the underlying probability distribution function, eq (1), the authors obtained for large N the normalized standard deviation,

$$\sigma_s' = \frac{1}{\gamma \ln(R\,\nu)} \left(\frac{R}{Y}\right)^{0.5} \tag{2}$$

where R is the return period in years, ν is the number of observations per year compiled at interval Δt and Y is the number of years of observations. Formula (2) is valid only for low probability levels and only for large samples $N = \nu Y$ of uncorrelated data. The latter implies that Δt should exceed approximately 24 hours, but because of little sensitivity on the confidence bands for H_s smaller values, as for example $\Delta t = 6$ hours, are often used.

Example.

Taking R = 50 years, Y = 5 years, ν = 365 observations per year and $\gamma = 1.2$
gives $\sigma_s' = 0.27$

Changing R and Y to 100 years and 3 years respectively
gives $\sigma_s' = 0.46$

The second method mentioned above is relevant to situations where data have to be obtained from hindcasting, which, due to the costs involved, restricts the number of data.

Rosbjerg, 1981, considered this case, where only maximum values η of H_s for independent storms exceeding a chosen level H_s' are taken into consideration, cf. figure 2.

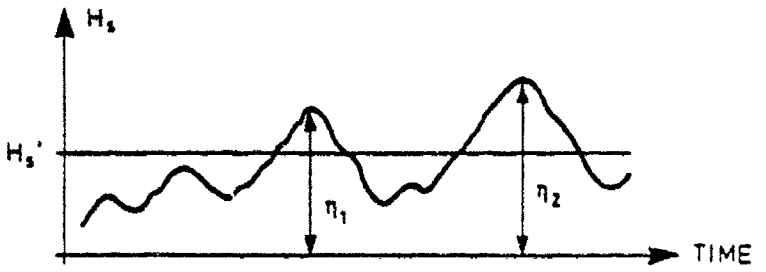


Figure 2. Data reduction by application of exceedence level, H_s'.

Rosbjerg assumed all the exceedences $\eta - H_s'$ to follow the exponential probability distribution,

$$P(H_s) \equiv P[\eta \leq H_s] = 1 - \exp\left(-\frac{H_s - H_s'}{\alpha}\right) \tag{3}$$

which is of the same type as the Weibull distribution, eq (1), with $\gamma = 1$.

The author also assumed the events η to occur at times corresponding to a Poisson-process with time dependent intensity. He arrived at the following expression for the R-year event defined as the value of η, which in average is exceeded once every R years,

$$H_s = H_s' + \alpha \ln \nu R \tag{4}$$

The corresponding absolute standard variation is

$$\sigma_s = \frac{\alpha}{(\nu Y)^{0.5}} (1 + (\ln \nu R)^2)^{0.5} \tag{5}$$

and the normalized standard deviation consequently

$$\sigma_s' = \frac{\sigma_s}{H_s} = \frac{\frac{\alpha}{(\nu Y)^{0.5}} (1 + (\ln \nu R)^2)^{0.5}}{H_s' + \alpha \ln \nu R} \tag{6}$$

The maximum likelihood estimate for α is

$$\hat{\alpha} = \bar{\eta} - H_s' \tag{7}$$

where $\bar{\eta}$ means average of η.

Nielsen et al., 1985, extended the analyses to include the Weibull distribution

$$P(H_s) = P[\eta \leq H_s] = 1 - \exp\left(-\left(\frac{H_s - H_s'}{\alpha}\right)^{\gamma}\right) \tag{8}$$

and found the following

$$H_s = H_s' + \alpha (\ln \nu R)^{1/\gamma} \tag{9}$$

$$\sigma_s = (\ln \nu R)^{1/\gamma - 1} \left[\frac{\alpha^2}{\gamma^2 \nu Y} + (\ln \nu R)^2 \frac{\alpha^2}{\nu Y} \left(\frac{\Gamma(1 + \frac{2}{\gamma})}{\Gamma^2(1 + \frac{1}{\gamma})} - 1 \right) + \frac{\alpha^2}{\gamma^4} (\ln \nu R) \cdot \ln(\ln \nu R))^2 \operatorname{Var}[\hat{\gamma}] \right]^{0.5} \tag{10}$$

ν is the average number of data per year and Γ the Gamma function.

The variance of $\hat{\gamma}$, $\operatorname{Var}[\hat{\gamma}]$, cannot easily be estimated, but by means of numerical simulation it is found that the term in (10) containing this quantity is highly dependent on the method for estimating the parameters in the Weibull distribution.

Petrauskas and Aagaard, 1971, found, by using a least square method, that the last term in (10) is insignificant. In this case the normalized standard deviation is

$$\sigma_s' = \frac{\sigma_s}{H_s} \cong \frac{(\ln \nu R)^{\frac{1}{\gamma} - 1} \left[\frac{\alpha^2}{\gamma^2 \nu Y} + (\ln \nu R)^2 \frac{\alpha^2}{\nu Y} \left(\frac{\Gamma(1 + \frac{2}{\gamma})}{\Gamma^2(1 + \frac{1}{\gamma})} - 1 \right) \right]}{H_s' + \alpha (\ln \nu R)^{1/\gamma}} \tag{11}$$

Nielsen et al., 1985, fitted the Weibull parameters by the method of moments, i.e. equating the first three moments of the distribution to those of the data, and found that the last term in (10) was of significance, namely in the order of 1/3 of the total standard deviation. The estimates on the parameter by the applied method of moments are given by

$$\frac{\Gamma(1 + \frac{3}{\hat{\gamma}}) - 3\Gamma(1 + \frac{2}{\hat{\gamma}}) \Gamma(1 + \frac{1}{\hat{\gamma}}) + 2\Gamma^3(1 + \frac{1}{\hat{\gamma}})}{(\Gamma(1 + \frac{2}{\hat{\gamma}}) - \Gamma^2(1 + \frac{1}{\hat{\gamma}}))^{3/2}} = \frac{\overline{\eta^3} - 3\overline{\eta^2}\bar{\eta} + 2(\bar{\eta})^3}{(\overline{\eta^2} - (\bar{\eta})^2)^{3/2}} \tag{12}$$

$$\hat{\alpha}^2 = \frac{\overline{\eta^2} - (\bar{\eta})^2}{\Gamma(1 + \frac{2}{\hat{\gamma}}) - \Gamma^2(1 + \frac{1}{\hat{\gamma}})} \tag{13}$$

$$\hat{H}_s' = \bar{\eta} - \hat{\alpha}\,\Gamma(1 - \frac{1}{\hat{\gamma}}) \tag{14}$$

$\overline{\eta^2}$ and $\overline{\eta^3}$ mean the average of sample values of η^2 and η^3, respectively, which are unbiased estimates of $E[\eta^2]$ and $E[\eta^3]$.

It should be noticed that the R-year event given by eqs (4) and (9) has a probability E of being equalled or exceeded in the specific lifetime L of the structure. For instance, if L is set equal to the return period R, this "encounter probability" E is as large as 63%. The relationship between R, L and E is given by

$$E = 1 - (1 - \frac{1}{R})^L \quad \text{or in case of R large} \quad R = -\frac{L}{\ln(1 - E)} \tag{15}$$

For design purpose R in eqs (4) and (9) should be evaluated with respect to E and L through eq (15). For example in a 50 years lifetime there is a 10% probability that the structure is hit by the 500 years' return period storm.

Eqs (6) and (11) make it possible to determine the necessary sample length when a prediction for a given return period with a prescribed accuracy and confidence is required. Following the normal distribution the products of σ_s' with 0.84, 1.28, 1.65 and 2.32 define the upper bound of spread corresponding to a confidence level of 80%, 90%, 95%, and 99%, respectively. For instance, the prediction of an event with 90% confidence and an uncertainty of no more than 0.20 imply that $1.28\ \sigma_s' \leq 0.20$. Inserting this in eqs (6) or (11) gives the corresponding number of years of observation Y for given ν and R.

Example.

The accuracy of estimates based on a restricted number of hindcasted data sets might be illustrated by the following example. The Delft Hydraulics Laboratory did a hindcast study for a specific deep water location in the Mediterranean Sea and found for a 20 years period the following 17 most severe storms, Table 1:

Table 1. Example of hindcasted storm wave data for a 20 years' period.

Rank i	Max H_s ($= \eta$) metres	Peak period T_p seconds	Average wave direction degrees
1	9.32	14.0	143
2	8.11	14.1	139
3	7.19	13.4	123
4	7.06	10.8	123
5	6.37	11.9	143
6	6.15	11.1	185
7	6.03	12.3	135
8	5.72	10.5	176
9	4.92	10.7	150
10	4.90	10.6	129
11	4.78	11.8	161
12	4.67	9.9	120
13	4.64	9.2	122
14	4.19	10.5	137
15	3.06	11.1	154
16	2.73	8.2	153
17	2.33	8.3	126

If we choose $H_s' = 4.0$ m we find N = 14 storms exceeding this level over a period Y = 20 years, which gives $\nu = 14/20$. According to eq (7) α can be estimated to $\hat{\alpha} = 2.00$ m. It can now be tested if the data follow the assessed distribution, for example the exponential type given by eq (3). In this case a straight line with slope 1:1 should be obtained by plotting $\eta_i - H_s'$ against $-\hat{\alpha}\ \ln(1 - P(\hat{\eta}_i))$, where $P(\hat{\eta}_i) = 1 - \frac{i}{N+1}$. (Gumbel plotting positions). Figure 3 shows that the fit is reasonable.

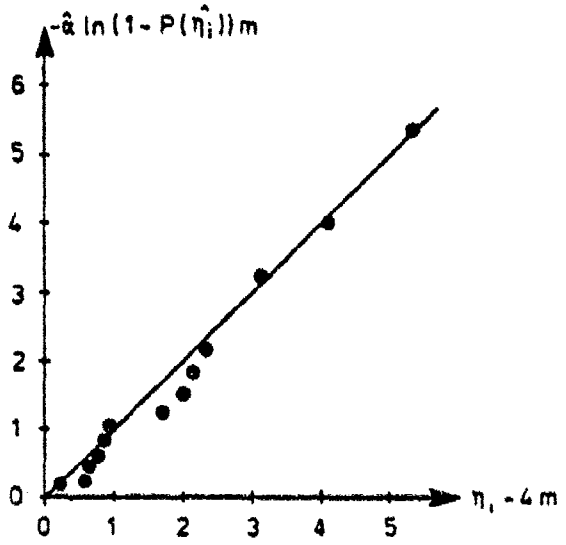


Figure 3. Test on exponential distribution of wave height exceedences.

Formulae (4) - (6) are then valid and the expectation values and the standard deviations can be calculated for various return periods, for instance

Return period R years	H_s metres	σ_s metres	σ_s'
50	11.11	1.97	0.18
100	12.50	2.33	0.19

Note that a change in the exceedence level H_s' for example to 3.50 m, which still gives N = 14, will change H_s and σ_s significantly since for R = 50 years $H_s = 12.39$ m, $\sigma_s = 2.47$ m, $\sigma' = 0.20$ and for R = 100 years $H_s = 14.12$ m, $\sigma_s = 2.92$ m, $\sigma' = 0.21$ m. This important problem is not discussed further here.

It is obvious that the 14 data points also fit a Weibull distribution.

If all the 17 data points given in Table 1 are considered, it corresponds to a exceedence level of $H_s' \cong 2.25$ m because the lowest value in the data set is $H_s = 2.33$ m. It turns out that in this case the data do not fit neither the exponential distribution, eq (13), nor the Weibull distribution, eq (8). However, if the exceedence level is not interpreted as the physically true cut-off level, but is

regarded a fitting coefficient only, like α and γ, then the 17 data points follow the Weibull distribution very closely, as demonstrated in Figure 4. The coefficients are in this case $H_s' = 0.73$ m, $\alpha = 5.27$ m and $\gamma = 2.80$, all estimated by the method of moments.

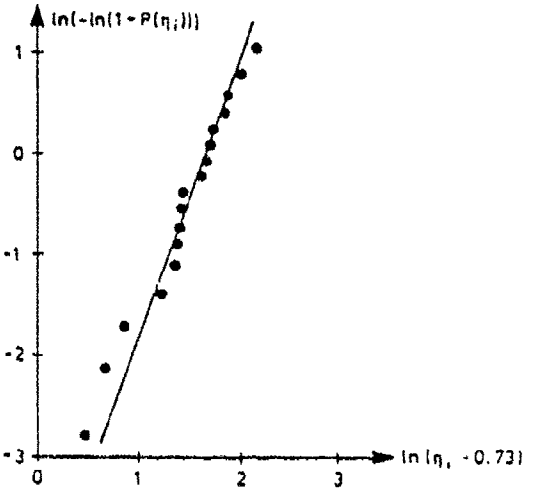


Figure 4. Data fit to the Weibull distribution. Gumbel plotting positions.

From eqs (9) - (11) we obtain the following corresponding values

Return period R years	H_s metres	σ_s metres	σ_s'
50	9.19	0.88	0.10
100	9.71	0.97	0.10

The Weibull distribution shown by the straight line in Figure 4 is a result of the chosen method of fitting. A least square fit or a visual fit will produce different lines and different estimates on the extreme events.

Thus it is concluded that also the choise of exceedence level and the method of fitting the data to a chosen distribution introduce uncertainty on the estimates of extremes.

ad C. and D. Errors due to the lack of knowledge on the true long term distribution and due to plotting positions.

Several probability distributions are used to describe extreme wave height statistics. These include for example the log-normal distribution, the extremal type I or Gumbel or Fisher-Tippett I distribution, the extremal type II or Fretchet or Fisher-Tippett II distribution, the Ward-Borgman distribution and the extremal type III or Weibull distribution. Although each of these distributions has a theoretical base, they cannot be evaluated and related to the extreme waves on a physical base. As a consequence they are only fit to the available data. Most often the scales used for the plotting are such that the chosen distribution lies on a straight line, simply because of the

more convenient visualization of the extrapolation. However, when extrapolating, one should always be aware of possible physical processes, such as for example wave breaking, which might interrupt the probability distribution at some probability level.

It follows from these comments that due to unknown extreme distribution errors can only be estimated by a sensitivity analysis in which various distributions are fitted. Table 2 shows such an analysis by the Delft Hydraulics Laboratory performed on the wave data given in Table 1.

Table 2. Example of influence of choise of extremal distribution and plotting position on low · probability wave heights. Data by Delft Hydraulics Laboratory.

Extremal distribution	Plotting position	Correlation coefficient	Return period H_s 50 year	100 year
Type I Gumbel	Gumbel	0.9875	11.0 m	12.2 m
	Gringorten	0.9852	10.3 m	11.3 m
Ward/Borgman	Gumbel	0.9872	9.8 m	10.5 m
	Gringorten	0.9920	9.4 m	10.1 m
Type III Weibull	Gumbel	0.9877	9.6 m	10.2 m
	Gringorten	0.9877	9.3 m	9.9 m

Although no accurate figures can be given it seems reasonable from this table and the above given example based on the distribution, eq (3), that due to unknown extreme distribution a normalized standard deviation σ'_D might be in the order of

$$\sigma'_D \cong 0.05 - 0.10.$$

In order to plot the data a position formula must be adopted. Many different plotting positions, all based on some statistical considerations, exist, but it is not easy or possible to select a specific one as the most correct. For this reason it is reasonable to estimate the error due to plotting positions by sensitivity analyses involving a number of reasonable plotting rules.

Table 2 gives an example where only two plotting rules are used, namely

Gumbel/Weibull
$$P(\eta_i) = 1 - \frac{i}{N+1} \tag{16}$$

and

Gringorten
$$P(\eta_i) = 1 - \frac{i - 0.44}{N + 0.12} \tag{17}$$

It is seen that significant deviations in the estimated extreme wave height occur due to the plotting rules. It is believed that a realistic normalized standard deviation σ'_p on extreme events will be in the order of

$$\sigma'_p \cong 0.05$$

ad E. Errors due to climatological variations.

An additional source of uncertainty is the natural variation of the wave climate. Le Mehaute et al., 1984, considered this difficult problem under the assumption of the natural climatology being ergotic and stationary and governed by the statistical law of Weibull distribution. By setting Y = R in eq (2) they found that the normalized standard deviation of climatological variations in R years at a particular location is given by

$$\sigma'_C = \frac{1}{\gamma \ln(R\nu)} \tag{18}$$

If we for instance estimate $\gamma \cong 1.2$ as proposed by the authors we find for $\nu = 365$ and R = 50 or 100 years $\sigma'_C \cong 0.08$.

Combined errors.

The above mentioned sources of uncertainty can be assumed mutual independent except for an unknown but probably weak correlation between the climatological variation and the data samples.

The total normalized standard deviation might then be estimated by

$$\sigma' \cong (\sigma'^2_M + \sigma'^2_s + \sigma'^2_D + \sigma'^2_p + \sigma'^2_C)^{0.5} \tag{19}$$

With reference to the foregoing discussion one can establish the following two examples:

Examples.

Direct wave height measurement. $\nu = 365$ observations per year. Y = 5 years. R = 50 years.

$$\sigma' \cong (0.05^2 + 0.27^2 + 0.07^2 + 0.05^2 + 0.08^2)^{0.5} = 0.30$$

Hindcasted wave heights. 14 data sets over Y = 20 years. R = 50 years.

$$\sigma' \cong (0.15^2 + 0.18^2 + 0.07^2 + 0.05^2 + 0.08^2)^{0.5} = 0.26$$

From this it is seen that, even with what is generally regarded reasonable lengths of data sample and observation period, the uncertainty related to the 50 year event is significant and in the order of $\sigma' \cong 0.25 - 0.30$. If we assume normally distributed random variables it means a 16% probability of the wave height being bigger than 1.25 - 1.30 times the estimated height.

The uncertainty increases significantly when the lengths of data sample and the period of observation are reduced to figures below those given above.

The difficulties in obtaining reliable estimates on design wave heights might also be illustrated by the following example from the Norwegian Ekofisk North Sea offshore field given by Professor Tørum of Norway.

In 1970 the 100 year design wave height was estimated to be 19.6 m. In 1972 it was 23.6 m; in 1977 28.0 m; in 1981 up to 34 m. And finally in 1984 the estimate was 28 m with an uncertainty of approximately ± 15%! This big uncertainty exists despite the large resources spent on wave recordings, wave statistics etc. in this prospective offshore area. These resources are much larger than those available for the design of breakwaters.

It is not only the wave height that is of importance but also

- the wave period
- the spectral shape
- the horizontal, directional spread of the wave energy / short crestedness of the waves
- the groupiness
- the direction of the propagation
- the duration / time history of the storms

Therefore the uncertainty related to the estimation of these parameters should also be evaluated. It takes a lot of work and research to perform such an analysis, also because generally it is the reliability of the "joint parameters" which are of interest. This problem is not discussed further here. However, it is obvious that it all adds to the uncertainty on design wave climate estimations.

If the breakwater is in "shallow-water" and the wave data are from an offshore location then we have to include the uncertainty related to shallow water effects such as:

- Refraction, i.e. change of wave direction and wave height due to oblique wave approach.
- Shoaling, i.e. change of wave height and wave length due to water depth variations perpendicular to the coast.
- Wave breaking, due to instability by decreasing water depth.
- Wave set-up, i.e. change of the mean water level due to changes of the wave radiation stress.

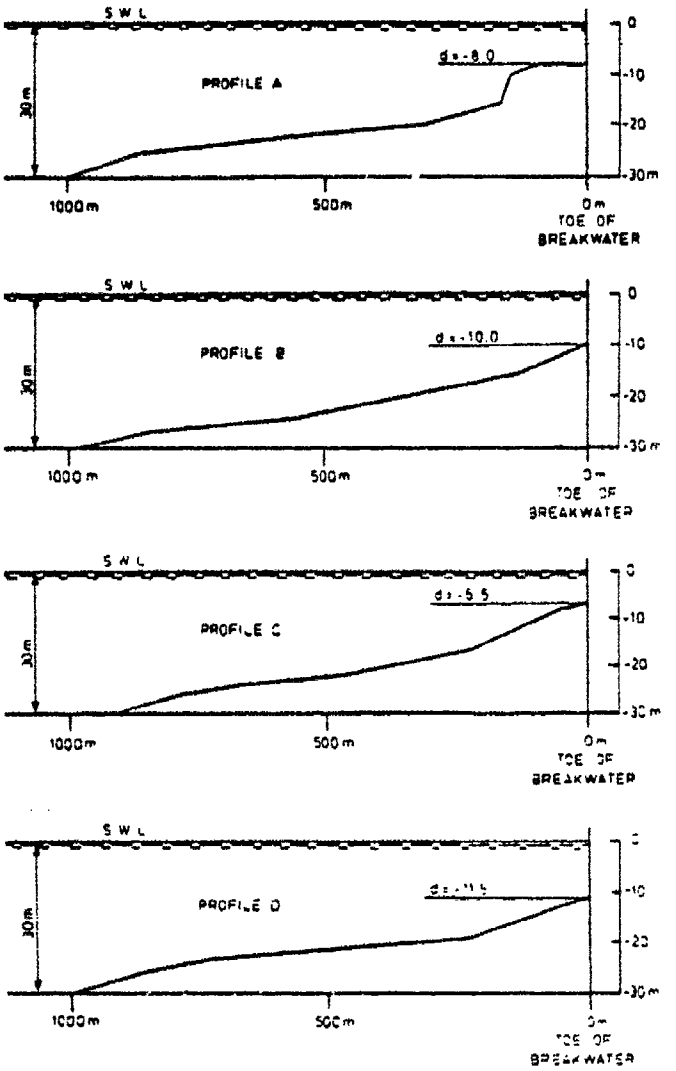
Besides these effects we also have:

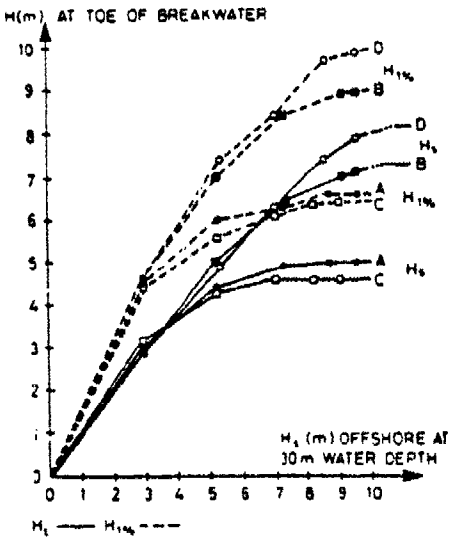
- Tidal water level variations.
- Barometric pressure variations.
- Wind set-up, i.e. wind induced change of the mean water level.
- Seiches.
- Currents.

The uncertainties related to all these parameters or phenomena are in general not well established except for tidal water level variations. Consequently a quantitive discussion on uncertainties is not possible. However, in the next paragraph we shall evaluate the importance of reliable data by a sensitivity analysis of the structural response to some of the parameters.

It has often been pointed out that estimates on design waves are much more reliable in shallow water than in deep water due to the depth limited wave heights. This is true but it should be mentioned that no wave theory exists which can predict with good accuracy the absolute wave height distribution and maximum wave heights in shallow water with breaking waves. Moreover, it should not be overlooked that the water level is also a very important parameter when breakwaters are designed for a certain amount of overtopping.

Another point which should be stressed is the sensitivity of shoaling/ wave breaking to variations in the sea bed profile. This is illustrated in Figure 5, where the wave heights of the incoming waves at the toe of a breakwater are determined for four different foreshore bottom profiles. The breaker index $\gamma_{H_s^{max}}$, defined as the ratio of the max significant wave height, H_s^{max} and the water depth, d at the toe, is also given in the figure together with the breaker index $\gamma_{H_{1\%}^{max}}$ related to the maximum value of wave heights, exceeded by 1% of the waves.





PROFILE	BREAKER INDEX $\gamma = \frac{H^{max}}{d}$ AT TOE OF BREAKWATER	
	$\gamma_{H^{max}}$	$\gamma_{H_{1/3}}$
A	0.63	0.83
B	0.73	0.90
C	0.71	0.98
D	0.70	0.87

Figure 5. Example of sensitivity of depth limited wave heights to differences in foreshore bottom profiles. Delft Hydraulics Laboratory.

The wave heights was determined by DHL in wind-wave flume model tests without the breakwater.

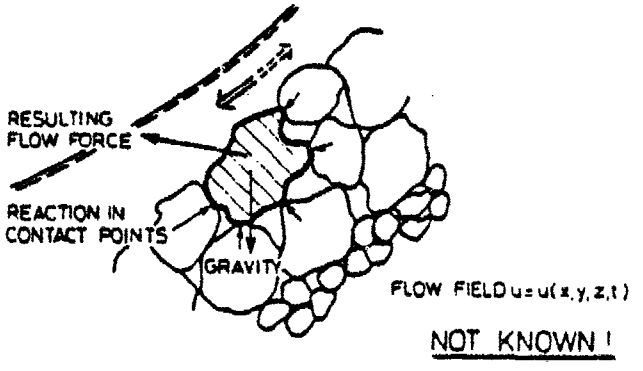
It is seen that a good estimate on the wave height in front of a breakwater in shallow water must be based on model tests with a correct reproduction of the foreshore topography. This means that in case of significantly varying bottom topography along the breakwater it is necessary either to test many sections or preferably to test the hole structure in a three-dimensional model.

4. SENSITIVITY IN STRUCTURAL RESPONSE TO THE ENVIRONMENTAL LOADS

The following is not intended to be a complete discussion as only a few, but important, problems will be discussed.

4.1 Hydraulic stability of the armour layer

The difficulties related to a purely theoretical stability analysis might be illustrated by considering the forces on an armour unit, see Figure 6.



GRAVITY $F_G = g\rho_w\left(\frac{\rho_s}{\rho_w} - 1\right) d^3$

FORM DRAG. $F_{DF} = C_F\, \rho_w\, d^2 |u| u$

SURFACE DRAG. $F_{DS} = C_S\, \rho_w\, d^2 |u| u$

LIFT $F_L = C_L\, \rho_w\, d^2 u^2$

INERTIA, FROUDE-KRYLOV. $F_I = C_I\, \rho_w\, d^3 u'$ (pressure grad. undisturb. flow)

INERTIA, ADD. HYDRODYN. MASS $F_M = C_M\, \rho_w\, d^3\, u'$ (change of flow field by the body)

COEFFICIENTS C are functions of Keulegan-Carpenter No. and Re No. and will vary considerably in time.

Figure 6. Forces on armour unit.

As a consequence stability formulae are semiempirical and formulated as an equality between a characteristic drag flow force and the stabilizing gravity force multiplied by unknown functions to take care of slope angle, friction, interlocking, wave period etc.

The various formulae show that the stability in terms of required mass, M of the armour unit is more or less proportional to the wave height in the third power. This very strong dependence put emphasis on the need for precise estimates on wave heights. This is depicted in Figure 7, where the relative variation of M in the range $H \pm \sigma(H)$ is shown. $\sigma(H)$ is taken as 0.3 $\hat{H}$, cf. paragraph 3.

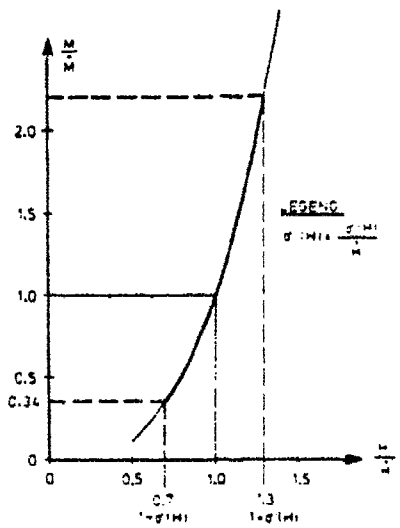


Figure 7.
Influence of wave height, H on required mass, M of armour unit.

The armour layer stability is also affected by the wave period T, but the variation with T is generally found to be much weaker than the variation with H. However, Gravesen et al. 1979 found a strong influence and proposed that the wave period is taken into consideration by using $H_s^2 \, L_p$ in the stability formulae instead of H_s^3. L_p is here the wave length corresponding to the spectral peak period T_p. As L_p is more or less proportional to T_p^2 this implies a dependence of M on T_p as schematized in Figure 8. As a characteristic standard deviation is chosen $\sigma(T_p) = 0.15 \, T_p$.

Gravesen et al.'s findings are related to an armour layer of cubes with slope 1:2 but surmounted by a vertical wave wall, which affects the stability in the case of larger wave.

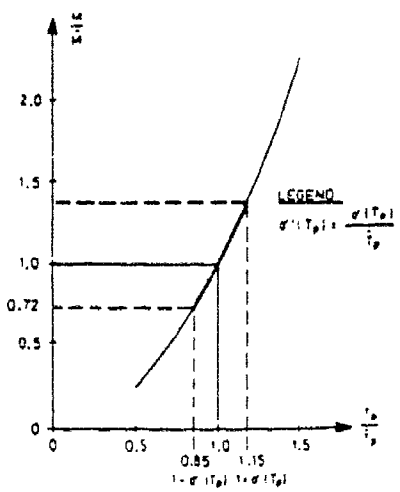


Figure 8.
Influence of peak period on required mass, M of armour unit as proposed by Gravesen et al., 1979.

A somewhat weaker but still significant dependence was found by Burcharth, 1979 in stability tests with Dolosse armour exposed to regular waves, Figure 9. The same reference also shows that run-up increases significantly with the wave period.

Figure 10 shows a replot of stability tests in regular waves with uniform stones, Dai & Kamel, 1969 and rip-rap, Ahrens, 1975. It is seen that the stability sensitivity to wave period is small in the case of uniform stones and large, but with opposite trends, for rip-rap.

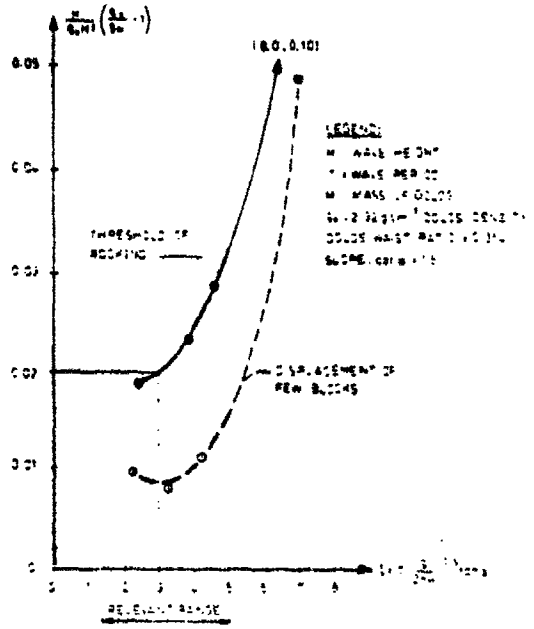


Figure 9.
Example of influence of wave periode on required mass of Dolosse armour units. Tests in regular waves. Burcharth, 1979.

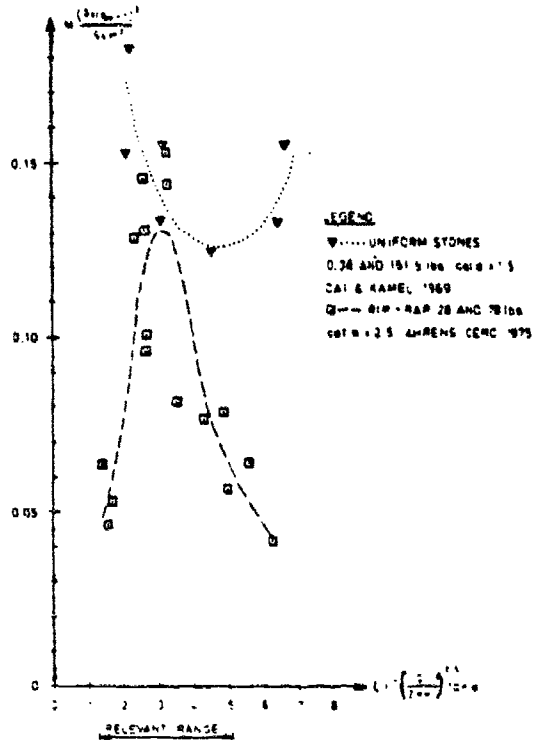


Figure 10.
Examples of influence of wave period on required mass of uniform armour stones and rip-rap. Replot of data from Dai & Kamel, 1969, and Ahrens, 1975.

In Figure 11 the data are normalized with respect to $\zeta = 3$ for easy mutual comparison of the wave period sensitivity. $\zeta = 3$ is a characteristic average value for rubble mound breakwater design wave situations. It is seen that an uncertainty on T (for example $\sigma'(T) = 0.15$) around the value $T_{\zeta=3}$ only gives relatively small variations on the required mass M.

This is somewhat contradictory to Figure 8 but might be explained by the influence of the wave wall as explained above.

Figure 11 also shows that the larger the porosity of the armour layer the more vulnerable the armour is to large wave periods (Dolos armour has the largest porosity and rip-rap the smallest). This is due to the "reservoir effect" of the pores as explained in Burcharth et al., 1983. A stability minimum seems only present for the relative impermeable rip-rap.

Note that the data in the Figures 9, 10 and 11 are from tests with regular waves.

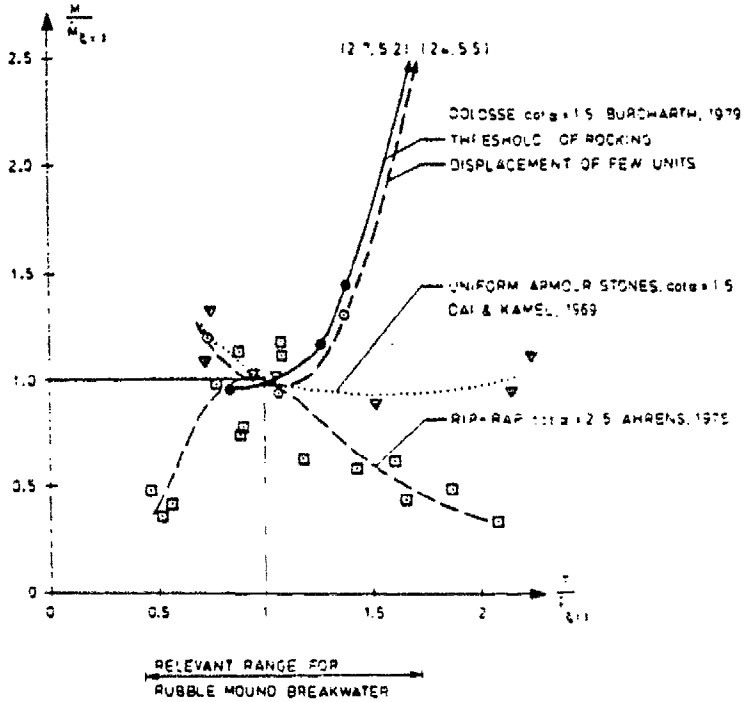


Figure 11. Example of influence of wave period on required mass of armour units and rip-rap. Regular waves. Data normalized with respect to the estimated values $\hat{T}_{\zeta=3}$ *and* $\hat{M}_{\zeta=3}$ *corresponding to* $\zeta = T\left(\frac{g}{2\pi H}\right)^{0.5} \tan\alpha = 3$.

The examples show that the effect of the wave period on armour stability is not clarified in general.

4.2 OVERTOPPING

The design conditions are often related to overtopping of the breakwater. This is the case where roads, reclaimed areas, berths, installations etc. are located behind and close to the breakwater.

Overtopping is very sensitive to variations in wave height and mean water level. Besides this also variations in wave period, wave direction and wave shortcrestedness affect the overtopping.

The sensitivity to the wave height can be illustrated by the example given in Figure 12, which shows some scale model test results from a rubble mound breakwater with a wave wall.

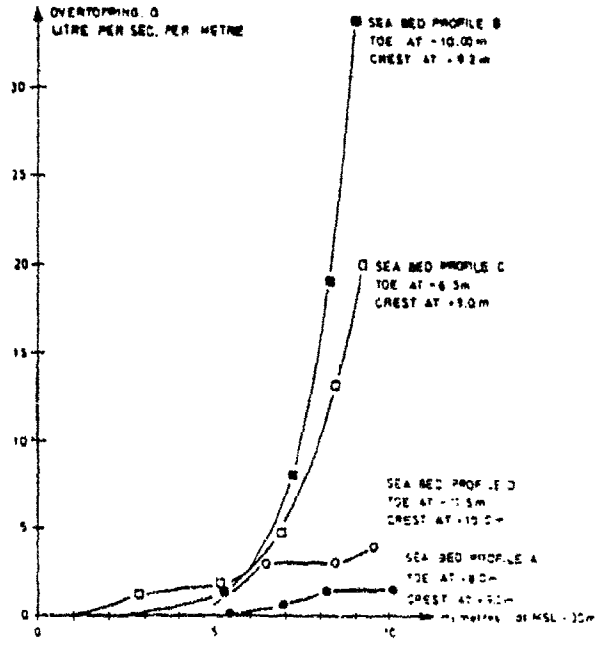


Figure 12.
Example of sensitivity of overtopping to wave height. Delft Hydraulics Laboratory. Sea bed profiles refer to Figure 5.

It is seen that the overtopping, Q increases exponentially when the wave height exceeds a certain value. A 10% increase in H_s can easily cause doubling of Q. The exponential growth of Q with H_s usually makes log Q a linear function of H_s.

Based on different scale model experiments Jensen et al., 1979, presented a more general description by means of the parameters QT_2/B^{*2} and $H_s/\Delta h$. T_2 is mean zero crossing wave period, B^* is a representative horizontal dimension and Δh is the vertical distance from still water level to the top of the crest or wave wall. By introducing Δh also the influence of water level is taken into account. Figure 13 shows an example given by Jensen et al.

Figure 13 clearly shows that even small variations in the still water level might cause significant variations in overtopping.

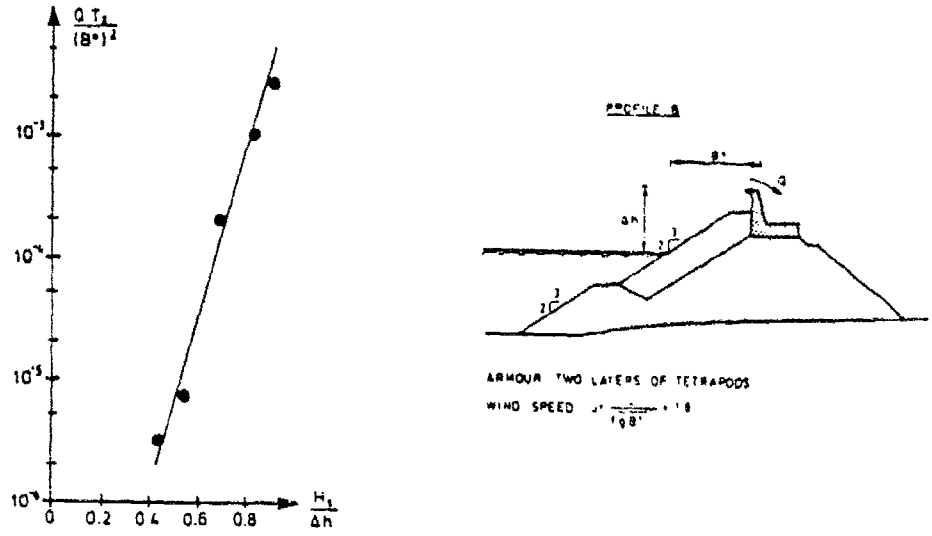


Figure 13. Example of sensitivity of overtopping to wave height and still water level. Jensen et al., 1979.

4.3 Directionality of the waves

Until to day nearly all breakwater model tests have been performed with uni-directional (2-D, long crested) waves. However, in nature the waves are directional (3-D, short crested) with horizontal spread of energy.

It is generally believed that 2-D waves is a good approximation to natural waves in shallow water due to the refraction which tends to make the waves long crested. However, Thunbo et al., 1984, found from a scale model experiment with a stone armoured breakwater with slope 1:2 in shallow water that 2-D waves caused 30-50% more damage than 3-D waves. This compares approximately to the necessity of a 40% increase in armour stone weight when going from 3-D waves to 2-D waves at the same damage level. Figure 14 shows some of the results.

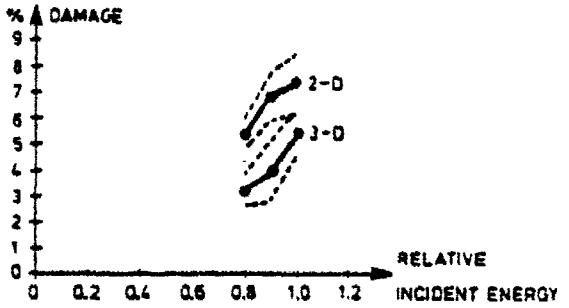


Figure 14. Example of comparison of 2-D and 3-D wave effects on stone rubble mound breakwater. Thunbo et al., 1984.

Shutler of HR, Wallingford reported from similar tests that no significant difference in 2-D and 3-D waves were found (scatter in the test results blurred possible differences).

It is concluded that there is still great uncertainty about the effect of wave directionality.

5. MODEL TESTS

Model tests are still necessary for practically all breakwater designs that depart from the very simple ideal design often tested in basic model studies of armour stability.

The reliability of model tests is therefore a question of great importance.

5.1 Reproduction of waves and data processing

The first point to discuss is the uncertainty related to the generation and analysis of laboratory waves. This problem was investigated by an IAHR Working Group, which was chaired by Joe Ploeg of Canada. The group consisted of representatives from some of the large hydraulic laboratories. Each laboratory performed the same experiment on a breakwater with the crest at MWL and exposed to some pre-specified waves. The wave climate in front of the breakwater and the water level variations behind it were recorded and analyzed. The results from the various laboratories deviated significantly and it was only after a great deal of thought that the reasons for these variations were explained. It turned out that the discrepancies to a great extent were due to differencies in the processing of the recorded data.

5.2 *Scatter in test data*

Another problem in model testing is the scatter in the test data signifying the response to the waves. This can be illustrated by some stability tests performed at the University of Aalborg with a Dolosse armour layer having a slope of 1 in 1.5 and exposed to irregular waves. For each of five different significant wave heights, H_s, 15 tests with identical wave trains were run with the object of studying the movements in terms of rocking and displacement of Dolosse. Very careful visual observations were made simultaneously by four people each covering a small area. A mirror system was used to obtain reliable observations in the splash and underwater zones. Each test was run for 20 minutes corresponding to approximately 1200 waves.

Some test results are shown in Figure 15, which illustrates the observed scatter related to the number of rocking and displaced blocks. These two modes of movement are relevant to the mechanical integrity of the blocks and the hydraulic stability of the armour layer.

Although direct recording of stresses in and/or recording of speed/acceleration of the blocks are much better than visual observations, the diagrams clearly illustrate the fact that reliable estimates of stability can be obtained only when tests are repeated several times. This is a fact which should not be overlooked.

It means that it might be necessary to apply a large safety factor if only a few tests are carried out, or to spend a lot more money performing many more tests than is normally the case at the moment. This is especially true for the complex, fragile types of armour units since it is seen from the Figure that the normalized standard deviation σ/μ for the numbers of displaced units is very large for small degrees of movements or damage corresponding to the design criteria for such units.

For large degrees of damage, i.e. failure situations, the scatter is reduced.

It should be mentioned that separation of rocking and of displacement in the "two" diagrams is not entirely meaningful and should be avoided in design diagrams. It is also important to remember that the scatter (e.g. in terms of the standard deviation) is dependent on the size of the test section.

5.3 *Scale effects*

The reliability of breakwater scale models has often been and still is seriously questioned and in most cases exclusively with reference to scale effects (thus forgetting the afore mentioned points). All scale models involve improper representation of some forces simply because only two types of forces at a time can be represented to scale. Therefore the question is "how much" is the model biased.

The two dominating forces in wave action models are gravity and inertia forces. Considering only these two types of forces the Froudian model scale law used for breakwater models ensures dynamic and kinematic similarity of the scale model and the prototype. Consequently viscous forces and surface tension are not reproduced to scale.

Viscous effects

For a wave exposed breakwater the flow is extremely unsteady. In some parts of the porous structure the flow will be turbulent or laminar all the time but in some part intermittent between the two flow-regimes, as discussed by Burcharth 1983.

The turbulent dragforces will scale like the inertia forces, because the viscous contribution is insignificant.

The flow-regime in granular structures is usually characterized by a Reynolds' number defined as

$$\mathrm{R} = \frac{V\,d}{\nu} \quad , \tag{20}$$

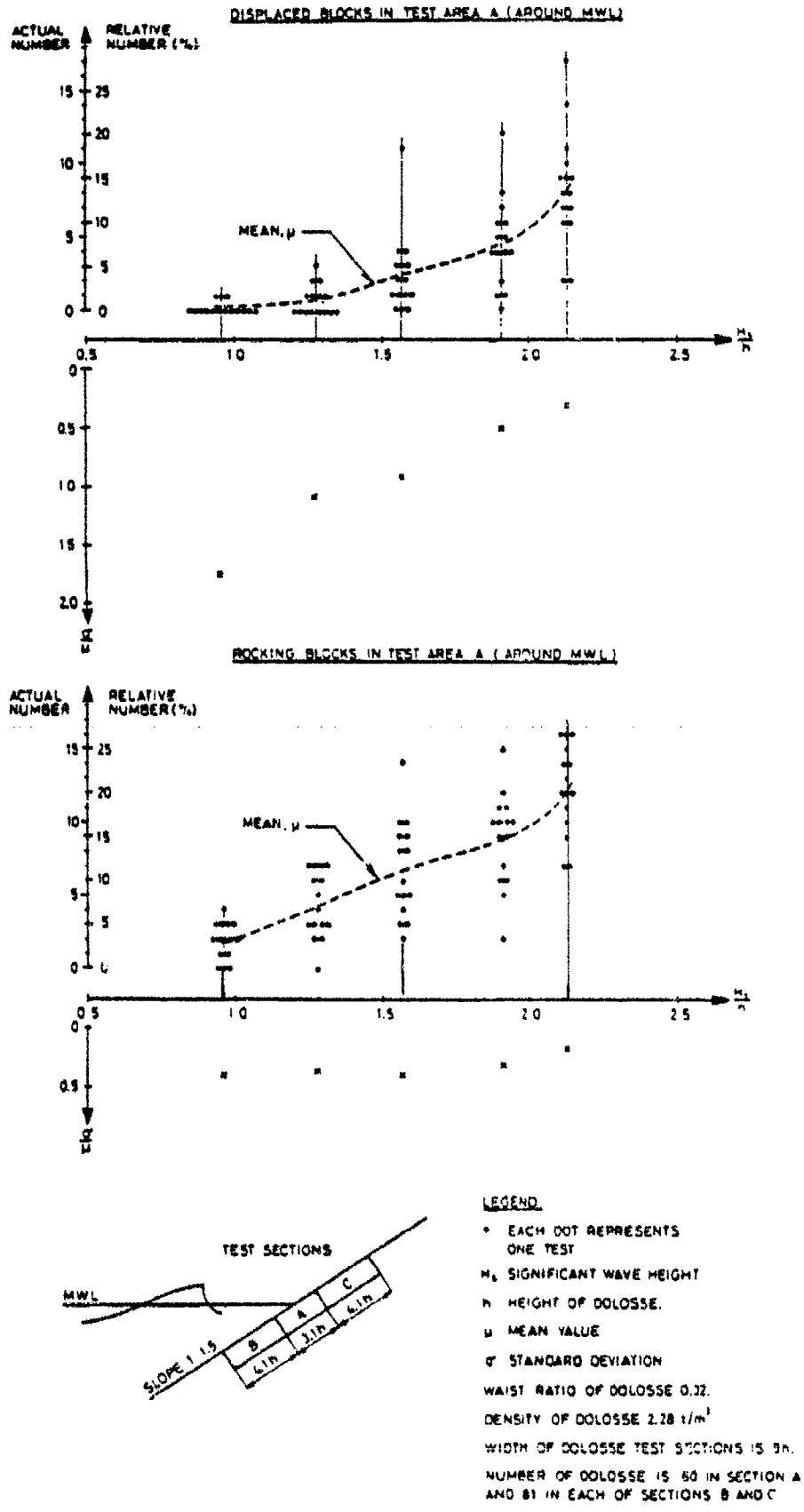


Figure 15. Example of scatter in armour stability tests. Burcharth and Brejnegaard, 1982.

where V is a characteristic flow velocity, d is a characteristic length and ν the kinematic viscosity of the liquid. When evaluating the unsteady flow in breakwaters it has become a tradition to use a constant figure for V which, more or less, is the maximum particle velocity of the incoming wave, i.e. $V = (g\,H)^{0.5}$, where g is the gravitational constant and H is the wave height. d is usually taken as a typical diameter of the armour units/filter layer stones/core material, thus characterizing the width of the flow channels.

The primitiveness of this approach is obvious, but it is difficult to come up with an alternative which is both meaningful and simple.

Many researchers have studied viscous scale effects in breakwater models and the state of the art might be summarized as follows:

- No "significant" scale effect is observed in the "hydraulic stability" of the armour layer if $R \geq 1 - 3 \cdot 10^4$ (d being a characteristic diameter of the armour units) and if the filter stones and the core material are geometrically to scale.

  However, it is important to notice that this statement is conclusive only in relation to mechanically strong armour units such as for example natural stones and concrete cubes. For the more fragile, complex types such as Dolosse and Tetrapods a scale effect which is not identified from visual observations of armour unit movements in the model might, when transferred to prototype, cause a very different amount of breakage. Timco et al., 1984, investigated this in some tests with Dolosse units with correctly scaled mechanical properties. They found that the influence of core permeability on the breakage of the Dolosse was very dependent on the geometrical scale.

- Run-up and overtopping are affected also by the porosity of the filter layer and the core. It has not been properly investigated how much changes in the size of the stones in order to obey the Reynolds' criteria stated above will bias run-up and overtopping.

- The reflection of waves from a breakwater scale model is practically independent of the permeability of the core, Timco et al. 1984.

- There is evidence that ultimate failures of rubble mound structures armoured with strong units can be studied with great accuracy in scale models. This statement is mainly based on a comparative study by DHI, Jensen et al., 1985, of the failure of the Thorshavn breakwater in the Faroe islands. This study is significant because of the availability of the prototype records of the waves in front of the breakwater throughout the damaging storm. The Reynolds' numbers in the model were about $4 \cdot 10^6$ for the armour stones and about $5 \cdot 10^3$ for the quarry run which eventually was exposed to the waves.

- Very little is known about scale effects related to the flow and the pore pressure in the more impervious parts of the breakwater such as the core (and the seabed if of sand). This means that for example uplift forces on concrete cappings and geotechnical aspects such as slip-circle stability and settlement cannot be properly evaluated in a scale model at the moment.

Surface tension effects

The surface tension determines the amount of entrapped air in breaking waves. As a consequence scale effects are present in scale models of forces from breaking waves and overtopping/spray. The shape (surface profile) of the waves in very small scale models is also affected.

Stive, 1985, studied the influence of air entrainment in a comparative scale model study of waves breaking on a beach. He recorded wave heights, set-up and vertical profiles of maximum seaward, maximum shoreward and time-mean horizontal velocities and found no significant deviations from the Froude scaling in a wave height range of 0.1 metre to 1.5 metre. This indeed indicates

that surface tension scale effects are insignificant even in small scale models except for phenomena where a very accurate reproduction of the profile of the breaking wave is important. The most important example is shock pressures on plane solid walls. A special problem related to shock pressures is the interpretation of the recorded pressures in the model, because the air compressability is not to scale. This problem has been discussed by many researchers, see for example Lundgren, 1969, but it still remains to check model data against prototype measurements before the uncertainty related to shock pressures can be evaluated.

However, the author believes that the order of magnitude of wave pressures on wave walls found from proper scale models is correct. This opinion is based on a study of breakwater failure where damage to the concrete capping with wave walls allowed a rather accurate determination of the wave forces involved. By means of results from scale model tests, performed by DHI, in which wave pressures on the wave wall were recorded, it was possible to estimate the wave climate. This estimate was in very good agreement with the wave climate established by hindcast from meteorological observations.

Effects of mechanical properties of armour units

The relative strength of armour units is dependent on the size of the units, Burcharth, 1981. This has to be taken into account when designing and interpreting the scale models. The importance of this has been demonstrated in a number of papers by NRC, Canada, see for example Timco et al. 1983, who also developed a method of producing concrete armour units with correctly scaled mechanical properties, Timco 1981.

There are different ways of tackling this strength problem in scale models, as discussed by Burcharth, 1983, but in the case of tests with large (in prototype), complex types of unreinforced armour units the method established by NRC seems to be the best. The reliability of the method has yet to be evaluated. This can be done only by comparison with prototype measurements. A promising full scale experiment with instrumented 48 t Dolosse set up by the U.S. Army Corps of Engineers, Vicksburg, might provide very useful data for such a comparative study.

6. STOCHASTIC DESIGN PROCEDURE

It follows very clearly from the foregoing discussion that our quantitive knowledge on the loads and the structural response is limited to such an extent that design based purely on theory is not feasible. It is obvious that it will take years before we have developed a reliable design theory. Until then scale model tests are by far our most important tool.

In this rather unfortunate situation it is reasonable to think of a stochastic or probabilistic design method. However, it is often argued that a probabilistic design procedure is of little value as long as the understanding of the physics is poor. It is of course true that such a design process never gives figures in which to place high confidence as long as we cannot describe the physics. However, it is worth while to recall that the less we know, the more important it is to try to assess the reliability. The probabilistic approach is the only one which gives 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. As engineers we must estimate these functions, just as we estimate safety factors.

To-day's knowledge makes it of course not very easy to assess the probability functions. This is obvious from the Figures 16 and 17, which show typical failure modes and the corresponding fault tree. It is seen that not only the distribution functions for a great number of individual parameters but also the joint distribution functions for correlated parameters must be estimated.

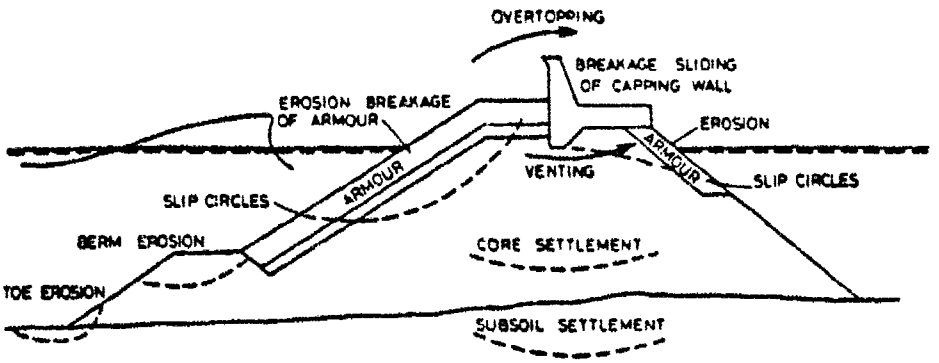


Figure 16. Failure modes of conventional rubble mound breakwater.

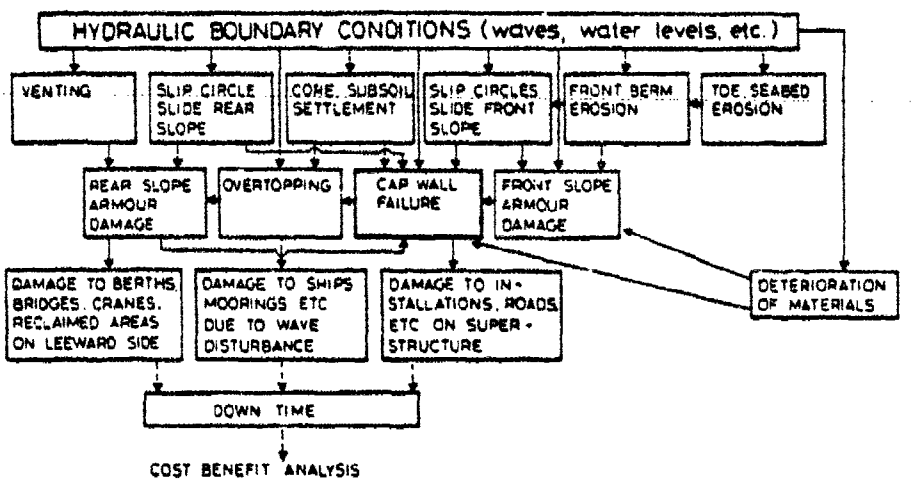


Figure 17. Simplified FAULT TREE for one section of a conventional rubble mound breakwater.

Despite all the problems one should go ahead and for a start restrict the stochastic design calculations to the vital parts of the structure only involving the most important parameters. An example is given by Nielsen et al., 1983, for the design of an armour layer. Systematical scale model experiments should be performed by capable laboratories to support this development.

In the present situation it is very important for the designer of rubble mound breakwaters to understand all the uncertainties he is up against. This might help him in developing designs which are uncomplicated in the sense of clear failure modes and load responses which are not very sensitive to exceedences in the estimated loads.

SUMMARY OF "COMMENTS FROM DR. HANS BURCHARTH"

MR. ORSON SMITH

MR. SMITH:

I'm with the design branch at CERC where we keep our head below the clouds and our thoughts close to the earth and deal almost exclusively with field applications. I had the opportunity to talk to Professor Burcharth quite a bit last week and had also previously read a number of his papers, so Gary Howell asked me to review some of his thoughts on prototype testing of dolosse and specifically the project at Crescent City.

Dr. Burcharth was asked early on in his visit if he thought predicting structural integrity of concrete armor units was really possible, and his answer to that was an emphatic yes. He felt, however, that this would not be in the form of a simple formula. In fact, we're a long way from predicting it in the sense that failures can be anticipated accurately in the design process. He felt that much prototype data and many laboratory experiments were going to be necessary and that a patient and methodical approach was required. To this extent he thought that the important variables must first be identified, and this should be done with an open mind, and that once these variables are identified they should be addressed one by one to the extent that is possible.

He felt the prototype measurements contemplated here at Crescent City are an important first step and that they are critical to future scale model experiments, specifically the calibration of those model tests. He hoped, however, that the Crescent City experiment could be performed for use in general research, even though it is a project-oriented experiment. It's a unique opportunity that will be valuable to many future investigations of concrete armor units.

He had hoped that the units could be placed in a position where the waves were as well defined as possible--this has already come up--and that the units should really be away from elbows, the head of the breakwater, and transitions to avoid the end effects of lateral flows that complicate the identification of variables even further. This led to his stated opinion that the unit should be placed in the center of the new dolosse section in a tight pack or a cluster, perhaps centered around the mean storm water level.

He also was concerned about defining the environment and felt that the waves should be measured quite near the breakwater as well as in deep water. He also felt that the wave direction should be verified by photographs and that the hydraulic pressures in the filter layer and near the cap of the breakwater should be monitored.

He gave a seminar while he was here last week and many of us attended that. He has also left us with a number of his more pertinent technical papers that I think will be distributed.

I might also mention he left a letter that summarizes these comments I'm to review today. I will try now to cover some of his work over the last seven years or more. It's summarized in a useful way in this paper Fatigue in Concrete Armor Units. It's dated October 1984. In that paper he reviews some of his other work by postulating the loads he feels might be important in any concrete armor unit.

There are three general types of particular interest to the instrumentation, and they are the static loads to gravity. He was emphatic that these should be measured because of their importance in future calibration of scale models. He also has identified that there are pulsating or gradually varying loads, usually due to wave action, as well as impact loads, and these are impacts between the individual units. These two dynamic loads are both important. Of course, the impact load is more severe, but the pulsating load, he feels, can be important, particularly when combined with the upper limit of the static loading.

The effect on the fatigue strength of the units is illustrated by this graph. (Graph shown here.) Here are two plots of representing fatigue strength versus the number of cycles, the lower one being the impact loads and the upper one being the pulsating loads.

He made references to some other measurements that have been performed at the Delft Hydraulics Laboratory with accelerometers. He said that accelerometers would be an important component of the instrumentation package, at least to the extent they should be discussed. I guess some are already in the plans.

These impact loads might be significant during placement; so he also offered for discussion the possibility of monitoring all the instruments during the actual placements of the units.

Here is another illustration from the October 1984 paper on fatigue. (Illustration shown here.) He again makes the point of the three important types of loading: static, pulsating, and impact. He has illustrated an example of what the time history might be like. If all three of these types of loading are to be monitored, this time history indicates the problems of calibration for the different types of loads.

I'd say it does present a challenge, but Professor Burcharth thought the effort was worthwhile.

At the bottom of the page, he shows a graph of what might be the total stress in terms of what could be measured at the individual armor units. (Graph shown here.) These little plots here represent the distribution of stress transfer functions, and this distribution is related to the random placement of the units and their quite different orientation from one unit to the other and the armor pack. I think this graph illustrates his point that, in order to define these distribution functions as best as we can, all the units should be placed closely together in the same environment, and that should improve the statistical confidence in all of the measurements.

In summary, Dr. Burcharth's feelings about the project overall were quite enthusiastic. He felt the data would be available both as prototype measurements and for calibration of future scale model experiments. He felt that this pioneer effort should avoid the complications that occur at the head and elbow and transition. He emphatically urged that static loads be measured and both the pulsating and impact loads be monitored. Besides that, he offered the possibility that as a further improvement to the possible confidence problems in the theoretical studies that the instruments be placed at both ends of the dolos and not just the one.

In conclusion, he expressed the wish that everyone involved in future theoretical work with concrete armor units could cooperate in the analysis of the data from this experiment and, from what I have been told, that's exactly the intent of CERC in this project.

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STRENGTH OF ARMOUR BLOCKS - EXPERIENCE AND TESTS AT SINES

Manuel A.G. Silva

1. Introduction

The presentation hereafter attempts at providing data based on the experience at Sines in a way amenable to allow a correlation with Crescent City.

Despite the vast literature available on the climatology and on the accidents that affected the West Breakwater, it is felt that a brief description of these will make the text self-contained and easier to follow.

The major part of these notes is otherwise devoted to the ways on which the collisions of armour blocks under a storm can be characterized and the tests to perform in order to assess the consequences of such impacts.

A sketchy reference to analytical work of relevance for the interpretation of the behavior of the blocks is also added. Topics like the influence of damage accumulation, the effects of thermal stresses at curing and time of contact at the impact are considered.

Last, a summary of recent decisions concerning the new breakwater being built at Sines is also presented.

It ought to be mentioned that great importance is attached, even though not emphasized, to the monitoring of the behavior of both the West Breakwater and the new breakwater for the coal terminal.

The cost of a program that includes continuous recording and inter

pretation of data obtained from instrumented blocks appears to be beyond the resources that Portugal can allocate. Given the worldwide interest on such data and the water depths and exposure at Sines, a concerted international effort may be worthwhile and is suggested so as to allow gathering and interpretation of the recorded results.

2. Background Information

The port of Sines is depicted in Fig. 1, where one can identify the West Breakwater, designed to support three oil berths and provide mooring conditions at the remaining quays. The breakwater reaches depths in excess of 50m and was severely damaged by storms in 1978, 1979. The storms are usually described by their periods(s) and wave heights (both maximum and significant, in meters) as follows:

February 78 - H_{max}= 11.4, H_s = 8.0, T_s = 12.6

December 78 - H_{max}= 10.8, H_s = 7.4, T_s = 12.0

February 79 - H_{max}= 17.6(?), H_s = 10.0, T_s = 13.6

The storms destroyed hundreds of dolos (42 tp) that constituted the breakwater armour and caused immense damage. The dolos were either made of plain or very slightly reinforced concrete. In 1979 a crash program for repairs was undertaken, based on placement of antifer cubes weighing 88 tp. These emergency repairs were tested in December 81 by a major storm (though more oriented towards SW than the previous ones) and behaved quite well.

The characteristics of the waves in December 81 were H_{max} = = 16.9 (?), H_s = 9.0 and T_s = 12.0 (?). The question marks evidence some uncertainty on the correct recording of the waves.

Hindcast studies performed at W.E.S. led the designers to pro

pose, for the future repairs, a design wave characterized by a recurrence period of 100 years, with T_p = 16 s and H_s = 14m.

In addition to the West Breakwater, another one is being built to protect the new coal terminal of Sines. This breakwater runs along depths of 25m and its armour is formed by cubes of 60 tp and 71 tp, the latter ones being made of heavy density concrete.

Henceforth, it wil be assumed that both the layout and the climatology are known, concentrating the text on tests and recommendations on the blocks to be placed.

3. Studies Performed for Sines

Perhaps the singlemost important cause of the Sines failures in 78/79 was the fracture of the dolos. Such fracture had not been anticipated in the hydraulic tests previous to the cons - tnuction due to known limitations of Froude similitude. This fact further motivated a series of studies on the behavior of antifer cubes that are described below.

A rational approach to the establishment of the impact strength of the blocks comprises (i) hydraulic tests to determine impact speeds and number and modes of collisions and (ii) tests, based on the data obtained, to assess the strength of the blocks.

Experiments with a significant number of blocks of large size (60 tp, 71 tp, 88 tp) are costly, difficult to conduct and last for a proctrated period period of time. As a consequence, a reasonable effort was placed on analytical interpretation of test results, mainly aimed at establishing the patterns and values of curing stresses under different casting procedures, at predicting the effects of cumulative shocks and at interpreting the effects of coaxial and edge to face impacts.

Three different types of tests were considered:

a) Drop tests from different heights on a thick slab;

b) Tipping tests like those performed in Sines in 1980 [1];

c) Collision tests with the cubes placed on small flat railroad cars as described in "Coastal Structures 83" [2].

Tests a) are physically difficult to perform and tests b) add to the same problem the question of tipping for an initial angle other than 45°. Both a) and b) raise the issue of appropriate compliance of the foundation, are hard to be repeated for cumulative damage assessment (due to strains developed while resetting the tests) and b) originates a complex stress distribution, fairly difficult to analyse. In addition, the blocks rock alternatively around consecutive edges, creating one more source of uncertain correlation with the breakwater [3].

The above set of reasons led to the decision of concentrating the efforts on tests type c). Again the intertwined subjects of costs, handling and obtaining data within oportune time advised that most tests be performed on smaller blocks, with care exercised to account for scale and curing effects [3].

Prior to starting the impact tests, the ranges of speed to be examined as well as scale effects and a study of concrete mixtures with slow hardening cement was performed.

Having to be selective, reference is emphasized, next, on the hydraulic tests that provided data on impact speeds.

4. Hydraulic Tests

Tests on hydraulic stability were performed in Lisbon (LNEC) and Delft (DHL). Reference hereafter, for simplicity and unless otherwise noted, is understood to correspond to the West Breakwater with qualitative statements applicable also to the breakwater under construction.

Most tests were run in models built at a geometric scale 1:78 and the climate simulated in agreement with hindcast studies. Aside from the standard characterization of hydraulic stability, these tests were used to provide answers to the questions: do the blocks move? If so, do they collide? If so, do they fracture? Conclusions were that rocking starts for a significant wave of approximately 7m and that the blocks collide. Breakage cannot be assessed from tests at scales of the order 1:78 for well known reasons. As a result, experiments were conducted at de Voorst (scale 1:12) with the material properties scaled down. The findings were not wholly satisfactory, primarily because of much higher abrasion than in prototype. The blocks never fractured (even for H_s = 17 m) beacause of the significant amount of energy dissipated through abrasion and, perhaps, due also to discrepancies in wave propagation properties.

Experience at Sines indicates, nonetheless, that the blocks fracture under severe rocking, a fact that tipping tests [1], however non representative of true behavior, had already shown. Altogether, the above data confirmed the wisdom to ascertain the impact strength of the concrete blocks.

Some cubes in the models were, then, instrumented with accelerometers (3D at de Voorst and 1D at Delft) in order to generate data leading to the impact speed, factor of utmost importance in these phenomena.

In order to gain information with some statistical legitimacy, most tests were run in series of four under the same conditions. A short summary of the major aspects follows.

4.1. Impact Speeds

As mentioned earlier, the results reported apply essentially to the West Breakwater rehabilitation profiles as typically illustrated in Fig. 2. Storms, henceforth identified solely by their significant wave height (H_s = 8, 10, 12, 14m), were

selected and four runs of 100 waves were applied for each H_s. The instrumented units were placed at different locations selected by the likelyhood of most severe rocking, thus origi - nating a family of parametric data.

The integration of the accelerograms led to estimates of impact speeds, v_i, measured at the mass center and averaged for each series of tests.

Fig. 3 shows a typical family of exceedence curves relating each impact speed v_i with the percentage of waves for which v_i is exceeded, for the imposed H_s.

The application of the theory of extremes, using an adequate Weibull distribution, gives the probability p that v_i be exceeded in a storm of 100 waves. Fig. 4 gives, in ordinates, (1-p), for one of the sequences of tests.

The probability p associated with the train of 100 waves is converted into p_n for the n waves acting during the actual duration of the storms (as defined by the climatology):

$$p_n = 1 - (1-p)^{n/100}$$

It is also possible to determine the number of blocks N that move under a prescribed storm by means of overlay and/or other techniques. Combining p_n and N it is immediate to estimate the number of blocks colliding at speeds exceeding a fixed value $\bar{v}$. If $\bar{v}$ is correlated with fracture, by a known cri - terion, the damage caused to the blocks by a certain storm can be assessed. This information is important for designers (in terms of stability and costs) and Port Authorities (in terms of costs and risks).

The table below shows, for a typical series of tests, the number of blocks that impact at speeds higher than the tabulated v's.

COLLISIONS AT SPEEDS $\geq$ v

H_s (m)	Number of Cubes in Motion	v (m/s)			
		1.0	2.0	2.5	3.0
8	68	-	-	-	-
10	67	6	-	-	-
12	51	46	18	7	5
14	77	74	27	6	2
% OF TOTAL NUMBER OF BLOCKS		26%	5%	3%	1%

For instance, in this series, 26% of the blocks moved with $v \geq 1.0$ m/s and 5% with $v \geq 2.0$ m/s.

In de Voorst, the tests showed that the accelerations could go up to 5g and v_i up to 3.5 m/s.

4.2. Importance of Location of Blocks

The highest impact speeds were found at the level (-6.0)m, whereas the location where the highest number of collisions took place is at (-3.5)m. These conclusions were obtained after averaging the readings in each series of four runs. Fig. 5 shows some of the data that allowed the obtaining of those conclusions.

It is remarked that Figs. 2 to 5 are adapted from reports submitted by DHL to G.A. Sines, with due permission of the latter.

5. Tests on Impact Strength

The dynamic strength of the antifer cubes involves a large number of parameters ranging from strain-rate sensitivity to scale effects, type of cement to casting conditions, abrasion to number of collisions prior to failure, impact speed to mode of collision, tensile strength to water/cement ratio, moisture content to curing conditions.

Attention is, hereafter, concentrated on the actual tests performed at Sines, without singling out most of those aspects for the sake of objectivity and simplicity. Experimental studies on the importance of thermal cracking, weather environment during casting and concrete mixtures were, however, made and reported elsewhere.

The tipping tests are not examined here (see [1] and [3]), but earlier comments compounded with those of W.G. Godden [5],

> "... correlation between theory and test is extremely difficult to attain, as is consistency in experimental data"

explain some guarded doubts of the author on the expectations to be placed on the validity of such experiments.

Emphasiz is, therefore, placed on the impact tests of cubes placed on rail cars, with the speed at collision regulated from the height of release of the cars. These tests are designated as translational tests [1,3].

The translational tests were performed on cubes of geometry and weight tabulated below. The concrete mixture was approximately the same used in the repair works, except that fine sand was unavailable when the cubes of 1 through 27 tp were cast.

Blocks Tested

Weight (tp)	1	3	9	27	88
Width at base (m)	0.77	1.11	1.60	2.30	3.49
Width at top (m)	0.71	1.02	1.47	2.13	3.23
Height (m)	0.71	1.02	1.47	2.12	3.21

Essentially three modes of collision were studied:

a) Face on face, requiring lengthy operations to ensure (?) a full face shock;

b) Face on face inserting a thick steel plate at the center, in order to know the area of actual impact and ensure coherence of data;

c) Edge on mid-face collision.

Fig.6 shows the site where the tests were made, together with pictures of collisions type b) and c).

The importance of damage accumulation was examined by repeating collisions at constant speeds (below a predetermined critical speed that is the lowest speed causing fracture at a single shock) till failure occurred.

The main results are described in [2] and not repeated here in detail. Major features are summarized in Figs. 7 through 9. In short, it can be stated:

i) Scale effects are important;

ii) Damage accumulation due to previous shocks significantly reduces the impact strength;

iii) Impact of the face of a block by the edge of another block causes only local crushing, unless the shock takes place near the border;

iv) Interposition of a thick steel punch ensures coherence of results (at the cost of more difficult correlation with breakwater response) and requires higher number of shocks for fracture;

v) Square punches create biased failure due to corner singularities of stress field and are advantageously replaced by circular indenters.

6. Analytical Studies

The interpretation of the results obtained and their extrapolation advised analytical studies that are partially mentioned in the sequel.

Aside from the rigid body type of analysis of the tipping tests [3], an approximate study of the edge on side collisions was conducted based on the theory for elastic half-spaces loaded impulsively [2]. The single shock problem by a rigid punch was also examined [2].

It is felt, however, that the key to the successful study of the behavior of the cubes rests on an accurate characterization of its material properties. Concrete has a large number of microdefects, even before external loads act on the structures. The gradual growth of the microcracks at the interface aggregate-mortar explains best the nonlinear behavior of concrete and is translated by a much simplified version of the damage theory [6].
Some relevant results obtained are presented next.

6.1. Critical Speed and Fatigue Effects

The stress-strain curve for concrete was fitted, the tangent modulus E was, then, obtained at each strain level and the critical impact speed v_{cr} at which crushing occurs was evaluated [7]:

$$v_{cr} = \int_0^{\varepsilon_m} c \, d\varepsilon = \int_0^{\sigma_m} d\sigma / \sqrt{\rho E}$$

where σ_m = maximum stress, ε_m = strain for $\sigma = \sigma_m$, ρ = mass density and $c = \sqrt{E/\rho}$. For instance, with $f'_c = 3.2 \times 10^4$ kN/m^2 and a known σ-ε curve, it was obtained $v_{cr} = 5.9$ m/s, while the impact speed v required to develop an impact stress $\sigma = 2.9 \times 10^4$ kN/m^2 was found to be $v = 0.8\ v_{cr}$.

The weakness of the concept of critical speed lies on the fact that, prior to fracture, a number of collisions affects the strength of the blocks which fail as a result of gradual damage growth. Making use of a logarithmic relationship between the fatigue strength f_N and the number of load cycles N, considering a strength reduction of 40% after a million cycles and using f_1 as 94% of the ultimate f'_c, the following table was found for concrete used in the repairs at Sines (f'_c = 254 Kp/cm2):

Effects of Cyclic Loading

Number of Cycles	Speed Causing Failure (m/s)
1	3.6
5	2.9
100	2.6
1000	2.2

These values, while taking accumulation of damage into account, do not consider the thermal cracking induced at curing. The actual speeds are consequently lower than those tabulated.

6.2. Time of Contact

It is sometimes attempted to gain information on the "force" P acting at collision by writing the equation of conservation of momentum. In general, the time of contact is estimated from linear wave propagation as $t = 2L/c$, where L = side of cube is half the length traversed to change initial compression into tension. The aforementioned nonlinear response of concrete, however, significantly alters that value. One dimensional theory of damage leads to the following analytical expression for the constitutive equation [8]:

$$\sigma = 2\left(\sigma_m / \varepsilon_m\right)\left[1 - 0.5\left(\varepsilon/\varepsilon_m\right)\right]\varepsilon$$

which, used in conjunction with the model depicted in Fig.10, led to the values of contact time, maximum stress and ratio of contact times (damage/linear) shown next, both for the case with no indenter and with a rigid punch inserted (coaxial, face on face collision).

Contact Times - Effect of Nonlinearities

	Impact Speed (m/s)	Contact Time x1000 (s)	Max. Stress (Kp/cm^2)	Ratio of Times (Damage/Linear)
No punch	2 4	1.16 1.23	109 199	1.65 1.75
Punch	2 4	1.68 1.85	150 258	2.40 2.64

The study showed that both the maximum stress and the time of contact increase with a decrease of the size of the indenter.

In the calculations, neither fatigue nor thermal cracking were introduced and, therefore, the actual values would require a corrective factor.

6.3. Scale Effects

Considerable attention has been extended to the study of scale effects.

It has been established that a significant weakening of the impact strength of the larger blocks vs small units [2,3] takes place.

For the sake of completness, it is herein included Fig. 10 that clearly illustrates this behavior. The figure also evidences the influence of damage inflicted by previous collisions, an experimental result in close agreement with predictions based on dynamic damage theory.

7. Future Work

In terms of tests to evaluate mechanical strength of cubes, it is envisioned (January 85) that the following will be made at Sines:

a) Conclusion of the translational tests with units of different sizes still available, in a way that wil make data like those shown in Fig.11 more reliable.

b) Evaluation of the true importance of thermal cracking by (i) testing and comparing cubes "normally cast" with cubes submitted to controlled temperature gradients, and, (ii) testing cubes made of different concrete mixtures.

c) Estimate of fatigue properties, following procedures similar to those proposed by Hans Burcharth for dolos.

It goes without saying that the efforts at Sines have to be geared towards ensuring safety at bearable costs. Such criterion means that the tests for the coal terminal cannot be so ambitious as to delay the works and the insight gained in 81/ /83 ought to be judiciously used. On the other hand, for the

future expansion of the port and rehabilitation of the West Breakwater it appears reasonable that the presence of a contractor at Sines be suitably used to generate data that will further enlighten future decisions.

Clearly there are three avenues to follow simultaneously:

I. Close monitoring of the behavior of the West Breakwater to gain data on the actual performance of the blocks and allow adequate correlation with tests.

II. Study of composition of the concrete mixture in order to lower thermal cracking and provide basis for analytical modeling of the response of the blocks.

III. Impact testing of cubes cast at a reasonable scale (perhaps with weights between 10 to 15 tp), together with tests of some prototype units.

As indicated at the beginning, there is a great paucity of information on prototype behavior. It appears that Sines provides a priviledged site for the collection of useful data by monitoring its breakwaters, even through in a manner less ambitious than at Crescent City. A project of this size is not compatible with a rational allocation of Portugal's limited resources and its wide interest thereby suggests a concerted international effort.

The field of fatigue-like effects remains open to further studies, notwithstanding H. Burcharth's findings for dolos. The importance of reducing microcracking at curing justifies also research on cost effective methods of achieving it, paying attention to the repercussion on costs of construction.

A reasonably accurate prediction of the number and type of collisions to be expected for typical storms and the accumulation of effects along periods of time with design and maintenance signification advise more systematic hydraulic tests and risk

analysis.

Attention to physical modeling of material properties, including e.g. tensile and compressive strength, mass density,wave propagation characteristics, abrasion, as well as adequate analytical models for approximate qualitative interpretation of the tests is also reccomended.

It is felt that a bird's eye view, seasoned with good engineering judgement, by specialists on materials, structures and hydraulics acting jointly should prove invaluable in the continuous effort to design and build safe breakwaters at reasonable costs.

ACKNOWLEDGEMENTS

Thanks are due to G.A.S. for authorising the use of proprietary material and to W.E.S. for the invitation to be present at the workshop. NATO Research Grant Nº 099/84 has allowed fruitful cooperation with Prof. D. Krajcinovic and is gratefully acknowledged.

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FIGURE CAPTIONS

Fig. 1 - Overview of Port and Planned Works.

Fig. 2 - West Breakwater - Location of Instrumented Blocks.

Fig. 3 - Impact Speeds vs % Waves for which they are Exceeded. Typical Set of Results.

Fig. 4 - Typical Probability Curves for Collision at Speeds up to $(v_i + 0.5)$ m/s.

Fig. 5 - Typical Findings of Variation of Mean Impact Speeds and Number of Shocks with Water Depth.

Fig. 6 - View of Site and Collisions Edge on Face and Side on Side.

Fig. 7 - Modes of Failure for Side on Side Collision (a) Without, and with (b) Square Punch, (c) Circular Punch.

Fig. 8 - Damaged Cubes After Side on Side (above) and Edge on Side (below) Shocks.

Fig 9 - Pattern Singularities due to Square Indenter. Broken Antifer Cubes (88 tp). Experiment at Reducing Thermal Stresses.

Fig. 10 - Simple Model to Investigate Time of Contact.

Fig. 11 - Effects of Scale and of Damage Accumulation.

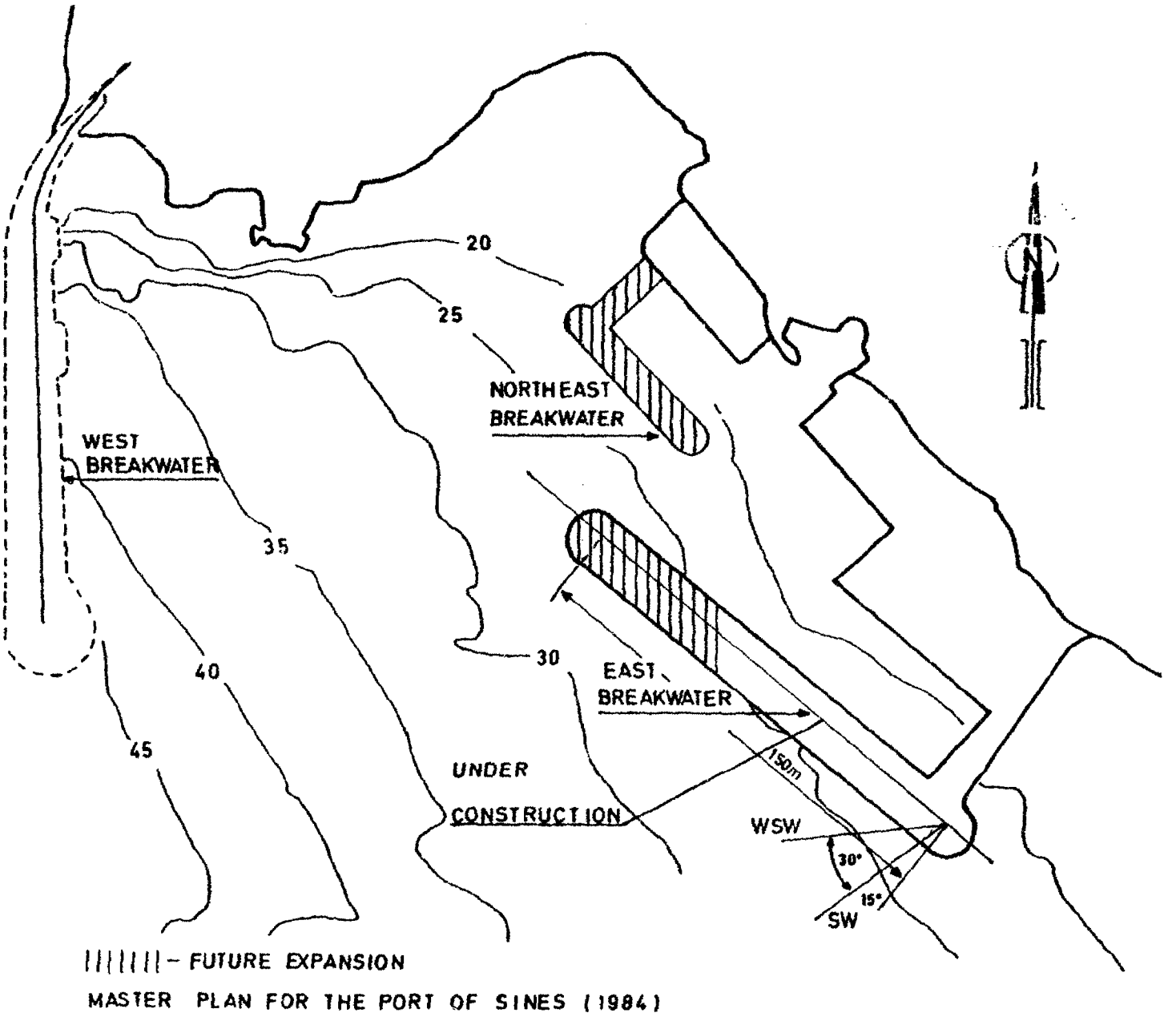
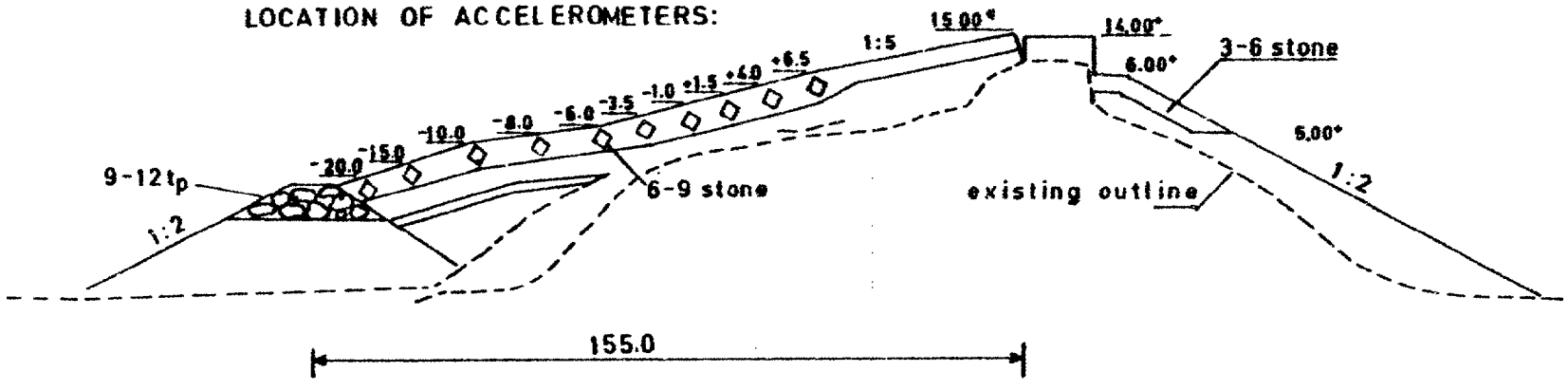

20
25
NORTHEAST
BREAKWATER
WEST
BREAKWATER
35
40
45
30
EAST
BREAKWATER
UNDER
CONSTRUCTION
150m
WSW
30°
15°
SW
N
FUTURE EXPANSION
MASTER PLAN FOR THE PORT OF SINES (1984)

Figure 1



101

WEST BREAKWATER: MODIFIED GENTLE SLOPE ALTERNATIVE

Figure 2

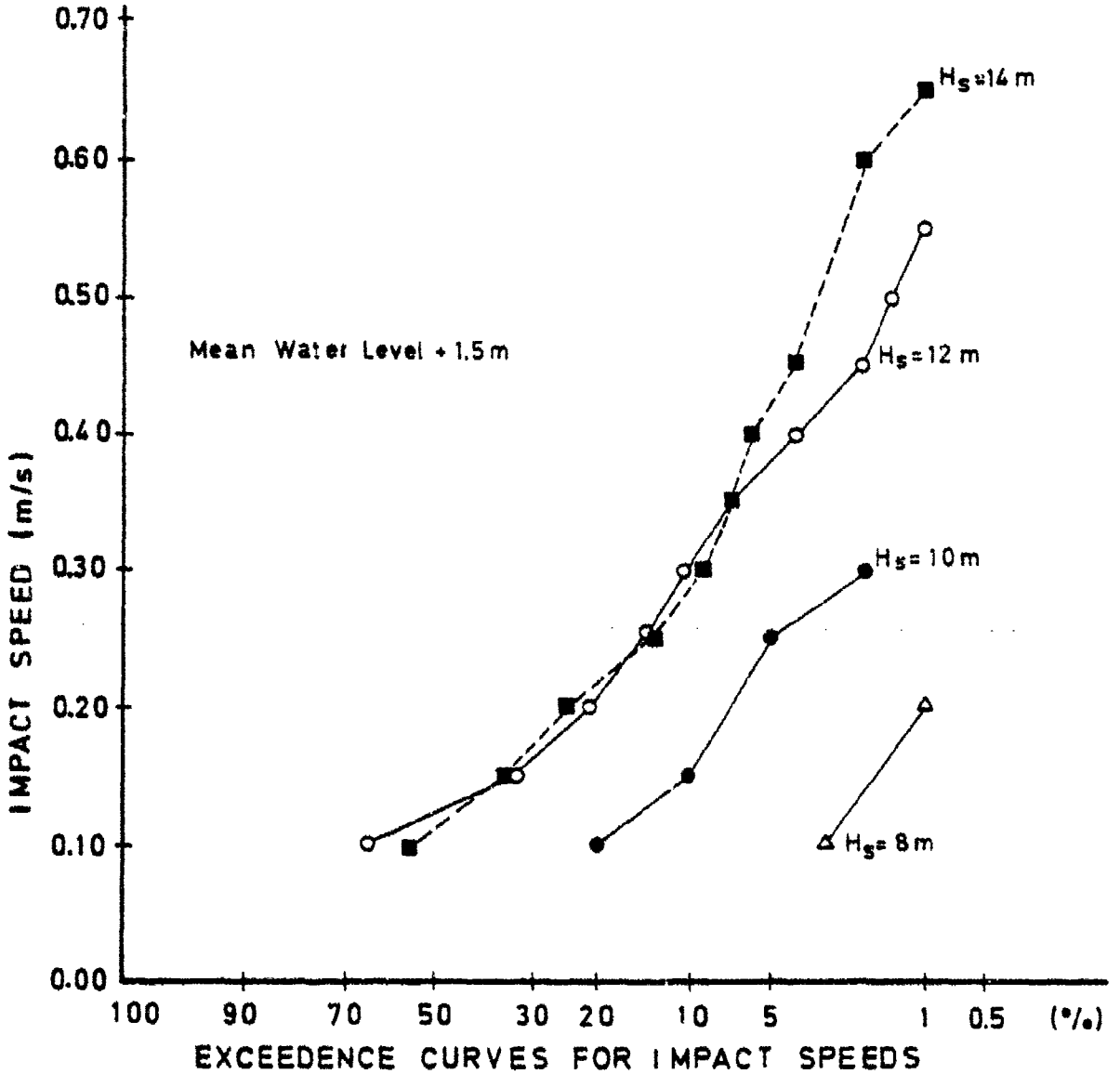
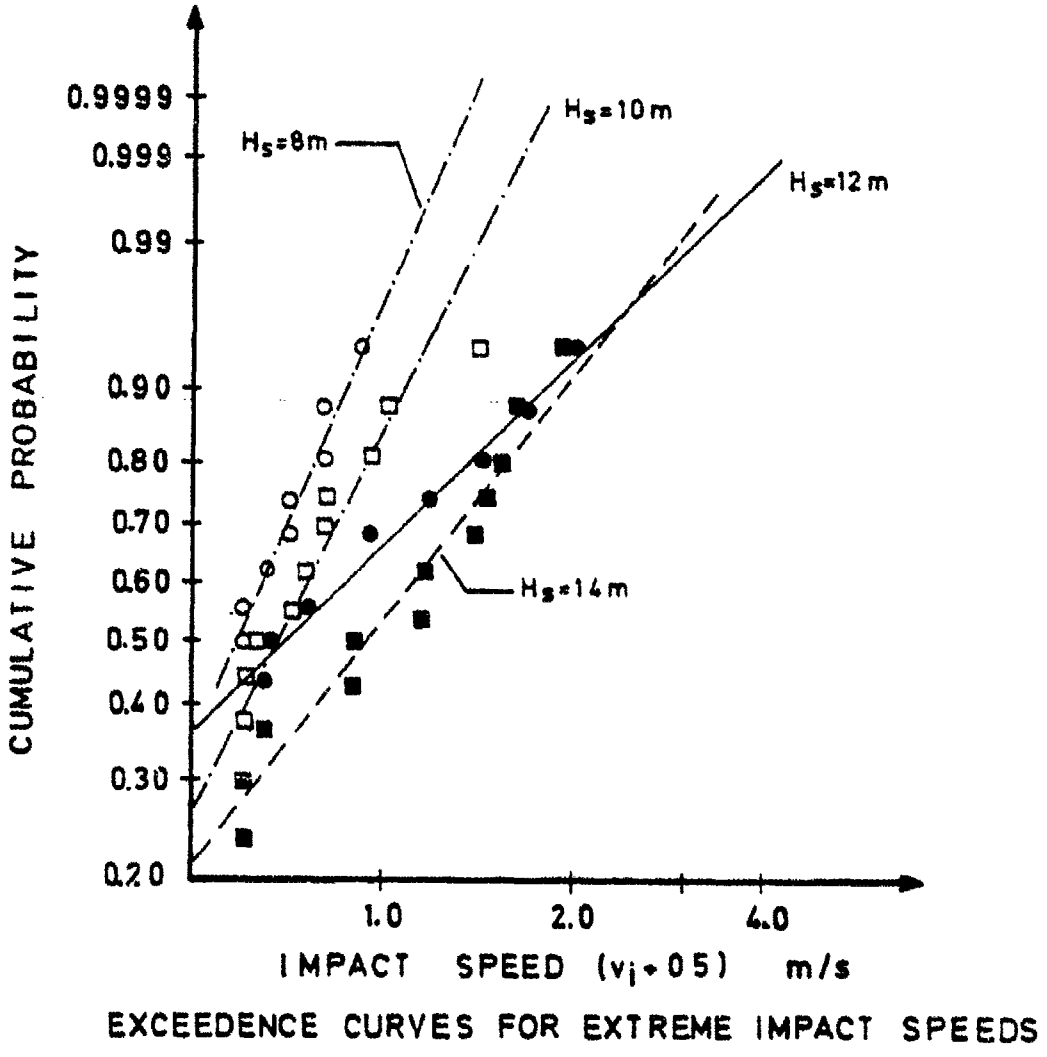
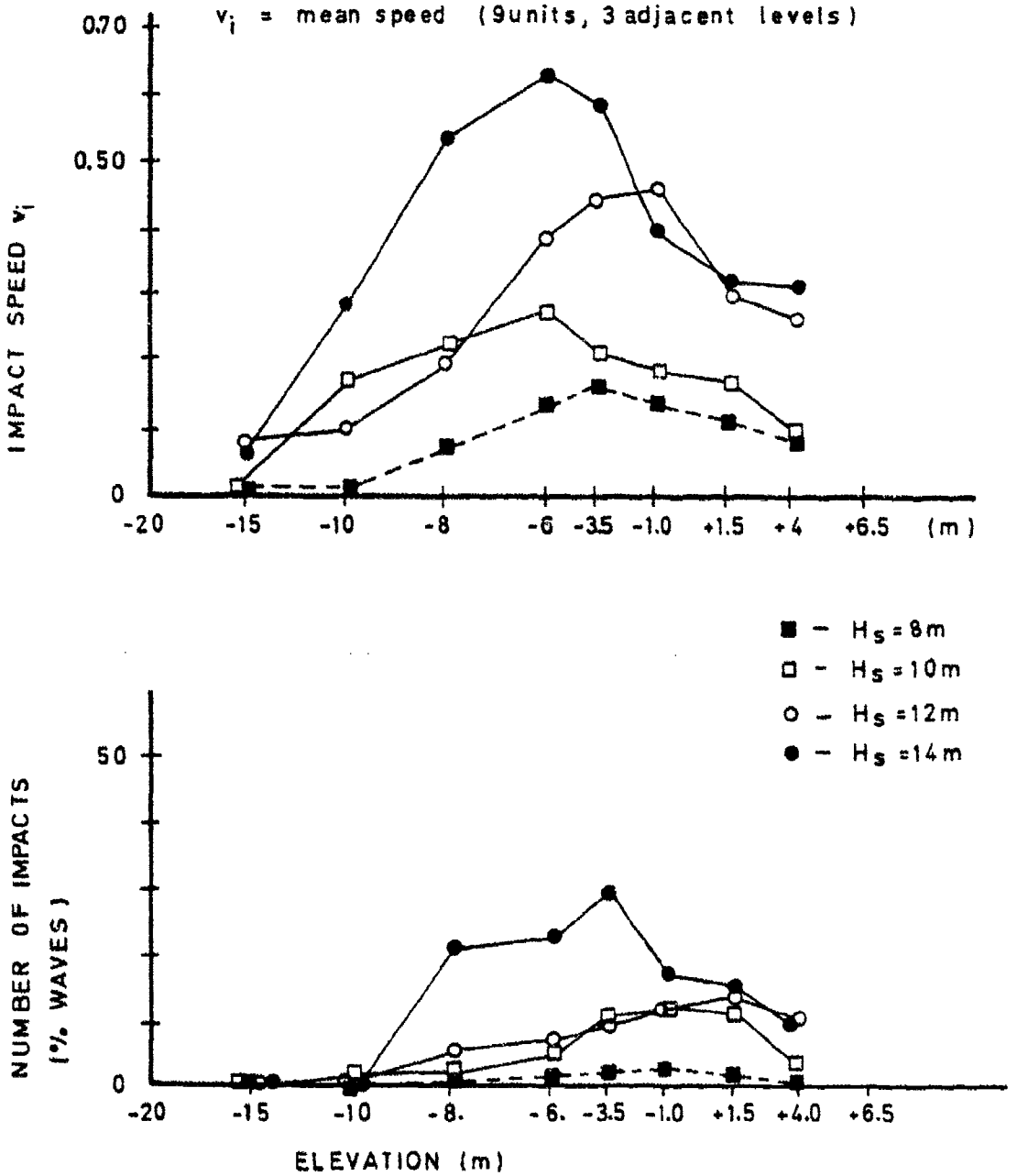

0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
IMPACT SPEED (m/s)
Mean Water Level + 1.5 m
Hs=14 m
Hs=12 m
Hs=10 m
Hs=8 m
100
90
70
50
30
20
10
5
1
0.5
(%)
EXCEEDENCE CURVES FOR IMPACT SPEEDS

Figure 3



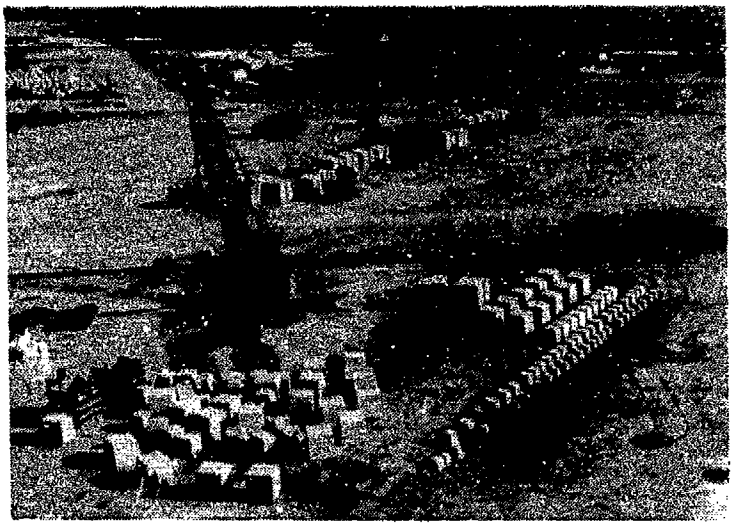
EXCEEDENCE CURVES FOR EXTREME IMPACT SPEEDS

Figure 4



DISTRIBUTION OF SPEEDS AND NUMBER OF COLLISIONS OVER ARMOUR LAYER

Figure 5





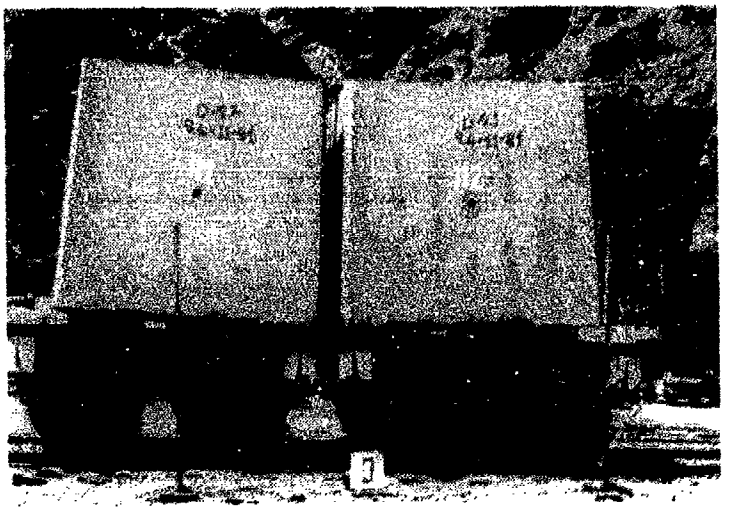


Figure 6

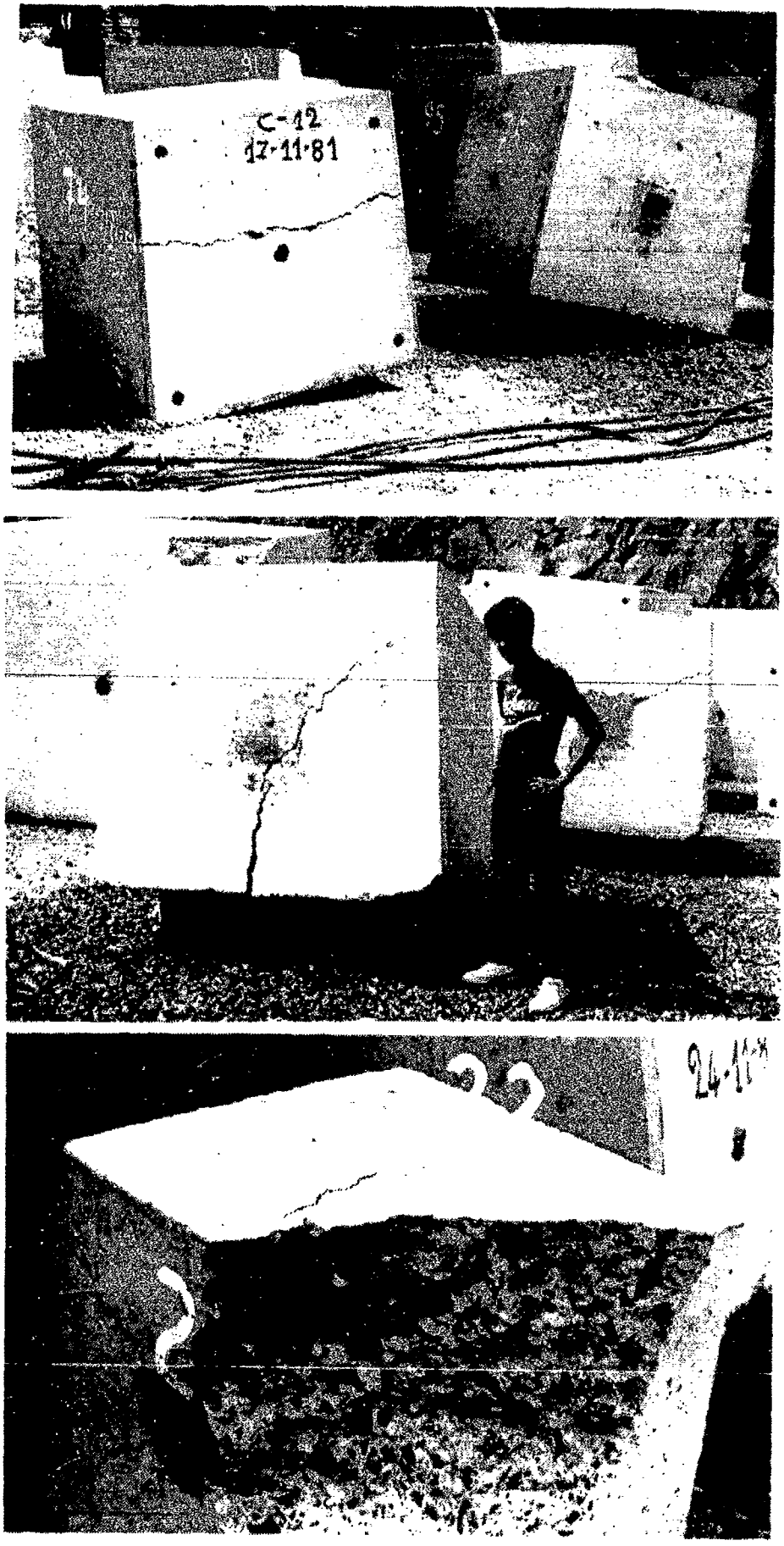


Figure 7









Figure 8

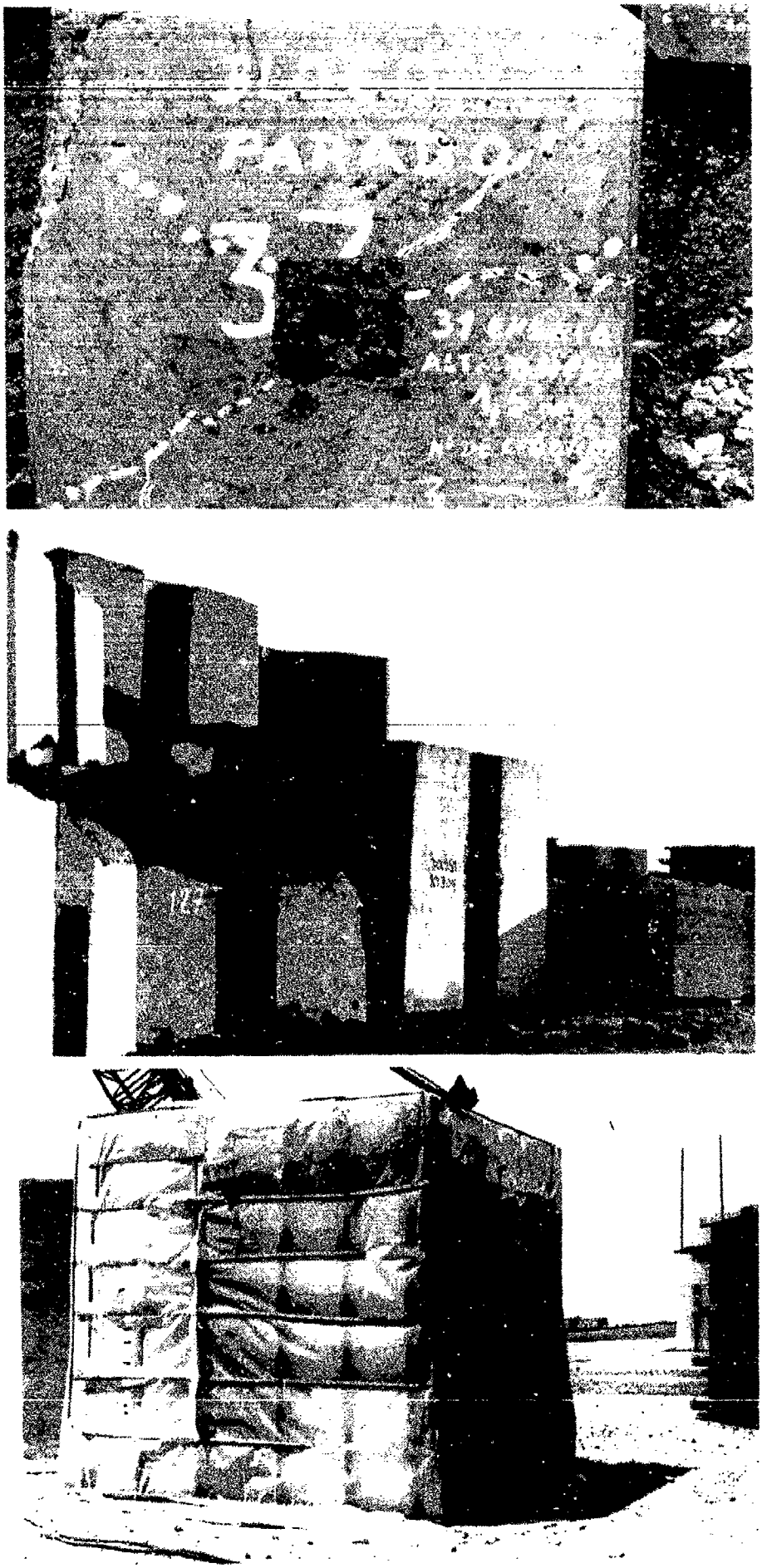
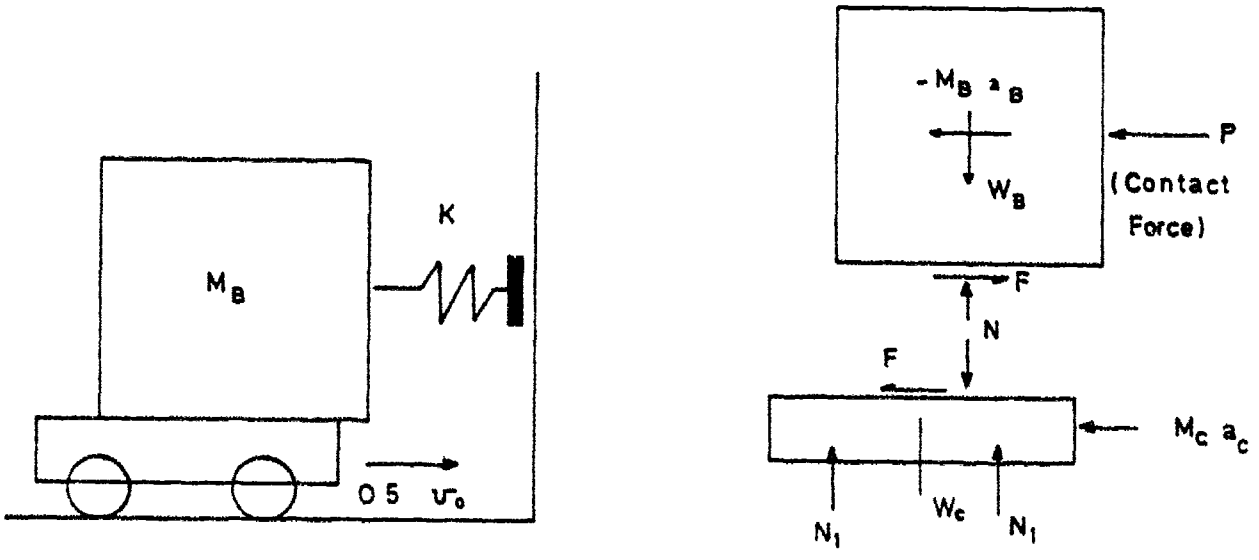
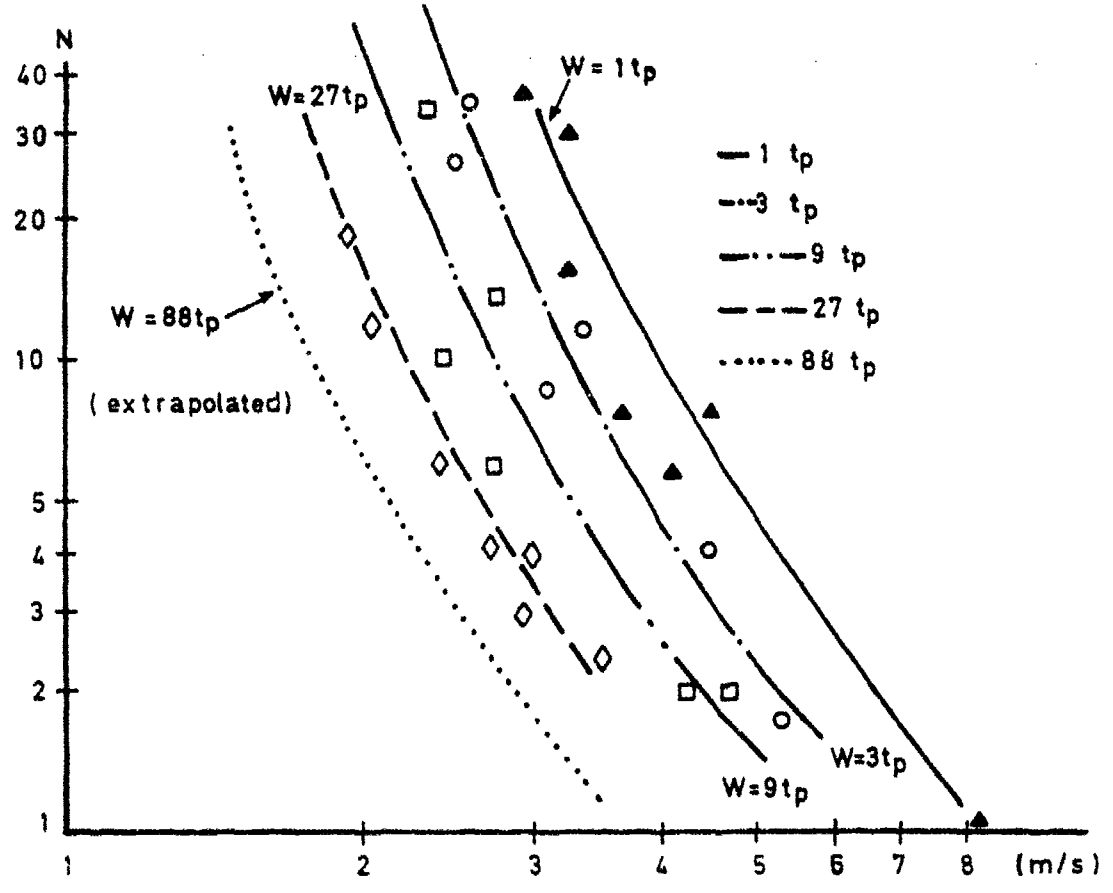


Figure 9



ANALYTICAL MODEL FOR COLLIDING SYSTEMS (k is a softening spring)

Figure 10



NUMBER OF SHOCKS VS SPEED AT IMPACT (1981)

Figure 11

DISCUSSION OF "PORTUGUESE EXPERIENCE-ONGOING STUDIES"

DR. MANUEL da SILVA

Now, what is going on at Sines? What are we expecting to see then? Well, we would like to test the existing units--but, for the reasons I mentioned before, were not tested--at speeds that would be close to against in our curves so it could be a better material for interpretation. We discussed this with Professor Burcharth before he came here. We think that we should perhaps study three impact speeds, at least for three pairs of cubes, which speeds, so as to give some statistical validity to the results. We would like to do that both with the units that were cast as normally we do, without the special care and with the thermal insulation, and see what the different behavior would be. At the end we would like to test some prototypes.

That's what we did in the past three years. Are there any questions?

MR. MAGOON:

Could I ask a question or make a comment? I believe on the cubes that there was a casting of cubes, about 60 tons, in Fishguard Harbor in Wales. To get rid of the problem, they cast a small square hole down the middle of the cube and poured water into it. They cooled the inside of the cube with a small hole.

DR. da SILVA:

It has been proposed for the entire cubes also. The problems had to do with the contracts and contractors.

PHYSICAL MODELS OF

FORCES ON

CONCRETE ARMOR UNITS

"MODELING OF CONCRETE ARMOR UNITS"

DR. ROBERT G. DEAN

DR. DEAN:

I decided I would make some general comments related to armor units. First, I would like to address some of the modeling considerations that seem important to me. These will be in areas of wave structures, the structure itself, and the armor units.

The second area I would like to cover today is understanding the mode of failure or damage, and we'll discuss several different possibilities. I'll go through those in detail, so I won't bother to enumerate them now. Then, finally, I would like to mention briefly some comments about quality control in field testing. I think all of these areas are those that some people here have had experience with before, but I would like to just address several issues, possibly just simply for the purposes of stimulating discussion.

With regard to modeling considerations, and I will keep my remarks brief. First of all, waves. I think that it's very important, especially for deep water, to model the wave spectrum very carefully. With regard to that, several aspects are that the narrower the spectrum is, then, simply because of the results of the narrow distribution, the greater the expected maximum wave height for a given number of waves. I think these results flow from the Rice distribution. In deep water, where the waves are not depth limited, it is important to model the spectrum carefully, and if there's some uncertainty about the width of the spectrum, then I think it is certainly desirable to do sensitivity calculations as to the stability of the breakwater relative to the spectral width.

The second consequence of the spectrum width is the dynamic wave set-up. I'm not sure if that term is familiar to everyone here, but when I speak of dynamic wave set-up, I consider that we have a train of waves that are uniform in height, that they will cause the set-up, and that set-up will be static. We've done some studies that show if there's a variability in wave height, such as with wave groups on a natural beach, then there's a dynamic wave set-up that we've found is roughly 50 percent higher than the static set-up would be associated with the highest wave in the group. The 50 percent is a rough value, but it's pretty close. Then, as some people have hypothesized, if there are several high waves in a group, then this succession of waves can be more damaging to the breakwater than can a series of waves of this lesser height. Of course, if you have a narrow spectrum, that would also result in a succession of high waves.

I would also like to mention briefly, although it's not indicated here, the effect of wave period. I think a lot of times we can learn quite a bit about breakwaters by looking at natural beaches. One thing we know about natural beaches is that, if we have long waves, they tend to push the sediment toward the beach and actually build up a berm; if you have short waves, they tend to pull the sediment off shore. I think that breakwaters work exactly the same way. It's important to recognize that on the faces of the breakwater the slope can actually work both ways. If we have long waves which tend to roll the units up on top of the structure, then the slope acts in our favor; but, if we have short waves, it tends to pull the units off the structure and then they act against us.

Wave direction. I think that you recognize that, in some cases, at least, if waves arrive oblique to the structure, then they may be less stable than if the waves arrive normal to the structure. I think the work that has been done in part here at the Waterways Experiment Station has indicated that if the scales are large enough and there's some guidance there, that the capability of modeling the structure itself, that is the matrix or the body of the structure itself, is reasonably good.

Modeling the structural properties of the armor units themselves is desirable, but it's very difficult and expensive, and we'll probably have to pursue other approaches to that.

The second general topic is understanding the mode of failure or damage. To avoid breakage by impact with other units. One would have to invoke a "no-rocking" rule of the individual units. We saw some slides that showed abrasion by rocks, which will be difficult to eliminate if those types of "missiles" are present. It may be possible to use more-resistant concrete. Imperfections in casting can also lead to failure and damage.

I'm not sure whether liquefaction has ever been really responsible for breakwater failures. I think the pressure gradients on a breakwater can certainly contribute to their damage or failure. Liquefaction as a phenomenon would probably not happen in a breakwater due simply to the general design and porous nature of the breakwater itself.

Some time ago, several of us looked at the Sines breakwater failure. One of the strong possibilities for the failure was that if there is a fairly steep slope and armor units which are, by virtue of their size and geometry, reasonably fragile, then the armor units on the bottom, to some degree, must resist the weight of the overlying armor units. Possibly this is even more enhanced if there's a parapet wall which reflects the waves and, perhaps, to some degree, lubricates the face on which the dolosse armor units rest. If this does happen, then it's easy to see that if the lower unit breaks, and again if the base on which the units rest is somewhat lubricated, the upper units are going to slide down, leading to a progressive type failure.

Unraveling is another possibility. Of course, we attempt to design the breakwater so that it will be, to some degree at least, self-healing for a limited number or limited degree of unraveling.

The third area I will cover relates to quality control in field testing. The only way that I could ever feel comfortable with the quality of the units that are produced is to break or stress to some proof limits a certain percentage of the units. I'm not sure what kind of a system should be used, but possibly a yoke with hydraulic jacks which exert force against portions of the dolos, whatever type of armor unit would be used. It seems there could be an adaptive testing program where one might start off to break one percent of the armor units, shown as the solid black line. If it was found that a small fraction of those tested units break below the proof limit or the design limit, then perhaps one could relax the numbers that are tested, at least for a while. I think there should be randomly selected periods when a certain number of the units are tested to a greater percentage. If, on the other hand, it's found that when one starts out testing one percent of them that a fairly large number breaks, then I think that the percentage tested should remain high. In a sense, that would act as an incentive to reduce the number which actually failed below the proof limit.

Finally, I would like to relate results of a model test that was done a number of years ago here at Vicksburg. I would like to fall back on saying that breakwaters are to some extent like beaches, and I think we engineers do not respect that resemblance. A number of years ago on the Atlantic Generating Station, the concrete outer units were 40-ton dolosse, the first underlayer was 16-ton rock and then 8-ton rock. This particular breakwater, in a two dimensional test, was purposely pushed to failure. It failed in a shape which is quite similar to a natural beach-concave upwards--and finally the breakwater stabilized in the 8-ton rock. It had build out a talus slope seaward of the original breakwater face.

That test always impressed me that when 40-ton dolosse were present, the outer layer was not stable, but after it had deformed and finally entered into the 8-ton rock layer, the profile became stable. I think that's even more impressive if we look at the weight ratio--40 tons versus 8 tons, that's a factor of five--and the shape factor, which the stability coefficient. I wouldn't argue about these numbers too much, but more or less a ratio of 8. So that gives a total stability ratio of 40, that is dolosse, to the 8-ton rock. Yet the profile did become stable for that condition.

I think that if one were to attempt to design to this concept, and there have been degrees of attempting to establish a more natural profile by perhaps building a berm seaward of the upper portion of the breakwater, one of course would have to look very carefully at the effect of tides and also what kinds of end conditions may exist at the terminus of the breakwater. I have looked at possibilities of designing cross section using a more natural profile, if you will, and the end conditions probably present the greatest problem.

Finally, I will mention that A. Torum, from Norway, is spending a year with us, and I mentioned to him yesterday that I was coming here. He told me that if he could make one request relative to prototype testing, it would be to measure the total forces and moments on a dolos. He sketched this out yesterday, so I thought I'd present it to you as well. I think that would represent a very challenging exercise but perhaps not impossible. For example, the petroleum industry has measured total forces and moments on an entire structure, so that is probably not too less difficult than would be the measurement of total forces and moments on an entire dolosse.

If there are any questions, I would be glad to try to answer them.

DISCUSSION OF "MODELING OF CONCRETE ARMOR UNITS"

DR. ROBERT G. DEAN

UNIDENTIFIED SPEAKER:

On your proposal for breaking certain percentage of dolosse to test them, would that be like a fatigue type of thing or just one stress?

DR. DEAN:

Well, that's a good question. I think that might depend on the type of wave climate in which the installation was located. For example, if there were many periods of high waves, such as Crescent City, I think one might look at fatigue. If it were in an area where perhaps an occasional hurricane were expected to be the cause of failure, then I think it would be more of a limits type test.

DR. McDOUGAL:

Since we don't have any strong theory to get us from environment to load, testing some of those on site is more a quality control or uniformity measure just to make sure you're not producing good units. So it's a quality control, and at the moment it's a little rough to relate whatever test levels you do your destructive or non-destructive tests at, to environmental loads, but at least it will insure a uniform product.

THE MEASUREMENT OF STRESSES IN MODEL DOLOS UNITS

R.D. Scott*, D.J. Turcke, P.Eng.**,

W.F. Baird, P.Eng.***, K.R. Hall, P.Eng.***

INTRODUCTION

Extensive damage has occurred in the armour layers of a number of breakwaters. In certain cases, the breakage of individual armour units has led to partial or complete failure of entire breakwaters. Their failures are often rapid and catastrophic which clearly demonstrates the inadequacy of current armour layer design procedures. A good design must incorporate both the hydraulic stability of the breakwater and its individual armour units as well as the structural integrity of the individual armour units and the breakwater armour layer.

Structural failures have occurred predominately in certain types of armour units, such as the Dolos or Tetrapod, but avoiding the use of these units does not eliminate the problems of armour layer design. These armour units, and others, offer other advantages in terms of hydraulic stability and in the economic use of materials, that units of lower stability, such as the rectangular block, do not have. An essential step in the safe and economic design of an armour layer is to design the individual armour units to resist the applied loadings. These loads are highly complex, ranging

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from simple static forces to short duration dynamic impulse forces. A variety of loadings may be experienced by the armour units in both hydraulic and non-hydraulic environments. Some concept of the nature and intensity of these forces must be obtained before one can achieve good armour unit design for structural strength.

In this study, a combination of physical and mathematical modelling was used to develop a procedure to determine the forces experienced by armour units. Physical modelling offers advantages such as the ability to conduct parametric studies and to collect large amounts of data at a relatively low cost. There are, however, disadvantages due to scale effects and difficulties in achieving complete similitude. Specifically, model Dolos armour units were instrumented with strain gauges, see Figure 1, and typical loadings determined from these strains. The instrumented units were subjected to a variety of load tests and ultimately were used in a model breakwater under simulated prototype wave attack.

MODELLING REQUIREMENTS

A first step in the modelling process was to look at scaling relationships and the types of model materials that would be suitable. This required the assessment of similarity relationships between prototype and model [1]. Dimensional analysis shows that for complete similarity the geometric scaling must be equivalent to the material scaling. However, as small deformations of the model units were expected, a certain level of strain distortion and differences in Poisson's ratio are permitted between the prototype concrete and the model material. As any strain measurement system used would have a limited resolution, it was necessary to choose a material that would give adequate strain levels in a small model

experiencing low load levels.

The general model material requirements that were identified are as follows:

(i) The material should have approximately the same density as concrete.

(ii) It must be linear, elastic, homogeneous and isotropic.

(iii) The material must have a strength that would produce minimum model strain levels in the order of 50 microstrains to ensure that the behaviour of the units could easily be monitored within the resolution of the instrumentation.

(iv) It must have reproducible mechanical properties.

(v) It must be easily cast and easily strain gauged.

A wide variety of materials were examined such as metals, cementitious materials and plastics. Concrete, itself, could be used except that under scales appropriate to model testing in wave basins the expected strain in concrete models would be too small to be accurately measured. Metals also do not have the appropriate combination of strength and density. However, there are a wide variety of thermosplastics and thermosetting plastics available, some of which met the criteria outlined above. Thermosetting plastics were found to be the best material for the following reasons:

1. Thermosetting plastics have a limited development of heat of polymerization which assures a homogeneous hardening process and results in a relatively consistent modulus throughout the material.

2. The relatively lower shinkage that occurs in epoxy compounds after casting results in a significant decrease in the internal stresses.

3. The density, elastic modulus and curing rate can be easily modified by adjusting the amount of hardener or adding an inert material (filler) dispersed homogeneously throughout the model unit.

The material chosen for this physical modelling study was a steel fibre reinforced epoxy which had a specific mass of approximately 2.0 and a modulus of elasticity of 5 GPa.

A large number of material tests were carried out on the epoxy, as per American Society for Testing of Materials (ASTM) specifications, in order to completely determine its mechanical properties and, hence, its suitability as a modelling material. Considerable experimentation was required to perfect mixing and casting techniques with the plastic. Tests undertaken included the standard compression, tension and flexure tests for plastics. In certain cases the specimens were strain gauged in order to assess the compatibility of the material and the strain gauges.

STRAIN MEASUREMENT

A Dolos model of an overall length of 110 mm was chosen, based on the size of wave basin available. Though the Dolos was selected, this method of stress/strain determination is by no means limited to this armour unit shape.

A critical component of the Dolos stress/strain measurement was the number, placement and selection of the strain gauges. An examination of the Dolos geometry shows that it may experience flexural, torsional, shear and axial loading conditions. These forces may be static, quasi-static or dynamic in time duration. Based on these and other requirements, and a

review of the current state-of-the-art in strain measurement, a suitable instrumentation system was developed. The strain gauge chosen was a constantan foil type in self-temperature-compensated form with a flexible, polymide backing and a 3 mm length. The constantan alloy has a high strain sensitivity which is relatively insensitive to strain level and temperature.

Six strain circuits were used to measure the deformations in the central portion of the armour unit. All the strain gauges were wired in full bridge circuits to maximum sensitivity and for temperature compensation purposes. This measurement system gave a complete indentification of all the strains at one location in the Dolos from which stress/strain distributions could then be extrapolated for the entire unit.

As moisture and abrasion can have a severe effect on strain gauge performance, an adequate protection system had to be developed. The system had to prevent damage to the gauges but not affect the deformation characteristics of the Dolos itself. In this project, the gauges were coated with Micro-Measurement M-Coat G, a polysulfide modified epoxy compound and M-Coat B, a solvent thinned nitryl rubber compound. These coatings did give excellent protection to the strain gauges; however, this is still an area of on-going development.

DATA ACQUISITION

The output voltage from the strain gauges was fed through a signal amplifier and either stored in analogue form on instrumentation recorders or was digitized and stored on a computer for subsequent analysis. The signals were conditioned by a combination of hardware and software filters. Tests have been conducted with data acquisition rates varying between 100

Hz and 100,000 Hz. The rate of sampling necessary to obtain good strain data is affected by the size of the model armour unit used and by the nature of the loading the unit is subjected to. Short duration dynamic loading events demand high data acquisition rates.

In any test conducted, the time history of all the strains on all units must be recorded simultaneously resulting in a large quantity of data. This data must be scanned and pre-processed in order to minimize numerical computations and to facilitate analysis.

MODEL STRESS/STRAIN DISTRIBUTIONS

Of primary importance in this procedure is the ability to take strain measurements obtained from a hydraulic model test and derive the stress/strain distributions throughout the armour unit. The Finite Element Method (F.E.M.) is used extensively at all stages in this work to compute stress distributions and to verify certain test results. The method is well established as a powerful tool capable of carrying out complex dynamic non-linear material and geometric analysis. The graphical display of analysis results permits rapid synthesis and verification of voluminous amounts of stress and displacement information.

The Dolos finite element model consisted of 246 20-noded isoparametric elements with 4110 degrees of freedom as shown in Figure 2. Graphical output for viewing results can show principal stress flow patterns throughout the continuum, for example Figure 3, or Von Mises contour stresses. Colour is used to identify the stress intensity.

The FEM technique was used to determine equivalent loadings that produced the measured strains in the armour unit and to verify results, see Figure 4, for simplified tests conducted on the armour units prior to

breakwater testing. In the analysis of the Dolos in a model breakwater, both the loads and the boundary conditions are highly variable as to location and intensity. Using numerical analyses simplified loads and boundary conditions were derived that gave strains at the Dolos mid-shank equivalent to those measured with the strain gauges. With the finite element model, stress/strain distributions throughout the entire Dolos could be determined from these simplified conditions.

SCALING TO PROTOTYPE

In order to develop an adequate design procedure, the scaling relationships between model and prototype must be fully known. The approach that provided the most satisfactory results was to first determine the model loads from the measured model strains by numerical analysis then scale these loads to prototype. Essentially, the instrumented unit is a type of "load cell".

To achieve complete similitude such that model strains can be related directly to prototype strains is difficult or impossible to achieve when model and prototype are of dissimilar materials. Such a similitude relationship would require that the model replicate all the fundamental material properties of the prototype. This condition would preclude the use of any material other than concrete.

By using the load cell analogy, one must accurately define the model's static and dynamic properties so that measured strains may be related to equivalent loads. These loads may then be appropriately scaled to the prototype desired. In this manner, the loads can then be applied to a prototype of any material, whether it is reinforced or not. This is analogous to standard construction design practice where one first

determines the loads a structure will experience then designs the structure to resist these loads.

TEST PROGRAM AND RESULTS

The tests reported herein [2,3] were conducted using a 110 mm Dolos, though larger armour units have been instrumented. Interest in this series of tests was primarily with the static and quasi-static forces experienced by armour units, though dynamic tests were done. Although, inter-unit impact and projectile collision forces are very important in assessing the strength of armour units, it was felt that wave force data should be collected prior to excessive unit motion.

An initial step in the testing procedure was to calibrate the instrumented Dolos by applying static point loads at several locations to place the unit under combinations of flexural, shear, torque and axial loads. The Dolosse were held in a restraining device during this process and the load was applied by means of a sensitive load cell. All the instrumented units using the epoxy modelling material exhibited linear load-strain curves. A typical test result is shown in Figure 5.

A series of tests were performed in which the instrumented Dolos was placed in an armour layer and the number of layers, armour placement density and slope of the layer were varied.

Short duration dynamic tests were conducted in a dry environment. These tests, as outlined in Figure 6, were done by dropping or rolling the Dolos onto one of its flukes or fluke ends. Figure 7 shows a typical response for such a test. These tests, when measured by high speed data acquistion equipment, have shown highly significant strains.

The final stage in the procedure was to measure strains in a model

breakwater subjected to simulated prototype wave attack. The instrumented Dolosse were subjected to a variety of forces ranging from static loads to quasi-static forces from fluid motion around the unit to hydrodynamic and inter-unit impact forces. The strain gauged units were placed randomly in the armour layer. Wave heights were initially low then gradually increased in increments until significant armour unit movement was observed.

Figure 8 shows an example of the output measured during a test conducted with regular waves. There was excellent correlation between the repeatability of the strain gauge signals and the wave period. The strain levels recorded exhibited a marked increase with an associated increase in wave height. Irregular wave tests were also run; a typical result is shown in Figure 9.

The instrumentation peformed well throughout all aspects of the test program and was not affected by the hydraulic environment of the breakwater nor by abrasion from collision between adjacent units. In addition, the lead wires did not impede the Dolos motion. Strain levels exceeding the rupture strain for concrete were recorded in high wave conditions where large armour unit motion was observed.

CONCLUSIONS AND CURRENT RESEARCH

It can be concluded that the strain gauging system used presents a viable means of determining strain levels and loads on a model armour unit subject to simulated prototype wave attack in a hydraulic flume. This method of determining prototype loads from model tests will enable the proper structural design of breakwater armour units. Using the design loads the optimum geometry of armour units, the size of the units and the need for reinforcement can be determined. The economics of alternative

means of strengthening armour units, such as by prestressing, could also be examined.

Current research is being directed towards: (1) increasing the sensitivity of the strain measurements and measuring the dynamic response of the instrumented armour units, (2) Determining the optimum number and placement of instrumented units, (3) Carrying out further material property tests on the epoxy used for modelling, (4) Investigating scale effects by using instrumented units of different sizes and materials and (5) Performing a statistical analysis of measured strain levels. In conclusion, it is believed that this work will lead to improved strain measurements and, ultimately, to a complete design procedure for armour units.

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2. Hall, K.R., Baird, W.F., Turcke, D.J. and Scott, R.D., "Design and Evaluation of an Instrumentation Package for Measuring Strain Levels in Model Dolos Units", Unpublished report prepared for Public Works Canada and the National Research Council of Canada, March 1984.

3. Hall, K.R., Baird, W.F., Turcke, D.J., "Structural Design Procedures for Concrete Armour Units", Proc. 19th Coastal Engineering Conference, Houston, Texas, 1984, pp. 2563-2579.

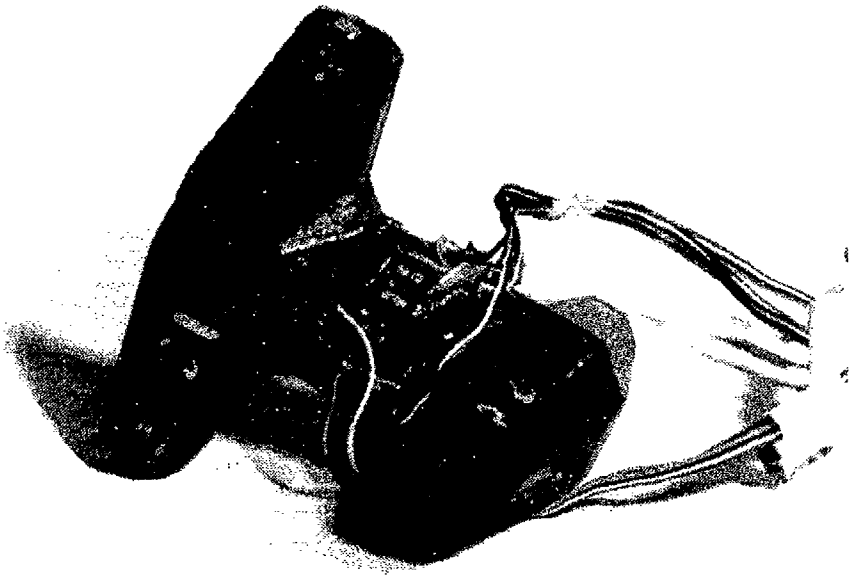


Fig. 1 - Instrumented Dolos Armour Unit

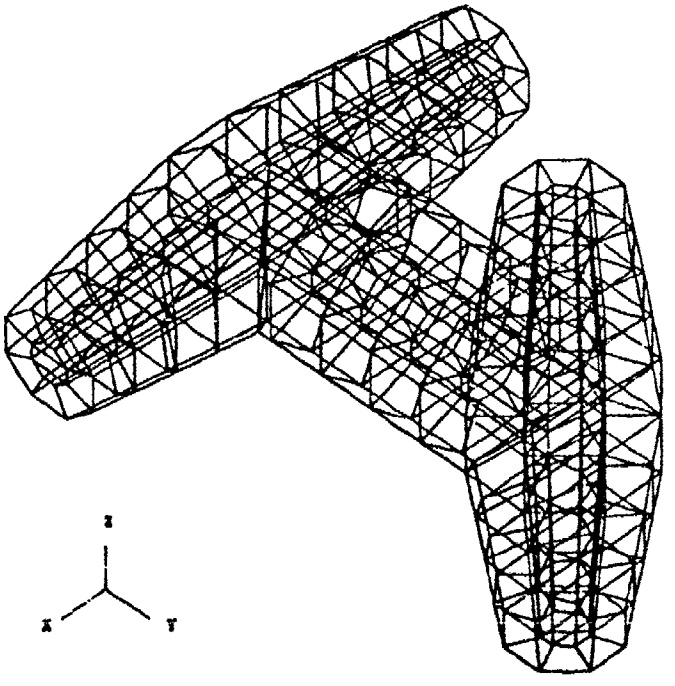


Fig. 2 - The Dolos Finite Element model

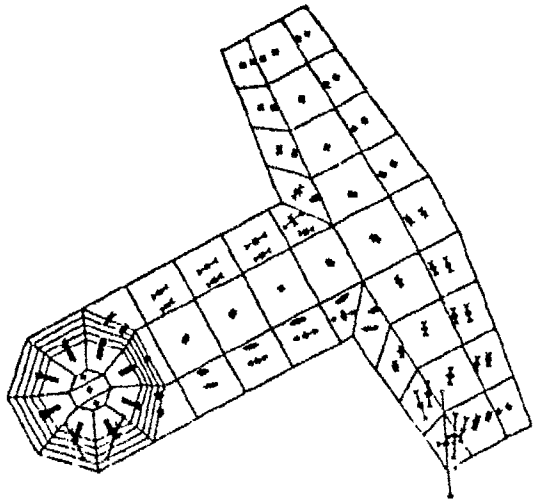


Fig. 3 - Principle Stress Flow under Self-weight Loading

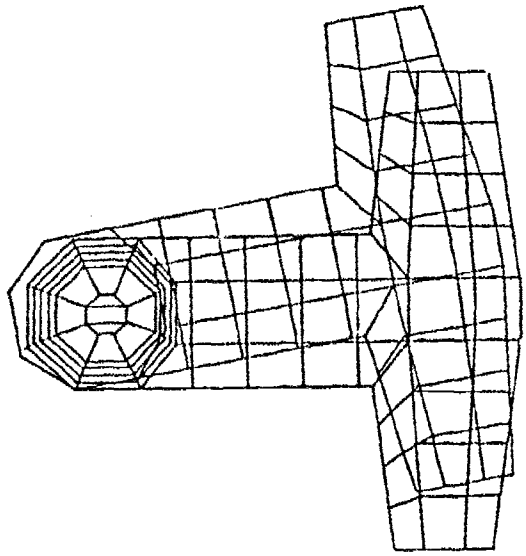


Fig. 4 - Drop Test Simulated with FEM

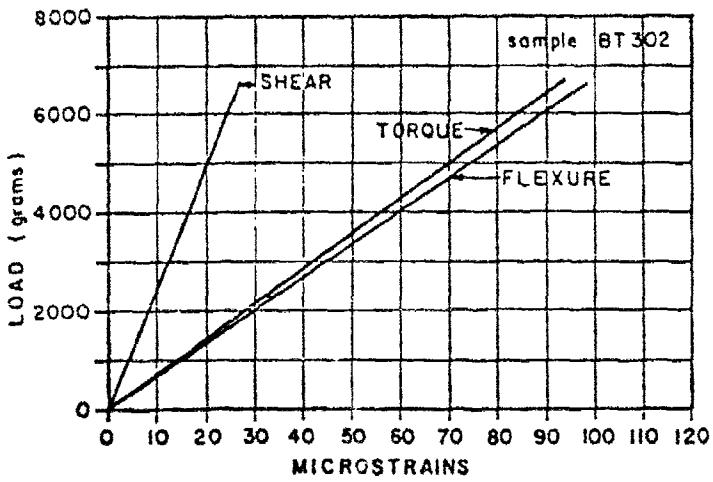


Fig. 5 - Load-Strain Plots for Static Loading

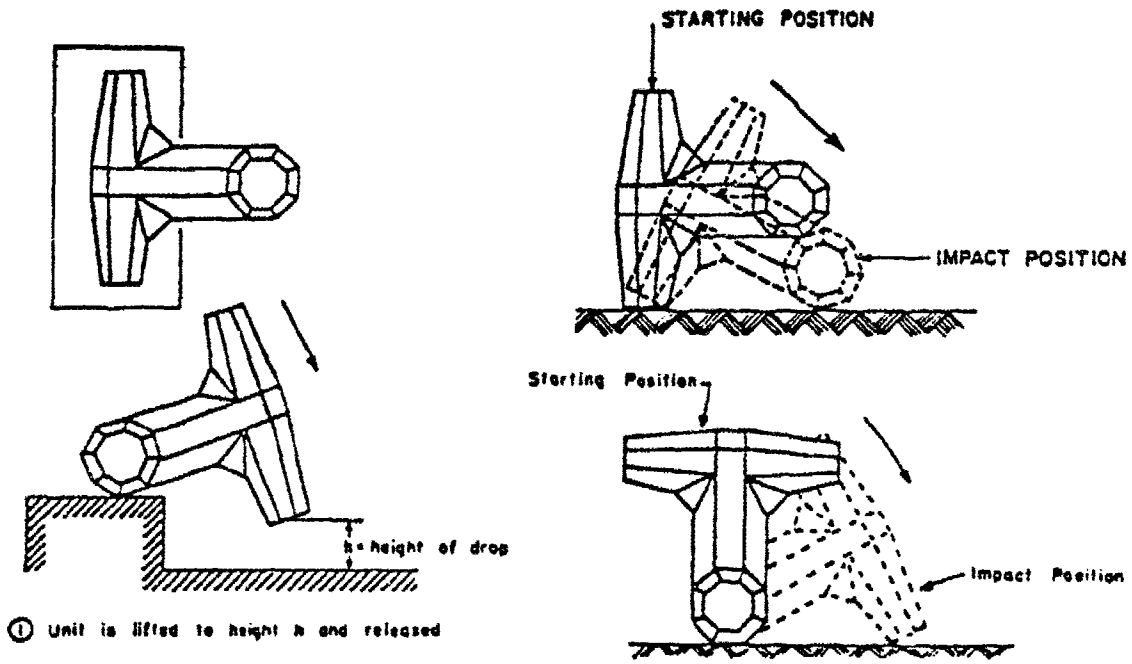


Fig. 6 - Dynamic Load Tests Performed

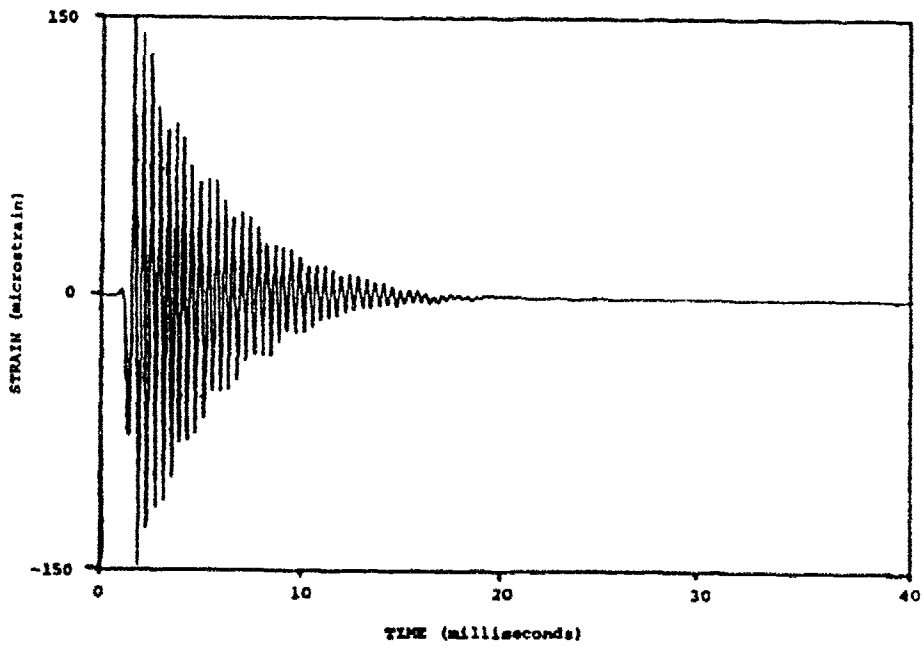


Fig. 7 - Dynamic Response for a Drop Test

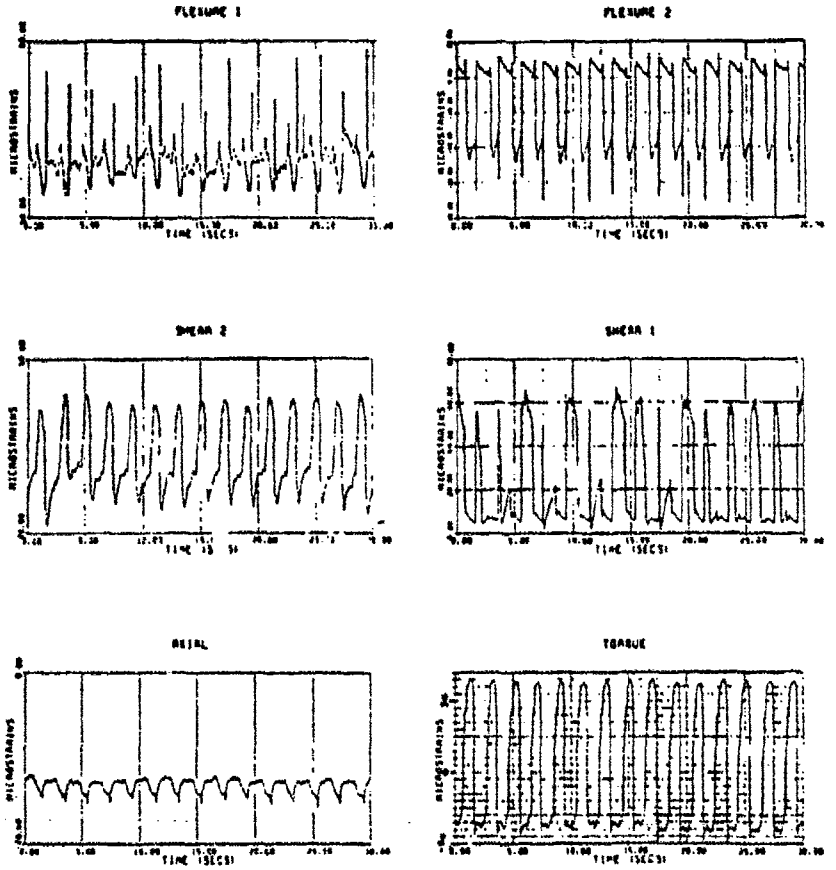


Fig. 8 - Strain Response for Regular Waves

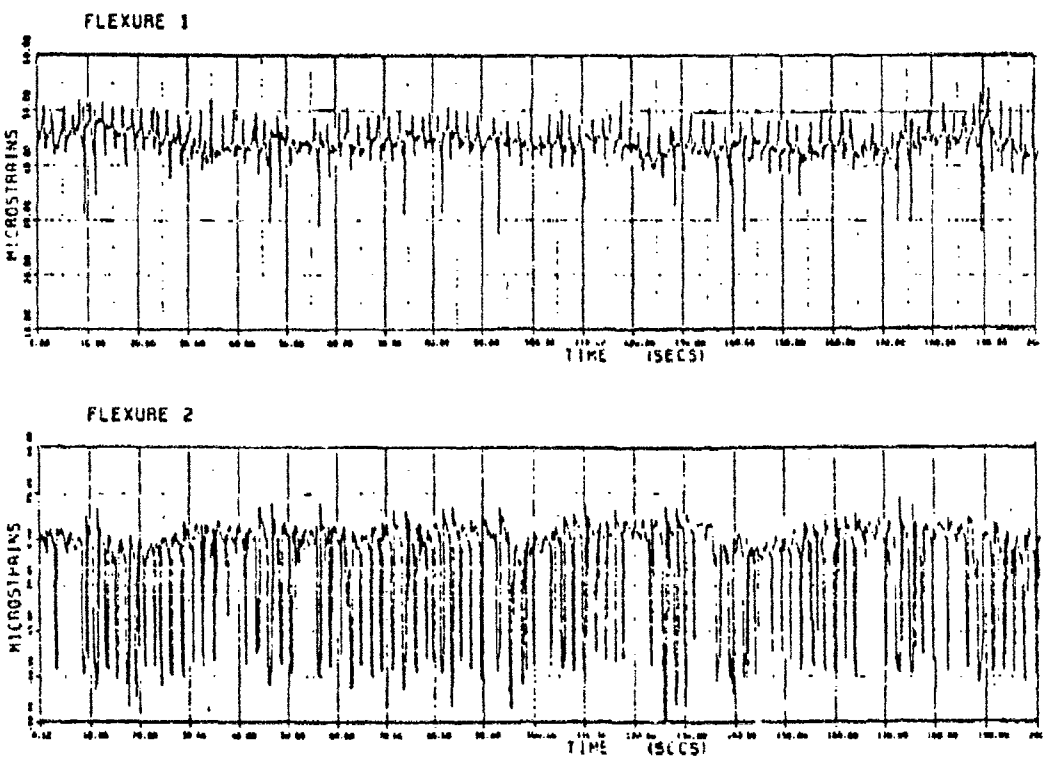


Fig. 9 - Strain Response for Irregular Waves

DISCUSSION OF "MODELING OF ARMOR UNITS"

MR. DOUGLAS SCOTT

MR. COLE:

How many strain gages do you have on the mid-line?

MR. SCOTT:

We have 24 gages total, six full bridge circuits, and one bridge measures axial. We have two shear bridges.

DR. McDOUGAL:

Two questions. One, Professor Dean brought up the effect of slope and stress, and you said you did those kinds of tests. How did you see the stresses go up as a function of the slope?

MR. SCOTT:

They did go up but how significantly is unknown. The tests aren't conclusive enough to really say. We noted the increase, especially , if the units were placed at the bottom of course.

DR. McDOUGAL:

The second question is you said you randomly placed units around the water level, water line. Presumably they had different orientations and contacts with the other units. How much difference in the response of those different units exposed to the same wave conditions did you see?

MR. SCOTT:

We had two layers of dolos units. In the units on the surface, we saw considerably more strain than the ones underneath.

DR. McDOUGAL:

Were the units in the top layer sort of independent of orientation relative to each other and to the wave?

MR. SCOTT:

Yes, there would be quite a bit of variability and particularly variability on the type of strain-for instance, which gages were recording the highest strain. Certain gages will be recording simple strain, while others a tortional strain. We found a lot of tortional strain, particularly once they were at the top of the armor layer.

UNIDENTIFIED SPEAKER:

With the assumption of linear behavior, aren't you limiting yourself as to results? You can't really predict the mode of failure if you're assuming linear behavior.

MR. SCOTT:

I guess that's something that should be elaborated more on. We're, in a sense, trying to use them as load cells. So we want linear properties for that reason. We don't want properties that are varying, are not linear, and are quite difficult to define.

We're trying to find out what simplified forces these units will see at models, scale those up to prototype, apply those to a prototype dolos unit, and look at stress distribution in the prototype. Based on that then you can make design decisions to equal reinforcement to--

UNIDENTIFIED SPEAKER:

Regardless of what forces apply to a prototype unit, the behavior for a linear and non-linear material is drastically different. Concrete is fairly reasonably linear up to cracking. Once you get tensile cracking, there's a non-linearity right there. That is a very real stress load.

MR. SCOTT:

That's if you want your dolos units to crack.

UNIDENTIFIED SPEAKER:

I think the problem is sort of complicated. You see so many cycles of loading area that if you want to design a good dolos, you're going to have to restrict yourself to the linear side and the rest of it suits the purpose.

MR. SCOTT:

That assumes you're not going to reinforce it. If you're going to say you're not going to reinforce the unit, it doesn't matter whether it's reinforced or not at the beginning. I mean if reinforcement is effective, you're going to crack the unit anyway.

UNIDENTIFIED SPEAKER:

With most traditional designs, even if reinforcing is strictly linear range, you don't have cracking. You can prevent cracking. All the standards.

MR. SCOTT:

In order to stress the steel, you have to crack the concrete.

DR. WALTON:

Could we save this particular point for discussion later? I would like to take a few more questions.

DR. LIGTERINGEN:

I think the development should show--static forces or quasi static forces which a unit may experience in the layer due to dead load and possibly due to the wave

force. The question I have is how you interpret the dynamic loadings which, in my opinion, are by far the most important in strength analysis of dolosse. The dynamic shock of the units you have, applied to the model, will impact completely differently from what happens in the prototype. So the strains which you measure are, in that respect, not relevant, in my opinion.

MR. SCOTT:

On what basis do you say that?

DR. LIGTERINGEN:

Because the impact phenomenon is different if you have one unit, as you have modeled it on another model unit, from what happens in prototype between concrete. The plastic failure at the interface in the prototype creates completely different impact phenomenon, impact momentum. The momentum is the same, but the maximum force in there, the maximum strengths which are developed from it, are different.

At Delft, we modeled dolosse with strain gages about five years ago, and we left that plan because we could not increase those dynamic stresses due to impact in the proper way. We could not find a solution.

DR. SOBEY:

To comment briefly, I didn't see any dynamic stress or strains in those time histories you showed. You went through them quickly, but it seemed nicely cycled between second harmonic and--but there didn't seem to be any impact loading. Also, for the particular case that I illustrated, I forget the wave we applied to it on that particular case, but the units were not moving all that much.

MR. MAGOON:

I think one of the questions we were asked to consider here was the best placement of units that are going to have 20 units or 20 bullets or whatever they're going to put out there. Given the limitation that we have, maybe one of the ways would be to in this type of model to try several tests of the unit configurations in something like Crescent City. Would that get a reasonable comparison? Assuming the absolute answer was not exactly accurate, would that be a reasonable way to go about testing, to go about trying to evaluate where to put 20 units?

MR. KENDALL:

I just was wondering isn't that something you listed at the end as part of your ongoing effort?

MR. SCOTT:

Yes, you are looking at prototype examples, using these models in the prototype situations where there have been failures, to see if we do record stress that exceed the failure in prototype.

MR. KENDALL:

Goes back to our old saying about optimizing the place in the models that tells you the most.

MR. SCOTT:

Yes, I think it would be an excellent means of establishing where we should place the prototype units.

"STRUCTURAL STRENGTH OF ARMOR UNITS"

DR. HAN LIGTERINGEN

Although I never start a presentation like this with a joke, some of you may think our solution not to use dolosse anymore is a joke. Although this solution may seem to be a bit too simple, on the other hand I have a feeling that in some parts of the world the dolosse or any other standard unit is too easily accepted as the best economic block that might be applied in breakwater engineering. I had this impression when talking about solutions we have found for several rehabilitations using cubes. People looked a little surprised to us and said, "Cubes? Really? Cubes? Those are traditional, old fashioned." Yes, traditional and old fashioned.

I think it was J. F. Agema. When Agema was designing the breakwater Europort in Rotterdam, he looked upon the economics of using tetrapods--other special cylinder type units, or cubes and concluded that, taking into account all aspects of fabrication, transportation, special care, which these cylinder units need, that you come up with the cube again as the most economic solution. So I don't think we should too easily discard it. On the other hand, we should not stop using dolosse altogether because in some cases it may have a good applicability.

In my introduction, I touch a little bit on all the aspects of these two days. In fact, I will be treating tests in models and prototypes. Computer modeling, and prediction of impact forces within the framework of implications for design and construction.

In the Delft Hydraulic Laboratory we analyzed a number of great failure cases from the Mediterranean and the Atlantic coasts: failures at Sines, Portugal; Gioia Tauro, Italy; Arzew, Algeria; Tripoli, Libya; and San Ciprian, Spain. Some of them use dolosse, some of them use tetrapods. In all cases we found, from the reconstruction of what happened, damage to the concrete armor units was an important contribution to the final failures. We used in those analyses correlation of the observation in prototype with the consecutive model investigations, and by doing so we improved the ideas on the critical limits of what you can expect with each of these types of units with respect to strength.

In the course of this analysis, we also came upon the gaps in the design procedures and the knowledge about concrete armor strength, and therefore we began a research group of more or less consisting of the same people aided with people from the Concrete Research Institute in the Netherlands and contractors. We felt that the contractors were important in this respect because the methods of construction are an important part of the whole process.

In the first phase of this joint research, we analyzed more in depth these failure cases. We came up with the state of the art in the concrete mechanics and in concrete technology, and we came up with a definition of priorities as far as further research is concerned. Now, elements of this research project are integrated into my further presentation, whereas Mr. Heijdra will go into some of the aspects of further research as we see them and which we would like to discuss with you.

The first main conclusion from these damage investigations were that stress is due to 1) settlement after construction and 2) rocking during storms. Rocking starts, in all cases which we investigated, at significant wave heights in the order of three to four meters, well below design levels as far as the hydraulic design of these breakwaters was concerned.

The second main conclusion was that temperature stresses during curing may introduce micro cracks, not only in cylinder blocks, but also in the larger, bulky types as the cubes.

The third conclusion is that either with cracking or without cracking, stresses which are left in the concrete due to curing, shrinkage, and settlement, lower the effective tensile strength in the units. What is left there to be actually taken up due to wave loads or due to an impact is a little smaller than five Newton per square millimeter.

The fourth conclusion is that breakage of units leads to progressive damage, which is something we should take into account in the interpretation of model test results where this breakage does not occur and, therefore, the results, as far as damage concerns, are conservative.

From these analyses we came up with global indications of the weight of specific units above which the structural strength becomes important: the dolos, beyond above 10 to 15 tons; tetrapods, at about 30 to 40 tons; and cubes, above 50 tons. It's a very limited conclusion.

If I look back to the damage to the two-ton dolos in the Cleveland breakwater as discussed and presented by Mr. Pope two years ago in Washington, we should be very careful with these crude figures. Still another conclusion of these investigations was that great uncertainties exist regarding the actual loads, the stresses and the effective stress.

Yet the designer cannot wait until all these questions have been finalized; therefore, I will discuss the implications for design and construction today with our present knowledge and experience. I do that taking into account that we have to include safety margins, and I don't say that we have a standard procedure at the moment. I'm very well aware that what WES is after--further development of a design procedure--is something which has to be worked upon. I will show you how we try to include elements to conclude aspects of concrete strengths to the best of our knowledge. I will do that for the different design phases.

A designer likes to compare a number of alternatives, often at the very global, superficial level in order not to limit himself too fast to a certain type of unit. Already in this stage it's important to take the concrete strength into account because, if you don't, you'll end up with finding a dolos or tribar as the most economic alternative, economic in breakage in this case. Now, as far as hydraulic damage is concerned, we have something like a Hudson's formula or Iribarren formula to make these evaluations. We don't have such nicely grouped empirical data as far as the structural strength is concerned. Yet, there's something we have. From a number of a prototype experiments, we have a global indication of the impact velocity which might lead to breakage. Under very specific conditions, that is a full test, for instance, of a unit on a horizontal plate, different from what probably will be experienced in the prototype in the breakwater slope itself. It gives an indication.

This graph indicates for dolosse the impact velocity which is allowable for a first hit or first impact to limit breakage as function of the weight. It indicates that for that whole range of units we have some indications that, an impact velocity of between 1 and 1 1/2 meters per second is the limit which these blocks may undergo on the breakwater.

Similar results have been obtained for tetrapods, which indicate that to use the same parameter, the impact velocity, for tetrapods, four to five meters per second would be the limit, whereas cubes show velocities in the order of five to six meters per second.

Another piece of information which is valuable in this stage of the design is the information we got from the model tests. The model tests also mean the large scale tests we did on the delta flume (Holland), where you see here a section of the emergency repair for the west breakwater of Sines in which two things are important. The white blocks, which you see in the middle here, were blocks where we tried to reproduce the concrete strength. It was not completely successful, as Dr. da Silva mentioned, but in any case it was good enough to get a good reproduction of the impact of these blocks when rocking one against the other.

The second important aspect is the two cables, which you see here, leading to accelerometers in two of the blocks placed around to water level, accelerations measured in three horizontal directions and providing us an indication of the level of the velocity's impact, which was on the order of one meter per second.

The third important aspect, which Mr. Heijdra will go into, is the correlations which we got between the Sines technique, measurement of rocking elements, and measurement of the overall pattern of motions. From these accelerometers, we came to the conclusion there was a good correlation among the three. That would mean that once you have established that correlation, you don't have to continue the acceleration measurements. You can stick to a simpler type of measurements and still have an idea of the level of impact velocities.

I will briefly go into four elements of detail design: motion behavior, strength analysis, the load determination, and finally the overall failure assessment.

As far as the motion behavior is concerned, the model studies for a specific design give us an indication of the level of accelerations of impact velocities, and integrating that with data on the spatial distribution of rocking, we can come up with an overall analysis as presented this morning by Dr. da Silva. If we look to the strength analysis, then, this is an example of how we measured, by means of strain gages, the forces on the tetrapod in the past. We have left this method of measurement because we have problems in interpreting the impact accelerations.

As for the actual effectiveness strengths of the units, at the moment we have available a mathematical model which, for a cube, computes the temperature development during curing, given a certain concrete mix, and, from that calculates by a finite element model the stresses in the concrete during curing, again as a function of the characteristics of the mix and the ambient temperature. The model has not been extended to be used for cylinder type units. That is part of that ongoing research.

These are the specifications for this particular case to 25 kilogram per cubic meters of low heat cement, 25 degrees starting temperature, and an ambient

temperature of 15 degrees. Now, this type of calculation may give us a first indication of what is the actual effective strength which is left in the particular unit. It's not sufficient because of the limitations which are, for instance, that we cannot at this moment take this relaxation completely into account. It's only approximate. Therefore, at the design stage I think we still have to rely on load measurements in the prototype. With load measurements, I again refer in particular to the impact loads. From our experience, we can say that static loads or quasi dynamic loads, the loads due to the wave in the up rush or down rush, are lower, lower by a factor of five to the impact loads which may be experienced by the blocks due to rocking.

I do not say that you should not look at all into those earlier loads, but this one, in any case, has something to be taken into account. When looking to the loads, we have this problem of interpreting what actually happens either in the model or in the prototype during a full test. In the model we attempt to have an indication of the impact momentum since we have the velocity and we have the mass of the block. We still don't have, from either the model or the prototype, a good indication of what is that side factor and what is the impact time, the side factor indicating he form of the impact and the time which is the actual time that the force is felt by the unit.

Burcharth has been studying this problem, and we're planning to continue that for other types of units, but it's a complicated matter in view of the fact that it's not an elastic impact. It's a plastic behavior at the contact surface and, therefore, we find at this moment that impact times, fortunately, are much larger than what the elastic theory would predict.

Ideally, in the design stage for a breakwater we would do prototype loading tests at full scale. If I show a cube as it has been tested as Sines, that is not an example of an ideal situation because in this case the emergency repair was already going on while these tests were being done. The same applies to the tests which were discussed this morning by Dr. da Silva. These also were impact tests which were done at the same time that the construction was going on.

The result of these tests was a nice crack. I have another example of prototype tests which were done during the design phase, and that is for the europort breakwater in Rotterdam, about 20 years back. We had cubes there, and we were very careful to look into their dynamic properties, but the cubes were investigated by means of fall tests prior to the construction. That 40-ton cube failed, and another failed after a certain fall height.

Now, this just gives you the picture of the elements, in fact the ingredients, we have in detail design at this moment. Model studies, complications of the effective stress and strengths of the units, load assessment from the hydraulic model, and finally, ideally, the load measurements at full scale in prototype.

Finally, I would like to say a few things about tender specification and supervision. Again, in the tender specifications, the designer should now take into account concrete strengths aspects if we are dealing with units which are in critical zone in that respect. That could be done in the definition of the concrete mix design, which could be done by specifying the fabrication methods, and it should be done, in our opinion, by concrete testing and full scale dynamic tests.

Regarding the first two, we have reservations in going too far in specifying the concrete mix and the fabrication methods. Just because it limits the number of pontractors which might be able to tender, it also eliminates the possibility of giving

the contractor his own, let's say, ingenuity in arriving at an optimum strength of those cubes or those units. So if we look here to a work which is just started, which was already discussed this morning. It's the east breakwater for the new port of Sines. It's here that my firm has taken the approach that we should specify only in general terms what is needed as far as the concrete strength is concerned, and ask the contractor to come up with a proposed mix and have a final acceptance of that on the basis of prototype tests and temperature measurements of fabricated blocks. So, in fact, we allow here a discussion with the contractor to come to an optimum design, which, I agree takes some time. On the other hand, I think it's most worthwhile in order to arrive at an optimum solution.

Just to show you briefly how that cross section looks: it's a very mild slope again. Breakwater is in 25 meters of water depth, significant wave heights for design in the order of 12 meters, and, therefore, even with this model slope, we still have 60-ton units, cubes, which should withstand the waves.

Then, finally, as far as supervision and monitoring is concerned, I will present a few aspects related to concrete strengths, underwater inspection, quality control, and monitoring after construction.

Supervision means that full tests which are specified in this process of coming up with a final mix should in any case be very carefully analyzed. As far as the construction is concerned, which starts after that, we think that it's mandatory that diver inspection of the breakwater, during construction, takes place. It's too often that, due to the rough conditions and all other kinds of practicalities, divers are only now and then allowed to look at what happens. I think it's very important, in those cases where the concrete strength is a problem or may be a problem, that the units are placed by the aid of divers and supervised by divers.

Of course, underwater video may help here, underwater video. I think in that respect the original construction of the Sines breakwater had very good inspection.

Finally, one should watch a too regular placement. That's the other side of the picture, especially with concrete cubes. The process of positioning is good, is diver assisted, then the contractor tends to end up with a neatly placed layer which is not good at all from the point of view of porosity. A warning there.

Then, finally, the monitoring after construction. You know that beyond working on constructed breakwaters all over the world, surveys resulted in replies from different countries on about 150 structures, and there was a lot of information on the design and it was lot of information on construction, but there was very little instruction on what actually happened to the breakwater afterwards. That's also the reason why one of the recommendations of that working group is that monitoring after construction is being more regularly implemented either by the government or by a port authority. It takes a little bit of money, compared to the overall construction costs, and that requirement should really be spent. In that respect, I think we should compliment the Waterways Experiment Station with their attempts to monitor this breakwater at Crescent City, although you cannot call it actually monitoring after construction. In any case, it's an exercise which will give us good information on what actually happens in the prototype.

Similar ideas are being considered by Belgium at the moment on their Seurbourg breakwater. Again, we should look to those results because we have then a set for dolosse and we have a set for cubes.

As far as the construction of the east breakwater in Sines is concerned, we will certainly propose a monitoring program to the client. Whether we will be able to come to a level of monitoring as it's envisioned here at Crescent City, I doubt. On the other hand, taking into account my earlier remark that there is a correlation between acceleration or velocities and, say, displacements or settlements, we might come up in the future also with easier monitoring methods on the basis of the knowledge we gain in this type of prototype experiments.

DISCUSSION OF "STRUCTURAL STRENGTH OF ARMOR UNITS"

DR. HAN LIGTERINGEN

DR. McDOUGAL:

Would you comment on instrumented hydraulic model tests in the light of instrumented prototype units?

DR. LIGTERINGEN:

I think our main problem in having model units instrumented is that the impact phenomenon is not well produced. As far as that is concerned, the prototype is a great advantage because there the impact phenomenon is very well reproduced. So, looking to prototype instrumentation, I think our preference for acceleration measurements is relaxed. Having strain gages in the prototype units has, in any case, the advantage that you not only measure stresses due to motions but also stresses due to dead loads and weight. So there's an advantage there. On the other hand, I like very much the combined approach which we have been seeing this morning of having both the accelerations and the strain gages.

DR. SOBEY:

You commented in the previous discussion that Mr. Scott's approach representing the elastic modeling material wasn't satisfactory. What, in addition, do you need to model? Is it plastic behavior, the rate of strain, or--

DR. LIGTERINGEN:

Yes, the rate of strain is very important. What we did in those tests in the delta flume is not only modeling the models of elastic but modeling the rate of strain, the number of other parameters. I think I have some of those data with me. If you are interested, I can give them to you. They are also reported, I think, in the Washington conference of 1983 of coastal researchers. So the only thing we were not able to achieve in those experiments was the abrasiveness. The model concrete was a little bit too soft and we ended up, after a test storm of 12 hours, with nicely rounded balls instead of the worn out cubes you spoke of. That's why we were not completely satisfied with this and so our investigations into finding solutions for a model concrete, let's put it like that, are continuing on the scales which we can achieve in the delta flume.

I think Dr. Heijdra will go into that a little bit more in detail.

DR. GALVIN:

I would like to comment on this interest in the impact. It seems to me that in the natural conditions, the static tests are fairly applicable. The reason I say this is the typical wave periods you get in nature are on the order of 8, 10, 12 seconds, and the rise time in a breaking wave, even on a breaking wave, is probably not less than a tenth of the wave period. So you're talking about times on the order of one second, whereas I think the dynamic effects that most structural engineers are worrying about are on the times of millisecond or something. Perhaps Mr. Cole or somebody could comment on that.

DR. LIGTERINGEN:

Yes.

DR. GALVIN:

So, there are two ways that the impacts are important. The impact of the wave on the armor unit and the impact of one armor unit on another.

DR. LIGTERINGEN:

I'm talking about level.

DR. GALVIN:

As long as you're not talking about all those flying missiles. Talking about armor units moving under water, there's a very large inertia in each of these units and, in addition, they've got the fluid forces, the drag forces, on the units which prevent them, I believe from moving very fast.

DR. LIGTERINGEN:

I don't think so. As I showed, the acceleration measurements on the cubes in the model gave us maximum integrated accelerations in the order of one to two meters per second. We have found maximum velocities on tetrapods which are moving in the order of two to three meters per second, and I think that the same applies to dolosse. So, I'm talking then about two phenomena.

The first is the rocking of these units in situ, small movements around their mean position. The second is the impacts of the unit which is hydraulically, by the waves, moved upwards or downwards over any distance longer than its characteristic dimension, and hits then, while finding its new place, hits then on its neighboring units. I'm thinking in both cases the velocities upon impacts are appreciable and, in our evaluations, creates far larger forces than, even that hydraulic impact of an impacting wave on a fixed unit.

That is exactly one of the points which I summarized in my conclusions. It's this under-estimation of what the magnitude of the movements in the layer is. Hydraulic engineers have been looking upon a breakwater as a static construction. Now, if you look to the single frame movies of a breakwater slope, then you see that around the design wave height it's a ground motion, it's in continuous motion.

DR. GALVIN:

I'm not denying that. It's just how fast is that motion?

DR. LIGTERINGEN:

Yes. Well, pretty fast.

DR. GALVIN:

Is it fast enough to be considered dynamic motion from the structural point of view?

DR. LIGTERINGEN:

Yes. As soon as it is a motion, you get an impact, colliding, two units, one against the other, and it becomes a dynamic problem even though the impact velocities may not be large enough to create breakage. It's a dynamic problem and, I think, should be looked upon as a dynamic problem.

DR. ZWAMBORN:

I would like to come back to the question of whether you can model the impact structure. I think what we're really looking at are reflective characteristics on the model material and possibly a crushing characteristic of concrete which is up to scale to get to correct deduction in the model before you can get the corrective production in the model. I don't know whether you looked at those aspects when you developed the materials. It's a material problem. You said in Delft you tried this and you came to the conclusion that it's not feasible.

DR. LIGTERINGEN:

Well, we haven't found the final solutions. I don't say it's not feasible but--

DR. ZWAMBORN:

This morning you said it's not possible. I think it's possible provided you can develop that sort of material.

DR. LIGTERINGEN:

Okay. Well, that's right. As soon as we can come up with that solution, we may go back to the strain gage track. Yes, I agree.

MR. SCOTT:

I would like to make a comment. We're not really looking at the forces that are involved in a dolos in lifting up and flipping over the breakwater. We're concerned with just the small motions.

DR. LIGTERINGEN:

The small motions give large impact velocities.

MR. SCOTT:

I question that.

DR. LIGTERINGEN:

I will show you the results.

DR. ZWAMBORN:

If the material characteristics are different, you have a slight yield. When the impact force may be a fifth or a tenth of impact forces without it, that makes enormous difference. If you are working on the base of stresses and loads, then you

may have a problem. I am glad you mentioned there's light on the horizon in terms of the correlation of movement. Again, the techniques of measuring movements on the order of magnitude are easier than when you have to instrument units. I also think it's interesting more and more people are writing papers about that now. One or two or three of ten units, it's still an enormous evaluation of breakwater which may have 20,000 or 30,000 units.

DR. LIGTERINGEN:

I agree, and in that respect, in fact, I had a question to Gary Howell this morning whether aerial photography is envisioned as far as the Crescent City breakwater. Because that would give an idea of spatial arrangements which could be correlated to impact velocities or whatever.

MR. HOWELL:

Yes.

DR. DEAN:

I was wondering whether these impact forces that cannot be modeled would be larger in the model or smaller in the model than they are in the prototype. Because even that is valuable information.

DR. LIGTERINGEN:

I think the impact forces are larger.

DR. DEAN:

In the model?

DR. LIGTERINGEN:

The impact time is much more short in the model.

DR. DEAN:

So the forces would be larger?

DR. LIGTERINGEN:

The forces are larger.

DR. DEAN:

Even that is valuable information.

DR. LIGTERINGEN:

That is true.

DR. DEAN:

That can tell you quite a bit about the prototype. It gives you an upper limit.

DR. LIGTERINGEN:

Yes.

"MOVEMENT AND DESIGN FORCES; DETERMINATION AND PREDICTION BY SCALE MODELS"

DR. G. HEIJDRA

DR. HEIJDRA:

I want to present our experience in movement and how to detect the damage in hydraulic models.

My presentation is titled "Movement and Design Forces for Armor Units; Determination and Prediction by Scale Models." We have already looked at predictions. I will emphasize the measurements techniques we are using at the Delft laboratories. I will spend a few minutes discussing the prediction methods for design forces. (Slides shown here).

I first want to talk about the hydraulic damage. We distinguish between hydraulic damage and rocking damage. Hydraulic damage is classified as the damage in which the units have greater movement than the diameter of the unit itself. Rocking damage is classified as the movement for the unit which is smaller than the diameter.

You have several opportunities to measure the hydraulic damage. First, you have visual observation. Second, you have the figure and movie opportunity. Third, you have the high speed camera technique, and fourth, you have the color photographs taken after each test run. In fact, I have mentioned some of the positive items or negative items for each measuring technique for these observations. For instance, you have a good global impression of what happens during a test run.

You see the total cross section or total, say, position of a half of the structure. On the other hand, you cannot do that for the whole test run, so you are restricted in time. I think it's more important because you have no records about it so you lose your information quite quickly. Overall, I think looking at the model tests very often a very important measuring technique.

Field techniques and the movie support part of visual observation, but it is restricted in the area because you can't have the overall part of the total construction.

With the high speed camera, you can have the opportunities of measuring the movements and the velocities of units rocked. It's a global instrumentation, a global possibility, but it gives you the idea. You can also have an idea about the load scheme, but you are restricted in time. I will show in the film some of the high speed shots.

Color photographs provide the measurements we're using in Delft quite often. They offer a total view of the cross section or total view of the head or elbow of the construction of the breakwater, giving the opportunity to see what happens with movements of the units more than one diameter. You can count the number of elements displaced, so after the tests you have an indication of how many units are moved away.

It's difficult to take such color photographs. I will show you on the next slides. They are taken from the head of the Sines breakwater, one of the largest models we have had in our laboratory. You must imagine that this is the dimension on the root of the breakwater of about six meters and it is height of 80 centimeters, so the area is very, very large.

Each unit represents in nature an armor unit of 90 ton. So you can imagine, I thought about 10 thousand units were placed on the head of the breakwater. You can imagine that color photographs are a restrictive measurement device for indicating the damage, but I think on the next slides you can see what happens.

You see quite a lot of damage. It occurs with the significant wave height of about 10 meters. Here you see what happens when you have significant wave height of about 40 meters.

We can get information about which unit has moved and also we get information about the rate of movements when we look at the photograph closely. It's a good method, especially when we combine it with acceleration measurements where we have the opportunities to correlate the movements on one side and the accelerations and impact velocities on the other hand. Together these give a good opportunity to assess the impact velocity and, from that, the forces on the units.

So the overlay technique has several advantages and several disadvantages. First, the advantages. It gives displacements of the total test run but also of the total area. It gives the slightest displacement of our visible. That does mean that when we have displacements of about one or two millimeters, we can watch the overlay photograph. So it gives us a good opportunity to see what happens about the movement.

The problem is that we can take pictures only after some time of the test run. So we don't know if it is settlements or one movement or rocking. I think what we are doing is that that we stop during a test run, several times, and we compare. Say, when we have a test run of about 12 hours in nature, then we stopped after four hours, after eight hours, and after 12 hours. When we compare the displacement after four hours and compare displacement after eight hours, we have an indication if units moved for the first time in the first four hours the same as displacement of units in the other four hours. So that gives an indication if we have to do settlement or rocking checks.

Another disadvantage of this technique occurs when we have too much hydraulic damage. Then the overlay technique is giving a good idea of the displacement itself because it's influenced by the blocks displaced to another area.

When compared with the Sines technique, we have the opportunity to see the difference between settlement and rocking. On the other hand, we have only a very restricted area. The second disadvantage is that we have an idea of what happens only near and above the water level because the technique is that we have a spot on an area where there's no water movement. So we can't see on the film what happens beneath the water line.

Our experience is that on very steep slopes there are problems because of the wave trough. The next wave comes soon, and we can only take a picture when we're in a wave trough, before the other waves come on the breakwater slope.

Another method we have is to measure the accelerations, or the bending moments. We measure the bending moment during the test run. We can do that in two ways. We can fix the model elements so we have an indication of the wave forces on the typical date, or we can use a free unit and get some information about the rocking phenomenon. We want an indication of the results to have an idea of what happened as a result of the wave or impact forces. We want to have an indication of the actual damage we can expect on the cross section of the breakwater. (Graph shown here).

On the horizontal axis you see significant wave height. On the vertical axis you have the rate of seriousness of the damage. It gives an indication of the percentage of units moved. It's not only the percentage of units moved on a cross section, but we also see what happens on a specific spot. For instance, when we have three or four percent damage throughout the cross section of the breakwater, it's not as dangerous as when units move from a specific spot. So from the classification model, we decide that for more than three or four displaced units, it has the classification moderate.

The diagram shows not only hydraulic damage, but we also want to know what happens when a certain impact velocity has increased. Here is the diagram for an impact velocity of three meters per second. This gives us the opportunity to, for one significant wave height, see what happens due to hydraulic and rocking damage.

When you see the acceleration measurements and strain gages for measuring the bending moments, you will see the advantage of this measurement technique is that it gives an indication of the velocity. Only an indication. There is some disadvantage in that it restricts in time and space because there is only one, two or three units for measuring the second. That is the disadvantage that was previously discussed: the characteristic of the collision is different. Other disadvantages are the scale for different measurements and the fact that it measures only one wave direction because the accelerometer. The units are too small to measure more.

The strain gages for measuring the bending movements have the same advantages and disadvantages. An additional disadvantage is that it is located in a predetermined cross section and we can use strain gages only in slender armor units and not in units like cubes.

(Slide shown here.) This cross section shows several locations of units where we measure acceleration. This shows our three units equipped with accelerators, and it shows you that only the middle one here by the water level has impact. It gives a good indication of the effects of impacts due to the wave forces. It shows the impact of rocking. It gives an idea of magnitude and what can give. It answers a question asked during Ligteringen's presentation. It shows the time of impact is much shorter than the time of impact due to the wave acting on an armor unit.

DR. McDOUGAL:

Is that wave plotted to the same time scale as that impact load?

DR. HEIJDRA:

Yes. It's the same scale.

DR. DEAN:

Those are actual measurements?

DR. HEIJDRA:

Yes, influence due to the collision is not on scale.

DR. HEIJDRA:

We can overcome the problem when the frequency of the accelerometer is much lower than the frequency of the impact. Then we have a good indication of the pulse. When we have a good indication of the pulse, we can translate to the real force acting in prototype, when we have indication of parameters. So the shape parameters. These two formations together give an indication of the impact force acting on breakwater armor unit.

Now I want to compare rocking and hydraulic damage. (Series of slides shown here.) On the horizontal axis you see the removed units to another color band or the rate of rocking and, on the other hand, the seriousness of rocking divided by the number of physical elements. Here is a very good agreement of these two kinds of damage. So when we have high hydraulic damage, we can expect that a lot of other elements are rocking.

So when we have a model test on this measurement technique with color photographs, we get an indication of what happens with units that are rocking. It's difficult to say that it's the same for all kinds of units. These results are from cubes, but I think it gives a good indication of what can happen.

I have another comparison of several measurement techniques. Here, on the horizontal access, you see acceleration measurements. The first class of the acceleration measurements shows movement of units during five percent of the incoming waves. For the second class, up to 30 percent of the units moved and, in class three, more than 30 percent of the units moved.

The Sines technique is more or less the same. Visual observation is difficult because you have your own impression and you have to compare that with the other results. When you see the results compared--six of the total of about 11, so more than 50 percent at the same classification visual observation and acceleration measurement.

The same applies for the Sines technique and the acceleration measurements because we have to add up these (reference to slide) for the total combination. The overlay technique and acceleration measurement is more or less the same. About 60, 70 percent give the same classification. Only the Sines technique and the overlay technique show problems. Only 30, 40 percent give you the same classification.

Due to the steep slope of the breakwater, the Sines technique gives some problems by analyzing the shocks.

I'm going to make some comments on breakable units because you are now seeing only the units that can't break. These efforts to make units that can break gives us quite a lot of information, especially information on progressive failure. Progressive failure occurs when an armor unit moves from one place to another and collides with the resting unit, which can break, too.

The problem is what kind of material to take. That collision characteristic is very difficult to scale. When we have the material, then we have material that we can use on only one scale. Because, especially with dynamic elastic models, we find different data for each material. So when we translate from concrete to the material in the model, we can use that material for only one scale.

The third difficulty is getting a good indication of the actual tensile strength or effective strength tensile. Dr. Ligteringen talked about the indication of shrinking and so on. The National Research Council in Canada has performed some tests with breakwater armor units that can break, but each of the units is good for only one test. When it breaks, it is thrown away. It's fairly costly.

I will make a few comments on the study Dr. Ligteringen presented. I have with me some descriptions of the study performed in 1983, and if you are interested you may have one (see Appendix A for description). The study aims to look for design rules for structural strength in a level that is equal to the hydraulic design level. Now we have only Hudson's formula to calculate the weight of the unit. Maybe there are some additional modified equations or diagrams. We developed some diagrams for stability of elements. I think that our aim for the study is to develop some design rules for the structural strength at the same level.

The study we carried out in 1983 was on loading, strength of concrete armor units, and concrete technology. I want to show you some of the results on loading. (Series of slides shown here). This shows the relation we found for the breakwater at Gioia Tauro, Italy, where prototype test situations reveal the damage. It is presented here in two sections, one and two: eight percent, 31 percent of the dolosse units at Gioia Tauro are equipped with 50-ton units. After a storm, a storm with signature significant wave height of about seven meters, the same as the design storm for that breakwater, eight percent of the units break for a typical section and, for another section, 31 percent of the units break.

DR. GALVIN:

That was the wave height?

DR. HEIJDRA:

The wave height was 7 meters. That was also the design significant wave height. They measured in the waveward of the storm, so what they have done in the model test is that we have the same wave data in our flume when we perform the test.

This is the hydraulic damage: keep in mind as determined by the color photographs: 2 percent, 12 percent.

Then rocking damage, which we try to analyze by Sines technique, by overlay technique. It gives in total, the six percent or 22 percent that you can have in the model. That is different from the damage in prototype, and it gives the indication that the measuring technique used in models does not provide the last answer you can give to the client.

The damage in the model increased somehow due to the effective unbreakable units, and we have done it for another breakwater for ourselves. When I say "some how", I mean that you can get some more damage up to about 28 percent in this example but not the real damage prototype. Maybe the problem is that it's very random and it's typical situation and so on.

Here is the example of Gioia Tauro. You will also see it in the film. That is the reason I present the data here.

DR. ZWAMBORN:

Is that the actual measured rocking?

DR. HEIJDRA:

It's the actual.

DR. ZWAMBORN:

So you haven't allowed anything for underwater?

DR. HEIJDRA:

This is only for the stages of near the water level and above, and we predict in the way the damage for rocking for the part underwater.

DR. LIGTERINGEN:

Estimated overall rocking of it.

DR. HEIJDRA:

Yes. The second part of the study dealing with the structural strength came out of the following figure. It gives the impression of the kind of calculation methods you can use for calculating the physical load due to impacts to get an indication if it breaks or not.

On the vertical axis, you see the critical loading. The horizontal axis gives the relative length of the crack. When you have vehicle mechanics, you have a horizontal line. Above the line it's unsafe, under the line it's safe. You can have a modified structural mechanics by this line where you have a reduction of the area, the cross section, due to the cracks.

The second graph gives an indication of the fracture mechanics. Up to a certain level, the structural mechanics gives you enough information to design the structural armor units due to the impact loads. Only when there is a very large crack on the outside do we have to use the fracture mechanics. I think that could be important.

There's only one problem here, and it is the same. We don't get an indication of the shape of the impacts in prototype. What we intend to do for the next program is to combine these two things, mechanical loading by measuring acceleration in hydraulic model and measuring the load time relationship of concrete. We have to do the latter in two scales. First, we want to have to test it on a larger scale to see if there are any scale effects. They expect that there are not scale effects from crushing of the concrete. When we have the load time relationship, we can get an indication of the shape of the impact, and with that figure we can analyze mechanical loading in the modeling.

The second point is that we want to calculate the actual strength of the concrete which is in the identical model. I think that the measurements show this morning by (another speaker). Offer a good possibility to check the calculations with the same results as what actually was measured.

In the future, we will also want to devote some more attention to the concrete technology and to get some design methods to design concrete elements related to the impact loading we could expect for breakwater armor units.

I would like to show the film now because it gives an impression of what can happen.

This film is about the breakwater at Gioia Tauro. I told you a few minutes ago that the Gioia Tauro breakwater was designed in 1970, using sixty tonne cubes. It collapsed in 1977 due to a flood wave of about eight meters caused by a land slide just in front of the breakwater. When designed again, the specific wave height was about eight meters. They used 50-ton dolosse armor units. The film spends some time on the measurements of that construction of Gioia Tauro. Thank You.

FILM NARRATION:

On September the 31st, 1979, and January the 1st, 1980, part of the breakwater at Gioia Tauro, in the southwestern part of Italy, was damaged during a storm. A large number of dolosse were broken by the waves. In order to observe what has happened during the storm, a small-scale model investigation was performed at the Delft Hydraulics Laboratory.

Part of the breakwater was constructed in a 50-meter-long and 1-meter-wide wave flume at a length scale of 1 on 45.

The dolosse had several colors in order to detect displacements. On December the 31st, at 20:00 hours, the first movements occured. A high speed camera was used in order to show the movements as they would have occurred in nature. At 23:00 hours the significant wave height had reached a value of 6.5 meters. A large number of dolosse were rocking at this moment.

In order to detect the rocking of dolosse, single-frame film exposures were made each time the water surface passed a selected level. By comparing two successive exposures even the smallest movements can be observed. The behavior of this part of the slope during the storm was as follows: At 16:00 hours on December 31st, at 18:00 hours, at 21:00 hours, 22:00 hours, at 23:00 hours, at 0:00 hours on January the 1st, the top of the storm.

Then the intensity of the storm decreased; consequently the rocking of the dolosse decreased.

At the top of the storm the significant wave height was 7.1 meters.

This figure shows the total number of dolosse displaced during the storm in a section of 45-meters length. It can be seen that the waves were breaking just on the breakwater slope. When the distance between the breaking point of the waves and the breakwater was increased, breaking waves no longer hit the structure, which resulted in much less damage. From the tests, it was concluded therefore that the shape of the foreshore was one of the main reasons for the large amount of damage. Therefore, a new design was tested in the wave flume and modified in order to get a structure able to resist wave attack as occurred during the storm of January 1st, 1980.

DISCUSSION OF "MOVEMENT AND DESIGN FORCES; DETERMINATION AND PREDICTION BY SCALE MODELS"

DR. G. HEIJDRA

DR. ZWAMBORN:

One figure you showed was the comparison of what I could call displaced units. I think you called it hydraulic damage and the rocking units, and that compared very, very well. Was that based on a flume test?

DR. HEIJDRA:

Yes, it was on a flume test.

DR. ZWAMBORN:

I would like to sound a warning that that is not generally applicable. I think you mentioned that, but I like to stress it because we used exactly the same measuring techniques in an extensive study we did. On the breakwater trunk, we found about 50 percent displacements and 50 percent rocking, compared very well. On the head, however, this was completely different.

MR. MAGOON:

I think that the mechanism at the head of the breakwater is completely different.

DR. ZWAMBORN:

I would like to stress that people should not go away from here with the idea you can equate it.

DR. HEIJDRA:

I think that is a good comment because what happens on the trunk is completely different also from the hydraulic point. I also think the formula of Hudson gives good information for the forces acting on the trunk, but only by multiplying. He felt the coefficient gives an indication of the weight on the head of the breakwater. It has no relation between what physically happens on the head of the breakwater and the formula you are using. I think that is the same comment you have for the rocking phenomenon that you have on the trunk compared to what happens on the head of the breakwater.

MR. LILLEVANG:

Was it the same wave train that was striking the thing through all of these hours that the man was commenting on and were they monochromatic?

DR. HEIJDRA:

No, sir. Varying waves.

MR. LILLEVANG:

Spectrum-type thing?

DR. HEIJDRA:

Yes.

MR. LILLEVANG:

Have you determined the reasons for the duration of the apparent stability preceding the motion and then finally damage--without a change in the waves?

DR. LIGTERINGEN:

There was an increase in wave height. These were different steps of wave heights.

MR. LILLEVANG:

Ah, well, that's what I asked.

DR. LIGTERINGEN:

No, it was not.

MR. LILLEVANG:

If it was the same wave all the way.

DR. LIGTERINGEN:

No, it was not. There was an increase in wave height. It actually simulated the hydrograph that occurred in the prototype. So with increasing wave height, we saw this increase, this starting of the rocking.

DR. HEIJDRA:

So they measured actually--you are talking about the film, aren't you?

MR. LILLEVANG:

Yes.

DR. HEIJDRA:

They measured in prototype with a wave board, on 70-meters deep water.

MR. LILLEVANG:

I'm glad that has been corrected because it was my impression that you had the same wave train hitting it throughout.

DR. HEIJDRA:

No, no.

DR. WHALIN:

It was a spectral wave. I don't know if the shape changed or not.

DR. HEIJDRA:

The spectrum wave in nature acted the same as what we have performed in--

MR. LILLEVANG:

Well, there were some of the waves in that situation which appeared to have virtually a double breaking effect, and we couldn't see enough of it to get an idea of what that phenomenon was. Was it something of a longer period moving at a higher speed catching up with a shorter period while in the spectrum so we had a doubling up of attack at a critical point? What happened there?

DR. HEIJDRA:

I think the most influencing phenomenon is the foreshore itself, because near what we have seen just in front of the trunk, there was a nick in the foreshore itself. On that nick, the wave breaks, and I showed you on the sheet the difference in damage on those sections. One section showed less damage and one showed greater damage. The greater damage was caused by the crushing breakers due to the nick on the foreshore.

DR. LIGTERINGEN:

The effect which you saw in the movie was indeed a short smaller-period wave which breaks and by being--

MR. LILLEVANG:

Caught up with it.

DR. LIGTERINGEN:

Caught up with longer wave, that is true.

MR. LILLEVANG:

There's a firm that has patented a procedure for doing that to create waves in surfing beaches for recreation--in the middle of Kansas--where they propagate a series of waves of progressively longer period with minimum input of energy to the wave machine. They design it to break at just the critical point where the surfers want to

ride a wave. These waves build up, and they break as one hellish great wave. Great fun is had by all at minimal cost.

DR. McDOUGAL:

We model their idea.

DR. LIGTERINGEN:

In fact, it once more illustrates the importance of spectral testing because it was those combinations of waves which created most the damage.

DR. ZWAMBORN:

I want to ask about the accuracy of the measurements of velocities. You integrate the accelerations, I presume. I think you did stress it was only approximate, very approximate. Can you give the reason for that? Is that because it's such a small scale or the basis of the technique?

DR. HEIJDRA:

The problem here in this model test was that the period of the accelerometer was in the same value of the period or the frequency of the impact, so it gives--

DR. ZWAMBORN:

You mean the resonancy frequency of the--

DR. LIGTERINGEN:

Was nearly the same.

DR. HEIJDRA:

We have now some accelerometers which check different time and different frequency, but the measurements we did earlier were the same value, and then we have some problems because it's increasing and we have oscillation.

DR. ZWAMBORN:

Does this apply to your data numbering measurements in Sines?

DR. HEIJDRA:

No, it doesn't apply for Sines.

DR. ZWAMBORN:

So what's the accuracy of that measurement then? Is it plus or minus one meter per second if you're--

DR. HEIJDRA:

I can't give you percentage, but I think it's not very far to mention it because you have no comparison with the actual failure that can happen. The only thing is when we want to measure with accelerometers that the natural frequency of the measurement devices is an order different from the impact time.

DR. LIGTERINGEN:

May I add to that that in all these experiments it's important to have the parallel to the actual flume experiments, the calibration of these units, instrument units, just under control conditions so you know at what height you let them fall on the plates, and you correlate those records with the ones you see in the prototype or in the models.

MR. MAGOON:

Maybe one observation. I think Dr. Galvin commented that the velocities of the units might be relatively small under water, and in the film you showed, it looked like there seemed to be a difference in the below water and above water. The units seem to be moving relatively slowly under water. Did I miss something, or is that a real effect?

DR. HEIJDRA:

No, in what we measured with the acceleration measurements, there was no difference in velocity of the impacts under water or above the water level.

DR. SOBEY:

This is under the trough or under the mean water level when you say "under water"?

DR. HEIJDRA:

Under water. So when you have a unit and the unit is impacted under water. That may be under the wave trough and the velocity is the same.

DR. ZWAMBORN:

Can we just get this clear? Are you talking about units sitting above water and units sitting below water? Do they give about the same? Because the question you put is really the units which are under water at the time act--

DR. HEIJDRA:

Yes.

DR. ZWAMBORN:

--with natural results. I think there are two different--

DR. HEIJDRA:

I think you have to compare impact results, and that's very difficult to compare. Statistically we found no difference in the impact velocity above water and below water.

DR. ZWAMBORN:

That could be when the trough is right down and the dolos itself or the unit is above water at that moment.

DR. HEIJDRA:

Yes.

DR. ZWAMBORN:

Above the actual water level.

DR. HEIJDRA:

Or below the actual water level.

DR. ZWAMBORN:

Because my experience, too, is that the units being under water at a particular time, the wave may be here, and this one is also under water--

DR. HEIJDRA:

Yes.

DR. ZWAMBORN:

--the movements are different, looks smaller. Looks smaller.

DR. HEIJDRA:

When we looked at the statistical registrations, we didn't find a difference.

DR. ZWAMBORN:

You would have to relate the actual instrumentation measurements with the actual water level at the time if you want to answer your question.

MR. MAGOON:

Just look at the ones at the very bottom of the flume that were rotating differently. I guess it's the deceleration you're worried about when it hits some object. So that the velocity was lower down at the bottom of the flume and right after that. That's what I was trying to observe.

"BREAKWATERS AND BREAKING WAVES"

DR. CYRIL GALVIN

DR. GALVIN:

If you remember, Dr. Ligteringen began his discussion with the statement that the way to solve the dolosse problem is to not use dolosse. The results of what I'm going to talk about today will only slightly modify that. Don't use dolosse when the wave heights exceed 20 feet or 6 meters. I would like you to think about whether or not that would have been good advise in your own personal experience. If you remember Dr. Whalin's opening remark this morning, he mentioned that one of the instigations for WES's research into this subject was three structures in the Bahamas with three different weight armor units. The two smaller ones were not damaged, as I understand it, and the larger one was damaged. Is that correct?

DR. WHALIN:

During placement, and they were on the same structure. On a floating plant, not due to waves. They were tribars.

DR. GALVIN:

I said armor units.

DR. WHALIN:

I don't think I said this morning, but they were tribars.

DR. GALVIN:

Okay. Well, the last speaker showed damage to some dolosse structures. The critical damage occurred when the wave height was 7.1 meters as I remember.

I would like to read the abstracts of the paper I'm referring to, at least the first paragraph.

(Reading) Based on the analysis of this paper, unreinforced dolosse at the water line with typical rubblemound structure are expected to break when wave heights exceed 14 for 3,000 psi concrete, to 20 feet for 6,000 psi concrete. For metric equivalents, 4.3 to 6.1 height meters. These critical conditions are independent of the size and the weight of the dolosse. Design dolosse weights at these wave heights were one on two slopes range from only 3 to 8 tons.

By the way, this discussion was presented in New York in May of 1981, and there is a preprint in print on it.

The basic idea is that waves exert a relatively moderate force when they're breaking on an armor unit cross section but that these moderate forces are resisted at point contacts with other units in the structure, and so the moderate force divided by a relatively small unit at one point of contact can result in critical stresses.

There are two kinds of critical stress: either critical compressive strength stresses, which would lead to abrasion, or critical tensile stresses, which could lead to failure in bending.

If we take generalized armor units where the wave impacts on a well draining breakwater and then perhaps look at just two of those generalized armor units, you can see what I mean. The breaking wave acts on the individual units over some equivalent area, say a star, to produce a force over the total unit. That force must be resisted somewhere within the units at some point contact. The point contact necessarily would cover a much smaller area, and that area could easily exceed the compressive strength of the concrete.

For example, if you have such a point contact, with a sphere, you would necessarily crush at the point. You would necessarily crush the concrete and abrade it away until the area was large enough to resist the force without the concrete crushing.

Likewise, if you have a typical dolos on the structure, rather just the generalized unit, there can exist cases where the wave will impact on the broad side of one dolos and be resisted at a point contact on the fluke of another dolos. In that case the critical section becomes some part of the fluke of the resisting dolos and it is possible to exceed the allowable stresses of the concrete and break it by bending. As has been brought out, the concrete is very weak in tensile strength. If you work this through, all of the factors involving that breaking--the moment inertia, the size of the inside radius of that section, or the size--they all depend, for the typical dolos that we've been talking about. These dolosse have dimensions which are all proportional to the length C, from one end of the fluke to the other.

The size of the dolos drops out and you're left with simply a result which depends only on the compressive strength of the concrete--to be exact, on the square root of the compressive strength of the concrete.

The point I would like to make today is that this is a testable prediction for any tests of dolos installation. Also, to the extent that it's born out by facts in the field, it suggests that really the dolos, as we know it, with the typical weight ratio, should be limited to use for conditions where the design wave height is below a certain size. That size would be dependent on the compressive strength of the concrete.

DR. da SILVA:

I would like to make a comment. Maybe I did not understand very well what you said. By the fact that the area is small would mean that the compressive stress would be higher. It should not mean the force would be higher.

Besides, there is another point. When the unit moves with a certin acceleration, I believe the acceleration is not in general very small, but you can say there are inertia forces also in calibration with the forces generated at the contact point.

DR. GALVIN:

It seems that it may not be as simple as I have drawn here. If a wave is going to impact on the broad fluke of the dolos, it must be resisted by force on this surface here, must be resisted somewhere within the structure, and the only way it's resisted is on point-to-point contact with other units.

DR. da SILVA:

You say it's abrasion or crushing in that area.

DR. GALVIN:

Okay, abrasion is one thing, but the other thing is the bending moment around this axis here.

DR. LIGTERINGEN:

Yes, that is correct.

DR. GALVIN:

So those are two different factors.

DR. LIGTERINGEN:

Yes.

DR. da SILVA:

Well, maybe I did not understand correctly. It seems that the force itself would be higher, the force is up higher, in the bending moment but strength is the same no matter if the area of contact is this big or larger.

DR. GALVIN:

Yes, I understand. It is a magnification of the force. Because of the large area here, you are receiving, say, a moderate force from the breaking wave on this large area so that the moderate pressure from this wave on this large area and so that concentrates down to--must be resisted at several small points.

MR. LILLEVANG:

There are intermediate losses. That's a massive piece and you don't take that of course into great account on the fluke and transfer it entirely over to the contact with the next dolos.

DR. da SILVA:

That was the second part of the first part, the second, because when it moves, it moves with sudden acceleration that--

DR. GALVIN:

It does not have to move. If it's a rigid structure, it will transmit that force.

DR. LIGTERINGEN.

Yes, it doesn't move. Dr. Galvin is looking upon the situation with no rocking so the unit which is impacted by the wave is steadily supported by the surrounding units, and what he is saying is that in looking to a direct wave load you should not only take into account the wave load on A, the shank or on a shaft, but also on the forces which are transmitted by units to a fluke. I think that is true. When I commented in my presentation that forces due to impacts of rocking units are larger than forces due to wave loads, then I must say we have to take into account this loading situation. Still we come to the conclusion that impacting of units gives higher forces on units.

DR. GALVIN:

If that's the case, then the conditions are even worse than the conclusion that I reached.

DR. LIGTERINGEN:

Yes. Right.

MR. SCOTT:

When you started out you said that you presented a relationship between the wave height and the dolos unit, but you must have used some sort of calibration in there because I think later on you said that the size of the dolos was proportional to the square of the strength of the concrete. I don't see how you got the calibration coefficient in there without dealing with the experience. Am I wrong on that?

DR. GALVIN:

There are two coefficients that enter into the analysis. The first of those coefficients is your judgment on what fraction of the cross sectional area is the affected area resisting the wave. We took 0.7. I don't know, it looks reasonable to me.

There is a second factor, and that is the relation between the critical tensile stress of the concrete and the compressive stress. We took a factor of five. Now, we took that out of design manuals which give it a range between four and seven. We figured the fact that the concrete sits in salt water and is subject to various forces that we have been talking about this morning, we maybe take the lower part of that range, but we took five, and the handbooks give from, I think, four up to seven.

DR. LIGTERINGEN:

Ours is even higher.

MR. SCOTT:

I guess I have two questions. You said initially that this point contact could be abraded down. Wouldn't that result in failures?

DR. GALVIN:

There are two questions. I perhaps shouldn't have brought up the abrasion part. he abrasion is independent. I will say on that just if you take a section of a dolos, a section of a dolos is an octagon. Here. Now, if you abrade off the corners of that octagon and come up with an inside cylinder, you have automatically weakened the strength of that dolos by ten percent. That's just because you have changed the moment of inertia of that section.

MR. MAGOON:

Could I ask one question? On that abrasion that you're talking about, what effect does that have on the impact that we've been discussing here? In other words, if it's a dynamic case and these two concrete bodies come together, what does that do to the actual transmission of the loading or the rate of change through the unit when the unit actually crushes?

DR. GALVIN:

Well, I think it makes it less instantaneous, I believe, but I don't know.

DR. LIGTERINGEN:

Lengthens the impact time.

DR. GALVIN:

So if you go by the impulse dimensions, you're lengthening Delta T, so you'd be reducing the force.

MR. MAGOON:

Any idea how much that might be?

DR. LIGTERINGEN:

A factor of three to five, according to our calculations, which is considerable. I mean it lengthens the impact time of five milleseconds, which you calculate on the basis of elastic theory.

DR. GALVIN:

Again, the basic analysis I'm going with is not moving structures. It's fixed structures.

DESIGN METHODS

AND

STRUCTURAL MODELING OF

ARMOR UNITS

GENERAL REMARKS ON "STRESSES IN DOLOS"

MR. OMAR J. LILLEVANG

Conventional assumptions regarding stress distribution within the dolos are probably not valid. Traditionally we have visualized the deflections under load, we've magnified what we visualized in sketches in order to make first cut, determined how the member might best be shaped, and then tested it mathematically for adequacy to survive whatever loads we decide are appropriate to apply so it can survive.

I began wrestling with this problem concerning the dolos in 1968 when I saw my first ones on a trip to South Africa that was made for the purpose of investigating the shape and use. With its constantly varying modulus along its fluke, going through what fracture specialists might see as a stress multiplying notch at the crotch between the fluke and the shank, and then a constant cross section or section modulus until the process is reversed but in a plain turn 90 degrees, suggests to me there must be some peculiar distributions of stress within this shape that were three dimensional. I considered what one might do then in a conventional two dimensional analysis in taking incremental steps, going away from the central access of the shape, down across the 45 degree inclination at the side with section modulus changing by infinitesimal increments of distance but in something other than infinitesimal amounts in the response to the thing to loading. It struck me that trying to design this with any kind of a forced adaptation to the way we usually do things in two dimensions might be an idle exercise at best and a dangerous one at worst.

I obtained a mold made out of a casting resin molder or casting rubber and cast several soft rubber dolosse in the thing and began playing with them. I have one in my briefcase. I meant to come up and play with it, but that would be like Captain Quigg with his roller bearings perhaps. The distortions in the thing are dramatically better illustrated in soft rubber than by sketching. I began to wonder about the location of these concentrations. We ought to expect them at sharp changes in direction, but which sharp changes in direction are most alarming and need dealing with? So I went out in my garage and I mixed up some patching plaster in a thin cream consistency and brushed it on with a little glue brush. It hardened quickly and I got a brittle coat that way. Then I flexed it again and, behold, here were the lines of stress at the surface developing on the thing. Very crude, very simple, but certainly a good way to wiggle into a problem of some magnitude.

Convinced now that it was a three dimensional problem, I then undertook to acquire photoelastic analyses in three dimensions of this strange shape, the dolos, and went to a firm in Malvern, Pennsylvania, that made a good case for long experience in doing stress analysis photoelastically in three dimensions by the system that, in their proprietary terminology, they called "freeze stress" or "stress freeze". They loaded the model pieces in an oven, brought it up to a temperature that was still within the elasticity range of the plastic, but not beyond it, sustained the load then while they cooled it over a period of about two days, brought it down to room temperature. By this technique they were able to freeze the stresses into the three dimensional shape and then slice it into thin slices which would be polished and treated as the classic manner of taking plastic sheeting and cutting out a profile and looking at it photoelastically.

Something like 58 different slices were cut from four versions of the dolosse. The classic shape, as developed by Merrifield with sharp intersection at the crotch between the fluke and the shank, another with a very slight easing in that crotch with a fillet that was almost not visible to the eye at the scale of these pieces. They had an H value, or height value, or a length value, or a C value--I've heard all kinds of terms used for the overall dimensions of dolosse here in the last day--molded in this very, very small radius fillet. They tested it that way and found a remarkable easing of critical stresses at the notch of the corners.

Then we took the chamfer that Orville Magoon sponsored, if he didn't create it, the one used at Humboldt the first time in the world that I'm aware of, and copied all over the world ever since. We tried that and found that that chamfer also was immensely beneficial, much better than this very tiny fillet. We heard critical comments by the stress analysts saying it still has abrupt changes in direction and those need to be eased as a matter of good stress distribution. They proposed a larger radius fillet, and we tested one that had a radius roughly equal to the offset of the chamfer.

We tested these pieces then first with surface stress techniques that they had of the full shaped uncut piece and learned which loading seemed to create the highest concentrations of stress at the surface. This, after all, is where cracks begin, normally, and we found that two types of loading represented the most severe probable stress concentration in the shape in any of the four versions. One was what they called a "tension." They simply took and loaded a tensile pull on the dolos; another one they put a torque on, so-called "torsion." Out of that we then got traverses of stress, both surface and at concentrations, and transected through the thing, at depth, indicating a very surprisingly rapid loss of internal stress as we left the surface. The more severe the crotch intersection was, the more rapidly that stress reduced as we went in, with the eased ones, and coming to an integrated hold with a whole load on the thing. Of course the transect showed a less steep gradient, but internally in the piece, at depths at least where reinforcing bars would be put if that--what the concept of strengthening the piece, the tensile stresses had gotten down to very nominal if not negligible values.

The results of the tests were prepared for a client, Public Service Electric and Gas Company of New Jersey, which at that time was planning the Atlantic generating station, of late fame, and the result was a text report with samples of illustrations from the test data; the appendix, which was a graphic presentation of all the test data, 58 I think it was; and the reports as submitted to me by Photoelastic.

I don't know whether D. D. Davidson or others have copies of these in their archives. Whether they do or not, if it would be of value to WES to have another set, I would be glad to leave these with you.

The results were presented in perhaps a more compact form and discussed from the standpoint of the designer's concerns in a paper presented at the Coastal Engineering Conference in Honolulu in July, 1976. I have a preprint of that (see Appendix B for paper). It was of course published in the transactions of that conference, but being expensive and usually only in corporate or large government agency libraries, I think the paper was not widely read beyond the population of people who normally get the Coastal Engineering Conference proceedings. This is a preprint of it, and you may photocopy it for distribution.

This workshop's purpose is to share views from individual perspectives on "measurements and analysis of structural response in concrete armor units." The immediate concerns of the conveners clearly relate to the full-scale instrumentation of the dolosse at Crescent City. Out of long-time habit, we keep referring to the design wave. I deeply hope that the investigators will be able to determine the wave characteristics that relate to each stress record that is acquired, but I'm pessimistic on several grounds. One, my hopes for survival of the data cable are high, but my expectations are low. No matter how ingenious we devise a system for the cables, they have a reputation for short longevity, and, Murphy's Law being what it is, they usually fail just when we need them. Secondly, the terrain of the sea floor, over which stress-inducing waves will approach and be altered by the sea floor before they wash over the test dolosse, is contorted. It has abrupt rises, sharp pinnacles, submerged escarpments, and the locations and shapes of them are imperfectly known. For every wave period and every azimuth, and varying momentarily as tide stages change, there'll be different wave conditions each fifty feet or more, or even less, along the breakwater's crest. Draw-down of receding waves, aggravated by reflection, will be varying by station to station, along the structure and they may be a critical factor in the stability of the armor. Present plans for wave recording will leave more disappointments than successes when the real nature of wave loads is sought from the record.

I told you I was hopeful but pessimistic, so don't take this as condemnation. I don't mean it as that. It's a challenge to extend every effort, and by effort I also mean expense--you can't get anything for nothing--in order to improve the odds of getting something that is meaningful rather than elegant.

The late Eric Merrifield (phonetic) once stated, and with his always benevolent exasperation, "I didn't conceive the dolos to be placed in a breakwater to rock." Perhaps that has been lost from our understanding. I'm still of the view that more effort is needed for understanding how to use the dolos and veer away from treating it as a peculiarly formed rock and trying to force it to act like a rock.

Also, we need a clearer insight as to when a broken armor piece should induce fright. Quarry stones break, and rarely does anyone panic over it. Again, there could be situations where a single broken dolos element or any armor element--whether stone, tribar, tetrapod--breaking could bring the thing down like a house of cards. We need to know when one situation or the other is present in our design and probably is project specific. Perhaps we've leapfrogged across the fundamental issue by looking for a stronger dolos instead of learning more about its significance as a particle in a large mesh.

I think we should design better and survive positively. For the designer's and modeler's use, we have a very real need to have a scale-down material for dolosse in testing. At least it needs to satisfy as closely as can be done in compromise with all such issues, end up being compromised in their answers. Compressive strength, that's obvious. Very closely related, I believe, is tensile strength. Elasticity, certainly, if it's a flexing structure we've got to know about that, and I'm a little bit dubious about this as a flexing structure. I think it's a shear fracture structure. All of you come back and say, all right, take components of stress that are in there so you can treat it as in flexure. All right, but I'm not sufficiently versed in mechanics of fracture to argue that point.

We need to have the hardness model, I think, very clearly. Who has gone on a project and not seen gouge marks and spalls where one piece has settled in against the other? It has restraining effect. It may not represent a great force, but if it's a passive resistance it has tremendous influence on the stability of the nestled-in and stabilized structure. So hardness of the surface, I think, has to be very important.

The density, obviously, but that could be controlled by weighting additives. I offer barium sulfate as one of the best.

There has to be an expense relationship, obviously. We heard yesterday a lot about how much something costs. An old boss of mine, and mentor, Raymond Hill, honorary member of the American Society of Civil Engineers, was once introduced by a friend who was needling him in the presence of a lieutenant general with whom he had just shaken hands, saying, "This is Raymond Hill, the highest priced engineer on the West Coast." The general looked quizzical: he knew something was going on between them. Mr. Hill spun his cocktail glass for a moment, and he said, "I don't know, General--glad to meet you--high priced, maybe; high cost, no." This is very important.

We tend to minimize the importance in value of what we do in the design phase. That is where the money needs to be spent in order that there be good, maintainable, dependable structures. Don't apologize for it. It's time to stiffen your back, face the guy down, and leave the project if he won't listen to you. Because if he won't listen to you then, he won't listen to you later, and you'll do nothing but get yourself in a jackpot. The time to spend money is during investigation. It's well-spent money. It may be expensive, but it is not costly.

Now, how about dynamic response of full-scale dolosse? A very simple test that has intrigued me for a long time would be to go down to the harbor where there's a high-lift crane available, suspend a dolos in whatever attitude you choose, and with extreme high speed film photography, stereo if you can, let it fall into a cushion of water deep enough that it won't fracture when it hits the bottom. The idea would be to instrument the thing for stresses, and keep increasing the falls until it finally breaks, if it does.

It's easy to calculate the velocity at which this interacts with the water. If you can't move the water, move the piece. I think that this is a test that could reveal some stuff with stresses inside of this thing, showing how it reacts to the dynamic force of moving water.

I leave the challenge of finding a better material, and I think this is something that deserves all the pressure that this group, and others, can apply to finding the money to solve an extremely important question. The answer may be it doesn't matter and the answer may be it matters a lot, but right now we're speculating, largely. Let's find out how to design these structures so the strength of the dolos is not that important as long as it's well made. Otherwise, what we're sitting here doing is trying to find a new shape, not the dolos.

DISCUSSION OF "STRESSES IN DOLOS"

MR. OMAR J. LILLEVANG

MR. MAGOON:

I would like to make a comment that whatever prototype location is selected, that you give careful consideration to the type of surveys Mr. Lillevang had directed of the bathymetry offshore of the Diablo Canyon. I think Mr. Lillevang has brought those. I believe that's probably the most elegant sounding and comprehensive sounding that I've seen at a location. I think that this would be very important regardless of what is done, but certainly later on when you model, you'll want to know what the bathymetry is. I would ask perhaps if Mr. Lillevang could very briefly discuss the surveys and whether they are available to this group.

MR. LILLEVANG:

I've brought along as a matter of busman's holiday interest, for those who would like to see it, a couple of video tapes of the model study we conducted using that bathymetry.

BEHAVIOUR OF DOLOS STRUCTURES PARTICULARLY IN REGARD TO UNIT STRENGTH

by

J.A. Zwamborn*

1. BACKGROUND

It has been generally accepted that dolosse have a high Hydraulic Stability which, together with the fact that they can and should be placed at random, have made dolosse very attractive for breakwater armouring, particularly in areas with sustained wave action.

Provided dolosse can withstand the static and direct wave loading and movements (rocking) are too small to cause significant impact loading, results of hydraulic stability model tests can be used directly to determine the required armour unit mass. Figure 1 shows the expected percentages displaced and rocking units as function of wave height based on tests with monochromatic waves. Variations in individual test results and the effect of irregular waves also play an important role and the results in Figure 1 can therefore only be used for a **first design.**

It is now wellknown, however, that significant movements and/or severe rocking of dolosse can take place under 'accepted' design conditions, that is for stability factors K_D, between 20 and 30. This is clear from Figure 1 which shows that, although percentages of displaced dolosse are less than 1 to 2 for K_D values of 20 to 30, an additional 2 to 4,5 per cent of the units were rocking or showed small movements. These movements could result in

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abrasion, crushing, spalding and eventually breakage of units. The question is, does this happen and, if so, how does this affect the stability of the armouring?

To find the answer to these questions, attempts have been made to correctly scale the structural behaviour of model dolosse but full similitude cannot be achieved and model tests with breakable armour units are very difficult to perform (Timco and Mansard, 1982). Tests with pre-broken armour units indicate that 15 per cent breakage in the top, bottom or total armour layer, and clusters of 5 broken units do not significantly affect the armour stability (Markle and Davidson, 1983). This would mean that a design based on a total damage (displacement plus rocking- see Figure 1) of, say, 5 per cent should be safe but this test technique also has serious limitations, mainly because of the artificial introduction of broken units and the fact that the growth of local damage areas cannot be correctly reproduced.

Another approach would be to determine the extent of the movements (rocking), the impact forces involved and the resulting stresses in the dolosse and to relate these to the ability of the units to withstand these, also under repeated impact loadings.

Measuring techniques have been developed to measure accelerations (Groeneveld, Mol and Zwetsloot, 1983) and stresses in individual model armour units (Hall, Baird and Turcke, 1984) but a large number of units would have to be instrumented for each test condition to get statistically reliable results. The same applies to similar measurements in the prototype which will have the added disadvantage of considerable practical and logistic problems.

Considerable progress has been made in determining the resistance of dolosse of different size, waist

ratio, concrete strength and with various types of reinforcement, to static and impact loading (LNEC, 1979; Burcharth, 1981 and 1984; Grimaldi and Fontana, 1984; Zwamborn, 1979). To apply the results of these studies effectively, however, details of the in situ loading conditions under near design wave conditions must be known.

In South Africa, model research on dolosse has been concentrated on the determination of damage, expressed as percentages displaced, continuously, intermittent and occasional rocking units, as functions of wave height, dolos packing density, dolos specific density and dolos waist-to-height ratio (Scholtz, Zwamborn and Van Niekerk, 1982; Zwamborn and Van Niekerk, 1982). For design, low percentages damage are used (typically 2 to 3 per cent displacement plus rocking) and several repeat tests, where possible, on the entire structure (three-dimensional tests) are made for the final design, including realistic design conditions (wave heights, periods and direction). These design criteria are then checked against measurements of overall prototype behaviour of completed dolos structures.

2. BEHAVIOUR OF SELECTED DOLOS STRUCTURES IN SOUTH AFRICA SINCE 1979

Hundreds of thousands of dolosse have been used all over the world with a large proportion (estimated in 1980 to be about 50 per cent) having been used in South Africa.

Details of the behaviour of ten South African dolos structures were given previously in a report entitled "Survey of Dolos Structures" (Zwamborn and Van Niekerk, 1981). This report included data until 1979/80; further data, for selected projects, covering the period 1979 to 1982, are included below.

Richards Bay Main Breakwater

The Richards Bay main breakwater is protected with 5t, 20t and 30t dolosse placed to slopes of 1 in 1,5 and 1 in 2 (Campbell and Zwamborn, 1977). This breakwater was completed in February 1976 and has been monitored at regular intervals since 1978. The monitoring was concentrated on the above-water part of the dolos armour, namely the 20 t dolosse on the trunk and the 30 t dolosse on the head of the breakwater. Techniques used were vertical and horizontal photography, visual observations and profiling with a spherical cage, made of reinforcing steel, with a diameter of 2,5 m (0,6h, where h is the dolos height - see Figure 2). Unfortunately, it was not possible to carry out underwater (diver) surveys because of strong wave action, bad visibility and the large number of sharks in the area.

The following survey and storm data were available for analysis:

Breakwater Monitoring		Storm Data						
Date	Type of Survey	Date	H_{s-r} (m)	T_{cl} (s)	Direction (°deep sea)	H_{s-b} (m)	H_{max} (m)	Tide (m+CD*)
25-6-78	Vertical photography (small scale)	3-9-78	4,8	11	224	5,3	10	1,6
28-6-79	Vertical photography (large scale)	24-7-79	4,8	8	185	4,3	8	1,8
3-8-79	Visual damage SATS							
28-11-79	Visual and photographic (CSIR)	17-4-80	4,8	8	200	5,3	10	1,5
May '80	Profiles (1,2m ball)							
4-7-80	Vertical photography (small scale)							
Dec '80	Profiles (1,2m ball)							
18-6-81	Vertical photography (large scale)							
July '81	Profiles (2,5m ball)	17-8-81	4,5	13	168	4,5	9	0,1
25-9-81	Horizontal photography							
5-12-81	Vertical photography (large scale)							
5-6-82	Vertical photography (large scale)							

* Chart Datum

Only storms with significant wave heights exceeding 4 m have been included in the table. The H_{s-r} values are wave heights recorded by waverider anchored in the 20 m water depth, about 1,5 km seaward of the south breakwater head. Wave periods and directions were recorded by wave clinometer. H_{s-b} values are the wave heights directly in front of the breakwater head, determined from actual measurements made in the original physical model during three-dimensional breakwater stability tests (Campbell and Zwamborn, 1977). Since the wave height measurements apply to a 6-hour period, $H_{max} = 1,9\ H_{s-b}$ (Longuet-Higgins, 1952). Most of the storms occurred at high tide. The depth at the breakwater head decreased from the original -17 to -12 m CD, because of siltation, but the bottom falls quite steeply at 1 in 25 to - 20 m C.D. about 300 m from the breakwater head. This means that at high tide (+ 1,8 m CD), the maximum breaker height can be about 0,9 x 13,8 = 12,4 m (Jackson, 1968).

Generally, the main breakwater was found to be in a good condition after more than 6 years of service although some maintenance will be done during 1985, mainly on the breakwater trunk on the seaward side.

Detailed analyses of a section of the breakwater head on which were placed 1045 30 t dolosse with a waist-to-height ratio of 0,36 were possible with the use of large-scale vertical photographs. Examples of these photographs are shown in Figures 3 and 4. The results of these analyses on the 'head section' are as follows:

(i) Comparison of Vertical Photographs 28-6-79 and 18-6-81
These photographs were taken with an aerial photography camera (0,23 m square negative) from a height of 300 m covering a period of two years (the photographs taken on 4-7-80 were at too small a scale for detailed damage

analysis). The most exposed part of the head (seaward of the line shown on the photographs) was analysed in detail for any changes over the two-year period by using large-scale (1 in 200) transparent overlays on which every dolos was identified and marked. The result of this analysis which, of course, covers only the above-water parts of the breakwater showed the following:

30 t dolosse - 3 units appeared to have disappeared
4 units had broken
4 units had moved (changed position) without breaking

Thus, 7 units of a total of 1045 placed on this part of the head were 'lost' (about half of this number were above low water when the aerial photographs were taken).

During this period two storms occurred in which the significant heights of the waves were 4,3 and 5,3 m and maximum heights up to 8 and 10 m, respectively.

(ii) Comparison of Vertical Photographs 18-6-81 and 5-12-81

The period covered by these photographs is six months. A comparison identical to that described above for the two-year period was made and the results showed the following:

30 t dolosse - 1 unit broken
1 unit moved, without breaking.

The remainder of the visible units were in exactly the same positions (of the total of 1045 units on the analysed part, about half were above water).

During this period one storm occurred with a significant wave height of 4,5 m at the breakwater or an expected maximum wave height of about 9 m.

(iii) Comparison of Vertical Photographs 5-12-81 and 5-6-82

These photographs, also covering a six-month period, were again analysed in the standard way and the result showed no change, except for one piece of dolos which

appeared on the June '82 photograph, just above the water line, where it was not visible on the December '81 photograph. Significant wave heights did not exceed 4 m during this period, but several periods with waves just below 4 m significant or about 8 m maximum did occur.

In **conclusion**, the main breakwater and particularly the head, was generally in a good condition although some repair was necessary along a 50 m section of the trunk (20 t dolosse). In-service damage/breakages amounted to **eight** 30 t units on the exposed head section over a period of three years, during which period three storms occurred, with maximum heights of waves of up to 10 m at the breakwater head. This damage to the above low-water section of the head (1045 30 t units in total, about 500 above low water) amounts to 1 to 2 per cent. However, underwater profiles showed no serious deterioration below the water surface.

It should be mentioned that a more reliable evaluation would have been possible if proper **as-built** profiles and vertical photographs made immediately after construction had been available. This should be included in any future breakwater contract!

Gansbaai Harbour

The Gansbaai harbour breakwater, described in Zwamborn and Van Niekerk (1981), was repaired between 1979 and early 1982 with 505 20 t and 1630 25 t dolosse, as shown in Figure 5. The performance of these dolosse was closely monitored by regular visual observations, including underwater diver surveys, during construction and after the occurrence of large waves.

The following data were made available for analysis by the Fisheries Development Corporation:

Breakwater Monitoring			Storm Data						
Date	Dolos breakages		Date	H_{s-r}	T_p	H_{s-b} (m)		H_{max}	Tide
	20 t	25 t		(m)	(s)	WSW	11°N of W	(m)	(m+CD)
			2-10-80	3,4	-	1,7	6,0	8,5	1,1
			3-10-80	3,4	-	1,7	6,0	8,5	0,9
13-10-80	6	-	12-7-81	5,4	14	4,6	6,9	8	0,6
			13-7-81	5,5	16	4,7	7,0	8,5	1,4
13-7-81	3	13	7-8-81	5,5	-	4,7	7,0	8	0,6
			18-9-81	4,5	-	3,1	6,8	8,5	1,2
			19-9-81	3,8	14	2,1	6,4	8,5	1,2
1-10-81	1	14*							
Total, during construction	10	27							
			12-7-82	3,4	12	1,7	6,0	8	0,6
			17-7-82	3,7	18	2,3	6,4	8,5	1,2
			29-7-82	4,2	14	2,6	6,7	8,5	0,9
July '82	1	2							
Total, after completion	11	29							

Only the peaks of the storms are given; the storms lasted from one to three days so that high waves occurred for considerable periods. The H_{s-r} values are significant wave heights recorded by waverider situated 1 200 m west of the breakwater, seaward of a 12 m deep shoal area, in water 26 m deep.

The peak periods, T_p, were also determined from the waverider data. The shoal area caused considerable refraction of the dominant westerly waves resulting in reductions in wave height for WSW'ly waves and, generally, increases in wave height for 11° N of W'ly waves. Unfortunately, the wave directions were not recorded

during the above storms and two wave heights at the breakwater (H_{s-b}) are therefore given in the table.

The conversion from H_{s-r} to H_{s-b} is based on actual wave heights measured in a 1 in 80 scale Gansbaai model. Not knowing the wave direction makes it difficult to decide what the maximum wave heights at the breakwater were during the above storms. Considering the duration of the storms, $H_{max} \geq 2H_s$ (Longuet-Higgins, 1952) which would give H_{max} values of at least 3,4 to 9,4 m for WSW and 12 to 14 m for 11°N of W waves. However, the water depth along the 25 t dolos section is only 9 to 11 m at low water or 11 to 13 m at high water while the bottom is almost level in the direction of the incoming waves. Theoretically, the limiting breaker height would therefore be about 0,6 x 13 = 8 m (Jackson, 1968). As the model tests also showed maximum recorded waves near the breakwater of 8 to 9 m, it may be accepted that during all the above storms the breakwater was attacked by waves up to 8,5 m high (H_{max}), except during the lower tides when $H_{max} \approx 8$ m.

From the above data it is seen that damage (dolos breakages) during the reconstruction period (1979 to 1982) amounted to:

20 t dolosse 10 breakages or 2%
25 t dolosse 27 breakages or 1,7%

After its completion, the breakwater armour withstood three further storms with only

20 t dolosse 1 breakage or 0,2%
25 t dolosse 2 breakages or 0,1%

Considering that these figures include both the above- and below-water parts of the breakwater, that working conditions at Gansbaai were difficult due to regular long swells and that large waves (8 to 8,5 m) occurred quite regularly at the breakwater (10 times in about 2 years) it can be concluded that both the 20 t and the 25 t units performed very well, with only minor breakages, with waves up to 8,5 m.

Koeberg cooling-water intake basin

The breakwaters forming the Koeberg intake basin are protected with 6 t, 15 t and 20 t dolosse placed at a slope of 1 in 1,5 (Zwamborn and Van Niekerk, 1981). Some 2 295 20 t dolosse were used to armour the main breakwater from chainage 750 to 912 m, which includes the head. The entire main breakwater has depth-limiting design conditions; the depth at its head is -8 m CD and waves with $H_s \geq 3,2$ m were assumed to start breaking on the head (H_{max} = 6,4 m). This means that the design waves (6,4 m breaking waves) occur, on average 15 days per year, which was taken into account in deciding on acceptable damage criteria.

The breakwaters were completed in April 1980. A photographic base-line survey of the armouring was made on 28 October, 1980 (from the air) and on 6th November, 1980 (from a boat) while an above- and under-water survey of the total number of broken dolosse (including construction damage) was made on 20th October, 1981.

The available damage survey and storm data were summarized as follows:

Breakwater Monitoring			Storm Data				
Date	Type of Survey	Damage (20t)	Date	H_{s_r} (m)	T_p (s)	H_{max} (m)	Tide (m+CD)
28-10-80	Vertical photography (small scale)	-					
6-11-80	Horizontal photography (base line)	0					
			12-11-80	5,3	18,3	6,7	1,6
			19-11-80	3,7	13,5	6,3	1
			8- 1-81	3,4	-	6,3	1
			21- 1-81	3,2	-	6,1	1
			24- 1-81	3,2	-	6,1	1
			30- 1-81	3,8	13,5	6,3	1
7- 2-81	Horizontal photography	4 (settled)					
			12- 4-81	4,4	13,5	6,3	1,0
7- 5-81	Horizontal photography	No change	26- 5-81	4,4	13,5	6,2	0,8
			6- 6-81	3,8	13,5	6,3	1
			21- 6-81	3,7	13,5	6,3	1
			28- 6-81	3,4	13,5	6,3	1
			13- 7-81	5,5	15,5	6,3	1,0
			19- 7-81	5,2	11,9	6,7	1,5
			22- 7-81	4,2	11,9	6,3	1
			24- 7-81	3,9	15,5	6,3	1
			29- 7-81	4,0	15,5	6,3	1
			4- 8-81	4,8	13,5	6,3	1
			7- 8-81	4,6	13,5	6,3	1
			11- 8-81	5,0	13,5	6,3	1
					Storms several days		
18- 8-81	Horizontal photography	No change	27- 8-81	3,4	13,5	6,3	1
			15- 9-81	4,7	11,9	6,3	1
			16- 9-81	4,7	13,5	6,9	1,8
			17- 9-81	5,1	13,5	6,7	1,5
20-10-81	Diver survey	31 broken					
			29-10-81	3,3	13,5	6,3	1
11-11-81	Horizontal photography						

Only the storms during which significant wave heights exceeded 3 m are included in the above table. H_{s-r} and T_p values were obtained from wave-rider records. The waverider was anchored in 23 m deep water 3 km offshore.

No wave direction data were available but previous records showed that 90 per cent of the waves at Koeberg are SW to WSW'ly. Moreover, for these directions the refraction coefficients are close to one (these directions are nearly perpendicular to the coast) and the recorded H_{s-r} values were also assumed to apply just seaward of the breakwaters.

The water depth at the south breakwater head was -8 m CD and the bottom slope about 1 in 100. Thus, the limiting breaker heights are about 0,7 x 8 = 5,6 m at low water and 0,7 x 10 = 7 m at high water (Jackson, 1968). In the above table, H_{max} values are limiting breaker heights, except where H_{s-r} = 3,2 m, when H_{max}=1,9 x 3,2 = 6,1 m(non-breaking; Longuet-Higgins, 1952).

Unfortunately, here again no survey was done immediately after completion of the breakwater. By the time the first photographic surveys were done in October/November 1980, some 10 storms in excess of 3 m (significant heights) had occurred (6 month period).

The diver survey was done in October 1981. It showed 31 broken dolosse (1,4%), of which 18 were under and 13 above low water level. However, the site engineers consider that most of these breakages had occurred during construction.

The five horizontal photography surveys show very little damage above low water level over a one-year period during which breaking waves of 6 to 7 m height occurred on 24 occasions. All that happened was that 2 sets of 2 dolosse settled 0,1 and 1,3 m, respectively, and this without breakage (the latter case is shown in Figure 6).

It was therefore concluded that the 20 t dolosse on the Koeberg breakwater head can withstand frequently occurring 6 to 7 m breaking waves without breakage.

Concluding Remarks

Prototype measurements of dolos behaviour have provided evidence that 30 t dolosse with a waist ratio of 0,36 placed on the Richards Bay breakwater head, can withstand 10 m waves with little movement and 1 to 2 per cent breakage. Applying Figure 1, the expected total damage would be 3 per cent displacement plus all types of rocking, or below 2 per cent, when excluding occasional rocking (heavy line in Figure 1).

The newly placed 20 and 25 t dolosse at Gansbaai (waist ratios 0,34 and 0,35 respectively) have withstood three storms with waves reaching 8,5 m with only 0,2 and 0,1 per cent breakages respectively (including the under-water part). For this wave height, Figure 1 indicates 2,5 and 2 per cent 'total damage', or 1,5 and 1 per cent excluding occasional rocking, for the 20 and 25 t units respectively. The 20 t dolosse at Koeberg (waist ratio 0,34) showed no damage over a period of one year when 6 to 7 m breaking waves attacked the breakwater on 24 occasions. For these wave heights, Figure 1 would indicate 1 per cent 'total damage' or 0,5 per cent excluding occasional rocking.

These observations provide evidence that well-designed dolos structures can withstand severe and sustained wave action with nominal damage, particularly as in the cases of Gansbaai and Koeberg the (depth limiting) design wave conditions occurred during the observation periods.

Further evidence has been provided by the occurrence of the approximately one-in-100 year storm in May 1984 which struck the Cape Town area. This storm reached a maximum H_s = 10,8 m with T_p = 15,5 s (recorded in 200 m water depth). Detailed damage analyses are still under

way but preliminary information showed 6 20 t and 11 25 t broken dolosse at Gansbaai (1,2 and 0,7 per cent) and 13 20 t broken units at Koeberg (0,6 per cent) as a result of the storm. The water level recorded inside the Koeberg basin during the storm was 0,8 m above normal (about + 2,6 m CD) resulting in breaking waves probably exceeding 8 m.

During the same storm, severe damage was caused to several coastal structures in the Cape Town area, notably the Table Bay main breakwater which lost 172 concrete blocks of 30 to 40 t and 600 9 t dolosse which were used as (temporary) protection (Figure 7).

3. EXPERIENCE WITH DOLOSSE IN OTHER PARTS OF THE WORLD SINCE 1979

Details of the behaviour of some 25 dolos structures spread around the world, including data until 1979/80, are given in the report "Survey of dolos structures" (Zwamborn and Van Niekerk, 1981). Some additional data on selected projects are included below.

San Ciprian Harbour

Nominal 50 t dolosse were used as armour on the San Ciprian breakwaters in Northern Spain. The main or north breakwater is 945 m long ending in 20 m water depth and the south breakwater is 1 012 m long ending in water 21 m deep. The dolos slope varies from 1 in 1,25 to 1 in 2 and the spring tidal variation is 4,6 m.

Certain deficiencies, particularly in the quality of the concrete and in the execution of the works, had been noted during a visit in 1979, and soon afterwards serious damage occurred to the breakwaters. As a result, in July 1980 the owners of the port commissioned a review of the design, model studies, construction methods and the quality of the works as well as an evaluation of the damage in view of possible remedial works.

From this comprehensive study (Losada, et al, 1980) it is concluded that the damage to the San Ciprian dolos armour may be attributed mainly to wave concentration due to refraction, hydraulic instability of the armour (for wave heights exceeding 9 to 12 m and wave periods exceeding 12 to 17 s) and breakage of the 50 t dolosse due to excessive movements, deficiencies in the quality of the armour units and the relatively small waist ratio (only 0,348 for these 50 t units). Significant waves up to 6 m (H_{max} = 1,9 x 6 = 11,4 m) were reported to have occurred on several occasions since construction while refraction studies indicate that 6 m waves of 15 s period could build up to about 10 m (H_{max} = 19 m) at the north breakwater. Since the water depth limits the breaker height to about 16 m (Losada et al, 1980) it is very likely that waves from 12 m, up to breaking waves of 16 m, have occurred already. It is therefore not surprising that, on average, some 10 per cent of 50 t dolosse were broken and in specific areas up to 100 per cent of the units failed.

Dolos Experience in Japan

Up to 50 t dolosse have been used over the past few years in Japan, in most cases with conventional reinforcing - Kametoku, 35 kg/m³ and Naha Port, 92 kg/m³ steel (Zwamborn and van Niekerk, 1981 and Sakou et al, 1981).

So far damage has been reported on only one project, Yoron Port on Yoron Island, where 223 50 t dolosse, reinforced with 35 kg/m³ steel, were placed temporarily to protect a vertical-face pier. The dolosse were placed directly on the rocky bottom at about -10 m. Considerable damage occurred to the 50 t dolos mound and to the adjoining pier structure as a result of three typhoons with waves of estimated height 8,4 m. The armour damage consisted of:

a)	broken dolosse	7	b)	exposed reinforcing	15
	cracked dolosse	4		extreme abrasion	26
		11 (5%)			41 (18%)

Damage (b) indicates excessive motion of the dolosse, alternatively poor concrete (the design strength is given as only 16 to 18 MPa and the 35 kg/m³ steel is a very light reinforcing, 0,5% by volume).

Nippon Tetrapod Co. Ltd.,subsequently carried out tests with dolosse up to 4 t, steel reinforcing varying from 0 to 151 kg/m³ and concrete strengths from 20 to 40 MPa to determine their influence on the static and dynamic behaviour of these units (Terao, et al, 1982). The tests showed that with a 100 per cent increase in concrete strength the first cracks appeared only after an increase of 30 per cent in static load and about 100 per cent in drop height (dynamic test). The first cracks appeared under the same load conditions for unreinforced and reinforced units but while the unreinforced units broke after an increase in load of only 20 per cent above the first-crack load, this value was about 100 per cent for the reinforced dolosse.

Gioia Tauro west breakwater

The west breakwater of the new port at Gioia Tauro, Italy (Grimaldi and Fontana, 1984), was protected with 15 t dolosse to a slope of 1 in 2. The water depth at the toe of the structure was -8 m but the foreshore drops down steeply, at about 1 in 4, to 50/60 m depth only 200 m offshore which means that there are no depth-limiting conditions. In fact, the particular shape of the foreshore results in plunging breakers directly onto the armour slope.

The dolos armour was damaged during storms which occurred on 31st December 1979 and on 1st January 1980. The waves were recorded by waverider, anchored in 70 m water depth. The maximum recorded H_s was 7,4 m with a corresponding H_{max} = 11,6m and T_p = 12,8 s so that the armour must have been attacked by breaking waves with a height of up to 12 m. According to Figure 1, the 'total damage' would be $>$ 15 per cent for this wave height or about 14 per cent excluding occasional rocking units.

Prototype observations indicate breakages of 42 per cent of the dolosse below and 33 per cent of those above water over a 200 m long section of the west breakwater although the armour profile did not change too much. Although this is more than predicted by Figure 1, special model tests of the actual structure carried out in the Delft laboratory showed comparable damage to the prototype when equating the rocking model units with the broken prototype ones (Den Boer, 1982).

Survey of existing Corps structures

A detailed review of seven US Corps of Engineers' dolos projects is included in a report by Markle and Davidson (1984). Unreinforced and reinforced dolosse from 2 to 43 t were used in these projects and recorded breakages varied from 0 to 1,9 per cent, except for Crescent City where 9 per cent breakage was recorded. However, this included broken units left after construction.

Most of these structures have experienced design wave conditions but it must be mentioned that all, except the Cleveland case, had depth-limiting design waves.

Concluding remarks

Problems with a number of dolos structures have been experienced subsequent to the major failure at Sines in 1978/1979 (Zwamborn, 1979). The damage described above (San Ciprian, Yoron Port and Gioia Tauro) can, however, be explained adequately, and in the author's opinion, could be expected considering the particular local conditions. This damage does, however, emphasize the conclusions reached on the basis of the previous dolos survey (Zwamborn and Van Niekerk, 1981), that is,

(i) special attention must be given to the design conditions of deep water structures,

(ii) a simple formula cannot describe the stability of dolos armour, and

(iii) most structures have been built without proper model tests, which should include representative irregular waves and three-dimensional effects.

The survey of the Corps Structures (Markle and Davidson, 1984) largely confirm the conclusions reached on the basis of the South African dolos structures, namely that large dolosse can withstand up to 10 or 12 m high breaking waves without breakage. In the case of Corps structures, the larger dolosse were mostly reinforced with a light conventional steel reinforcement, the long-term advantage of which would need further investigation.

4. DOLOS ARMOUR DESIGN CONSIDERATIONS

For the design of a dolos structure the main requirement is to ensure adequate armour stability with an acceptable percentage of armour unit deterioration (e.g. breakage) under realistic and representative design conditions which must include the cumulative effects of expected wave action for the design life of the structure.

Different approaches to achieve this are discussed below.

'No movement' design

Recent static loading tests on 15 t and 30 t unreinforced dolosse have shown that these dolosse can carry 4 to 6 times their own weight under the worst possible loading condition (Figure 8) without breakage (Grimaldi and Fontana, 1984). Thus, considering a more realistic loading condition applicable to the actual breakwater situation (top dolos supported by at least two bottom units), up to about 8 layers of dolosse could be used without danger of failure of the lower layers due to the dead weight of the armouring.

As no particular problems have been encountered handling unreinforced dolosse up to 50 t in the casting and

placing process (breakages did not exceed 1 to 2 per cent) and because the direct wave forces are of the same order or less than the dead weight forces, a dolos armour consisting of several layers could be designed safely on the basis of hydraulic model test results, if these show that there is no significant movement (rocking).

In determining the 'no movement' criterion detailed tests would have to be done, taking into account:

(i) realistic and representative design conditions, particularly for waves (wave spectrum/groupiness),

(ii) possible variation (reliability) of design conditions,

(iii) storm duration and cumulative effects,

(iv) near-shore effects,

(v) variation in test results,

(vi) the extent of the structure, and

(vii) three-dimensional effects.

Because the 'no movement' criterion basically means that the design must be based on H_{max}, the result could become rather uneconomical. For instance, using Figure 1, a K_D factor of less than 5 is indicated. A similar result is found assuming $H_{max} = 1,6\ H_s$, namely a K_D of 20 used for a design based on H_s would become 5 when based on H_{max}. Because this would be of the same order as that for cubes, a 'no movement' design approach for dolosse does not have particular advantages.

'No breakage' design

The other extreme would be to ensure that the dolosse are unbreakable under prototype loading conditions. In this

case the design can be based on conventional hydraulic model tests which have to prove that armour displacement will be within acceptable limits, for instance, Figure 1 indicates K_D factors between 20 and 30 for 1 to 2 per cent displacement. Detailed tests, as described above, will be necessary to ensure a safe design. The results of these tests could also be used to optimise the design, that is to minimise the total cost (capital investment plus maintenance cost).

This approach is the easiest with regard to model testing but there are two major problems, namely, to determine the prototype loading conditions and to make the dolosse unbreakable for these conditions without losing the cost advantage of the units' high stability.

Extensive tests have been done with different types of reinforced dolosse (Burcharth, 1981 and Grimaldi and Fontana, 1984). The test results showed limited improvement in impact strength with from 30 kg/m³ up to 120 kg/m³ (0,4 to 1,6 per cent by volume) steel fibres, that is,an increase in drop heights from about 20 to 150 per cent at 'failure' (major damage and/or breakage). Conventional steel reinforcement of 77 to 138 kg/m³ (1 to 1,8 per cent by volume), however, was found to make the dolosse virtually unbreakable. Although first crack formation occurred at drop heights only about 50 per cent higher than for unreinforced units, serious damage (major cracking and spalding which exposed the main reinforcing bars) occurred for drop heights 4 to 8 times those of the unreinforced units (0,8 to 1,6 m for 30 t dolosse at Gioia Tauro). Moreover, tests at Gioia Tauro, where a 30 t dolos with a waist ratio of 0,37 and reinforced with conventional steel reinforcement (77 kg/m³ or 1 per cent by volume) was dropped on the breakwater core and showed no serious cracks up to a drop height of 10 m. Although the in situ loading is not known, one would intuitively consider that this dolos would be strong enough (mechanically)

to withstand the in situ forces caused by movements/rocking.

Drawbacks of conventionally reinforced dolosse are additional cost (50 to 100 per cent more expensive which could make dolosse unattractive in many applications) and the possibility of corrosion causing serious deterioration of the units with time.

As the Corps of Engineers has built several structures with reinforced dolosse (for example, at Humboldt, using 45 kg/m³ or 0,6 per cent conventional reinforcing), close observation of any deterioration due to corrosion would be of great value for future use of steel reinforcement.

'Optimum' design

It seems obvious that there should be an 'optimum' design between the two extremes discussed above whereby a certain amount of movement/rocking is accepted while, at the same time, the dolosse are made strong enough to withstand these limited movements without increasing the cost of the units too much.

As early as 1972 it was suggested that the waist-to-height ratio (r) of larger dolosse be increased according to:

$$r = 0,34\sqrt[6]{\frac{W}{20}}$$

where W is the mass of the dolos in tons, to compensate for the higher stresses occurring in larger dolosse (Zwamborn and Beute, 1972). A simple analysis showed that, when using this formula, dolos stresses would remain about the same with increased dolos mass (Zwamborn, et al, 1980). The more rigid structural analysis by Burcharth (1981) supports this finding, as may be seen from Figure 9, which shows that the increased "r" (below water case) falls between Burcharth's drop and pendulum formulas. The beneficial effect on the structural strength of the

dolos was confirmed by prototype tests on 15 and 30 t units at Gioia Tauro (Grimaldi and Fontana, 1984).

It is obvious that, to get greater unit strength, even larger waist-ratios could be used. However, stability tests with dolosse with waist ratios of 0,33, 0,38 and 0,43 showed a gradual reduction in stability for the larger waist ratios, which was to be expected. The reduction in stability from r = 0,33 to 0,38 was relatively small but the test results indicate a reduction in K_D of some 50 per cent from r = 0,33 to 0,43 (Scholtz, Zwamborn and Van Niekerk, 1982). As a waist ratio of 0,43 corresponds to an 82 t dolos unit, there is still considerable scope in using the waist ratio in the optimi ation of the design.

Structural performance of dolosse can also be significantly improved by proper mix design and good quality control. Tests by Burcharth (1981) and Terao et al (1982) confirm the importance of high rupture strength and low Young's modulus concrete (Terao found an increase of 50 per cent in the breaking drop height for 4 t dolosse for a 100 per cent increase in concrete strength). The beneficial effect of increased rupture strength is also evident from Figure 9 which shows that for a small increase from, say, 3,5 to 4,0 MPa, a 45 t dolos should be as strong as a 20 t one, even if the waist ratio is not increased.

A significant increase in critical drop height (about 85 per cent) was obtained at Gioia Tauro using 5,5 l/m³ plasticizer (Rheobuild 877). The 30 t units were two months 'young' when tested. Depending on the cost, the addition of plasticizers in the mix design could therefore be considered.

Single central scrap rail reinforcement, similar to that shown in Figure 10, was used in the original East London dolosse, mainly to lift the dolosse out of the mould. This type of reinforcement has been used at Oranjemund,

to avoid/retard breakage of too light units which had to be used (Standish-White and Zwamborn, 1978). So far, this offshore water intake structure has performed satisfactorily.

In the redesign of the Gioia Tauro breakwaters (Grimaldi and Fontana, 1984) it was decided (1979) to introduce single-scraprail reinforced 30 t dolosse in the more critical areas on the breakwater heads for extra safety and to reduce maintenance (27,4 kg/m steel or 0,35 per cent by volume, Figure 10). Subsequently, the designers (Polytecna Harris of Milan) developed the socalled **double-V rail reinforcement** (Figure 11) which consists of a frame with four scraprails in the dolos trunk and one scraprail each in the flukes (53 kg/m³ steel or 0,7 per cent by volume). This solution proved to be both very effective and economical; the critical drop heights were found to come fairly close to those of the conventionally reinforced units, the possibility of corrosion was minimized and the extra cost for the reinforcing was reasonable (extra cost for double-V reinforced dolosse being about 60 per cent in Italy and about 26 per cent in South Africa).

Various relatively cheap methods to improve the strength of dolosse have been discussed above. It is not possible to define the exact dolos strength required when a certain amount of moving/rocking is allowed to occur under design conditions. Prototype observations, discussed above, provided **reasonable proof** that dolos structures designed on the basis of a few per cent (2 to 3) total damage (displacement plus rocking) performed satisfactorily. There is no doubt that strength improved (e.g. scraprail reinforced) units will be able to withstand considerably greater movements/rocking but more observations and (full scale) tests will be needed to confirm the need and effectiveness of these improvements and to arrive at the **'optimum' design.**

STRUCTURAL TESTS ON DOLOSSE

In the foregoing sections, frequent reference has been made to structural test results which provide invaluable data regarding the relative strengths of dolosse of different design, concrete quality and reinforcing.

A standard drop test on a solid concrete base has been proposed for dolosse by Burcharth (1981), details of which are shown in Figure 12.

Burcharth found that trunk breakage occurred for drop heights of the centre of gravity of 0,12 to 0,17 m using 1,5, 5,4, 10 and 20 t unreinforced dolosse. These drop heights improved by about 50 per cent using conventionally reinforced dolosse (75 kg/m³ or 1 per cent by volume, compared with 45 kg/m³ or 0,6 per cent used at Humboldt).

The results of similar drop tests carried out at Gioia Tauro, Italy, on 15 t (r = 0,32) and 30 t dolosse (r = 0,37, see Figure 11) (Grimaldi and Fontana, 1984), as interpreted by the author, summarized as follows (mean of repeat tests):

Dolos mass (t)	Age (months)	No. Repeat tests	Reinforcement type (kg/m³) Plasticizer type (l/m³)	Failure or major damage	
				Drop Height L (m)	Impact Velocity V (m/s)
15	20	2	-	0,15	1,7
30	20	1	-	0,42	2,9
30	2	3	-	0,21	2,0
30	2	2	Rheobuild 877 (5,52)	0,39	2,8
30	2	2	Steel fibres (50/74)	0,24	2,2
30	2	2	Twisted " (30/50)	0,29	2,4
30	3	1	Steel " (120)	0,50	3,2
30	2	3	Single rail (27,4)	0,5	3,2
30	2	1	Double rail (51,3)	0,9	4,2
30	2	1	Double-V (53)	1,3	4,7
30	2	4	Conventional (77/138)	~1,2	4,6

The results in this table show the considerable improvement in dolos strength with time (pozzuolana cement was used), the positive effect of the larger waist ratio of the 30 t units, the limited benefit of the steel fibres and the effectiveness of, particularly the double-V rail reinforcement.

These drop tests were done by increasing the drop heights from about half the critical height for the first drop by steps of 20 to 60 mm for subsequent drops, that is, by about 10 per cent of the critical drop height. Burcharth (1984) has shown that repeat impact loadings reduce the ultimate dolos strength, for example, 10 repeat impacts on a solid base reduce the ultimate stress to 80 per cent. Thus, as a result of the test procedure, the above drop test results can be considered to be somewhat conservative for a single loading case.

The standard drop test is carried out on a rigid base and failure is virtually due to impact loading only. In a breakwater armouring, a dolos will either drop on underlayer stone or on another dolos, both of which will probably move under the impact. A dolos breakage test which more closely represents conditions on an actual breakwater, should therefore include a realistic yield, comparable to movement of the underlaying dolos and/or stone. Further drop tests and free fall tests were therefore done at Gioia Tauro onto a 50 to 1000 kg rock fill bed and onto the breakwater core (Figure 13). The mean results of these tests, as interpreted by the author, are as follows (30 t dolosse, see Figure 11):

Age (months)	No. Repeat Tests	Type of test	Test bed material	Reinforcement type (kg/m^3)	Failure or major damage	
					Drop Height L (m)	Impact Velocity V (m/s)
2,5	2	drop	rock	-	2,4*	5,9*
3,5	1	drop	rock	Twisted fibres (75)	2,4*	5,9*
2,5	3	free fall	rock	-	1,5	5,4+
3,5	2	free fall	rock	Twisted fibres (75) and Steel fibres(95)	2,5	7,0
2	1	free fall	rock	Double-V (53)	2,6	7,1
2	2	free fall	core	Double-V (53)	2,5/5,0	7,0/9,9
2	1	free fall	core	Conventional(77)	10	14

*Maximum possible lifting height, no visual damage.

The most significant result of these tests is that the critical drop heights are **considerably greater** than for the rigid-base case. The drop height for the 30 t **unreinforced** dolosse is seen to be about **10 times greater** which, because this test configuration is considered much more comparable with reality, means that unreinforced dolosse should be able to withstand considerable movements/ rocking without breakage, a fact which is borne out by prototype observations.

Because tests on rubble are difficult to control fully and the inclusion of a certain yield in the tests is considered essential to get results more directly comparable with the actual breakwater situation, a test technique which includes an **adjustable yield** has been developed for full scale tests on 9 t dolosse at Cape Town (May/ June, 1985).

RESEARCH NEEDS

In a paper "Dolosse, past, present, future?" (Zwamborn, et al, 1980) the following research needs were listed:

"(i) more information is required on realistic design wave conditions and representative reproduction of these conditions in, preferably, three dimensional models;

(ii) research into the structural behaviour, under extreme load conditions, of artificial armour units, in general, and dolosse, in particular, is needed; such studies should include surveys of existing dolos structures, hydraulic model tests to determine the loads on the units, stress analysis for these load conditions and structural tests to determine acceptable degrees of movements;

(iii) for deep-water conditions, structures with a high reserve stability should be developed; and

(iv) model test techniques should be investigated further and standardized as far as possible to ensure compatibility between test results of different laboratories".

Progress has been made with most of the above items but much work still has to be done and it is believed that the above needs still stand today.

The most important questions emerging from the foregoing discussions on dolos armouring are:

- **what is the extent of dolos movement under 'design' conditions,**
- **what are the resulting in situ loading conditions and stresses in the dolosse,**
- **can the dolosse withstand these loads without breakage (serious deterioration),**
- **what are the effects of dolos breakage on the overall armour stability?**

It is suggested that future research should be aimed at answering these specific questions while, at the same time, taking into account the overall research requirements listed above (items (i) to (iv)).

The problems are complex and different approaches including detailed structural analyses, measurements of movements, forces and stresses in model and prototype units under various load conditions and specific model and prototype tests will be necessary. Because of the stochastic nature of the problem, the South African approach will continue to concentrate on improved methods to record dolos movements in hydraulic model tests and to relate these to overall behaviour of actual dolos structures.

A structural test technique is being developed (dolos break test) which is expected to be more comparable to the breakwater situation. Moreover, 9 t 'test dolosse', having different types of scraprail reinforcement, will be placed along a 30 m long section of the west breakwater in Table Bay as temporary protection after the May 1984 storm damage. These 9 t units are much too light for this situation and their behaviour during the coming winter season (1985) will be closely monitored for comparison with model data.

CONCLUSIONS

It is concluded that carefully designed dolos structures can withstand large waves with only a few per cent damage. Shortcomings in the design and/or construction can, however, result in significant damage or even local failure of the structure.

Dolos armour design could be based on 'no movement', 'unbreakable' units (e.g. heavily reinforced) or 'limited movement' using strength-improved dolosse. The latter is considered to provide the 'optimum' design.

Prototype tests on dolosse have shown that scraprail-reinforced units (particularly the double-V reinforced ones) can withstand considerable impact forces while a realistic yield (comparable to the breakwater situation) will increase the ultimate drop heights by a factor of about 10.

Research is needed to establish the extent of dolos movements under 'design' conditions and to relate these movements to *in situ* loading, stresses and breakages/ serious deterioration.

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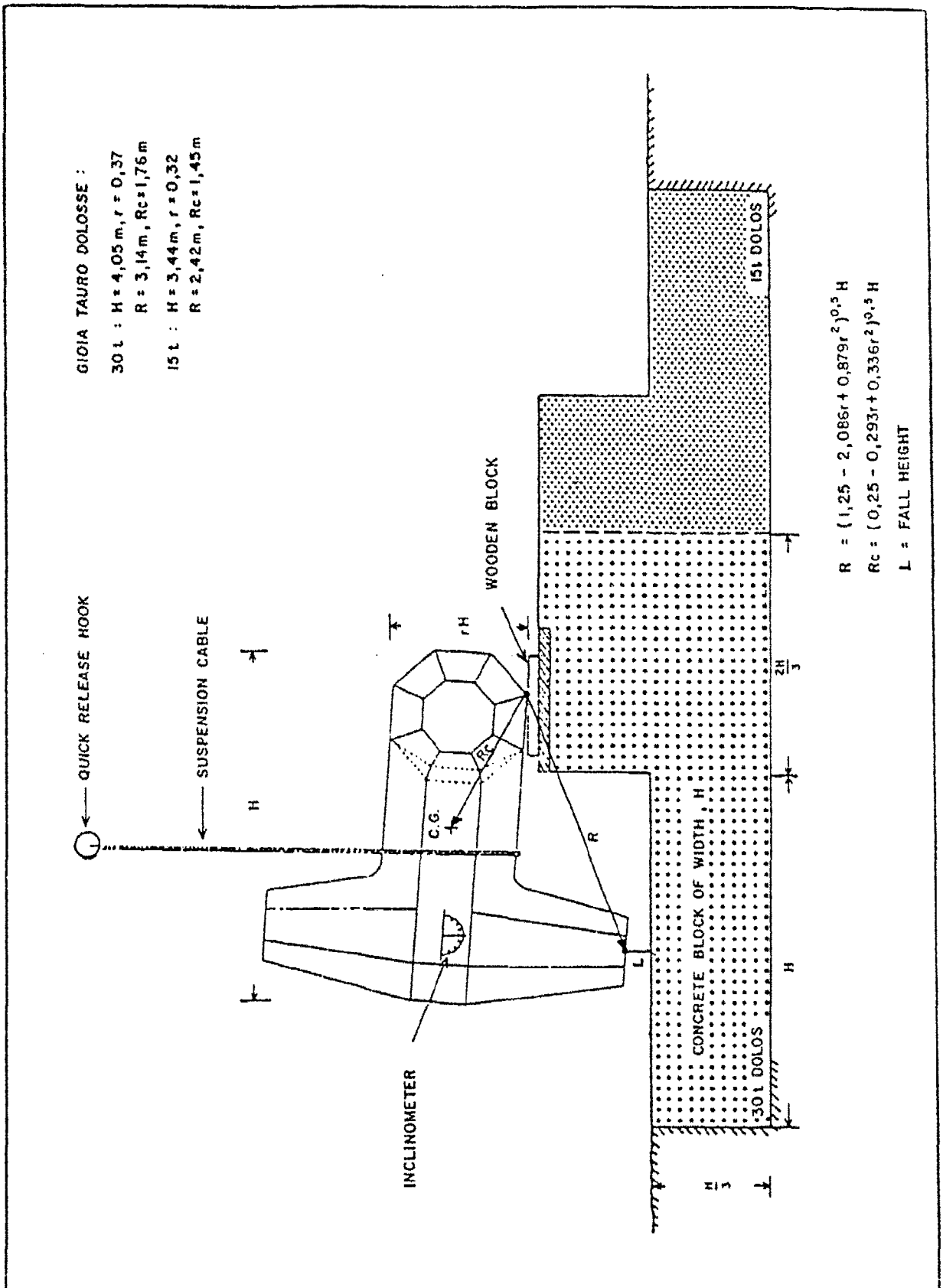
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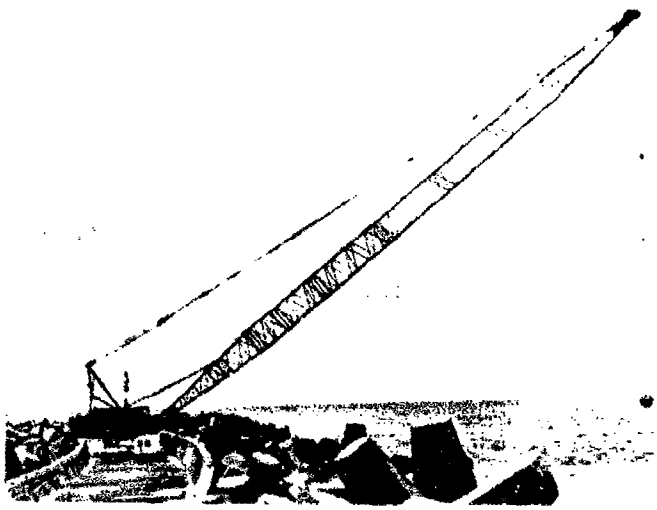
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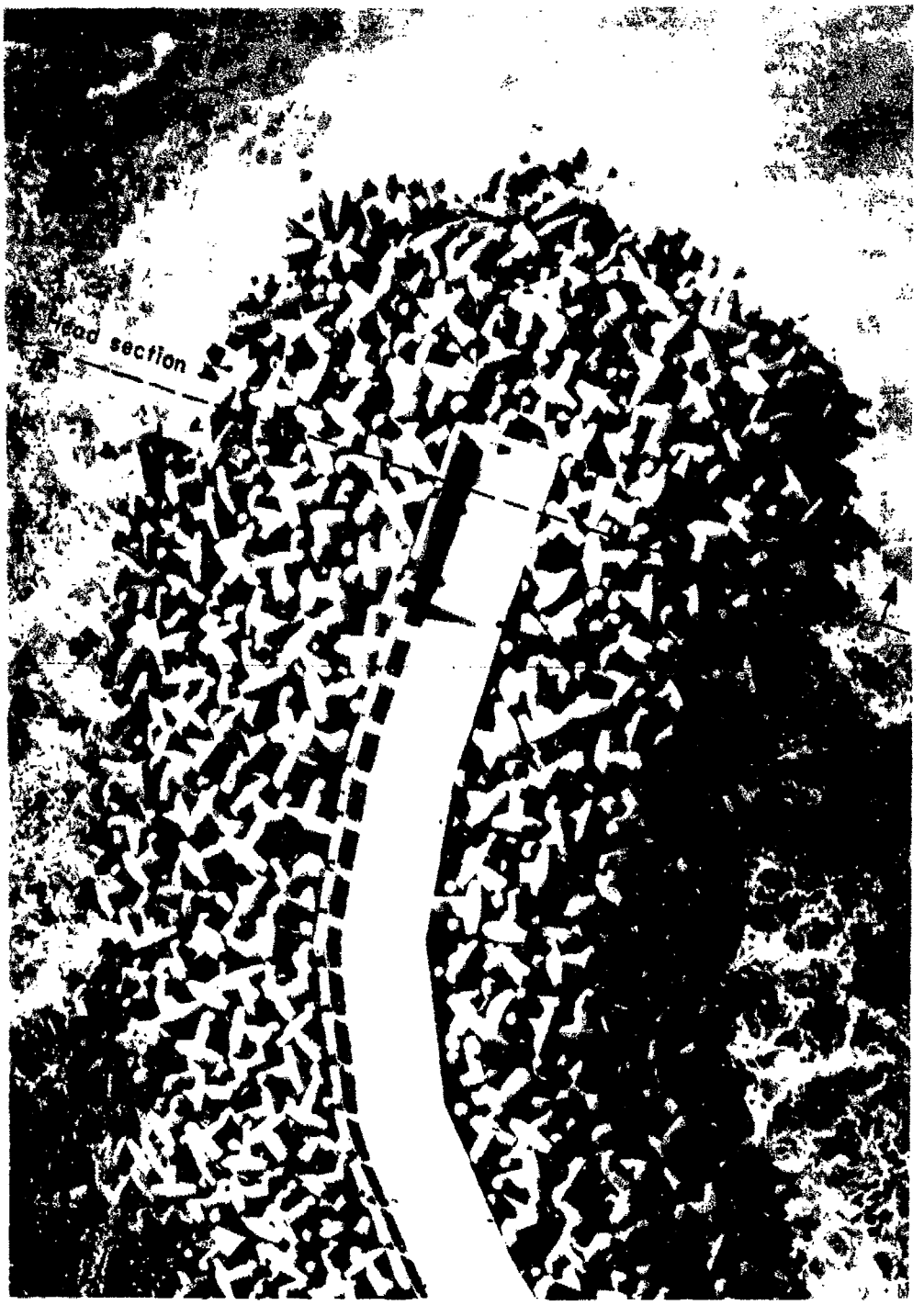
TRACED: JBT CHECKED: DATE: REF:	BEHAVIOUR OF DOLOS STRUCTURES STANDARD DROP TEST ACCORDING TO BURCHARTH	FIGURE 12

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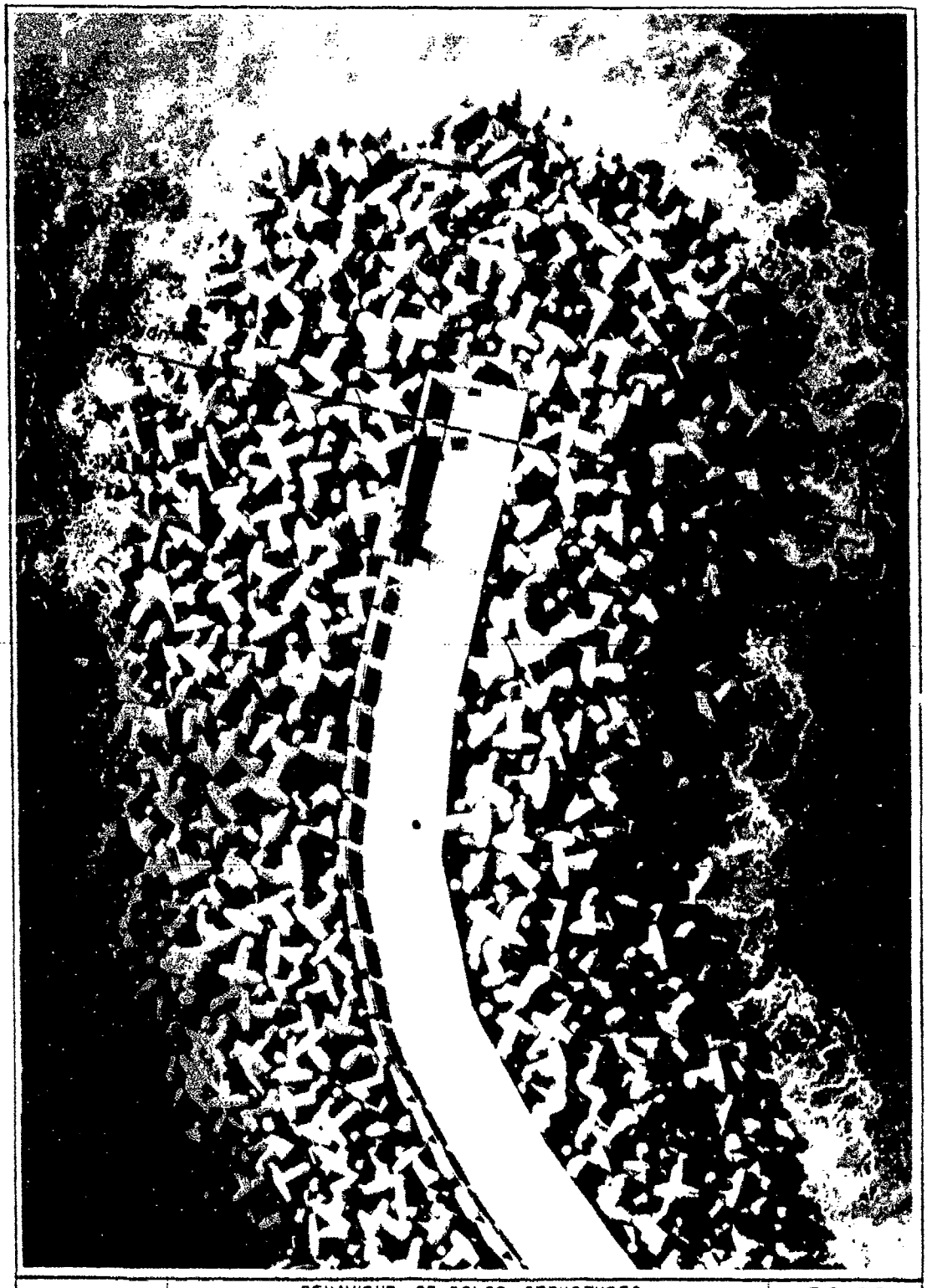




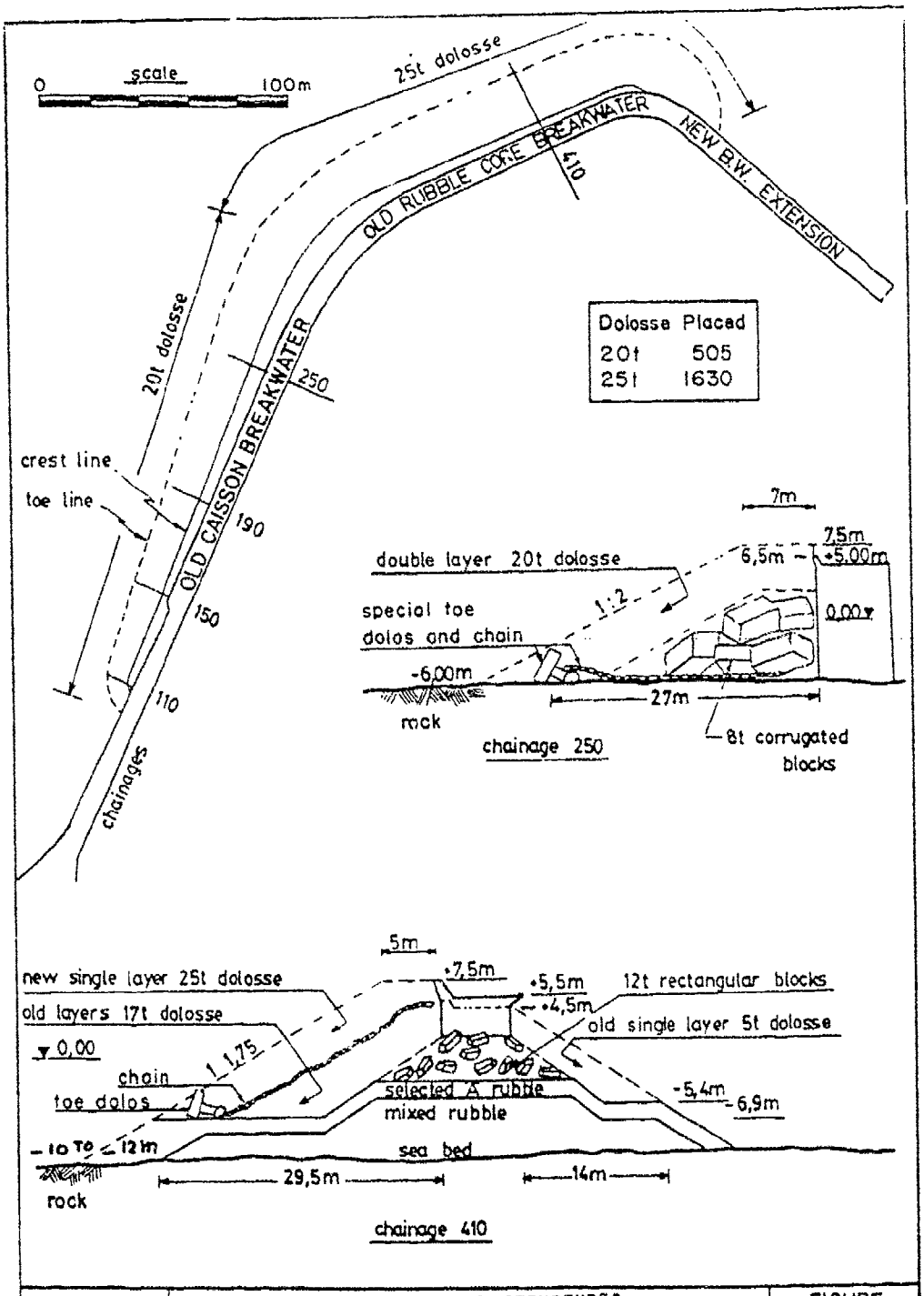
TRACED JM CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES MEASUREMENTS OF BREAKWATER ARMOUR PROFILES	FIGURE 2
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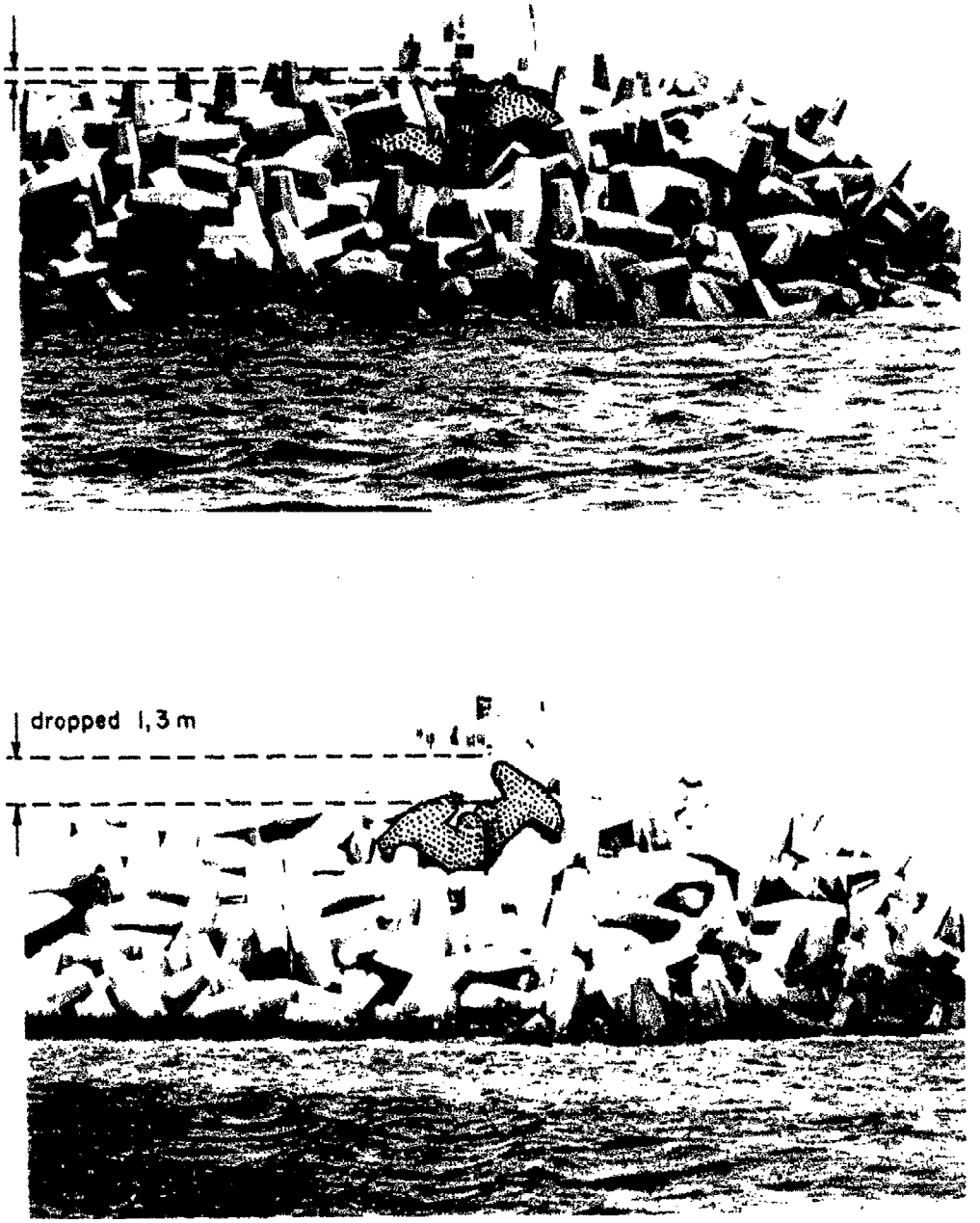
TRACED CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES RICHARDS BAY MAIN BREAKWATER HEAD, ON 18-6-81	FIGURE 3
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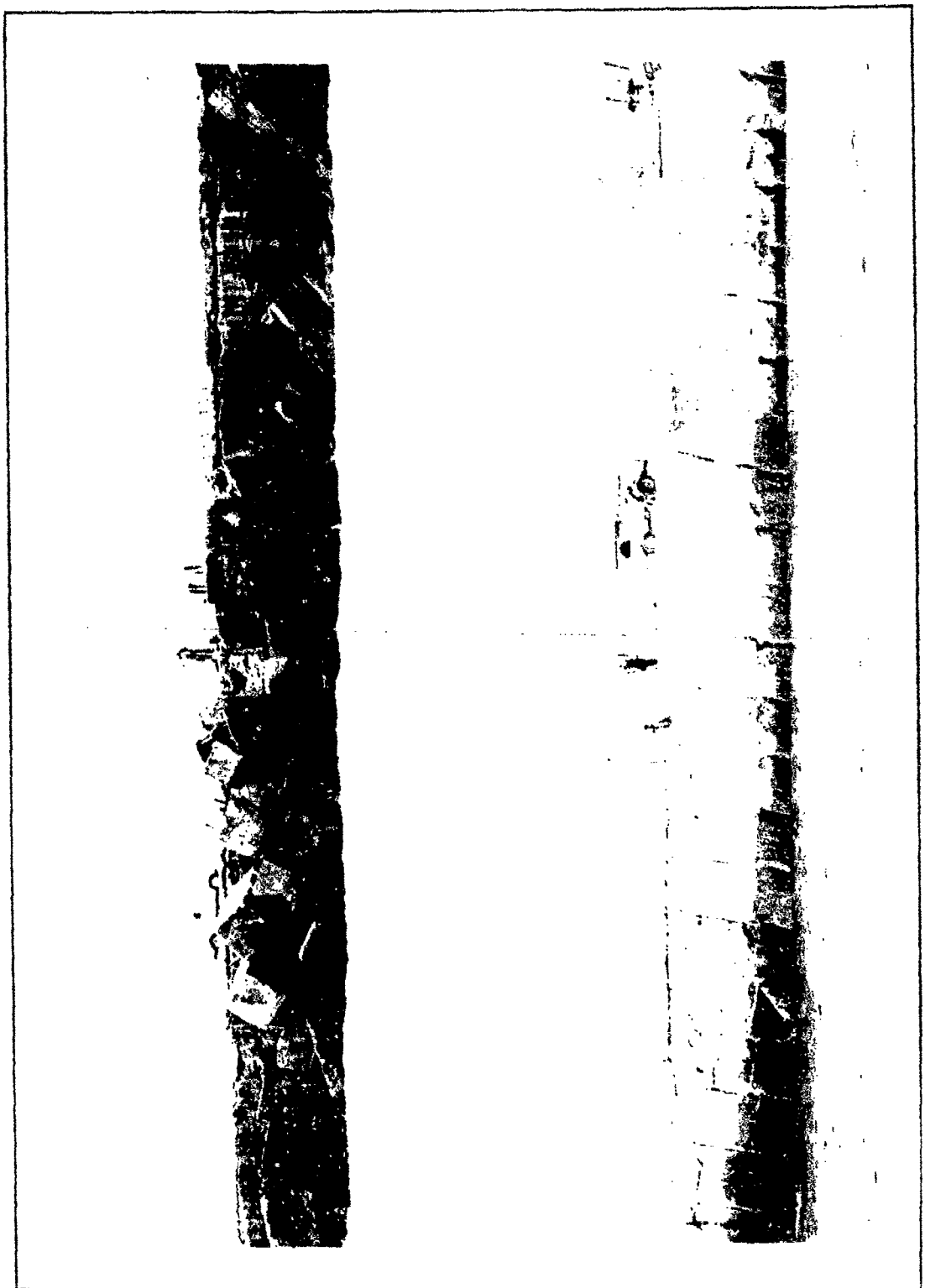
TRACED CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES RICHARDS BAY MAIN BREAKWATER HEAD, ON 5-6-82	FIGURE 4
	NATIONAL RESEARCH INSTITUTE FOR OCEANOLOGY	



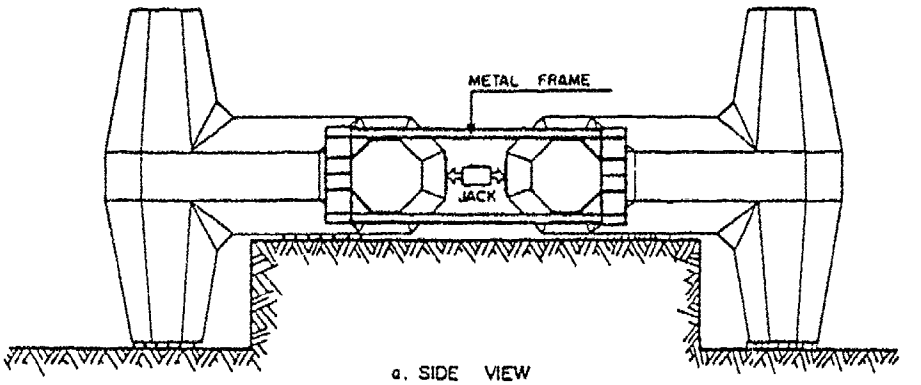
	BEHAVIOUR OF DOLOS STRUCTURES REPAIR TO GANSBAAI BREAKWATER	FIGURE 5
	NATIONAL RESEARCH INSTITUTE FOR OCEANOLOGY	



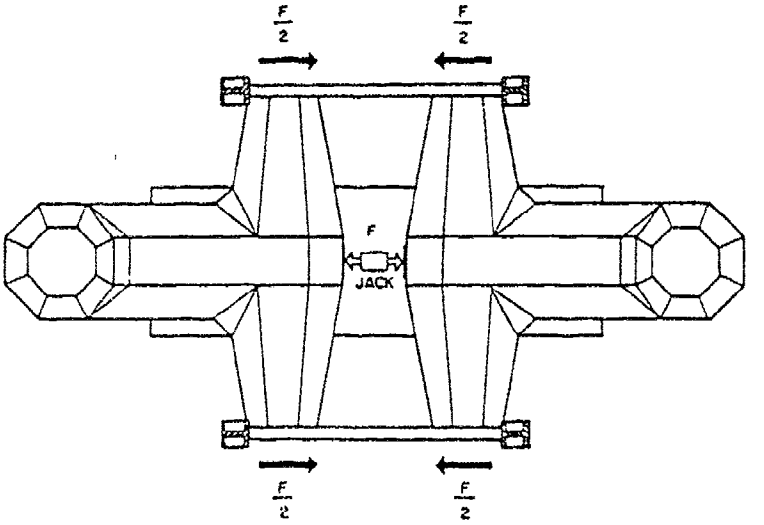
TRACED CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES KOEBERG MAIN BREAKWATER HEAD SECTION NOVEMBER 1980 TO FEBRUARY 1981	FIGURE 6
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TRACED CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES TABLE BAY MAIN BREAKWATER BEFORE AND AFTER MAY '84 STORM	FIGURE 7
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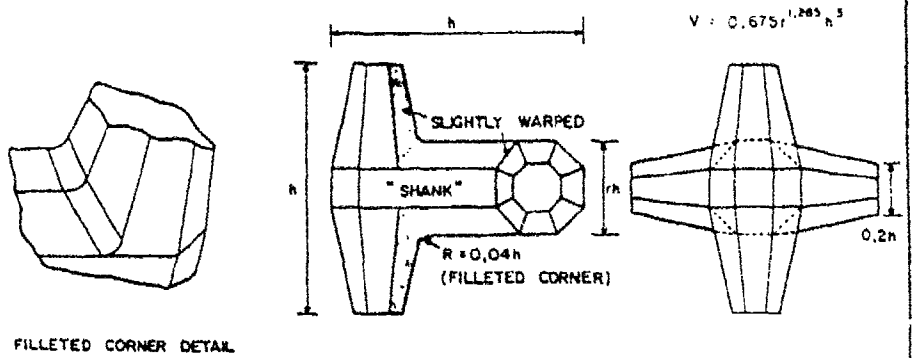
a. SIDE VIEW



b. PLAN VIEW

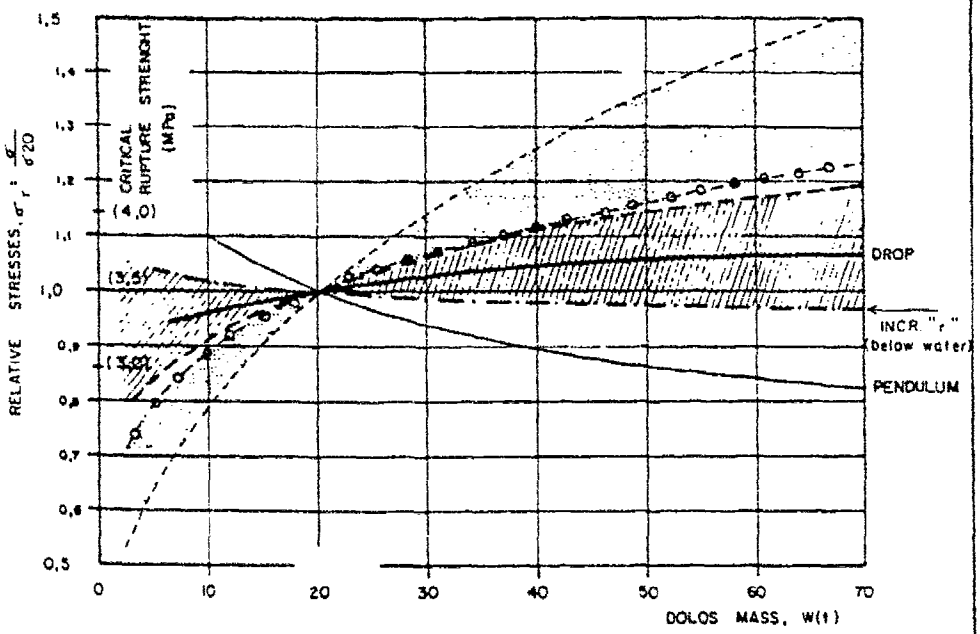
TRACED: HRS CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES STATIC LOAD TEST ARRANGEMENT	FIGURE 8

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FILLETED CORNER DETAIL

a. BASIC DOLOS SHAPE



b. STRESS INCREASES WITH DOLOS MASS

ZWAMBORN ET/AL (1980)

r = CONSTANT (ABOVE WATER)

r = CONSTANT (BELOW WATER)

INCREASED WAIST RATIO (ABOVE WATER)

INCREASED WAIST RATIO (BELOW WATER)

INCREASED WAIST RATIO: $r = 0,34\sqrt[4]{\frac{W}{20}}$

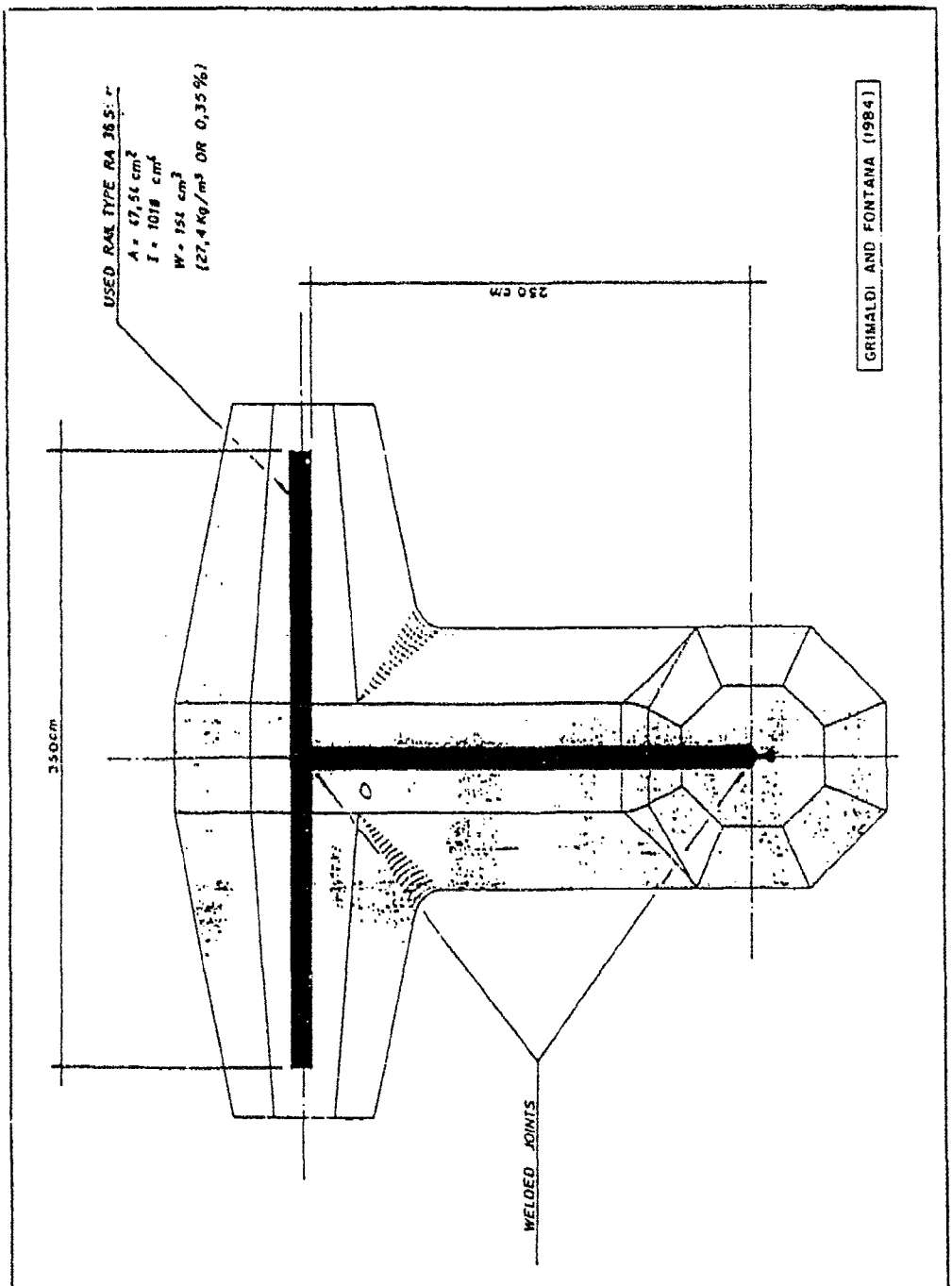
BURCHARTH (1981)

o o o r = CONSTANT (DROP AND PENDULUM TEST)

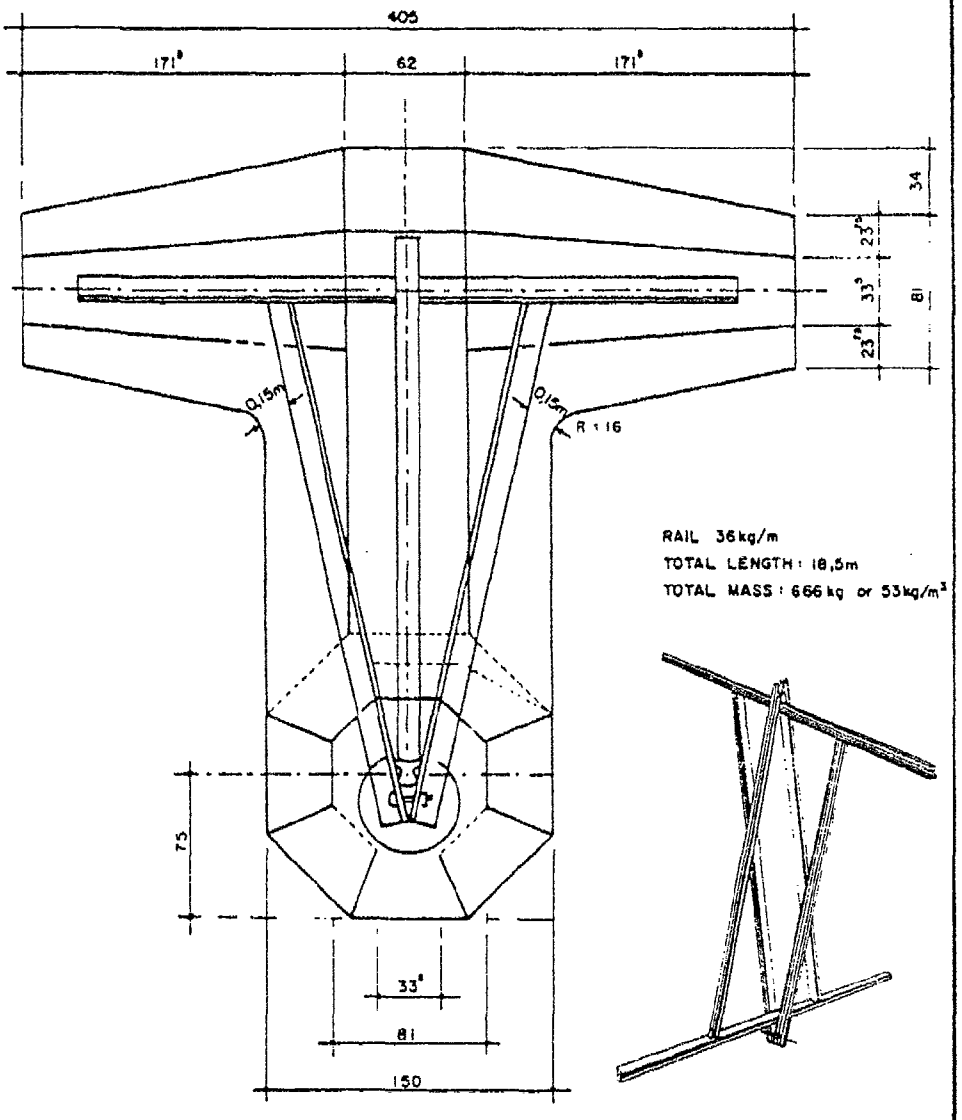
——— INCREASED r (DROP TEST)

——— INCREASED r (PENDULUM TEST)

	BEHAVIOUR OF DOLOS STRUCTURES COMPARISON EFFECTS OF INCREASED WAIST RATIOS NATIONAL RESEARCH INSTITUTE FOR OCEANOLOGY	FIGURE 9

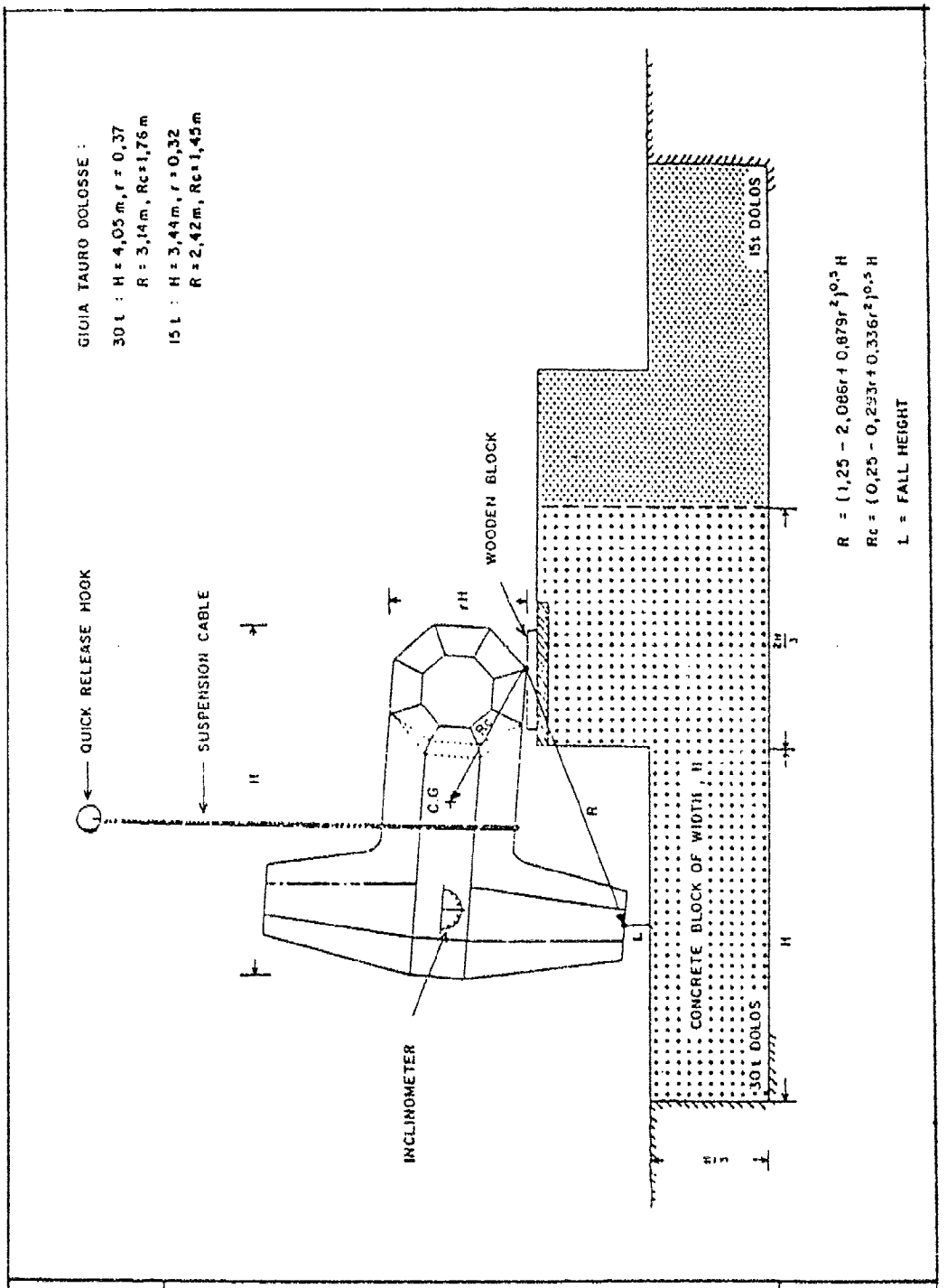


TRACED: HRS CHECKED DATE: REF..	BEHAVIOUR OF DOLOS STRUCTURES SINGLE RAIL-REINFORCED 30t DOLOS	FIGURE 10
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GRIMALDI AND FONTANA (1984)

TRACED CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES DOLOS OF 30t WITH DOUBLE-V RAIL REINFORCEMENT	FIGURE 11
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TRACED. JBT CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES STANDARD DROP TEST ACCORDING TO BURCHARTH	FIGURE 12

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FREE-FALL TEST ON 50-1000kg STONE BED



FREE-FALL TEST, DOUBLE-V REINFORCED DOLOS ON BREAKWATER CORE



DOUBLE-V REINFORCED DOLOS AFTER 8m FREE-FALL

TRACED CHECKED DATE REF	BEHAVIOUR OF DOLOS STRUCTURES FREE-FALL TESTS WITH 30t DOLOSSE AT GIOIA TAURO, ITALY	FIGURE 13
NATIONAL RESEARCH INSTITUTE FOR OCEANOLOGY		

OUTLINE OF STUDY
ON
STRUCTURAL STRENGTH OF ARMOR BLOCKS

May, 1985

Taiji Endo

Hydraulic Laboratory

Nippon Tetrapods Co.,Ltd

INTRODUCTION

Since a water depth at any proposed site for breakwater construction becomes deeper and incoming waves become larger, the weight of armor units to be used to protect the breakwater should be increased. In the case of large armor blocks used in an area of large waves, a structure for the dissipation of waves consisting of large armor blocks would be required to be stable against any wave. At the same time the strength of individual blocks making up the breakwater would become much more important than in the case of small blocks. NTC (Nippon Tetrapod Co.,Ltd.) has conducted studies using samples having different concrete strengths and weights and also concrete samples reinforced utilizing polyethylene fiber to obtain basic data concerning strength properties of Dolosse and Tetrapods.

1. DOLOSSE

Experiments were conducted using samples of reinforced concrete weighing 4 tons and plain concrete weighing 4 tons, 0.4 tons and 0.04 tons respectively. Data was obtained on stress distribution at various points on Dolosse under static and dynamic loads, the influence on the structural strength of Dolosse by changes in sample weight and concrete strength, and impact loads when dropped on a concrete foundation.

The results of the experiments were reported at the 18th International Conference on Coastal Engineering in Cape Town and are given in Attachment 1. Attachment 2 discusses the outline of 40t and 50t Dolosse used in the breakwaters at Naha Port, Okinawa Prefecture. 92 kg $/m^3$ iron bars were used for reinforce -ment.

2. TETRAPODS

Drop tests were performed on unreinforced Tetrapods weighing 4 tons, 0.4 tons and 0.04 tons respectively, of which design standard concrete strengths were σ =210 kg/cm², 300 kg/cm² and 400 kg/cm². Data was obtained on concrete surface strain at the base of the legs, the influence on the structural strength of Tetrapods by changes in sample weight and concrete strength, and impact loads when dropped.

A Drop test on a 4 ton Tetrapod reinforced with polyethylene fiber was also conducted.

The results of the tests are given in Attachment 3.

3. HYDRAULIC TEST USING WEAK STRENGTH MODEL

NTC developed gypsum concrete having a specific gravity from 2.3 to 2.4 with low compressive and tensile strengths. A 1 : 40 model of a 40 ton Dolos made from this material was tested to investigate the resulting damage.

The result of the test was presented in the 31st Japanese Conference on Coastal Engineering in 1984 (the Japan Society of Civil Engineering) and Attachment 4 has the details.

4. MEASUREMENT OF STRESS GENERATED IN BLOCKS UNDER WAVE ACTION

NTC carried out a hydraulic model test utilizing 50 kg Tetrapods jointly with the Central Research Institute of Electric Power Industry using their large wave channel.

Stress measurement was made using strain gauges.

Attachment 5 gives the details of the test.

PROTOTYPE TESTING OF DOLOSSE TO DESTRUCTION

T. Terao[1], K. Terauchi[2], S. Ushida[3]
N. Shiraishi[4], K. Kobayashi[5], H. Gahara[6]

ABSTRACT

The Dolos, a type of armor unit, has been used widely for breakwater and shore protection works in the world. However, it has been reported that the armor layers of several breakwaters have been damaged by wave action, and it is probable that the breakage of Dolos has been the cause of that failure.

In this paper, static and dynamic tests using Dolosse units are described. 4t reinforced units and 4t, 0.4t and 0.04t unreinforced units were used.

In these tests, concrete surface and reinforcing bar stress of Dolos, and impact load were measured.

The results of these tests were as follows:
(1) From the both tests i.e. the static load test and the drop test, stress was greatest in the corner between the chamfer and the stem. Cracks occurred at this point.
(2) In the static load test, comparing the results of both units with reinforced and unreinforced chamfer, it became clear that the reinforcement of the chamfer could reduce the magnitude of the stress concentration.
(3) In the drop test, the drop height which made cracks was almost constantly independent of the weights of the units. And it could be considered that there was little influence of increasing the concrete strength as to the breakage of Dolos

1. INTRODUCTION

The Dolos is a type of concrete armor unit that has a high degree of interlocking capability. Dolosse have been

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[5]Manager of R & D section of Nippon Tetrapod Co., Ltd.
[6]R & D section of Nippon Tetrapod Co., Ltd.

used at many port and harbor locations (1, 2). However, recently, it has been reported that the armor layers of several breakwaters have been damaged by wave action (3), and it has been considered that the breakage of Dolos is one probable cause of this damage. Consequently, the problem related to the structual strength of Dolos has been discussed. O.J. Lillevang and W.E. Nickola (4) examined the stress distribution of Dolos model with some shapes of chamfers under static load by using the three-dimensional photoelastic stress analysis, and suggested the shape of the chamfer to reduce the concentration of the tention stress. H.F. Burcharth (5) did the drop and pendulum tests using 1.5t to 20t Dolosse, and proposed a method for the design of impact loaded Dolosse. C. Galvin and D.F. Alexander (6) proposed a theoretical relationship between wave height and concrete strength of armor units. And there were some papers of tests related to the breakage of Dolosse prior to using them to breakwaters, for example, S. Barab and D. Hanson, C.A. Walter and D.R. Clark (7, 8).

In the case of a composite type breakwater with armor layer which are filled completely with armor units of the same size, it is considered that the lowest units will be subject to the static load caused by the dead weight and the units of the exposed side will suffer from the impact load resulting from rocking.

As armor units in these two situations are prone to some damage, we made static load and drop tests using Dolosse and also measured the stresses in some parts of units.

2. TEST CONDITIONS AND PROCEDURE

Assuming the load conditions, two different types of tests were performed. The static load test was performed to simulate the condition of a dead load of units caused by settlement, and the drop test was instigated to simulate the impact resulting from rocking under wave action. Fig. 1 shows the test methods.

4t reinforced and 4t, 0.4t and 0.04t unreinforced units were used in these tests. The waist ratio was constant at 0.32. Table 1 shows the test program, Fig. 2 shows the geometry of units, and Table 2 shows the mix proportions of concrete. Tensil strength test results of steel bars and bar arrangement drawing are given in Table 3 and Fig. 3, respectively.

In the static load test, the vertical fluke of the unit was fixed by a support equipment. There were two different loading conditions. One was imposed on the mid point of the horizontal fluke and the other was on the tip point. A hydraulic jack was used for loading. Photo. 1 shows the

situation of the static load test.

In the case of the drop test, the horizontal fluke was supported in a way to keep the stem level. Then, the vertical fluke was lifted up to a predetermined height and dropped onto a concrete slab of 1 meter thickness by use of a quick release device. Drop height started at 2 cm and increased every 2 cm. Some of the drop test units were provided with load cells at the bottom of the vertical fluke to measure the impact load. Photo. 2 shows the situation of the drop test.

In both tests, several strain gauges were placed on the reinforcing bar and the concrete surfaces of each test unit in order to measure the strain.

3. TEST RESULTS

3-1 Static Load Test

3-1-1 In the case of imposing a load on the mid point of the horizontal fluke

Stress concentrated on the corner between the chamfer and the stem due to the bending force. Cracks occurred at this point. Photo. 3 shows the breakage of Dolos. From the results of unreinforced units shown in Table 4, it is considered that the ultimate imposed load which caused cracks increased slightly as the compressive strength of concrete increased. Fig. 4 shows the relationship between the concrete surface stress and static load.

In the case of reinforced units, cracks appeared in that corner under the static load which was almost as large as the results of unreinforced units. Fig. 5 and 6 show the stress distribution of the reinforcing bar using the units with the chamfer reinforced and unreinforced, respectively. Stress concentrated on the corner revealing themselves as corresponding cracks.

In the case where the chamfer was not reinforced, the reinforcing bars placed at the stem yielded under a smaller imposed load compared to that of the reinforced chamfer. It is apparent that reinforcement of the chamfer is effective.

3-1-2 In the case of imposing a load on the tip point of the horizontal fluke

The results of cracking were different between 92 kg/m^3 and 151 kg/m^3 reinforcement units.

In the case of the 92 kg/m^3 reinforcement unit, cracks occurred in the corner between the chamfer and the stem, and

progressed toward the stem at 45°. Ultimate breakage was identified as shear rupture due to bending and torsion forces. Photo. 4 shows the cracks of the stem. Fig. 7 shows the stress distribution of the reinforcing bar. From the result of the relationship between the reinforcing bar and static load shown in Fig. 8, the stem and chamfer bars placed at the corner section ultimately yielded at about 170 KN.

While in the case of 151 kg/m^3 unit, cracks appeared in the corner with a small imposed load, and thereafter new cracks occurred and progressed inthe stem at 45°. Ultimate breakage was identified as sheer rupture due to tortion force. Fig. 9 and 10 show the stress distribution of the reinforcing and static load, respectively.

3-2 Drop Test

Cracks occurred in the corner between the chamfer and the stem identical with the results of static load test.

From the results of unreinforced units shown in Table 5, it is considered that the drop height which crack occurs is almost constant independent of the weight of the units and concrete strength. Photo. 5 shows the broken unit.

In the case of the reinforced units, stress concentrated on the corner and cracks occurred at this point, too. But the units didn't separate into two pieces. The stress distribution of the reinforcing bar is shown in Fug. 11.

Impact load and impact time were also measured by using load cells. Fig. 12 shows the relationship between the impact time of the load and the drop height. Fig. 13 shows the relationship between the impact time of the load and the weight of the unit. From these results, it can be assumed that the impact time of the load is almost constant independent of the drop height while using the same weight of the unit, and the ratio of the impact times is almost equal to the ratio of their characteristic length i.e. Dolos height.

From the results of the relationship between the maximum impact load and drop height shown in Fig. 14, it is considered that the impact load is proportional to the square root of the drop height and the ratio of the impact loads is equal to the square of the ratio of their characteristic lengths under conditions of the same drop height.

As the ratio of the concrete surface strain is almost equal to the square of the reciprocal of the ratio of their characteristic lengths under conditions of the same impact

load, the maximum strain on the concrete surface is proportional to the square root of the drop height as shown in Fig. 15. This results in the stress of the concrete surface being constant independent of the weight of the unit under conditions of the same drop height.

4. CONCLUSIONS

Stress distribution, the influence of the concrete strength and weight of unit for the breakage of Dolos, and impact load were obtained through these static load and drop tests/

The result of these tests were as follows:
(1) From the both tests, i.e., the static load test and the drop test, stress was greatest in the corner between the chamfer and the stem. Cracks occurred at this point.
(2) In the static load test, comparing the results of both units with reinforced and unreinforced chamfers, it became clear that the reinforcement of the chamfer could reduce the magnitude of the stress concentration.
(3) In the drop test, the drop height which made cracks was almost constant independent of the weights of the units. And it could be considered that there was little influence of increasing the concrete strength as to the breakage of Dolos.

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2 Sakou, T. Shiraishi, N. and Kobayashi, K. (1981) "JAPANESE PRACTICE WITH CONCRETE ARMOR UNIT", ASCE, New York, New York.
3 Magoon, O.T. Sloan, R.L. and Foote, G.L. (1974) "DAMAGES TO COASTAL STRUCTURES", 14th ICCE, Copenhagen, Denmark.
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5 Burcharth, H.F. "A DESIGN METHOD FOR IMPACT-LOADED SLENDER ARMOUR UNIT" Bulletin NR. 18 Laboratoriet for Hydraulik OG Havnebygning, Aalborg, Danmark.
6 Galvin, C. and Alexander, D.F. (1981) "ARMOR UNIT ABRASION AND DOLOS BREAKAGE BY WAVE-INDUCED STRESS CONCENTRATIONS", ASCE, New York New York.
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8 Walter, C.A. and Clark, D.R. (1981) "TESTING OF PRECAST CONCRETE DOLOSSE ARMOR UNITS", ASCE, New York, New York.

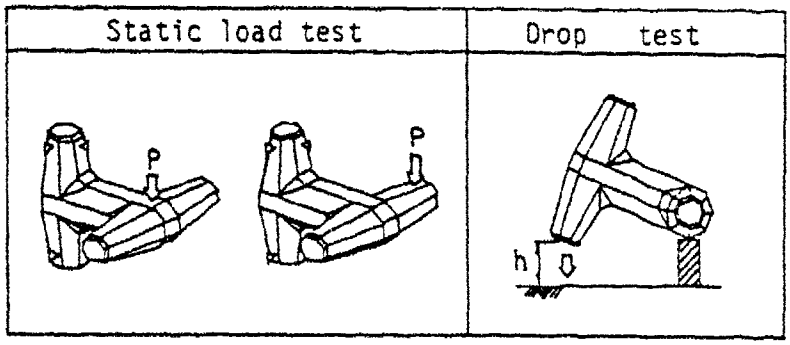
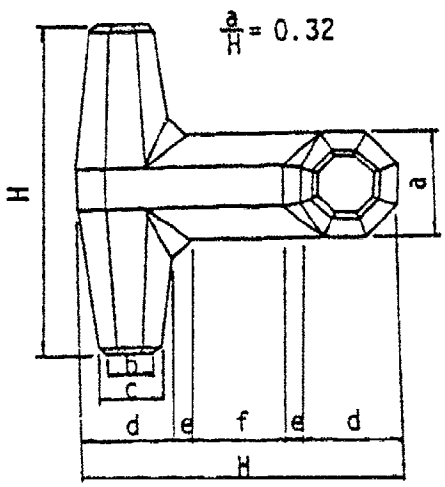


Figure 1 Test Method

Table 1 Test program

Test		Weight of unit (t)	Reinforcement (kg/m³)	Concrete strength (MPa)
Static load test	Imposed on the mid point of the horizontal fluke	4	75	20.6
			92	
			151	
			0	20.6
				29.4
				39.2
	Tip point		92	20.6
			151	
Drop test		4	92	20.6
			151	
			0	20.6
				29.4
				39.2
		0.4	0	20.6
		0.04	0	20.6



Weight (Ω)	H	a	b	c	d	e	f
4	2239	716	386	458	669	125	651
0.4	1038	332	178	212	310	58	302
0.04	482	154	82	98	144	27	140

(mm)

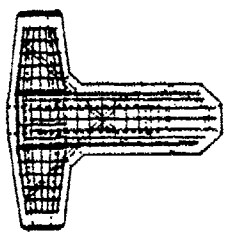
Figure 2 Geometry of units

Table 2 Concrete mixture

Concrete strength (MPa)	Slump (cm)	Max. diameter of agg. (mm)	W/C (%)	S/A (%)	Cement (kg m^{-3})	Water (kg m^{-3})	Sand (kg m^{-3})	Aggre-gate (kg m^{-3})	Additive (kg m^{-3})
20.6	10	25	55.5	38.5	251	139	743	1186	0.628
29.4	10	25	44.5	35.0	320	142	680	1184	0.800
39.2	10	25	34.5	34.5	421	145	612	1162	1.053

Table 3 Test results of reinforcing bar

Standard	Diameter (mm)	Strength	Results (MPa)
SR-24	13	Yield strength	294
		Ultimate tensile strength	428
	16	Yield strength	331
		Ultimate tensile strength	478



reinforcement: 13mm and 16mm bars
concrete cover layer: 65mm

Figure 3 Bar arrangement drawing

Table 4 Static load test results (unreinforced unit)

Weight (t)	Design compressive strength (MPa)	cracking static load (kN)	Breaking static load (kN)
4	20.6	61.7	73.5
	29.4	71.5	80.4
	39.2	80.4	93.2

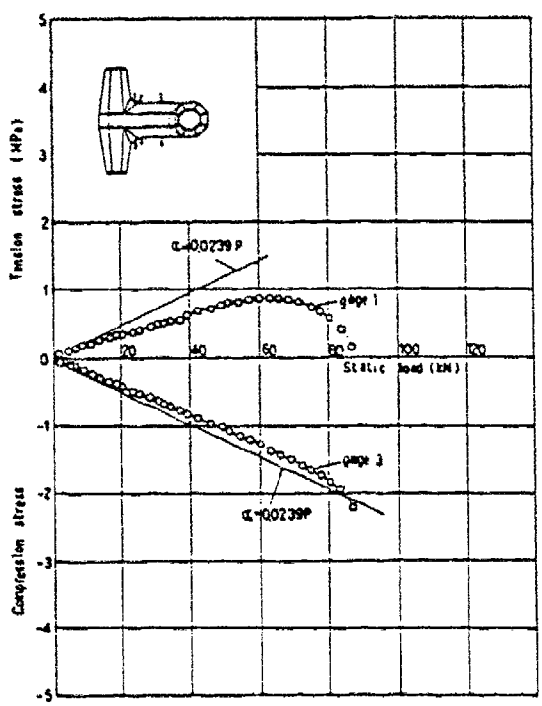


Figure 4 Relationship between concrete surface stress and static load

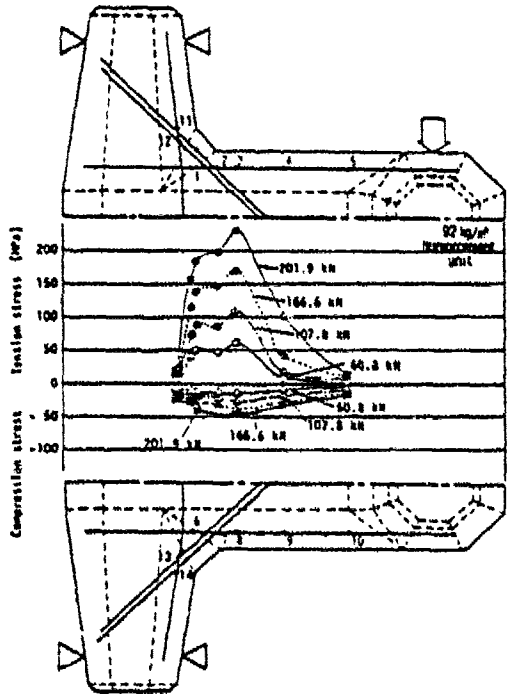


Figure 5 STRESS DISTRIBUTION OF REINFORCEMENT
(Measured on the mid point of the horizontal flute)

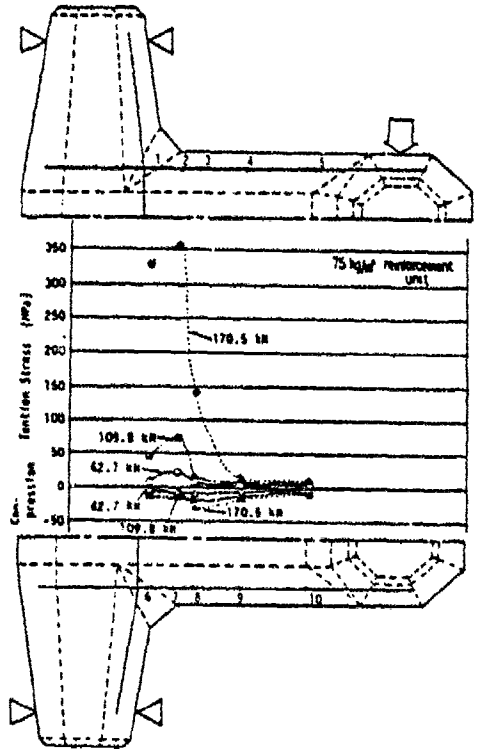


Figure 6 STRESS DISTRIBUTION OF REINFORCEMENT
(Measured on the mid point of the horizontal flute)

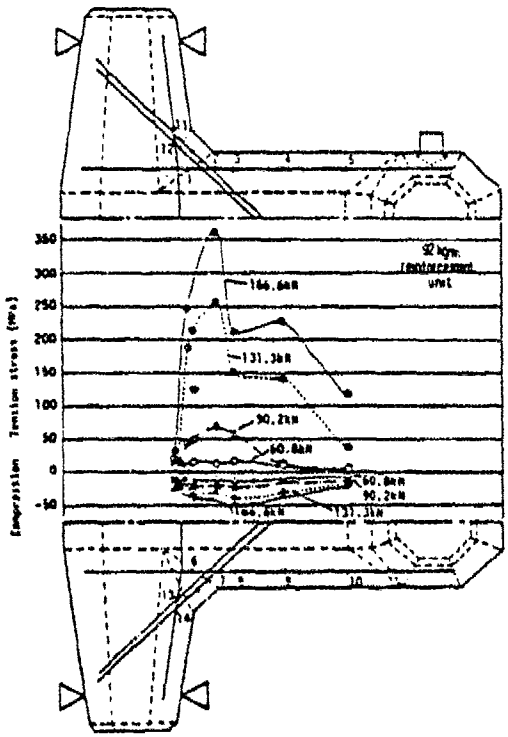


Figure 7 STRESS DISTRIBUTION OF REINFORCEMENT

(Imposed at the tip point of the horizontal flume)

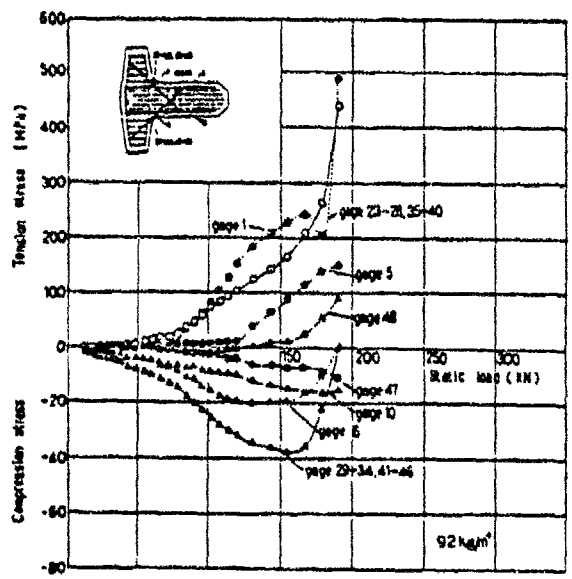


Figure 8 Relationship between reinforcing bar stress and static load

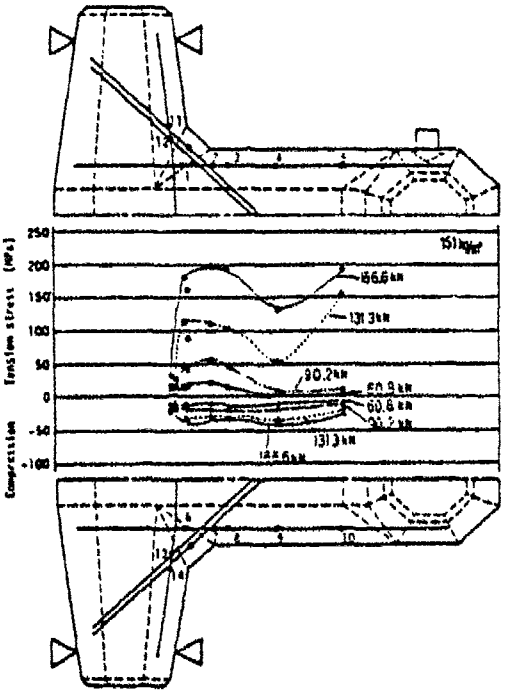


Figure 9 STRESS DISTRIBUTION OF REINFORCEMENT
(measured on the tip point of the horizontal flute)

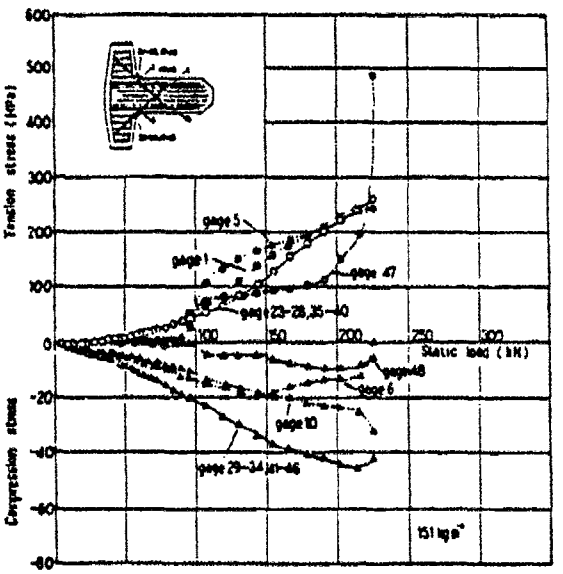


Figure 10 Relationship between reinforcing bar stress and static load

Table 5 Drop test results (unreinforced unit)

Weight (t)	Design compressive strength (MPa)	Cracking drop height (cm)	Breaking drop height (cm)
4	20.6	12	14
	29.4	10	16
	39.2	14	18
0.4	20.6	12	16
0.04	20.6	14	18

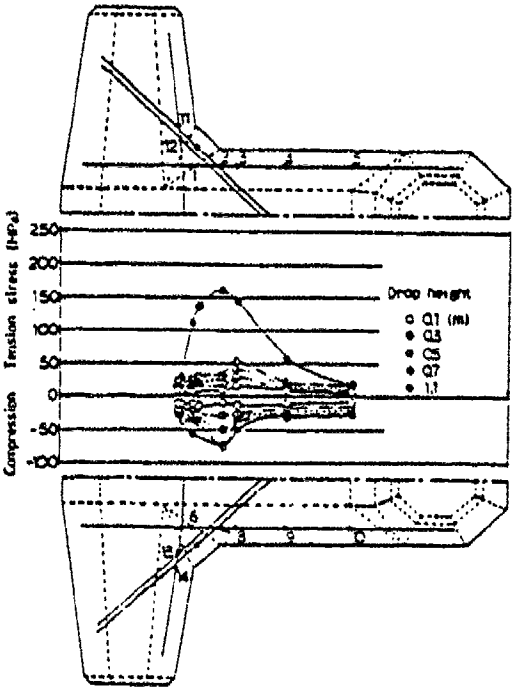


Figure 11 Stress distribution of reinforcement (Drop test)

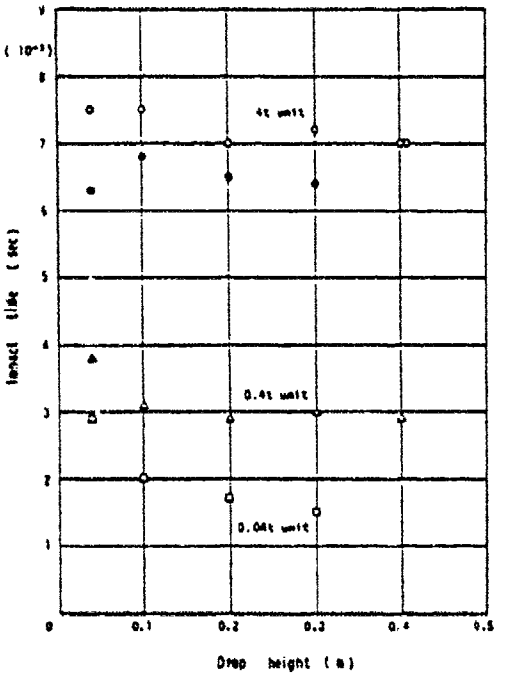


Figure 12 Relationship between impact time and drop height

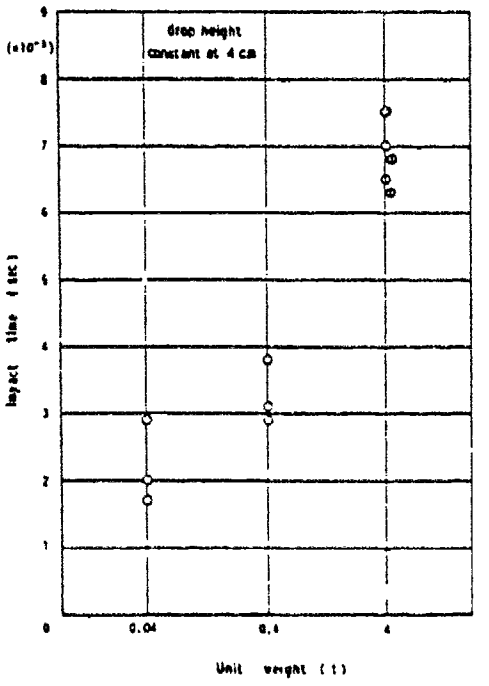


Figure 13 Relationship between impact time and unit weight

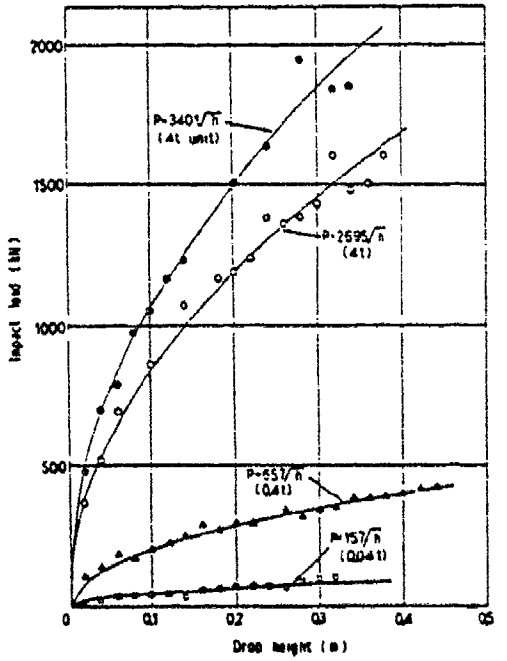


Figure 14 Relationship between impact load and drop height

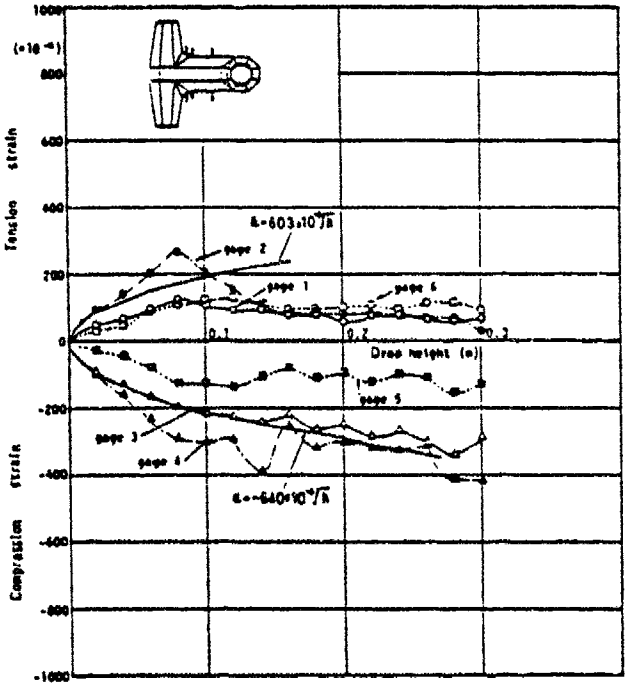
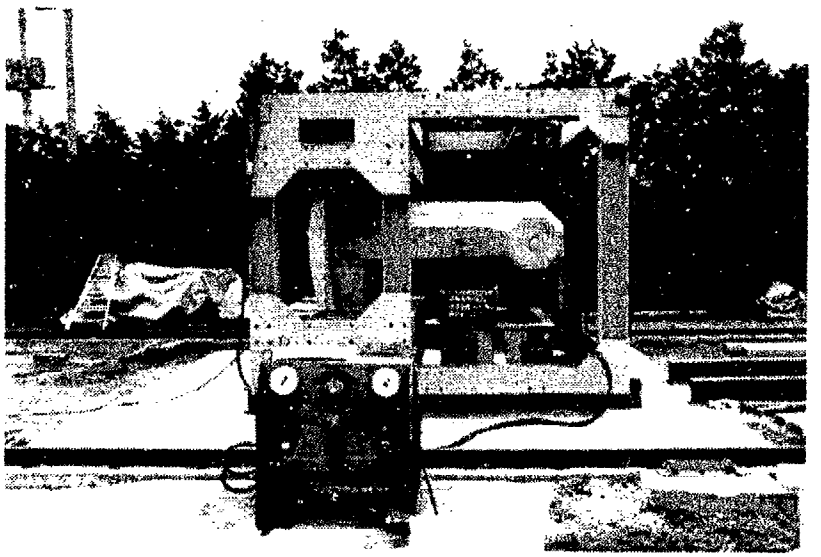


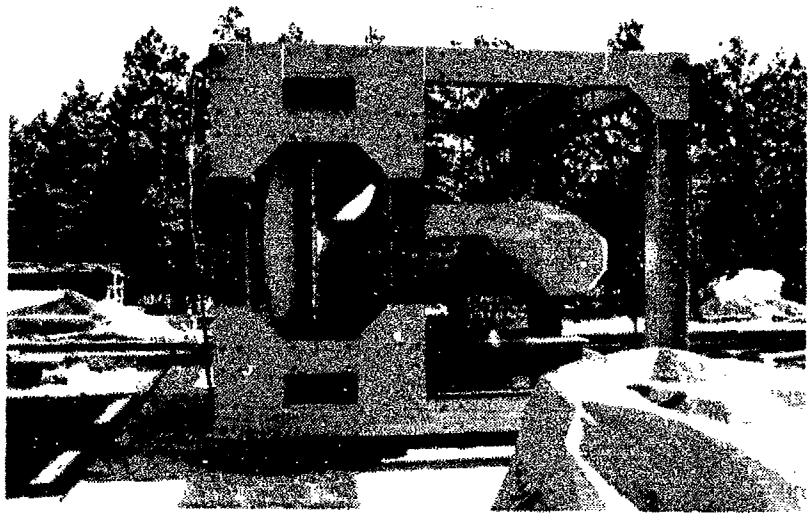
Figure 15 Relationship between concrete surface strain and drop height



Photograph 1: The situation of the static load test



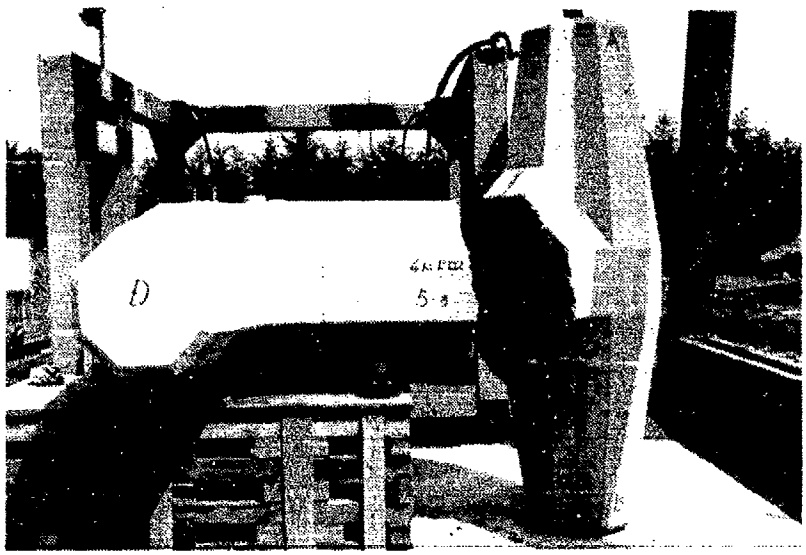
Photograph 2: The situation of the drop test



Photograph 3: The breakage of Dolos (Static load test)

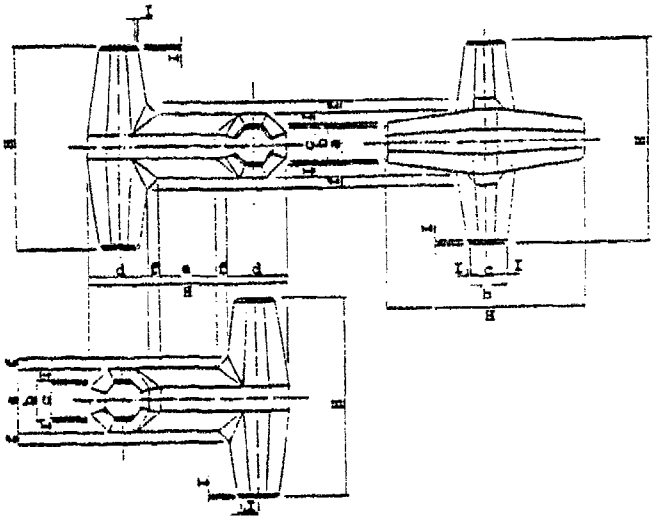


Photograph 4: The cracks of the stem (Static load test)



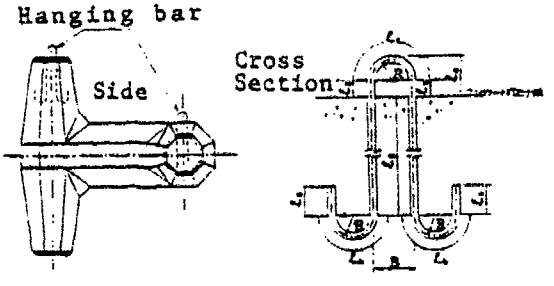
Photograph 5: The breakage of Dolos (Drop test)

DETAIL DESIGN OF REINFORCEMENT OF DOLLOSSE AT NAHA PORT, OKINAWA



Type (t)	Volume (m^3)	Mold Area (m^2)	H (mm)	a (mm)	b (mm)	c (mm)	d (mm)	e (mm)	f (mm)	I (mm)
40	17.391	50.041	4820	1543	986	830	1441	1401	270	78
50	21.739	58.067	5200	1663	1062	894	1552	1509	291	84

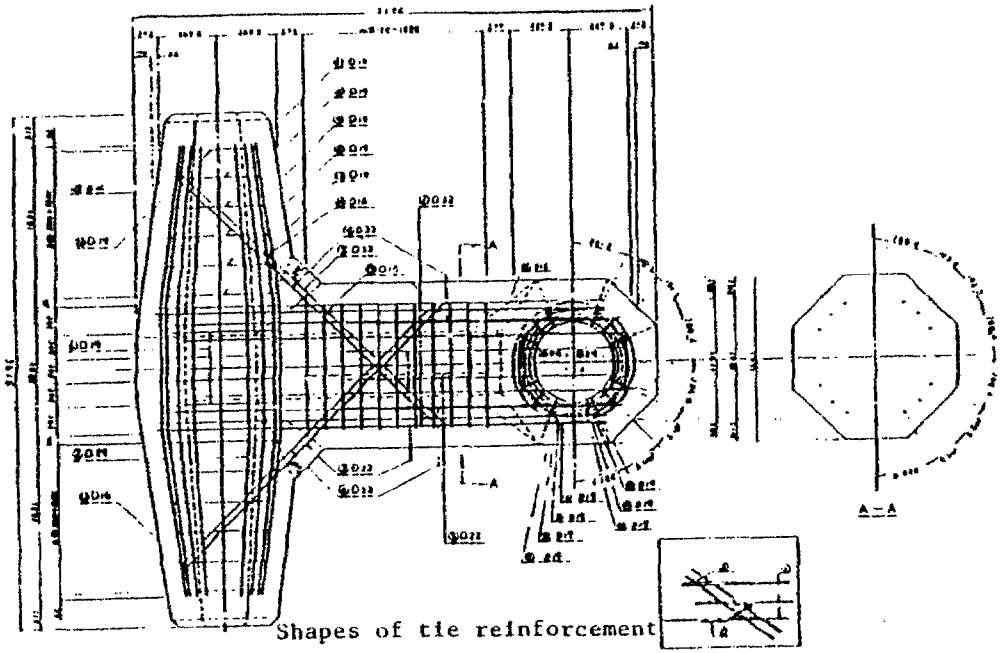
Fig.1 Plan of Dolos.



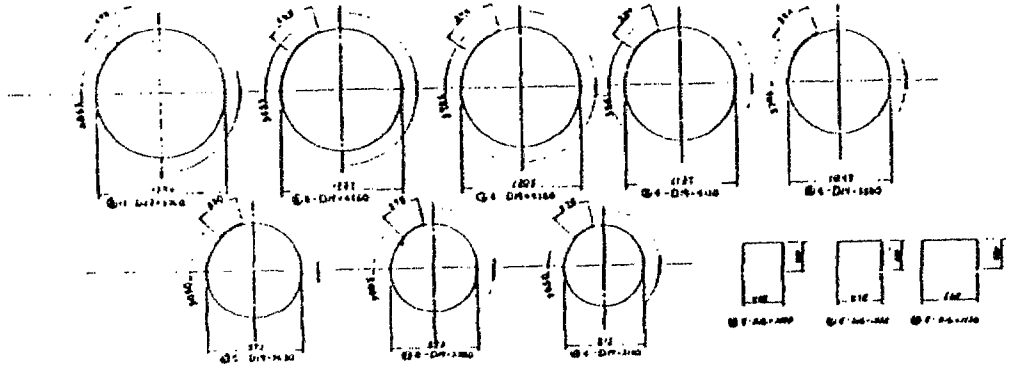
Type	Diameter (φ)	Property	Unit weight Wo	R	B	ℓ_1	ℓ_2	ℓ_3	ℓ_4	ℓ_5	ℓ_6	Length L	Quantity N	Weight of hanging bars W_1	Weight of reinforce-ment W_2	Total weight of bars $W(=W_1+W_2)$
t	mm	JIS G 3101	kg/m	cm	cm	cm	cm	cm	cm	cm	cm	m		kg/N	kg	kg
40	48	ss41	14.2	12.0	24.0	37.7	15.0	130.0	37.7	23.0	29.4	4.50	2	127.80	1594.879	1722.679
50	55	ss41	18.7	14.0	28.0	44.0	17.0	140.0	44.0	27.0	33.8	5.00	2	187.00	2020.512	2207.512

* ss41 : Structural steel limit ultimate tenside strength $\geq$ 41kg/mm^2 (JIS G 3101)

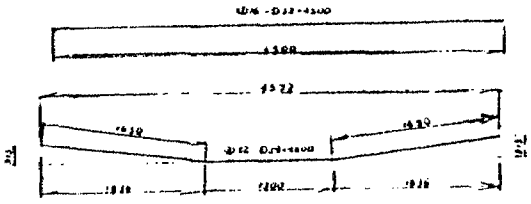
Fig.2 Plan showing hanging bar in Dolos with details of weight..



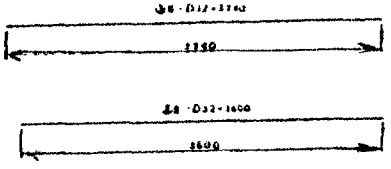
Shapes of tie reinforcement



Shapes of longitudinal reinforcement



Shapes of ③ , ④ reinforcement



Material list

No.	Diameter (φ)	Length (mm)	Quantity	Unit weight (kg/m)	Weight (kg)	Total weight (kg)	Remark
①	D 32	4500	16	6.23	28.035	448.560	
②	D 25	4600	32	5.04	23.184	741.888	
③	D 32	3740	8	6.23	23.300	186.400	
④	D 32	3600	8	6.23	22.428	179.424	
						1556.272 kg	
⑤	D 22	4760	33	3.04	14.470	159.170	
⑥	D 19	4460	8	2.25	10.035	80.280	
⑦	D 19	4380	4	2.25	9.855	39.420	
⑧	D 19	4130	4	2.25	9.293	37.172	
⑨	D 19	3880	4	2.25	8.730	34.920	
⑩	D 19	3630	4	2.25	8.168	32.672	
⑪	D 19	3380	4	2.25	7.605	30.240	
⑫	D 19	3130	4	2.25	7.043	28.172	
						442.226 kg	
⑬	D 16	3000	4	1.56	1.560	6.240	
⑭	D 26	3050	3	1.56	1.838	4.914	
⑮	D 16	3160	4	1.56	1.810	10.850	
						22.004 kg	
*	φ 55	5000	2	18.76	93.500	187.000	
		D 32	814.384 kg				
		D 25	741.888 kg				
		D 22	159.170 kg		2020.512 kg		
		D 19	283.056 kg		(SD-30)		
		D 16	22.004 kg				
		C 55	187.000 kg				
			2207.512 kg				
	Property		JIS G 3112 SD-30				
	Property(Hanging bars)		JIS G 3101 SS-41				

* Hanging bars

Fig.3 50ton Dolos arrangement of reinforcement.

OUTLINE OF TESTS ON TETRAPODS TO DESTRUCTION

DROP TEST FOR TETRAPODS

Plain concrete Tetrapods weighing 4 tons, 0.4 tons, and 0.04 tons having design standard strengths of 210, 300 and 400 kg /cm were used in the tests ; A 4 ton Tetrapod reinforced with polyethylene fiber,of which the design standard strength was 210 kg/cm, was also tested.

Whole blocks were suspended at a predetermined height and then dropped in a manner where their three heels would reach a concrete foundation simultaneously. Strain in the concrete and the resulting damage at various locations was studied. Table 1 shows the test cases and results.

1. PLAIN CONCRETE TETRAPODS

In the case of plain concrete Tetrapods, cracks appeared regardless of the concrete strength. These cracks extended upwards from the locations between each leg in the 4 ton sample and either between each leg or in the center of a leg in the 0.4 ton and 0.04 ton ones. Heels of some Tetrapods landing on the foundation received damage. The drop height causing cracks was approximately 20 cm irrespective of the sample weight and concrete strength. This phenomenon demonstrated that an increase in concrete strength did not significantly contribute toward improving resistance against cracking. The crack width was independent of the weight and concrete strength,and less than 0.01 mm after one drop -- barely detectable using acetone. The crack width, however increased from 0.1 to 0.7 mm with each drop until the block broke. The concrete surface experienced compressive strain on the upper side of the leg and tensile strain on the lower side. The magnitude of the strain was proportional to the square root of the drop height.

Impact time on the 4 ton sample was estimated to be from 0.007 to 0.008 sec from the shock wave profiles obtained from the experimental results. Fig. 1 shows the relation between the drop height and concrete strain.

The frequency of free damped oscillations of the impact strain was constant at 500~600 Hz for the 4 ton sample irrespective of drop height.

concrete strength and location of the gage. It was substantially in inverse proportion to the scale ratio of the Tetrapod height; 900 ~1000Hz for the 0.4ton sample, and 1.700 ~1.800 Hz for the 0.04 ton sample. The damping factor for the samples ranged from 0.01 to 0.05.

The drop height was increased by factors of 2 cm starting from ground level. Please note that these results are different from those obtained in tests where Tetrapods are dropped only one time to find the heights at which cracks first appear.

2. TETRAPOD REINFORCED WITH POLYETHYLENE FIBER

In the case of samples made of concrete containing 1.5% and 3% polyethylene fiber mix respectively, major cracks occurred in the groin or in the center of the base legs with numerous minor cracks all over the legs, but this did not lead to further disintegration the result of the fiber holding the structure together. However slight deformation and increase of the contact area with the ground surface resulted. The positions of the breaks were quite complex, and more significant in the case of the 3% mix. The drop heights at which cracks occurred were approximately constant despite some variation, and the value was about 12 cm irrespective of the mix ratio. This value was insignificant compared with the drop height of 20 cm where cracks appeared in the plain concrete blocks. While the reason for this is not clear,one reason could be that compaction was not sufficient at the time of concrete pouring. Since polyethylene fiber has a stretch modulus of 2.2×10^4 kg/cm, it is not possible to achieve greater drop heights before cracks occur simply by increasing the polyethylene mix ratio.

When the results for the 1.5% and 3% polyethylene content were compared with those of plain concrete the 42 cm and 60 cm break heights showed a significant increase over the 26 cm value found for the plain type.

As shown in Fig.2, the surface strain on the concrete on the tensile side had either the same or a somewhat decreased value with increase in drop

height. However, on the compressive side, the strain increased and behaved as if in 3 stages with increase in drop height. These three stages were ; (1) the stage where concrete resisted the impact force. (2) the stage where polyethylene fiber held together after the crack occurred,and (3) the stage where polyethylene fiber broke or was pulled out.

While polyethylene fiber cannot be expected to increase the resistance to cracking if mixed in the concrete, it can significantly increase resistance to impact and hold together after cracks occur.

Fig.1 Relationship Between Concrete Surface Strain and Drop Height

case	Details Condition	Details on Ground	Test Piece	Design Compressive Strength
4	Equal Impact on 3 legs	W×L×D 5m×6m×1m	4t Tetrapod ⑯-1	300kg/cm

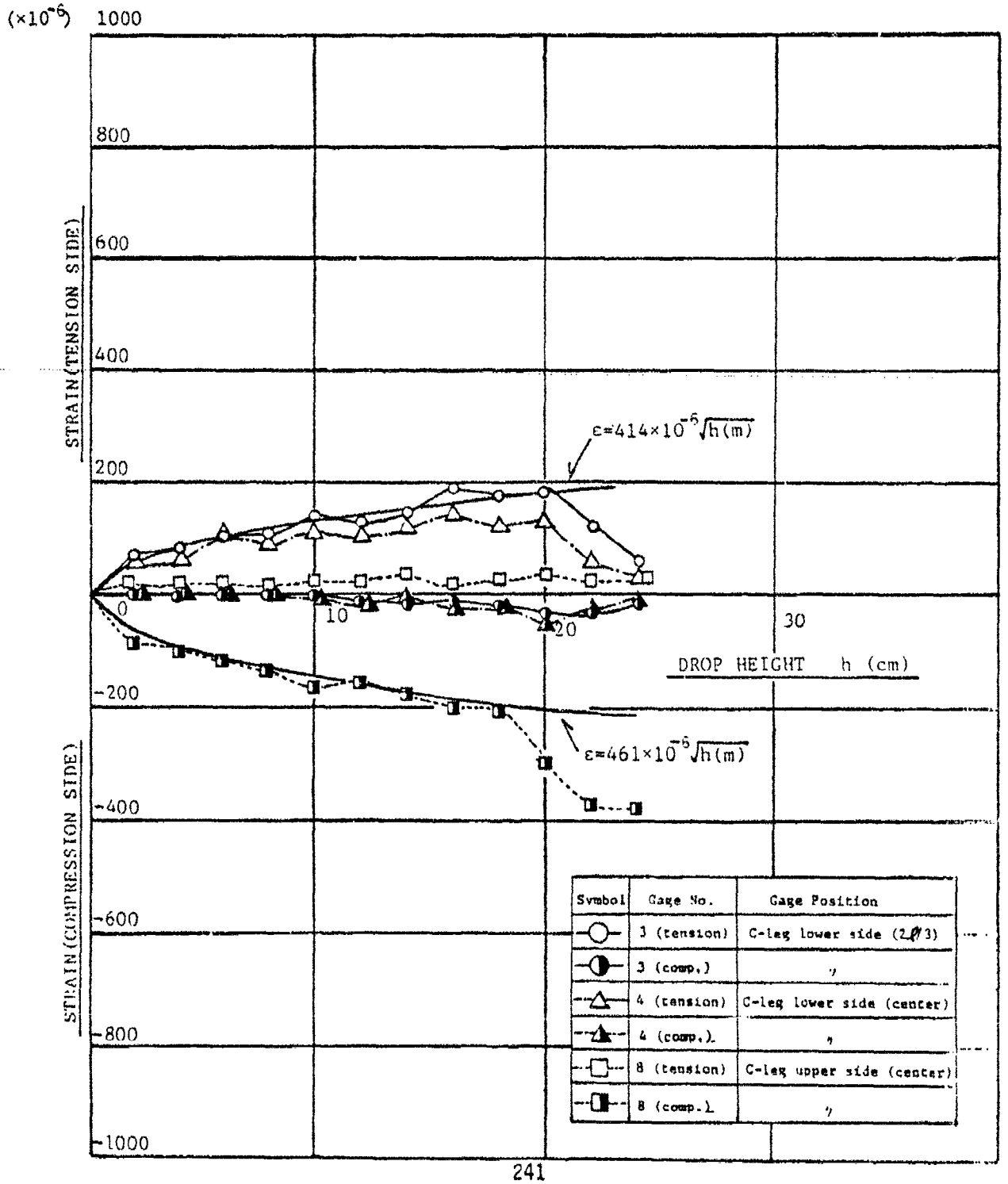


Fig.2 Relationship Between Concrete Surface Strain and Drop Height

Case	Details on Drop	Details on Ground	Test Piece	Design Compressive Strength
4	Equal Impact on 3 legs	W×L×D 5m×6m×1m	4t Tetrapod BF (Vf:3%) -2	210kg/cm

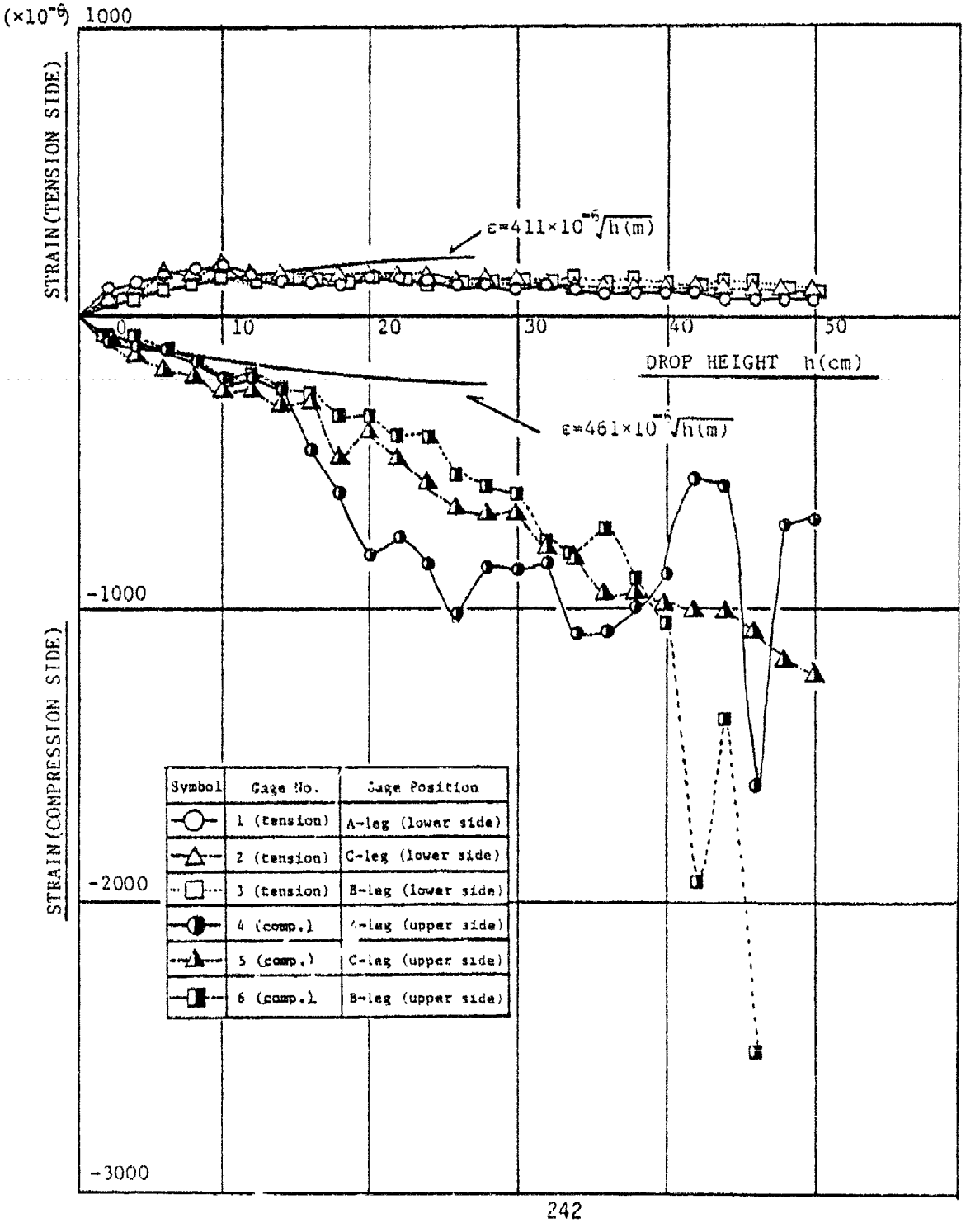


Table-1. Details And Results of Drop Tests

Test Item	Case	Item of Measure-ment	Test Piece	Details on Ground (Concrete)	Details on Drop	Weight of Test Piece	Design Compressive Strength (kg/cm^2)	Details on Test Piece					Test Results	
								Test Piece No.	Load Cell		Strain Gage		Cracking Drop Height (cm)	Breaking Drop Height (cm)
									Present	Absent	Present	Absent		
Impact Test	4	Impact Stress	Tetra -pod	Concrete (5mX6mX1m)	Equal Impact on 3 Legs	4	210	⑮-1					20	24
								-2					18	22
								-3					26	28
								BF**(1.5)-1					16	42
								-2					12	42
								-3					10	42
								BF(3.0)-1					14	54
								-2					10	-
								-3					12	66
							300	⑯-1					18	24
								-2		*		*	22	26
							400	⑰-1					22	28
								-2					22	26
						0.4	210	⑱-1					16	20
								-2					12	16
								-3					18	26
								-4					12	24
							400	⑲-1					16	22
								-2					24	28
						0.04	210	⑳-1					25	29
								-2					16	20
								-3					22	28
								-4				*	18	30
							400	㉑-1			*		20	28
								-2					20	34

** BF: Polyethylene fiber

HYDRAULIC TEST ON STRENGTH-REDUCED MODEL TO INVESTIGATE DAMAGE

1. PURPOSE

Discussion on the structural strength of large concrete armor blocks prompted by the destruction of a Dolos Breakwater at Sines, Portugal among those concerned both in Japan and abroad. In order to find a solution to this problem, the authors tried to reproduce similar damage to concrete armor blocks by hydraulic model testing.[1)]

The authors then developed a material with low strength (the term strength refers herein to both compressive and tensile strength) of a specific gravity 2.3~2.4, and conducted hydraulic testing using armor units made of this material. The mechanism of damage was subsequently investigated.

2. MATERIALS AND METHODS

The study consisted of roughly two parts;
(1) development of the strength-reduced material, and (2) hydraulic model testing using armor units made from the material produced. In developing the material, the authors aimed at reducing the overall strength by decrease of the bond strength between aggregates and matrix and the strength of matrix per se. The mix ratio was adjusted to maintain a specific gravity of 2.3.

This material was used to make 1 : 40 scale models of 40 ton Dolos blocks. A composite breakwater section consisting of composite armor units (a caisson) was built in a test channel using these strength reduced blocks. The damage suffered by individual blocks and the accompanying deformation of the structure was observed visually in order to evaluate the process and structure deformation by waves.

3. DISCUSSION

3-1. Properties of Stength-reduced Material

From the preliminary investigation, the authors decided to use copper as the main component for strength-reduced material, and gypsum plaster and water for matrix. Various degrees of strength to the material could be obtained while maintaining the specific gravity at 2.3 by controlling the mix ratio. Naturally the material did not dissolve in water after hardening. The final strength was achieved in about 7 days if subjected to high temperature curing in the atomosphere. The strength, though slightly, increased over an extended period of time.

The static modulus of elasticity (Em) of the strength-reduced material was approximately 7.2×10^3 kg/cm² (=720 MPa) while that of the elasticity (Ep) of the general purpose cement concrete was about 2.0×10^5 kg/cm² (=19.6 GPa). Therefore, the relation

$$Em/Ep \fallingdotseq 1/28$$

held. This is somewhat greater than the target value of 1/40. The Poisson ratio (μp) for the strength-reduced material was $\mu m \fallingdotseq 1/2.5$, while that of the cement concrete 1/5 to 1/7. Thus the following relation held.

$$\mu m / \mu p \fallingdotseq 2$$

i.e. it was twice the target value 1.

3-2. Result of Hydraulic Model Test

Cracks appeared in one of the Dolosse in the bottom layer of the structure due to its own weight combined with that of other blocks placed above in the structure after construction had been completed but before any wave testing had started. This was because the Dolos was sitting in a position that

received torsional stress.

Damage and cracks in the blocks were conspicuous in the bottom layer at the front of the breakwater. The crown height decreased by about 4 cm (1.5 m in the prototype) because of an overall shrinkage in the cross section and damage to the blocks. The wave height in front of the breakwater was H = 15.5 cm (6.2 m in the prototype) at maximum. The unit weight of the Dolosse used was found to be excessive against this wave height in view of the stability. The material developed was weak versus repeated loads.

Although the result of the hydraulic model test on the developed material could not be applied to the site, the results at least indicated the direction for future study. For example it would be possible to ascertain the position in a cross section where a Dolos could receive damage as well as assess its properties. It would be also possible to estimate potential overall damage resulting from the breakage of several Dolosse and compensate for it.

4. REFERENCE

1) Kazûo Yamada and Hiroshi Okamoto : Hydraulic Test on Strength-reduced Model to Investigate Damage. Proc. of 31st Japanese Conf. on Coastal Engg., 1984. pp. 557~561. (In Japanese) .

Table 1 Result of physical property measured

Item	Expected	Measured	Note
Specific Gravity (ρ)	2.3 ton/m³ (22.54 KN/m³)	2.305 ton/m³ (22.59 KN/m³)	
Compressive Strength (σc)	4.5 kgf/cm² (441 kPa)	4.55 kgf/cm² (446 kPa)	
Tensile Strength (σt)	0.45kgf/cm² (44.1 kPa)	0.58 kgf/cm² (56.8 kPa)	$\sigma t = \frac{1}{10} \sigma c$
Young's Modulus (E)	5.0×10^5kgf/cm² (490 MPa)	6.74~8.06 $\times10^5$kgf/cm² (664~790 MPa)	
Poisson's Ratio (μ)	0.2	0.35~0.49	$\mu = \frac{\varepsilon v}{\varepsilon t}$

εt : Transversal Strain εv : Vertical Strain

Table 2 Details on hydraulic test conditions

Wave period	2.0sec (12.7 sec)			
Wave height	① 7.8cm (3.1m)	② 14.3cm (5.7m)	③ 16.0cm (6.4m)	④ 15.0cm (6.0m)
Water depth at a toe of the slope	28.8cm (11.5m)			
Height of the crown of the structure	12.0cm (4.8m)			
Armor blocks	625g Dolosse (40t Dolosse)			

* ( ) ; In the prototype

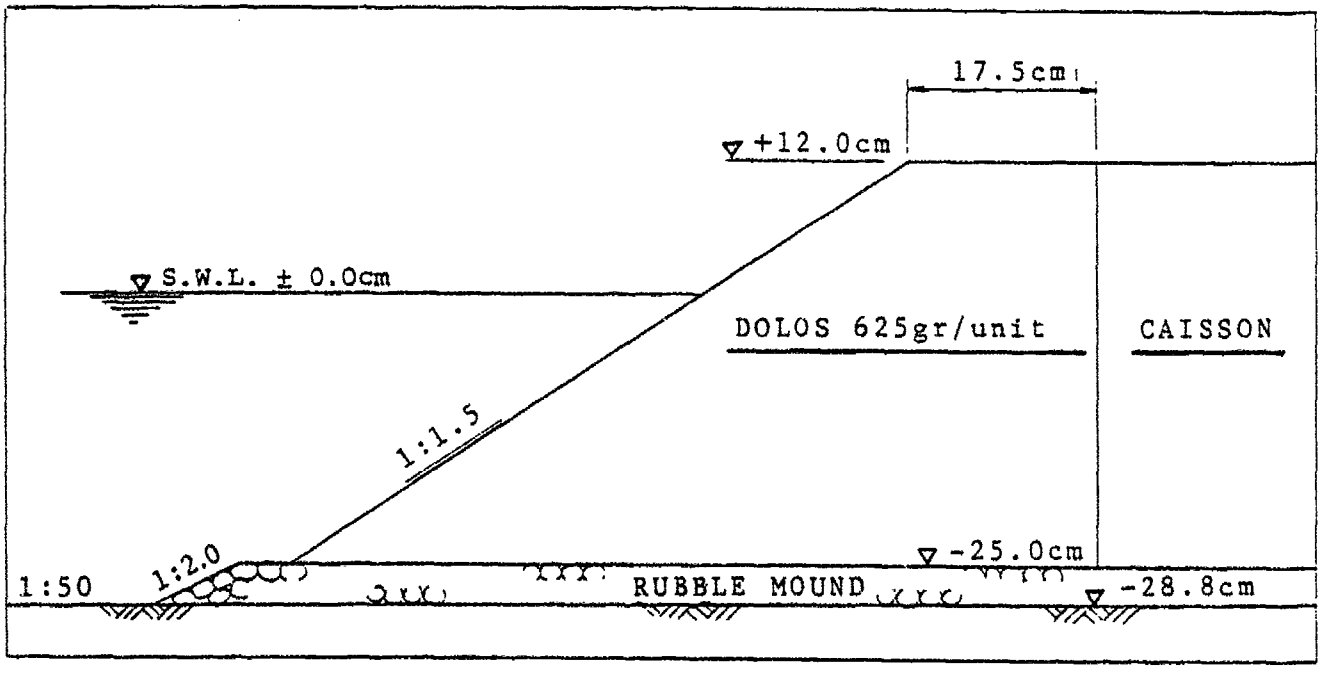


Fig.1 Hydraulic test section

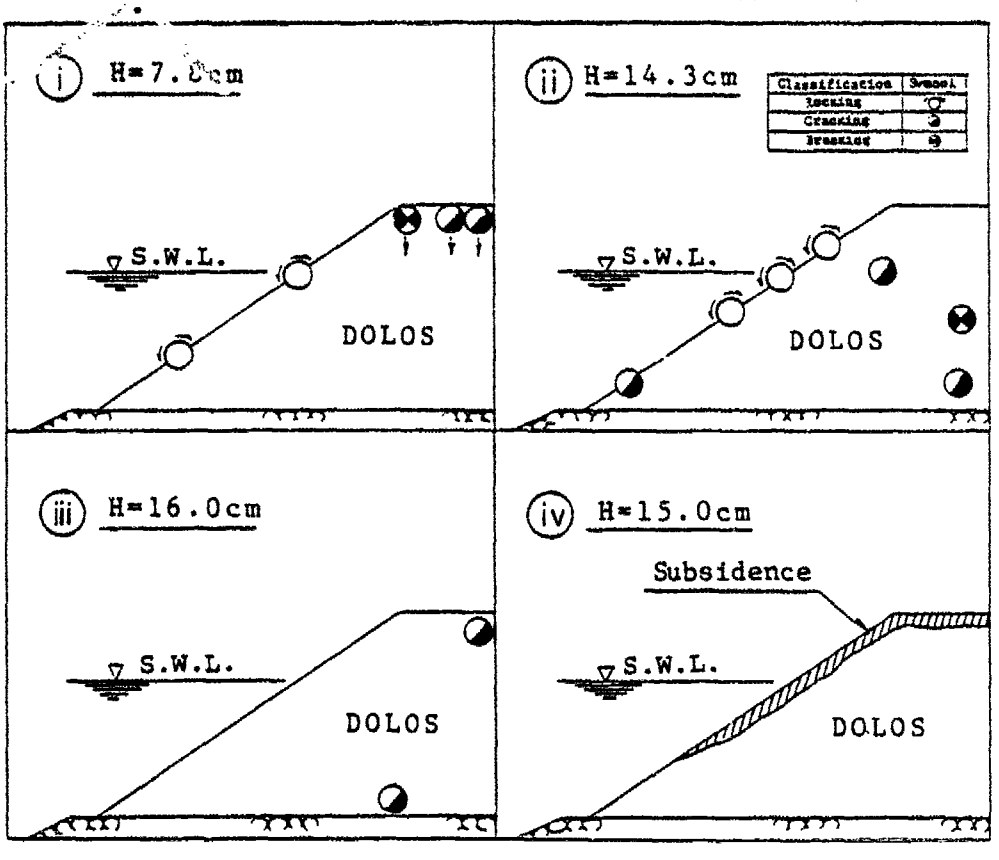


Fig.2 Result of observation during hydraulic test

Attachment 5

EXPERIMENT TO MEASURE STRESS IN TETRAPODS UNDER WAVE ACTION

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T.Endo[4], Y.Noguchi[5] and A.Matsumoto[6]

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2 Manager, Structural Hydraulics Section, Environment Department, Civil Engineering Laboratory, Central Research Institute of Electric Power Industry
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4 Deputy General Manager, Hydraulic Laboratory, Nippon Tetrapod, Co., Ltd.
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6 Engineer, Akita Branch, Nippon Tetrapod Co., Ltd.

PREFACE

Recently, rubble mound breakwaters constructed in deep water have received damage, e.g. the breakwater in Sines, Portugal. Coastal engineers are interested in the details, so in 1983, a Working Group was set up with in the Permanent Technical Committees of PIANC to find the cause of damage and produce countermeasures.[1)]

Concerning the damage to the breakwater in Sines, the results of research organizations in many countries suggested that the problem could be to do with structural strength of Dolos units as well as hydraulic stability of the breakwater against waves.[2)]

In order to study stress occurring in blocks, Terao et al[3)] carried out static load tests and drop tests using prototype Dolosse while Burcharth[4)] performed drop tests incorporating a pendulum device.

Delft Hydraulics Laboratory[5)] and the National Research Council of Canada[6)] measured stress in model Tetrapods and Dolosse respectively made of new model materials under wave action using new measuring techniques.

Meanwhile, the Waterways Experiment Station, U.S. Army Corps of Engineers, held an international Work Shop in order to establish design standards for structural strength of Dolosse and plans to measure stress in 50t Dolosse in the breakwater under wave action in Crescent City.

In this hydraulic model test, in accordance with these trends, measurements of stress in blocks were made to examine structural strength and produce data on the design standard for W.E.S.

This hydraulic model test was conducted using 50kg Tetrapods, which were selected based on a similitude relationship of an impact force on concrete blocks, and an attempt to determine the behavior of Tetrapods under conditions of wave heights greater than 1.1m utilizing the large wave flume of the Central Research Institute of Electric Power Industry.

This was the first time anywhere that use of such a large wave channel for stress measurement was tried and it is expected that this is a precursor to many tests on conditions of waves and structures. The authors hope this study will help to understand better the subject of blocks and contribute to the W.E.S. program.

1. INTRODUCTION AND OBJECTIVES

In recent years very large armor blocks have been used for many coastal structures. However surface stress in large blocks is very complicated under wave action. If stress in blocks became clarified , the results could be applied to design of structures utilizing such large blocks.

From this view point,a hydraulic model test was conducted to measure the surface stress in blocks under wave action.

In the test section,50kg Tetrapods were placed in two layers at random and surface stress in 2 Tetrapods in the vicinity of each other just above the still water level was measured using strain gages.

Usually scaled down Tetrapods are selected for model testing but because of the difficulty in reducing mass density,modulus of elasticity and coefficient of restitution in accordance with similutude relations between model and prototype , concrete Tetrapods weighing 50 kg were utilized in this special large scale test.

During the test,block behavior was observed visually and on video in order firstly to determine any relation between the wave action and block motion and secondly block motion and stress generated.

2. LAW OF SIMILITUDE

A similitude relation on an impact force is discussed hereinafter.

When a block collides with other blocks or a solid body, the impulse induced by the impact force is assumed to be equal to the change in momentum, so the following equation is validated.

$$\int_0^{\tau} Pd\, dt = m\ (v-v') \qquad (2\text{-}1)$$

where Pd ; impact force

v,v'; velocity of the block just before and after collision respectively

e ; coefficient of restitution

m ; mass

τ ; duration of impulse

Burcharth[4] and Aoyagi[7] showed that the duration of an impulse is the time which elapses for a longitudinal shock wave to travel from the point of impact to a free edge of the concrete block and back again.

From eq. (2-1)

$$Pd \propto \frac{mv\ (1+e)}{\tau} \qquad (2\text{-}2)$$

is derived, so the impact force is proportional to the speed of collision.

In this test, the motion of Tetrapods is assumed to be governed according to Froude's law,

$$vm/vp = \lambda^{1/2} \qquad (2\text{-}3)$$

where λ is a linear scale of a model Tetrapod and the subscripts m and p refer to the model and prototype.

As concrete was used for the model material, the modulus of elasticity, density and coefficient of restitution are the same as those of the prototype. So ;

$$Em/Ep=1 \qquad (2\text{-}4)$$

$$\rho m / \rho p = 1 \qquad (2\text{-}5)$$

$$em/ep=1 \qquad (2\text{-}6)$$

Therefore the relations ;

$$Cm/Cp = \frac{Em/\rho m}{Ep/\rho p} = 1 \qquad (2\text{-}7)$$

$$\tau m / \tau p = \frac{lm/Cm}{lp/Cp} = \lambda \qquad (2\text{-}8)$$

hold, where C shows the speed of the longitudinal wave in the concrete and l indicates the distance between the impact point and a free edge of the block.

From the equations above, the similarity ratios of the impact and elastic forces are obtained as follows.

$$Pdm / Pdp = \lambda^{5/2} \qquad (2\text{-}9)$$

$$Pem / Pep = (\sigma m / \sigma p)\ \lambda^2 = (\varepsilon m\ Em / \varepsilon p\ Ep)\ \lambda^2$$

$$= (\varepsilon m / \varepsilon p)\ \lambda^2 \qquad (2\text{-}10)$$

where Pe ; elastic force

σ ; stress

ε ; strain

The similitude relations of the stress and strain between the model and prototype under wave action are given by the following formula, as eq. (2-9) is equal to eq. (2-10) .

$$\sigma m / \sigma p = \varepsilon m / \varepsilon p = \lambda^{1/2} \qquad (2\text{-}11)$$

3. MODEL OF TETRAPOD

3-1 Weight of Tetrapod Used

The results of the drop test carried out by Nippon Tetrapod Co.,Ltd. (hereinafter referred to as N.T.C.) using from 736g to 4t Tetrapods are shown in Fig.3-1. The following equation should be valid in order for the similitude relation mentioned in the previously chapter to hold true.

$$\varepsilon = \alpha v \qquad (3\text{-}1)$$

in which ε ; impact strain

v ; collision speed ($\propto \sqrt{h}$, h ; dropping height)

α ; a constant factor

Fig.3.1 shows α values calculated by eq. (3-1) utilizing measured values of maximum impact compressive strain generated at the top of a leg (on the compressive side) with a linear scale factor based on 40kg Tetrapods. This figure indicates that α becomes constant in the case of Tetrapods heavier than 40 kg , so if 50 kg Tetrapods were used in the test, eq. (2-11) could be applied. Therefore in the hydraulic experiment, 50 kg Tetrapods were used.

3-2 Construction of Tetrapod Model

4 test Tetrapods of 50 kg were constructed with concrete using gravel of size 25mm.

Stress was measured at three locations 120 degrees to each other on the top center of each leg. Totally 12 points per one Tetrapod were examined as shown in Fig.3-2.

Strain gages (length=10mm) on the surface of the test Tetrapods were covered with waterproofing cement.

Photos 3-1~ 3-3 show a Tetrapod model completed and the condition during placement.

Characteristics of the test Tetrapods were measured and tabulated in Table 3-1. These values were close to standard concrete.

Table 3-1 Characteristics of the test Tetrapod

	Moist condition	Air-dried condition
Unit weight	50.6 (kgf)	49.7 (kgf)
Specific weight	2.36 (gf/cm³)	2.31 (gf/cm³)
Static modulus of elasticity	3.08×10^5 (kg/cm²)	2.45×10^5 (kg/cm²)
Dynamic modulus of elasticity	3.60×10^5 (kg/cm²)	3.68×10^5 (kg/cm²)

Before commencement of the experiment, preliminary drop tests in which two legs were supported on a foundation were performed and the distributions of strain were measured. The results, shown in Fig.3-3 indicate that the strain distribution was constant independent of the dropping height. In this hydraulic model test, measured strain at the 3 points on each leg were used to obtain the maximum strain in one leg using Fig. 3-3.

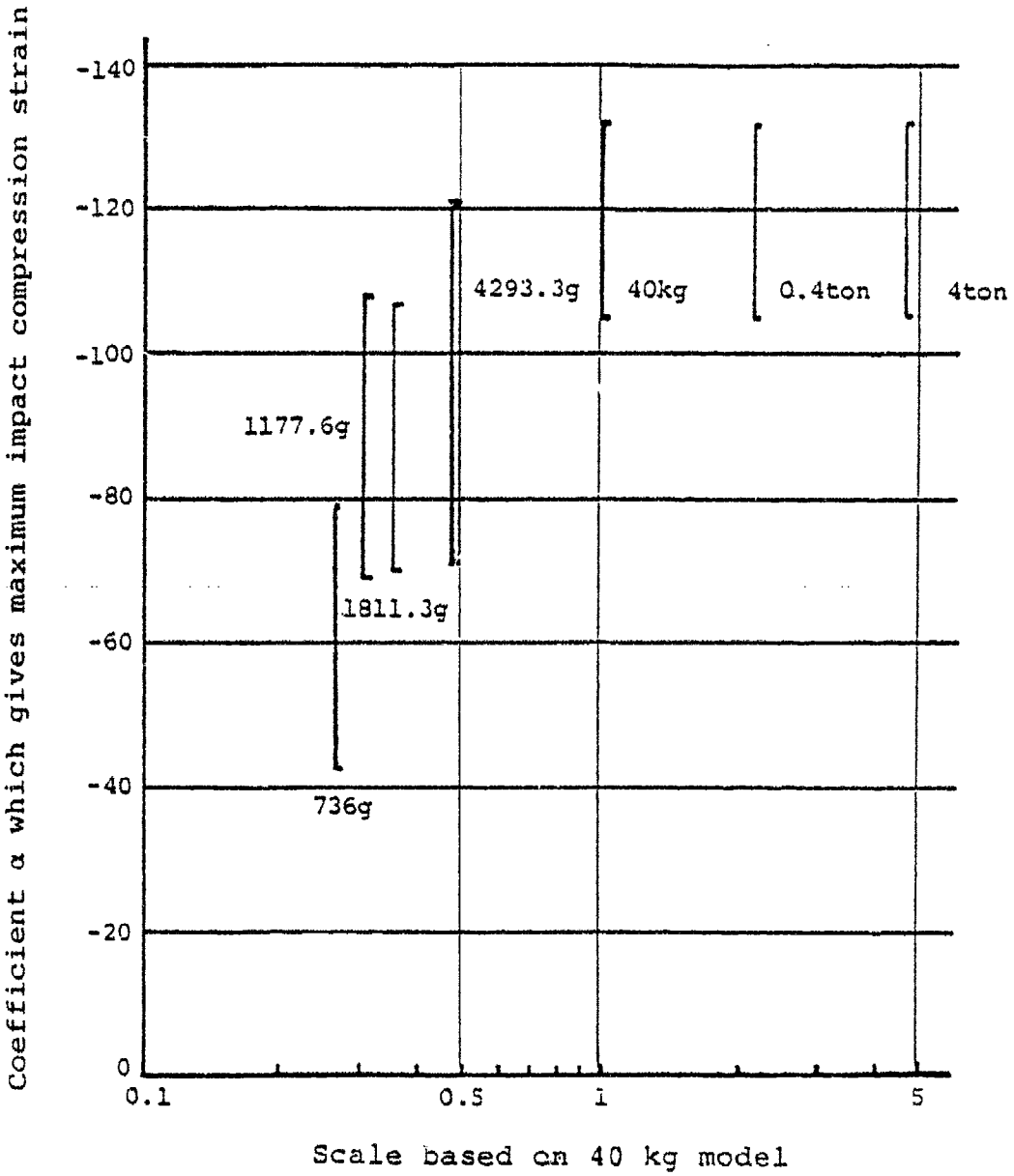
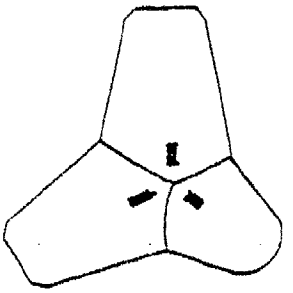
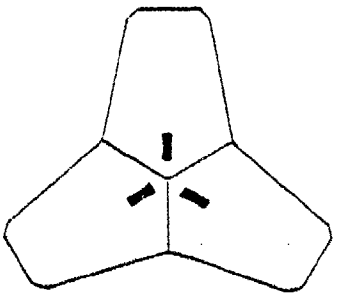


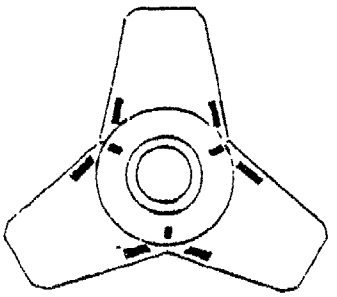
Fig.3-1 Scale Effect Concerning Impact Strain on Tetrapod



Side



Base



Plane

Fig. 3-2 Locations for strain measurement

Fig. 3-3 Distribution of impact strain on the base of one leg

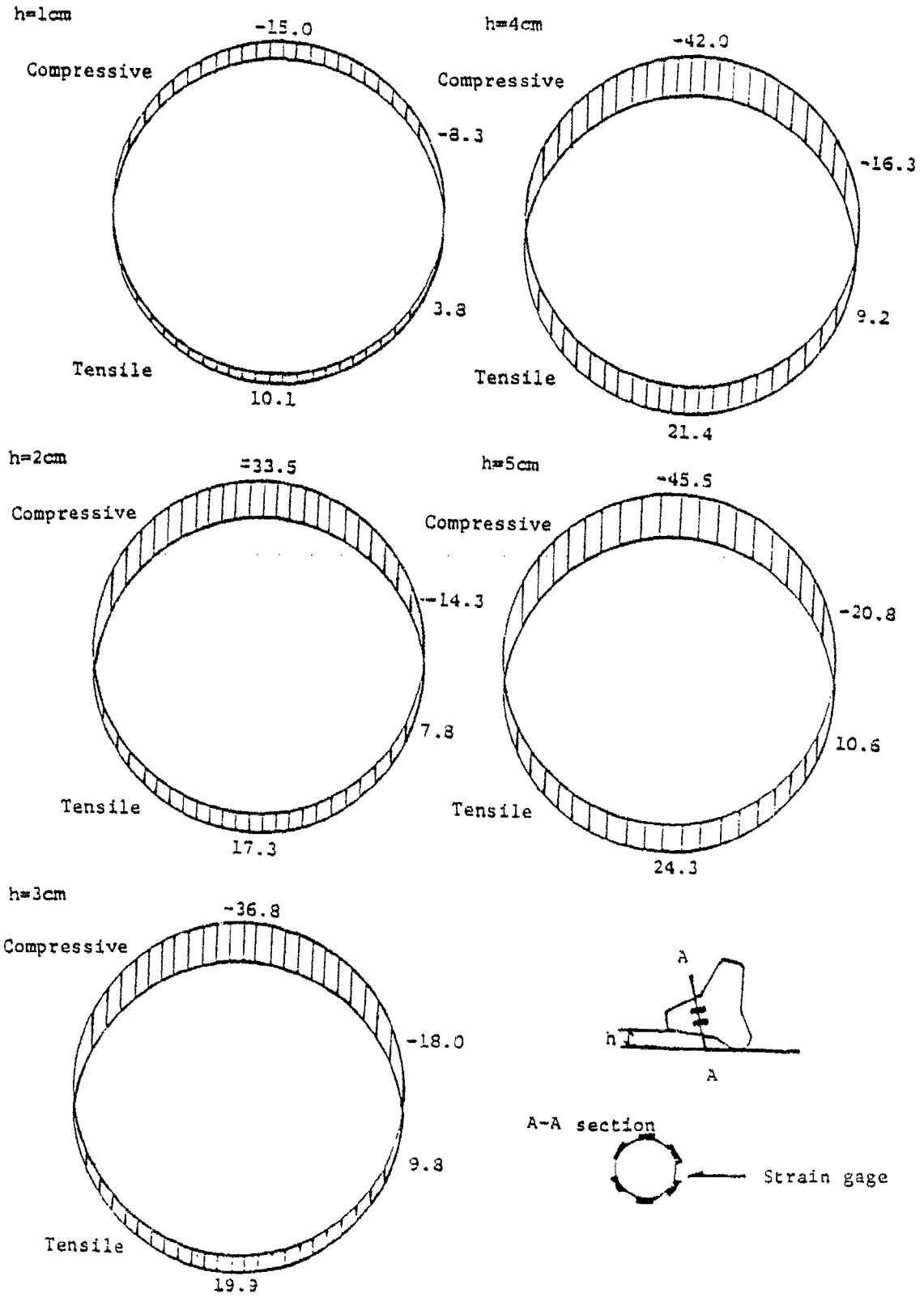




Photo 3-1 Strain gages on the surface of the test Tetrapod

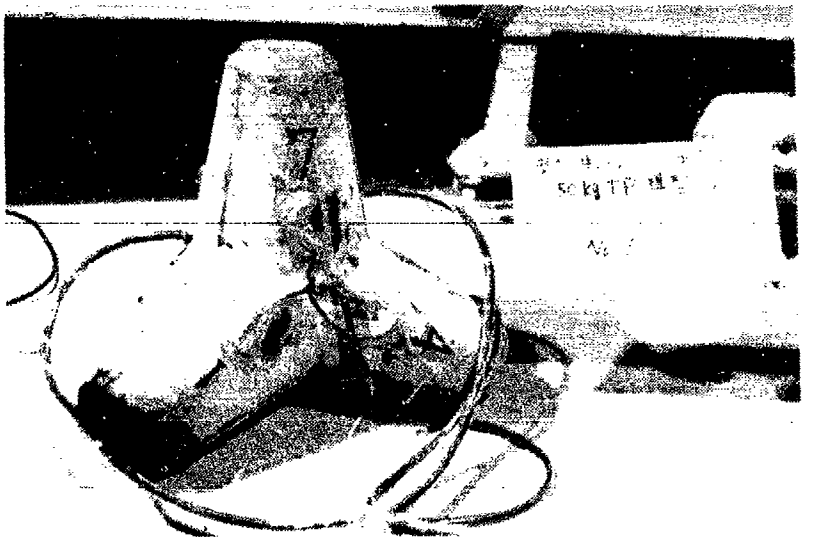


Photo 3-2 50 kg Tetrapod model



Photo 3-3 The condition during placement

4. TEST EQUIPMENT AND TEST METHOD

4-1 Test Equipment

(1) Wave flume

A large wave channel, 205m long, 3.4m wide and 6.0m deep, at the Central Research Institute of Electric Power Industry (hereinafter referred to as CRIEPI) was used in order to generate large waves for the 50kg Tetrapods utilized.

Figs.4-1 and 4-2 show the details and a diagram of the section respectively.

(2) Measurement system

The apparatus for measurement used in the experiment is listed below.

*Dynamic strain amplifier ; This was used for amplifying electric current from strain gages.

*Data recorder ; This was used in order to record analog data through the dynamic strain amplifiers.

*Multi-channel oscillograph ; Strain was reproduced on a multichannel oscillograph chart.

*Video system ; This was used to record Tetrapod motion.

4-2 Test Method

A test section was constructed at a distance of 130m from the wave paddle in the flume.

The Tetrapods under study were placed alongside 2 adjoining blocks on the surface of the slope slightly above the still water level where they received the maximum impact force.

Strain was recorded by the data recorder and reproduced on a strip chart at a speed ratio of 1 : 32 that of the recording speed for the duration of the impact itself ,which was in the order of 1 millisecond to obtain a clearer result. Strain on the chart was measured by a scale.

4-3 Test Cases

Test cases are summarized in Table 4-1. In the table,

h (m) ; water depth at a toe of the Tetrapod slope

T (sec) ; wave period

Lo (m) ; wave length in deep water

Ho' (m) ; equivalent deep water wave height

H (m) ; wave height at the toe of Tetrapod slope with the structure not in place

ξ ; surf similarity parameter

($= \tan\theta / \sqrt{H/Lo}$, $\tan\theta$; gradient of Tetrapod slope)

Ns ; stability number ($= \gamma r\, H^3/W\ (Sr-1)^3$)

γr ; specific gravity of Tetrapod ($2.3t/m^3$)

W ; Tetrapod weight (50 kg)

Sr ; γr ratio to the specific weight of water

($2.3/1.0 = 2.3$)

Waves were generated for 100sec in cases 1 and 4 and 60sec in cases 2 and 3. This was decided after taking effect of re-reflected waves from the wave paddle in the wave tank into consideration.

Photos 4-1~4-3 show waves hitting Tetrapods.

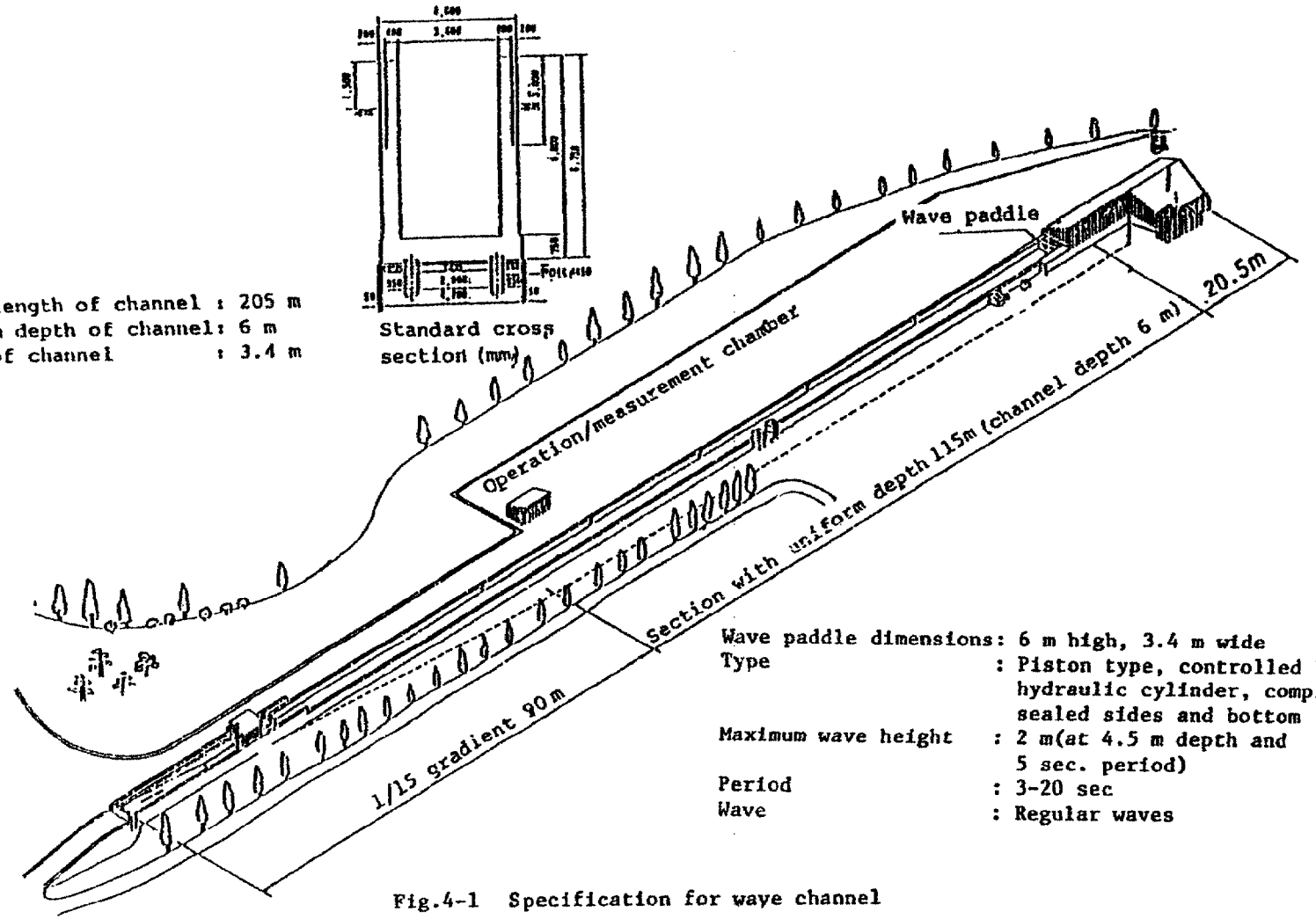


Fig.4-1 Specification for wave channel

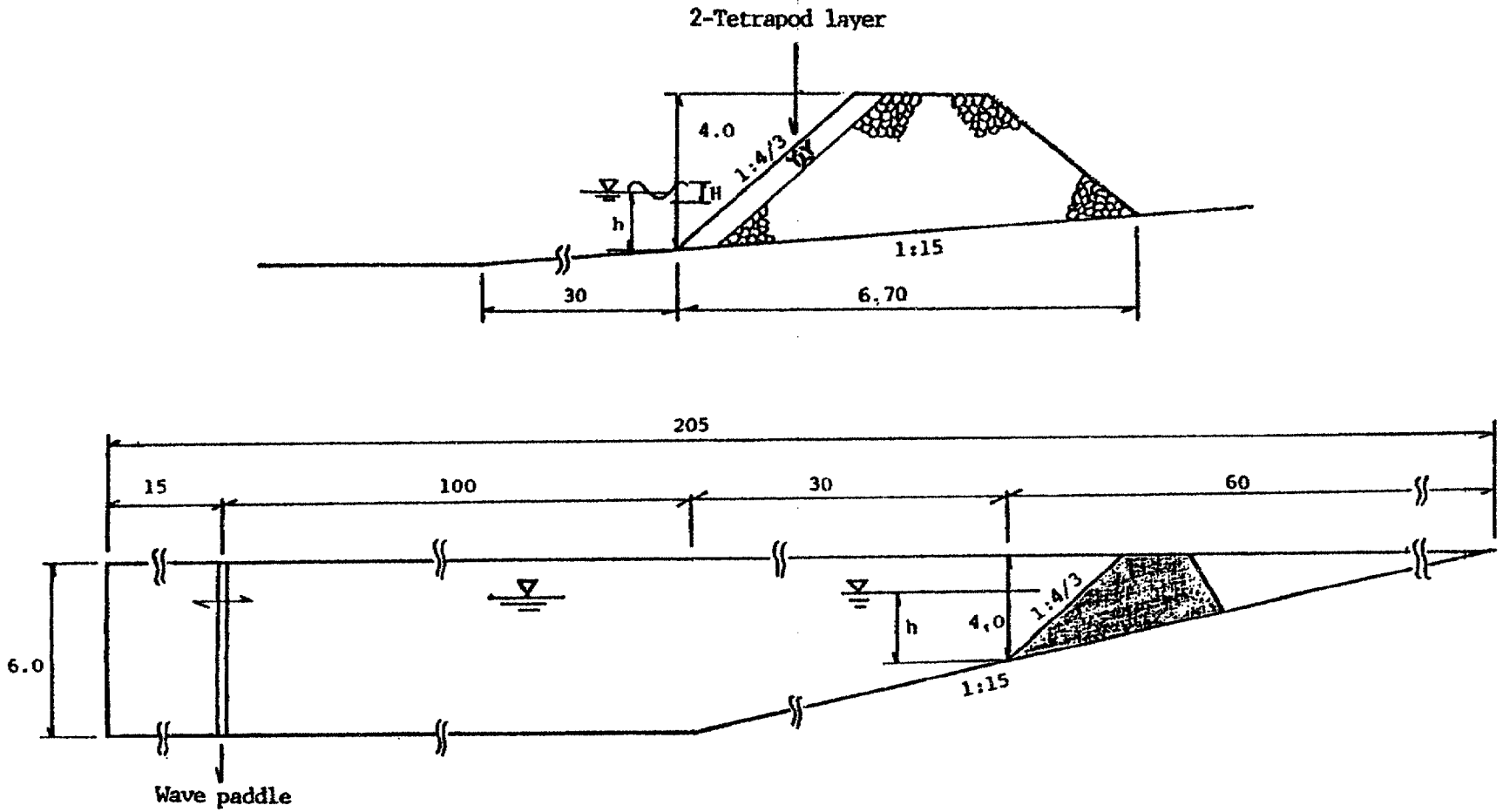


Fig.4-2 Sketch of test section (unit:m)

Table 4-1 Test Cases

Case No.***	h (m)	T (sec)	Lo (m)	No (m)	H (m)	ξ	Ns	Number of Times*	Tetrapod Number
1－1	1.7	4.0	24.96	0.96	1.04	3.67	23.6	3	T.P. No.1.4**
1－2	1.7	4.0	24.96	1.05	1.19	3.43	35.3	2	〃
1－3	1.7	4.0	24.96	1.14	1.33	3.25	49.3	1	〃
1－4	1.7	4.0	24.96	1.19	1.38	3.19	55.0	1	〃
1－5	1.7	4.0	24.96	1.24	1.43	3.13	61.2	1	〃
1－6	1.7	4.0	24.96	1.29	1.45	3.11	63.8	1	〃
1－7	1.7	4.0	24.96	1.33	1.48	3.08	67.9	1	〃
1－8	1.7	4.0	24.96	1.38	1.50	3.06	70.7	3	〃
1－9	1.7	4.0	24.96	1.43	1.51	3.05	72.1	3	〃
1－10	1.7	4.0	24.96	1.47	1.51	3.05	72.1	3	〃
1－11	1.7	4.0	24.96	1.51	1.51	3.05	72.1	1	〃
2－1	1.4	6.0	56.15	0.87	1.55	4.51	78.0	1	T.P. No.2.3
2－2	1.4	6.0	56.15	0.93	1.55	4.51	78.0	1	〃
2－3	1.4	6.0	56.15	0.98	1.55	4.51	78.0	1	〃
2－4	1.4	6.0	56.15	1.04	1.55	4.51	78.0	1	〃
2－5	1.4	6.0	56.15	1.06	1.55	4.51	78.0	3	〃
2－6	1.4	6.0	56.15	1.08	1.55	4.51	78.0	3	〃
2－7	1.4	6.0	56.15	1.11	1.55	4.51	78.0	5	〃
2－8	1.4	6.0	56.15	1.14	1.55	4.51	78.0	3	〃
2－9	1.4	6.0	56.15	1.17	1.55	4.51	78.0	3	〃
3－1	1.5	6.0	56.15	1.09	1.60	4.44	85.8	1	T.P. No.2.3
3－2	1.5	6.0	56.15	1.15	1.60	4.44	85.8	3	〃
3－3	1.5	6.0	56.15	1.17	1.60	4.44	85.8	5	〃
3－4	1.5	6.0	56.15	1.20	1.60	4.44	85.8	3	〃
3－5	1.5	6.0	56.15	1.22	1.60	4.44	85.8	1	〃
3－6	1.5	6.0	56.15	1.25	1.60	4.44	85.8	3	〃
3－7	1.5	6.0	56.15	1.28	1.60	4.44	85.8	3	〃
3－8	1.5	6.0	56.15	1.31	1.60	4.44	85.8	3	〃
4－1	1.5	4.0	24.96	0.90	0.97	3.80	19.1	1	T.P. No.3.4*****
4－2	1.5	4.0	24.96	0.96	1.04	3.67	23.6	2	〃
4－3	1.5	4.0	24.96	1.05	1.18	3.45	34.4	1	〃
4－4	1.5	4.0	24.96	1.33	1.35	3.22	51.5	1	〃
5	Drop Test							——	T.P. No.3

* ; These numbers indicate numbers of times per test.

** ; T.P. No.1 ～4 are Tetrapod number.

*** ; Test tetrapods were rocking after normal placement in test cases 1～3.

**** ; Test tetrapod No.3 were rocking easily and repeatedly in test case 4.

***** ; Test tetrapod No.4 were falling down easily in test case 4.



Photo 4-1 Waves hitting Tetrapods (Normal placement)



Photo 4-2 Tetrapod placed so as to fall down



Photo 4-3 Tetrapod after falling down easily:

i.e. after falling down a distance equal to approximately ½ of the block height

5. TEST RESULTS

Test results are classified into three categories according to Tetrapod reaction to the test.

5-1 Tetrapods Rocking after Normal Placement

Measured impact strain is shown in Table 5-1. εt max and εc max are used for the maximum values measured in tensile and compressive side when collisions occurred. Behavior of Tetrapods due to wave attack is shown in Fig.5-1.

Actual motion was not visible under wave action, but was obtained by a comparison of the positions of Tetrapods just before and after waves hit. Then strain less than 3×10^{-6} could not be measured precisely due to noise inference.

In case 1, the impact strain was not measured in Tetrapod No.4, but was measured in case 1-10 (during the third wave generation) in Tetrapod No.1.

In case 2, strain was not measured in either Tetrapod No.2 or No.3. Nevertheless waves of which height and period were 1.55 m and 6.0 sec respectively hit the structure in the condition shown in Fig.5-1(b).

In case 3, impact strain was found to be $3\sim26\times10^{-6}$ in the tensile side of Tetrapod No.2. Tetrapod No.2 was then observed to fall down in case 3-7 (duringthe the 2nd wave generation) . On the other hand, in Tetrapod No.3, impact strain was not obtained before Tetrapod No.2 fell down. After this point, tensile strain was measured and found to be $11\sim25\times10^{-6}$.

5-2 Tetrapods Rocking Easily and Repeatedly

In this category, the movements of a Tetrapod are show in Fig.5-2. Its reactions to waves were as follows ; leg ① crashed into Tetrapod A and returned to its previous location due to gravity. Then leg② collided with Tetrapod B. The distance leg① moved was about 20cm. The measured impact strain, εt max and εc max, is summarized in Table 5-2.

5-3 Tetrapods Falling Down

In this category, the measured impact strain values, ε t max and ε c max, are shown in Table 5-3.

In test cases 1-11 and 3-7 (during the second wave generation) , Tetrapods rolled down toward the toe of the Tetrapod slope (see Fig.5-1) .

In cases 4-1, 4-2 and 4-3, as shown in Figs. 5-3(a)~(d), each Tetrapod moved down a distance equal to approximately ½~1 of the block height.

In case 4-4, one of two test Tetrapods moved a distance of ½ a block size and rolled down toward the toe (see Fig.5-3 (e)) .

In case 5, a Tetrapod was pushed down manually from the crown of the structure in a condition of still water.

Examples of impact strain reproduced on a strip chart are shown in Fig.5-4. They were measured in case 4-1 where a Tetrapod fell down for distance of ½ its height. This impact strain can be seen as triangular pulses.

Table 5-1 (a)Measured impact strain for Tetrapods rocking after normal placement (T =4.0sec)

Case No	h (m)	T (sec)	Lo (m)	Ho' (m)	H (m)	ξ	Ns^3	The maximum impact strain ($\times 10^{-6}$) Tetrapod No.1 εt max	Tetrapod No.1 εc max	Tetrapod No.4 εt max	Tetrapod No.4 εc max	Notation
1 − 9	1.7	4.0	24.96	1.43	1.51	3.05	72.1	0	0	0	0	
1-10-1~3	〃	〃	〃	1.47	〃	〃	〃	26	-32	0	0	occurred during the 3rd wave generation
1 −11	〃	〃	〃	1.51	〃	〃	〃	Falling down		0	0	

Table 5-1 (b)Measured impact strain for Tetrapods rocking after normal placement (T =6.0sec)

Case No.	h (m)	T (sec)	Lo (m)	Ho' (m)	H (m)	ξ	Ns³	The maximum impact strain ($\times 10^{-6}$) Tetrapod No.2		Tetrapod No.3		Notation
								εt max	εc max	εt max	εc max	
3-4-1~3	1.5	6.0	56.15	1.20	1.60	4.44	85.8	0	0	0	0	
3-5	〃	〃	〃	1.22	〃	〃	〃	3	-4	0	0	
3-6-1	〃	〃	〃	1.25	〃	〃	〃	12	-19	0	0	occurred during the 1st wave generation
3-6-2	〃	〃	〃	〃	〃	〃	〃	26	-25	0	0	occurred during the 2nd wave generation
3-6-3	〃	〃	〃	〃	〃	〃	〃	9	-10	0	0	occurred during the 3rd wave generation
3-7-1	〃	〃	〃	1.28	〃	〃	〃	10	-13	0	0	occurred during the 1st wave generation
3-7-2	〃	〃	〃	〃	〃	〃	〃	Falling down		25	-20	occurred during the 2nd wave generation
3-7-3	〃	〃	〃	〃	〃	〃	〃			19	-30	occurred during the 3rd wave generation
3-8-1	〃	〃	〃	1.31	〃	〃	〃	Not measured		17	-22	occurred during the 1st wave generation
3-8-2	〃	〃	〃	〃	〃	〃	〃	after falling		11	-10	occurred during the 2nd wave generation
3-8-3	〃	〃	〃	〃	〃	〃	〃	down		20	-27	occurred during the 3rd wave generation

Table 5-2 Measured impact strain for Tetrapods rocking easily and repeatedly (T.P.No.3)

CaseNo	h (m)	T (sec)	Lo (m)	Ho' (m)	H (m)	ξ	Ns*	The maximum impact strain ($\times 10^{-6}$)		Notation
								ε t max	ε c max	
4-1	1.5	4.0	24.96	0.90	0.97	3.80	19.10	20	-20	
4-2-1	〃	〃	〃	0.96	1.04	3.67	23.60	29	-23	occurred during the 1st wave generation
4-2-2	〃	〃	〃	〃	〃	〃	〃	30	-30	occurred during the 2nd wave generation
4-3	〃	〃	〃	1.05	1.18	3.45	34.40	19	-20	
4-4	〃	〃	〃	1.33	1.35	3.22	51.50	27	-30	

Table 5-3 Measured impact strain in Tetrapods falling down

Case No.	h (m)	T (sec)	Lo (m)	Ho' (m)	H (m)	ξ	Ns^3	The maximum impact strain ($\times 10^{-6}$)		Notation
								εt max	εc max	
1 −11	1.7	4.0	24.96	1.51	1.51	3.05	72.10	62	-60	T.P.No.1 fell down
3-7-2	1.5	6.0	56.15	1.28	1.06	4.44	85.80	92	-80	T.P.No.2 fell down
4 − 1	1.5	4.0	24.96	0.90	0.97	3.80	19.10	38	-93	T.P.No.4 fell down
4-2-1	〃	〃	〃	0.96	1.04	3.67	23.60	58	-61	T.P.No.4 fell down
4-2-2	〃	〃	〃	〃	〃	〃	〃	45	-64	T.P.No.4 fell down
4 − 3	〃	〃	〃	1.05	1.18	3.45	34.40	11	-28	T.P.No.4 fell down
4 − 4	〃	〃	〃	1.33	1.35	3.22	51.50	40	-65	T.P.No.4 fell down
5	Drop test							36	-44	

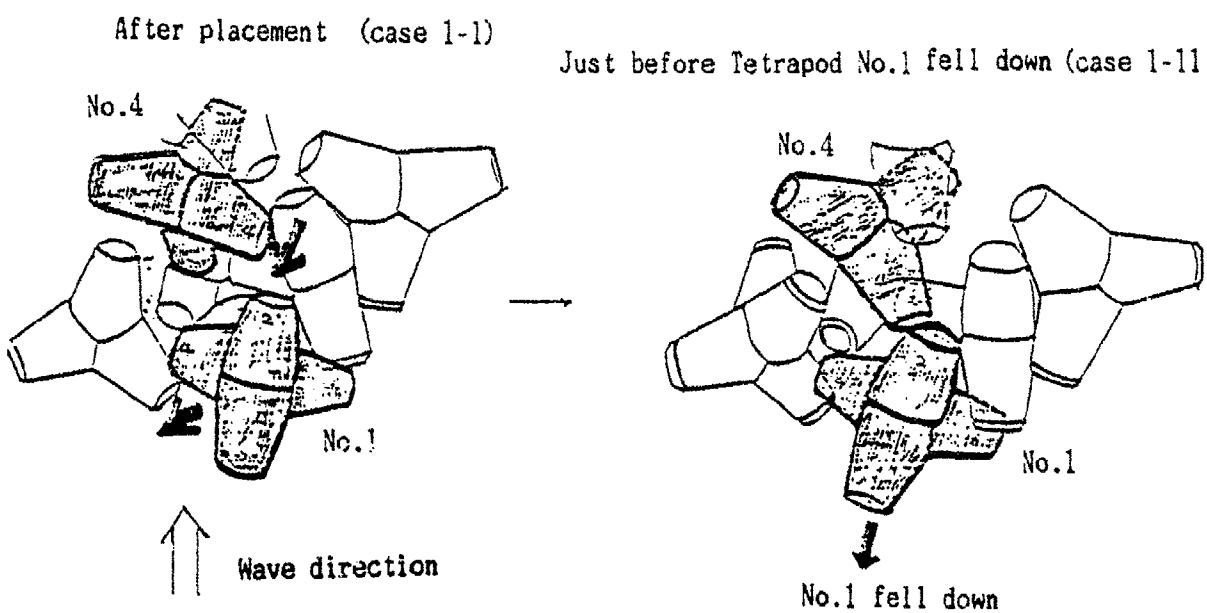
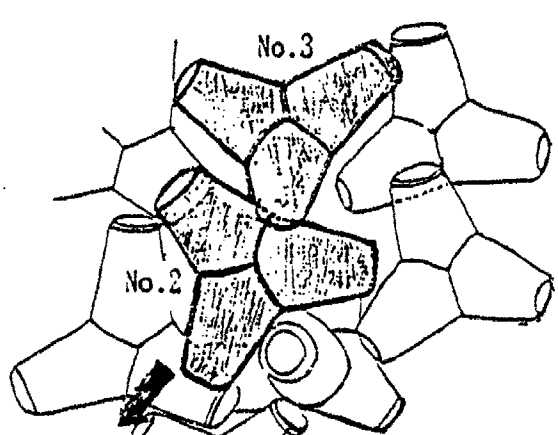


Fig.5-1 (a) Behavior of Tetrapods in case 1



No.2 fell down in case 3-7
(during the 2nd wave generation)



Wave direction

Fig.5-1 (b) Behavior of Tetrapods in cases 2 and 3

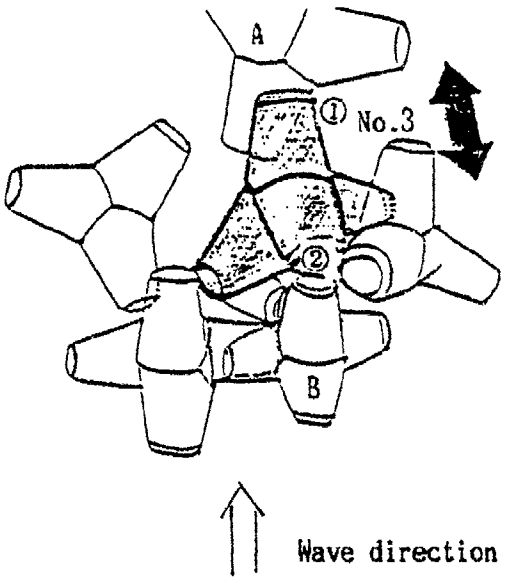


Fig.5-2 Behavior of Tetrapod in case 3

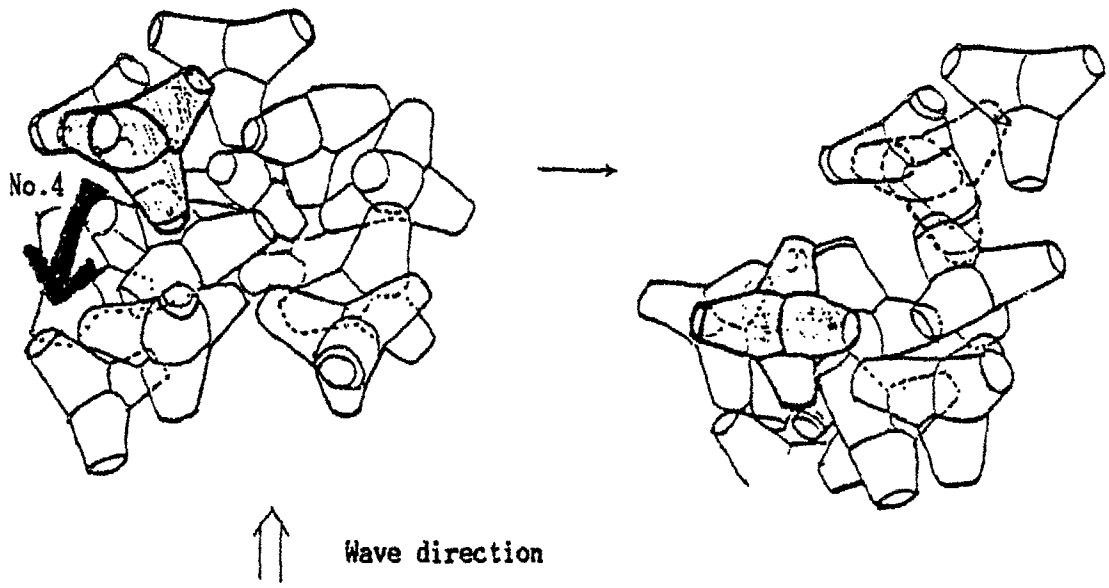


Fig.5-3 (a) Tetrapod No.4 falling down in case 4-1

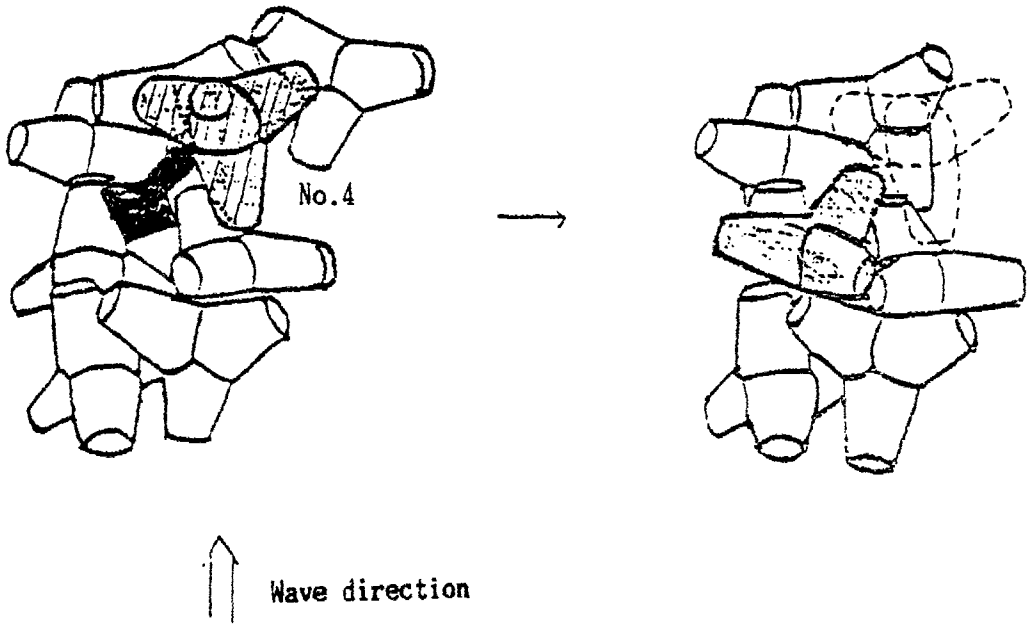


Fig.5-3 (b) Tetrapod No.4 falling down in case 4-2 (during the 1st wave generation)

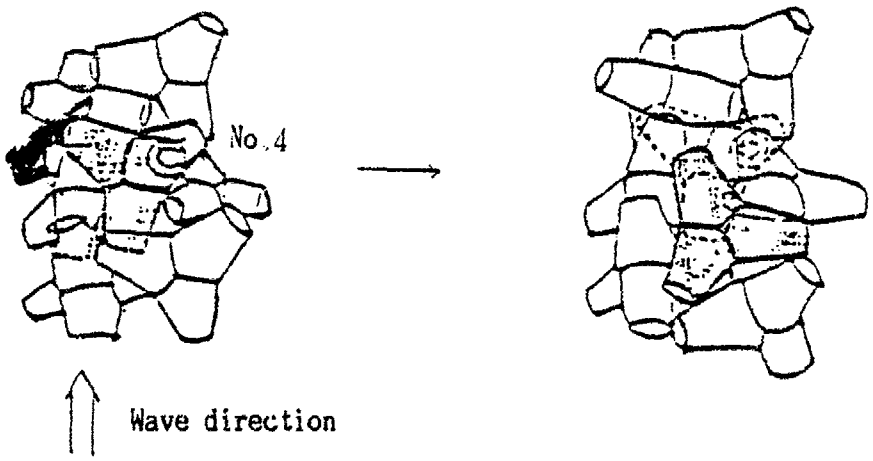


Fig.5-3 (c) Tetrapod No.4 falling down in case 4-2 (during the 2nd wave generation)

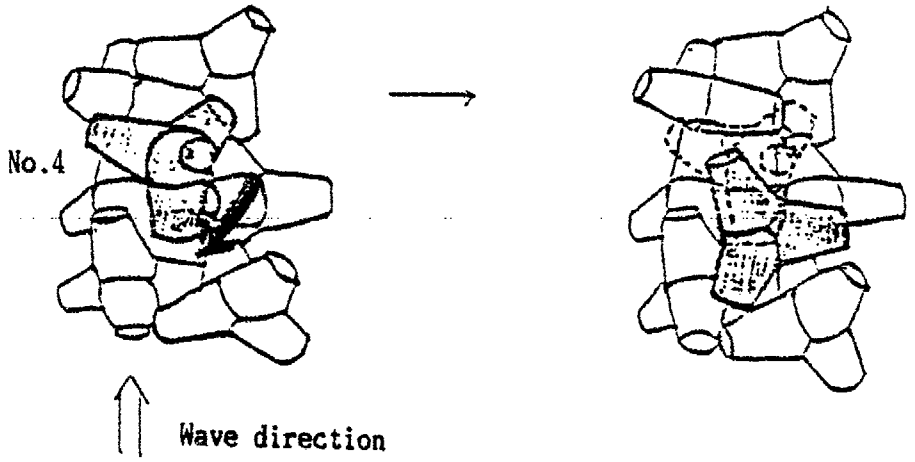


Fig.5-3 (d) Tetrapod No.4 falling down in case 4-3

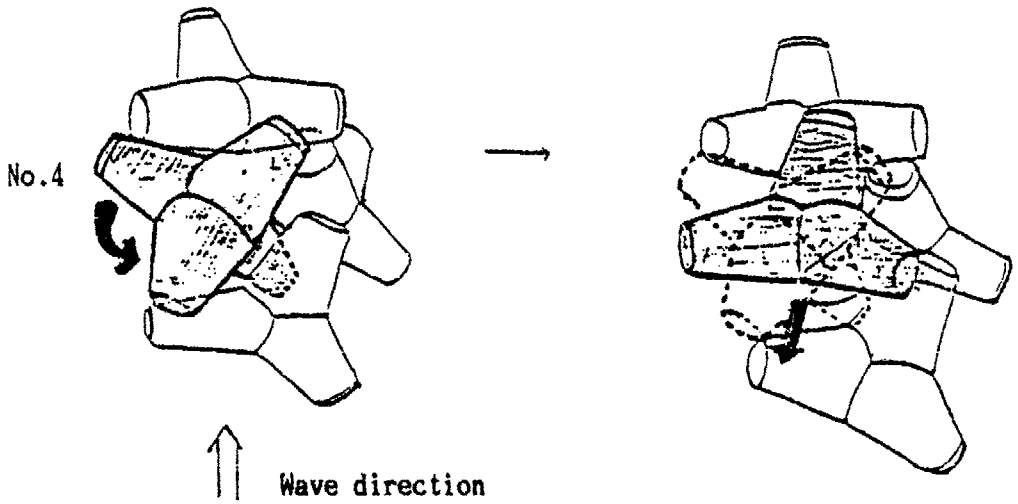


Fig.5-3 (e) Tetrapod No.4 falling down in case 4-4

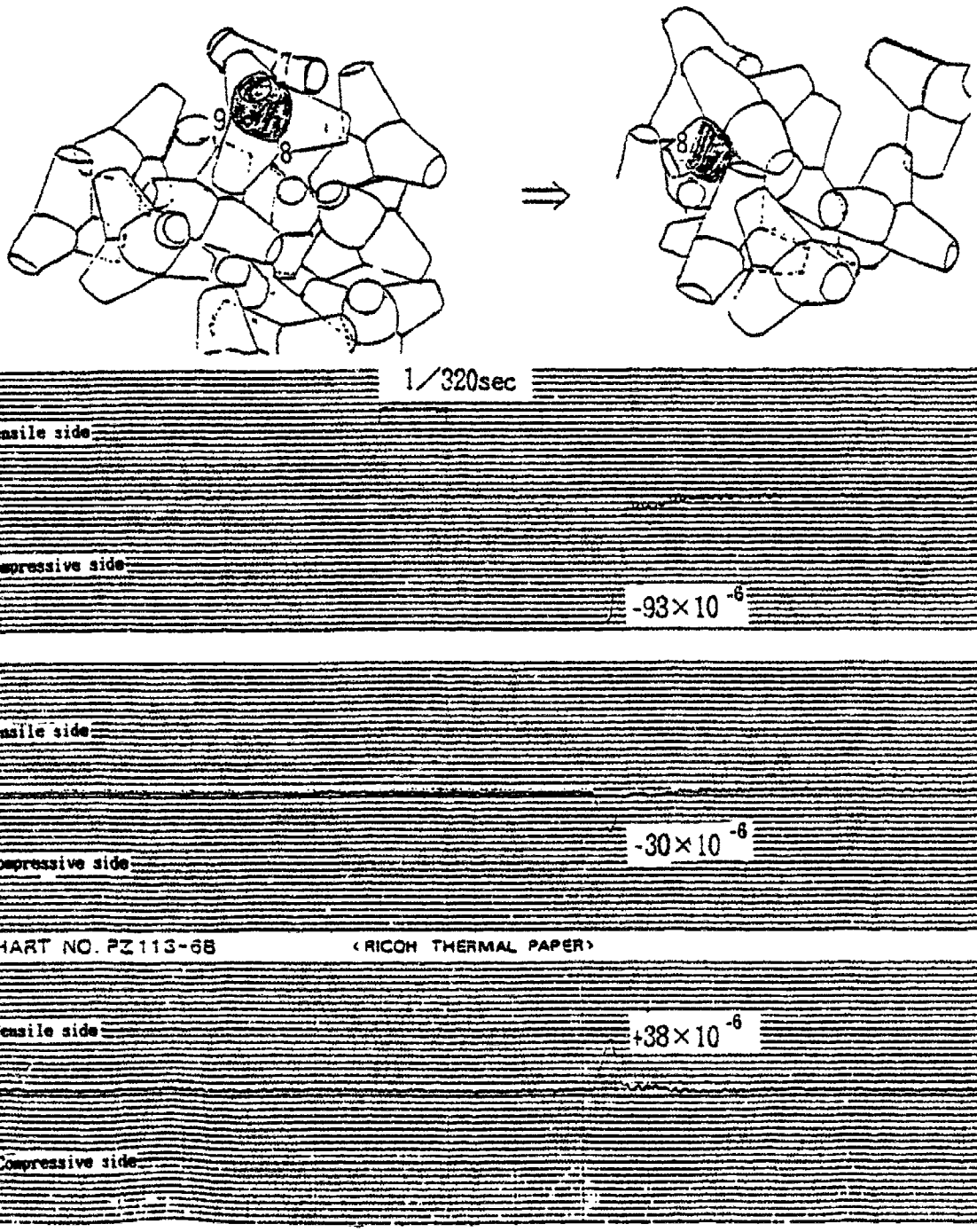


Fig.5-4 Examples of Impact Strain

6. DISCUSSION

6-1 Impact Strain and Wave Properties

(1)Tetrapods Rocking After Normal Placement

Because impact strain was significantly influenced by Tetrapod placement, correlations between impact strain and the wave properties could not be clarified. However even when waves (Ns^3 =70~85) hit Tetrapods, impact strain was small ($\varepsilon t\ max = 3 \sim 26 \times 10^{-6}$) only when test Tetrapods did not fall down. Such strain only occurred in conditions where the incident wave heights became very large or adjacent Tetrapods fell down. Before this occurred, because of slight rocking, impact strain could not be measured.

In case 3-6 (during the second wave generation) , impact strain was measured, as $\varepsilon t\ max = 26 \times 10^{-6}$, and impact strain became small even if waves were continuously generated. This meant that a Tetrapod was inclined to move a more stable position from an unstable one under wave action.

(2)Tetrapods Rocking Easily and Repeatedly

In this category, behavior of Tetrapods followed the wave action repeatedly. In the case of repeated rocking, maximum impact strain in the tensile side was estimated by applying the distribution, as shown in Fig.3-3, to measured values. Their mean value $\bar{\varepsilon}t$, standard deviation SD, maximum value $\varepsilon t\ max$ were calculated and are shown in Table 6-1.

From Table 6-1, a correlation between impact strain and wave properties was not obtained because there was little data and the magnitude of strain in the Tetrapods was influenced significantly by their linkage to each other under wave action.

In this category, measured impact strain when leg① crashed into Tetrapod A was larger than that when leg② collided with Tetrapod B (mentioned in 5-2) . It can be considered that when incident waves attacked a test Tetrapod the block accelerated, while on the other hand, when it returned, its behavior depended only on gravity.

(3)Tetrapods Falling Down

In the catgory where Tetrapods fell down a distance equal to ½~1 their height, ε tmax was 11 ~58×10^{-6}.

When Tetrapods rolled down toward the toe of the Tetrapod slope, ε t max was 40~92×10^{-6}.

On the other hand, in case 5 (still water condition) , ε tmax was 36×10^{-6}, i.e. not so large although the falling distance was large. It can be seen that in the category where Tetrapods fell down under wave action, they were accelerated by waves and their impact speeds became larger than that in case 5.

Table 6-1 Measured impact strain for Tetrapods rocking easily and repeatedly (T.P.No.3)

CaseNo.	h (m)	T (sec)	Lo (m)	Ho' (m)	H (m)	ξ	Ns^3	Impact strain in tensile side ($\times 10^{-6}$)			Number of Times
								$\bar{\varepsilon}$	SD	ε tmax	
4 − 1	1.5	4.0	24.96	0.90	0.97	3.80	19.10	14.2	4.9	22.8	1
4 − 2	〃	〃	〃	0.96	1.04	3.67	23.60	20.2	6.2	37.5	2
4 − 3	〃	〃	〃	1.05	1.18	3.45	34.40	15.6	5.3	25.0	1
4 − 4	〃	〃	〃	1.33	1.35	3.22	51.50	16.0	6.0	25.9	1

6-2 Estimation of Impact Strain in 50t Tetrapods

From the test results, impact strain in 50t Tetrapod under wave action was estimated using eq. (2-11) as follows.

$$\varepsilon_{50t} = 3.12\,\varepsilon_{50kg} \qquad (6\text{-}1)$$

where ε_{50t} ; impact strain in 50t Tetrapod

ε_{50kg} ; impact strain in 50kg Tetrapod

In the category where Tetrapods were rocking after normal placement, the maximum impact strain in 50t Tetrapod in the tensile side was about $\varepsilon_{50t} = 81\times10^{-6}$. In the category where Tetrapods were rocking easily and repeatedly, ε_{50t} of 117×10^{-6} was obtained.

Generally speaking, the critical impact strain εc where concrete breaks is 200 ~300 $\times10^{-6}$. Comparing the estimated results for ε_{50t} with εc, it can be considered that Tetrapods would not break by rocking.

In the category where Tetrapods fell down a distance equal to ½~1 their size, the maximum ε_{50t} is approximately 180×10^{-6}.

When Tetrapods rolled down toward the toe of the Tetrapod slope, the maximum ε_{50t} was about 290×10^{-6}. Therefore, in categories like the two above, cracks on a Tetrapod surface might occur.

7. CONCLUSIONS AND FUTURE DIRECTION OF STUDY

The following conclusions were derived through the experiment.

(1) When 50kg Tetrapods were used, the similitude relations on impact stress and strain between the model and prototype were satisfied under wave action.

(2) Tetrapods would not break due to usual rocking under these test conditions.

(3) In the case where Tetrapods fell down under wave action, impact strain occurred was larger than that in the still water condition. It can be seen that Tetrapods were accelerated by wave down rush and their collision speeds became larger. In this case, the maximum value of measured impact strain was very close to the critical tensile strain, so cracks on a Tetrapod surface might occur.

Because behavior of Tetrapods is various under wave action, the motion of Tetrapods due to waves could not be obtained from the tests.

In addition, even if large scale models are used, various cases cannot be examined. Therefore hereafter the authors will carry out similar experiments in simpler conditions using smaller models in order to find out the behavior of Tetrapods, impact load and stress under wave action.

Impact load depends on collision speeds, but a correlation between impact load and stress was not obtained from the test results. Hereafter, the authors will investigate this correlation. If the relation between behavior of blocks and impact stress becomes known, a structural analysis of Tetrapods will become feasible.

ACKNOWLEDGMENT

The authors wish to express their gratitude to Toshimi Fujimoto and Shyozo Saito of the Central Research Institute of Electric Power Industry for their most vallable assistance in this study.

The authors would also like to thank Associate Professor Hitoshi Nishimura and Associate Professor Yasuhiko Yamamoto of University of Tukuba and Professor Hiroshi Seki, Waseda University, and Masao Uekita of National Research Institute of Fisheries and Engineering for many valuable suggestions and contributions during the analysis of the experiment.

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"STRUCTURAL ANALYSIS OF ARMOR CONCRETE MEMBERS TO DISSIPATE WAVE FORCES"

DR. OSAMU KIYOMIYA

DR. KIYOMIYA:

My presentation is on a new type of breakwater called arc shaped slit caisson; the subjects of my presentation consist of three parts. The first subject is the loading test of arc shaped members in order to know the mechanical properties of the members. The second subject is finite elements analysis considering non-linearity of the materials such as concrete and bars. The third subject is the field observation of prototype arc shaped type caisson to understand the behavior of the structure under offshore circumstances. Though my presentation is a little different from the subject of the concrete armor units, I think that the analysis of the concrete and the measurement of concrete structures offshore will help the analysis of concrete armor units.

The new types of breakwaters are proposed by the engineers in the Port and Harbor Research Institute. Japanese engineers now have two ways to resist wave forces and to secure a calm water area. One is due to the concrete blocks which Dr. Endo explained to me. Another is this new type breakwater.

The reason these new type breakwaters are proposed is to construct breakwaters cheaply offshore where the water depth is from minus ten meters to minus fifty meters. In the deep water, the large and many concrete blocks are required. These new type breakwaters save on the total cost of construction.

Today, I will mainly mention the arc shaped slit caisson in these new type breakwaters. This is another type, another kind of breakwater.

This figure (see Appendix C for figures and related information) shows the outline of the arc shaped slit caisson. The several arc shaped members are attached at the front of the caisson and consist of the wave dissipated chamber. Wave energy dissipates in this chamber. The length of the caisson is twenty-one meters, its height is ten point five meters and the radius of the arc shaped member is seven meters.

This type of caisson has been constructed at two locations in Japan. The construction cost of this caisson is about one point two times that of the vertical face caisson with blocks. However, these breakwaters do not need the armor arrangement of the wave dissipating concrete armor blocks even when this type of caisson faces the ocean.

These figures show the state of waves and wave force for the arc shaped members. These members are subjected to wave forces repeatedly from the outside and from the inside. When the wave breaks in the chamber, impact load due to the wave breaking applies to the arc shaped members.

These two figures show the design wave forces for the arc shaped member. These patterns of the wave forces are obtained by an indoor hydraulic model test. The impact wave forces apply the inside of the members during wave dissipating and when wave height is maximum wave height and (unintelligible) is unit weight of water.
Impact wave forces obtained by the hydraulic test is also shown in this figure. The

design wave force is decided considering impact wave forces and distribution of wave forces.

The figure shows the reflection coefficient of the arc shaped slit caisson and wave dissipating concrete blocks setting before the breakwaters. This data is obtained from the hydraulic test. Where KR is the reflection coefficient, LS is the wave length and B is the width. Circle dots represent the base of the arc shaped slit caissons and triangle dots represent the armoring concrete blocks. Reflection coefficients of the arc slit caissons is almost the same as that of the armor in concrete blocks.

DR. GALVIN:

What is the slope in the armored blocks? One to four?

UNIDENTIFIED VOICE:

One to one and a third.

DR. GALVIN:

Okay, thank you.

DR. KIYOMIYA:

The first subject concerns the load test of the arc shaped member. Loading tests are carried out at the testing floor and wall. The test specimen is a quarter circle in the side view and is designed to a scale of approximately zero one five of the prototype member. The concentrated load is applied at the midspan by a hydraulic jack with fifty tons capacity. The purpose of the loading test is to know the mechanical properties of the arc shaped member in the elastic and plastic range of materials because these properties, due to the alternate load, are not well-known.

This figure shows details of the test specimen. This figure shows the location of instrumentation. The applied load is measured by a load cell attached at the tip of the jack. The deflection of the test specimen, and the strain in the reinforcing bars and on the concrete surface are measured. Contact points are attached on the surface of the concrete at the interval of ten centimeters to measure width of cracks. The locations of cracking and developing cracking are sketched by visual inspections.

This table presents the summary of the loading tests of the arc shaped members. When the load is applied from the inside to the outside, the failure mode of the member is bending. The ultimate strength is about seven point four tons. When the load is applied form the outside to the inside the failure mode of the members is crushing of concrete and shear. The ultimate strength is greater than that of the loading from the inside. The ductility of the loading from the outside is smaller that than of the inside. The ultimate strength, the failure mode, is different according to the loading direction. When load is applied alternately, the mechanical properties are compounded by each of the mechanical properties according to the loading direction. This figure shows load percentiles. This figure shows crack formations in the ultimate specimen. This figure shows the ratio of the average stiffness; and also this figure shows the equivalent viscous damping coefficient.

The second subject is on the finite element analysis of the arc shaped member concerning the nonlinearity of material and plastic hinge. I present the comparison between the computed results by the finite element method and experimental results.

Modeling of the reinforced concrete member is carried out for the concrete, the steel, and the bond between the concrete and the steel, as shown in this figure. The concrete is replaced by plane stress element, bars are replaced by the truss element that transmits axial stress, and the bonds are replaced by the spring element that transmits axial and shear stress.

Strength and stress-strain relationships of materials are shown in this figure. The biaxial strength envelope is chosen to the model of the concrete. Under biaxial compression, the strength envelope of the concrete is determined as Drucker-Prager's Yield Law. The stress-strain relationship of the concrete is assumed to be tri-linear. The state of stress reaches the brittle fracture surface, a crack is assumed to be formed. This model is called the sheared cracking model. After the cracking has occurred, the modulus of elasticity of the concrete is reduced to zero, perpendicular to the cracking direction. Moreover, shear modulus G is reduced according to crack width. However, models for tension stiffening in the concrete after cracking is not taken into account in this model. The stress-strain relationships of the bars and the bond are assumed to be bi-linear as shown in this figure.

The finite element mesh layout for the arc shaped member is shown in this left figure. Because of the symmetry conditions, half of the test specimen is analyzed.

I will mention the comparison of the the load deflection curves and crack patterns between the computed results and the experimental results. The load deflection relationship of the arc shaped member obtained by the computation is accordant with that obtained by the loading test as shown in this figure. The next figure shows the relationship, the applied load and the tensile strain in the reinforced bar. The computed results also agree with experimental results.

This figure shows the computed and experimental crack formation. The propagation and width of the cracks cannot be estimated precisely since the sheared cracking model is used in this study. The zone of crack formed and developed can be investigated in the sheared model. The crack formation of the loading test is shown in Number 4 figure. This shows without load, loading test. The black area shows the location of crushing of the concrete, sheared part shows the location of the cracking of the concrete. The location of the cracking and the crushing are in fairly good agreement between the computations result and the loading test result. The comparison shows okay.

The third subject is on the field observation of the arc shaped slit caisson. The site of the field observation is at about four hundred fifty kilometers north of Tokyo. The purpose of the field observation is to confirm the performance of the arc shaped slit caisson at an offshore environment and to know the behavior of the arc shaped slit caisson for waves. The depth of the water at the construction site is minus eleven meters and about one kilometer off the land. In winter season and in typhoon season, large ocean waves reach this caisson directly. The design wave height of the observation site is nine point five meters.

The measurement items are wave pressure for the caisson and each member, the vibration of the caisson and the stresses in the members. The deployment of the measurement transducers is shown in this figure. Eight wave pressure transducers are installed on the surface of the caisson, thirty-three reinforced bar gages are installed in the members, three velocity meters are installed on the top of the breakwater to record the vibration properties of the caisson.

Several records have been obtained since 1980, however, the maximum wave height at each record is smaller than the design wave height. Its values are almost three to four meters. This figure shows the examples of the obtained records. The measurement is carried out during ten minutes and sampling interval of record is zero point two seconds to zero point eight seconds. In this case, impact wave force and stress are not observed because the breaking of waves have not occured in the members in the chamber.

This figure shows stress interval and wave pressure. This figure shows maximum wave height and displacement. This figure shows the wave record and displacement record. This was the last figure.

The main conclusions of my research are as follows.

First, mechanical properties of the arc shaped members such as collapse process, ultimate strength, and crack formation depend on the loading direction according to loading tests.

Second, the finite element model mentioned in my presentation concerning the nonlinearities of materials and plastic hinge explains the results of the loading test as to the arc shaped members.

Third, field observation of the arc shaped slit caisson is offshore and confirms the efficiency of the caisson.

Further research works are now planned to know the particular test of the member, to know the degradation strengths of concrete in water. The values of the concrete strengths is ten to twenty thousand, weaker than in the air, and to develop the maintenance system and repairment system.

DYNAMIC STRESS ANALYSIS OF CONCRETE ARMOR UNITS

Joseph W. Tedesco[1] and William G. McDougal[2]

ABSTRACT

Concrete armor units are commonly employed for the protection of shorelines and rubble structures. Their design is primarily based on hydrodynamic stability. However, their structural response to wave loading is poorly understood. In this study, a simple model is presented to estimate impact loads due to wave slamming on the concrete armor unit, dolos. A nonlinear dynamic analysis indicates that the units will experience a structural failure at hydrodynamically stable wave conditions.

1. INTRODUCTION

It is surprising that 40- or 50-ton concrete armor unit structures are frequently constructed without reinforcements. This is particularly true when the structures are long and slender and are subjected to a variety of static and dynamic loads. The design and selection of units appears to have been motivated by hydraulic stability rather than structural integrity. This situation may be historical, in that, for the simple units or quarry stone, a structural analysis may not be necessary because these types of units will experience a hydraulic fialure before a structural failure. Unfortunately, this may not be true for the more recent, novel-shaped units which exhibit high stability coefficients, often at the expense of structural integrity.

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This condition is amplified by the fact that current design practices are allowing relatively large movements of the individual armor units. We have now come to the point where the structural stability of the armor units is a significant aspect of the design consideration.

The following comment has been made several times at this workshop, "If you can tell a structural engineer what the loads are, he or she can determine the structural integrity of the armor unit." That statement may be fairly accurate. The difficulty is quantifying the loads on the units. This is demonstrated by the diversity of approaches that the attendees of this workshop have employed to examine the problem. It helps to put some perspective to those different approaches by trying to categorize them by their methodology.

The first type of approach is referred to as bench tests. Examples include roll-over tests, drop tests, and pendulum tests. This approach includes everything from the small epoxy models, which were examined by Hall, et al., (1985) to Burcharth's (1981) intermediate size units, all the way up to prototype testing of large units out on the casting site. This is a useful line of research to pursue and is a worthwhile contribution to our understanding of the problem.

The second category we refer to as wave tests. Several examples of this approach have been presented at this workshop such as the epoxy models examined in Canada (Hall, et al., 1985). In these models, strains or displacements were monitored while the units were exposed to waves. The scale of this approach extends all the way up to the proposed prototype work at Crescent City. Again, this approach is a necessary component in the overall research program.

A third category we have termed wave-induced forces. An example of this approach was given by Galvin and Alexander (1981) in which an effort was made to estimate wave forces. It has been suggested in several presentations at this workshop that wave forces in themselves are probably not the critical load. The critical loads may be due to the motion of the structures rocking, banging together, or the projectile effect of broken armor units. However, all of those rigid body motions are wave-induced. If we had a better understanding of the wave-structure interactions, we could better estimate rigid body motions. Even though all of these interactions are wave-induced, it is useful to separate these forces into two categories. The actual hydrodynamic forces on the units and the unit interaction forces. These will be termed wave loads and impact loads, respectively.

If the applied wave loads are large, then the dolos may displace as a rigid body. When the wave force is released, the unit will fall back to its static equilibrium position. This may lead to the development of large impact forces between units. The wave load is accumulated over a longer period of time than the free fall. This accumulated energy is quickly released on impact over a small area on the unit. This is both a time effect and an area effect which may lead to the development of very large local stresses.

One common thread that should run through all of these methodologies is that they all be associated to some type of structural analysis. To do tests and not relate them with a structural model, which allows comparison with other types of tests, is not an optimum use of the test results, particularly for destructive tests.

A long-term objective of armor unit research should be to identify critical conditions in terms of quantifiable wave and structure parameters. This

will also require an understanding of the modes of failure. Several modes of breakwater failure have been presented at the workshop. Over a certain range of wave conditions, it may be hydrodynamic stability that controls the design of the unit, while over a different range than structural stability may control. These objectives are also the long-term objectives of this study. However, the present study is in its infancy and only a very simple model is developed. The unit that we have selected to examine is the dolos, although the methodology is not unit specific. This armor unit was selected because it is representative of units with characteristics that are susceptible to structural failure; high hydrodynamic stability and long slender members.

In the study reported herein, a very simple model has been developed to estimate wave forces on a stationary dolos. Next, a nonlinear dynamic structural analysis is presented for several sizes of armor units. Results for structural stability are presented for one dolos orientation. These results are scaled by wave conditions associated with the hydrodynamic stability problem. This reveals wave conditions over which hydrodynamic or structural failure controls overall stability.

2. WAVE FORCE MODEL

There have been several efforts to quantify dolos loads as a function of design conditions. Fang (1982) simultaneously employed Iribarren's equation and Hudson's formula to estimate the wave force on a dolos. He determined that the bending tensile stress is linearly proportional to the wave height and that an unreinforced dolos would fail at a wave height of 22.4 feet. Galvin and Alexander (1981) also determined an expression for the wave load. A breaking wave force was estimated which applied over a portion of the

dolos. This stress was then concentrated over the points of contact between units. This effect, termed stress magnification, yields an interesting result; failure is independent of the dolos size. For 6000 psi concrete, the structural failure wave height was 20 feet. This result is of the same order as the Fang (1982) wave height.

In the present study, the stress magnification concept is also employed. However, the force is estimated from the slamming of the wave onto the structure. Wave slamming results in an impulse type of load. Kirkgoz (1982) measured the wave-induced pressure on a vertical wall for breaking waves. A typical result is shown in Figure 1. The force very quickly rises to a peak and then more slowly drops off. The magnitude of the peak is a function of wave conditions but is also dependent upon the elevation with respect to the still water level.

The dolos problem is, of course, different than waves breaking on a wall. For units near or above the still water level, it is more a case of a blunt body penetrating a free surface. A similar situation exists in many ocean structures with horizontal members near the free surface. As a result, a small amount of information exists on wave slamming on horizontal members. It is this body of literature that we are going to draw upon to try to estimate wave forces on dolosse. We will examine a unit with the seaward flukes horizontal and parallel to the wave crest. A slamming analysis for horizontal cylinders will be used to estimate the force as the dolos penetrates the free surface.

Kaplan and Silbert (1976) and Kaplan (1979) have developed models for both the horizontal and vertical forces due to impact. The models include the effects of buoyancy, pressure gradients, momentum flux (including added mass),

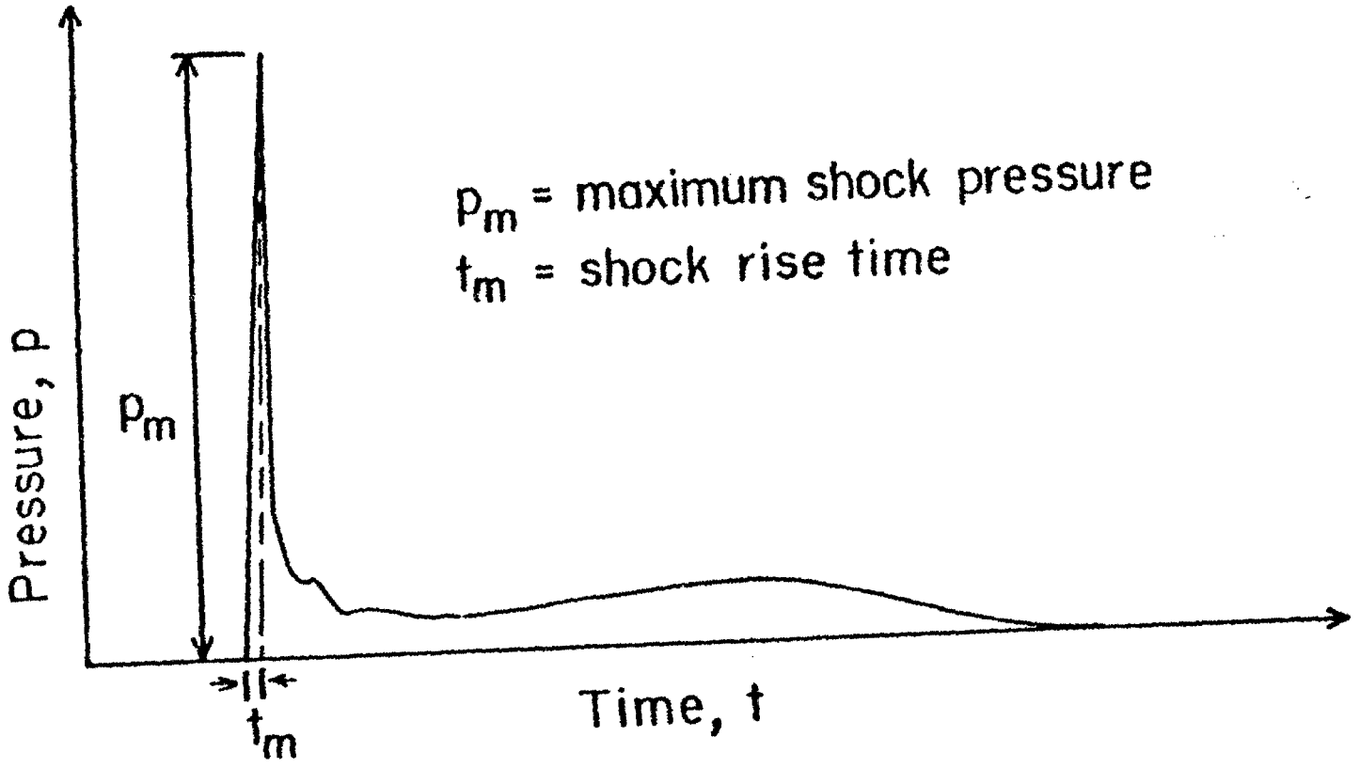


Figure 1. Definition sketch for shock pressure on vertical wall (Kirkgoz, 1982)

and drag. Results are in reasonable agreement with measurements for the horizontal force but less so for the vertical.

Experimental results suggest that the impact force, F, may be expressed in a simpler form

$$F = \frac{1}{2} C_s \rho d l U^2 \tag{1}$$

in which ρ is the fluid density; d is the diameter of the cylinder; l is the length of the cylinder, U is fluid velocity; and C_s is a slamming coefficient (Sarpkaya, 1978). The slamming coefficient is a function of the immersion depth of the cylinder and, therefore, a function of time. A time-dependent slamming coefficient has been empirically developed (Campbell and Weynberg, 1980)

$$C_s = \frac{5.15}{1 + 19 \frac{Ut}{d}} + \frac{0.55Ut}{d} \tag{2}$$

The maximum value for C at t = 0 is 5.15 and this value is appropriate if a static structural analysis is performed (Sarpkaya and Isaacson, 1981). For dynamic analyses, a value of 3.2 is suggested. Therefore, Eq. (2) is scaled accordingly for use in the present dynamic analysis.

The above formulation is, of course, only valid when some portion of the cylinder is immersed. However, when the cylinder is totally immersed, the formulation is inappropriate. Experimental results of Sarpkaya (1978) indicate that this formulation is only valid up to the point where the top of the cylinder is just below the free surface. Therefore, at this depth (when the slamming coefficient is at minimum) the forces are assumed to no longer be impact dominated and a drag formulation is adopted.

$$F = \frac{1}{2} C_D \rho S d l U^2 \tag{3}$$

in which S is the fraction of the cylinder which is immersed and C_D is a drag coefficient. It is noted in Eq. (2) that the minimum value for the scaled slamming coeffient is approximately 0.5. This is of the same order as the drag coefficient for a smooth cylinder. Therefore, for purposes of calculation, this minimum value of C_s is used for C_D. Several representative time histories of force are shown in Figure 2. For a given wave and cylinder size, the duration of the force is a function of the position of the cylinder relative to the still water level. The peak impact force is not a function of elevation because the horizontal velocity is assumed to be constant. The tailing off of the force depends upon the ratio of the cylinder diameter to the wave height.

3. TEST CASES

A technique for estimating the hydraulically stable stone weight as a function of the wave conditions was presented by Iribarren (1938). There have been a number of modifications made to the original relationship. The version given in the Shore Protection Manual (1984) and most widely used in the USA is Hudson's formula (Hudson, 1953, 1959)

$$W = \frac{H^3 \gamma_r}{K_D (\gamma_r/\gamma_w - 1)^3 \cot(\alpha)} \tag{4}$$

in which W is the weight of an individual stone or armor unit; H is the wave height at the structure; γ_r is the weight density of the armor unit; γ_w is the weight density of the fluid; α is the slope of the structural forces; and K_D is an empirical stability coefficient. This coefficient is to account for a variety of influences not explicitly included in the formula, such as wave

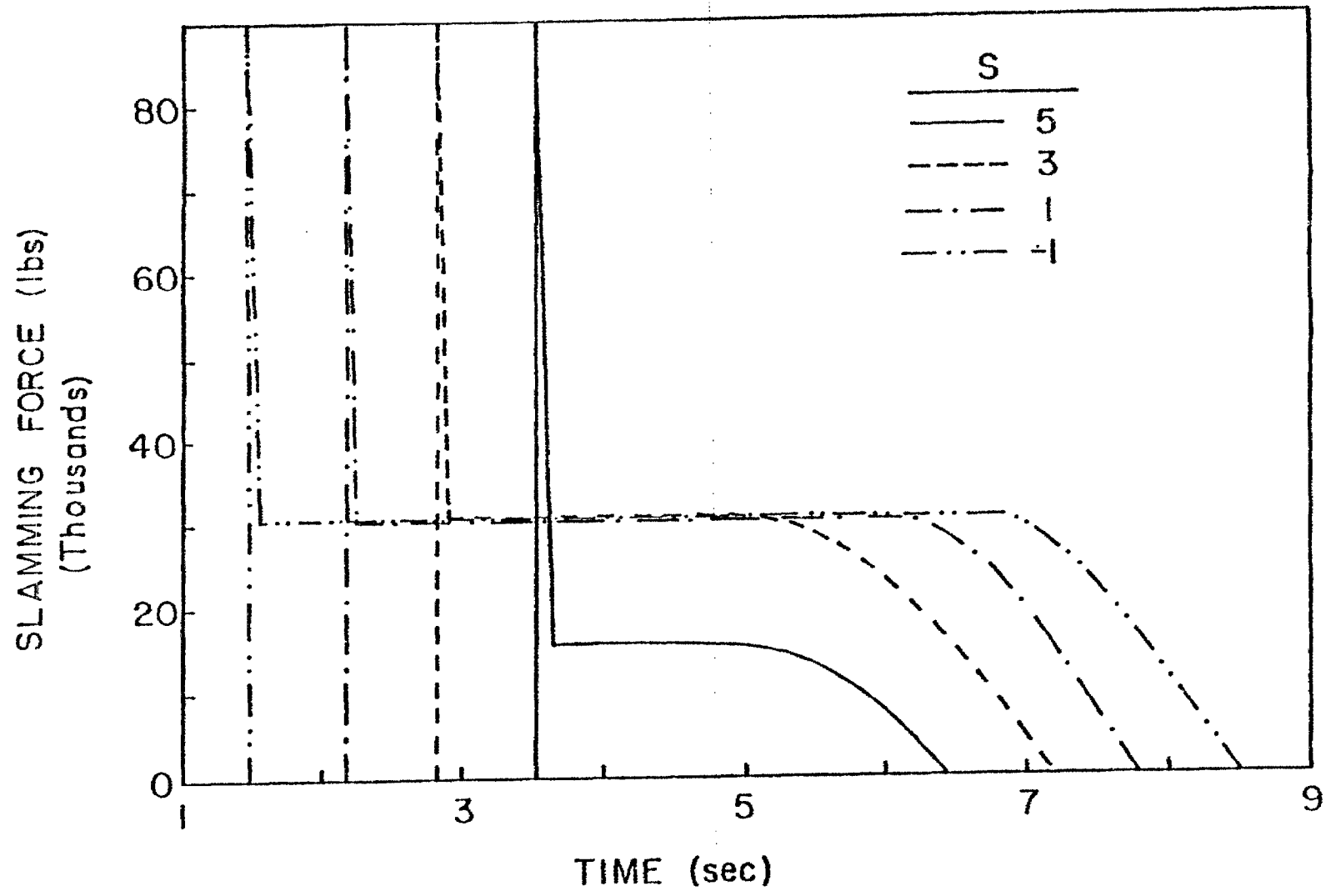


Figure 2. Time history of slamming force. (S is the elevation of the cylinder above the SWL in feet. The wave height is 10 ft.)

length, water depth, water level, degree of overtopping, location of unit, and so forth.

Three different sizes of dolosses are examined. These are summarized in Table 1. Also tabulated are the maximum stable wave heights calculated from Eq. (4) for each dolos, assuming a structural slope of 1V:2.5H. The wave periods are determined from the Iribarren number, ξ, defined as

$$\xi = \tan \alpha/(H/L_o)^{1/2} \tag{5}$$

in which L_o is the deep water wave length. It has been observed that a "resonance condition" develops for $\xi \approx 2.5$. This resonance occurs when the wave breaks on the structure when the down rush is at its lowest point (Bruun and Bunbak, 1977). Since this condition corresponds to a critical condition in terms of hydraulic stability, wave periods are scaled by this value. This value is denoted as T_ξ.

Table 1. Three dolosse examined.

	Small	Medium	Large
W (tons)	16.71	32.43	44.57
d (ft)	2.86	3.60	3.98
l (ft)	11.02	13.89	15.29
γ_r (pcf)	156	156	156
H_H (ft)	24.1	30.1	33.4
T_ξ (sec)	10.85	12.11	12.77

The wave heights are scaled by the maximum stable wave height for the dolos unit, H_H, as determined by Hudson's formula. Results are only presented for the case in which the depth is equal to the breaking depth as given by

$$h_B = H/\kappa \qquad (6)$$

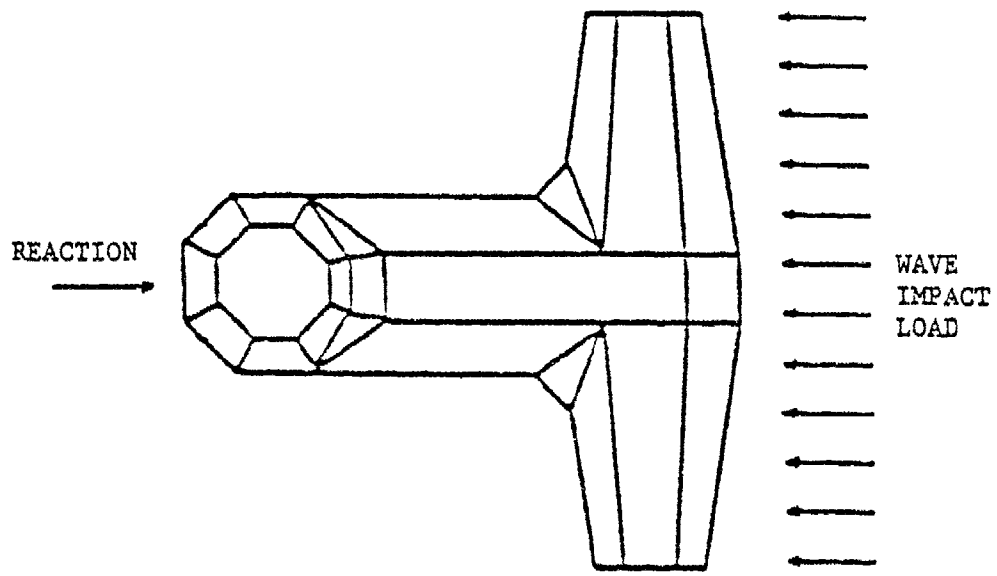
in which h_B is the breaking depth and κ is a breaking index approximately given as 0.78.

The actual loads experienced by a dolos unit are a function of the wave conditions, dolos location on the structure, and dolos-dolos interactions. These complex conditions are poorly understood. Therefore, in the present study, simplified conditions are assumed.

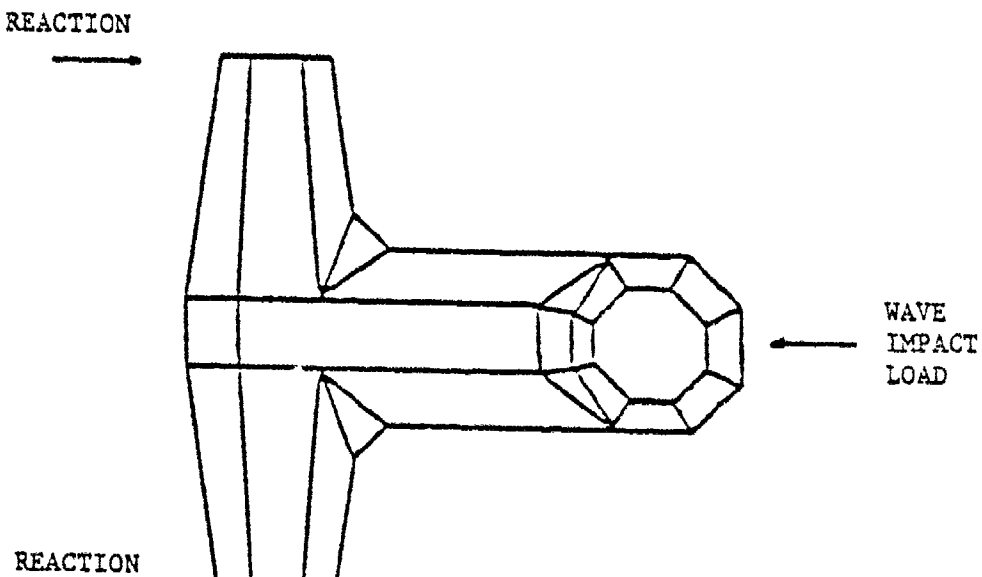
The case under consideration is depicted in Figure 3. The dolos unit being investigated is positioned among other units so it is supported at the tips of its flukes which are vertical and perpendicular to the wave crest. The wave impact forces are applied to the remaining flukes which are horizontal and parallel to the wave crest. No rigid body motion of the units is permitted in the analysis. This configuration was selected because it was perceived to be conducive to the development of large bending moments in the vertical flukes. However, it should not be assumed that this configuration represents the worst case. The self weight of the unit, which in certain circumstances can induce tensile stresses in the flukes in excess of 30% of the maximum allowable stress (Baird, 1981), was not included in the analysis.

4. ANALYSIS

Due to the complex geometry of the dolos, the dynamic nature of the wave loading, and the highly nonlinear material behavior of concrete, the finite element method of analysis (FEM) is employed in this study through implementation of the (ADINA, 1981) computer programs. An isometric view of the FEM model, representing one-half of the dolos sectioned through the mid-depth of the shank, is presented in Figure 4. The model is comprised entirely of



a) PLAN



b) PROFILE

Figure 3. Loading Configuration: (a) plan; (b) profile.

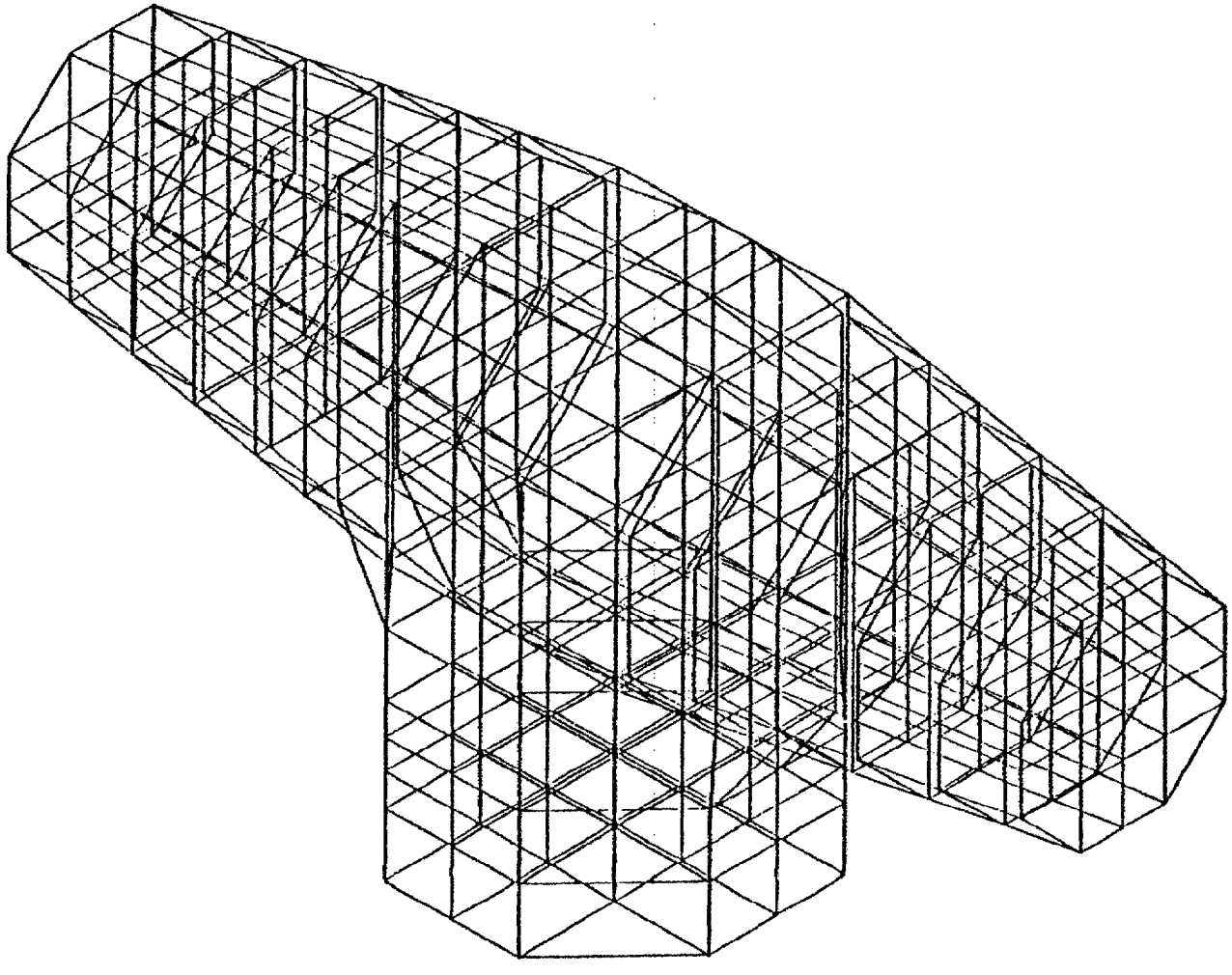


Figure 4. Isometric View.

three-dimensional, 21-node isoparametric elements (ADINA System Verification Manual, 1983).

Concrete structures exhibit a complex response with various important nonlinearities such as a nonlinear stress-strain behavior, and tensile-cracking and material-crushing failure. For this reason, linear analysis of concrete structures is generally considered inadequate in many engineering problems. The material model selected for use in this study is a hypoelastic model based on a uniaxial stress-strain relation that is generalized to take biaxial and triaxial stress conditions into account (Bathe and Ramaswamy, 1979). The model employs three basic features to describe the material behavior: (i) a nonlinear stress-strain relation including strain softening to allow for weakening of the material under increasing compressive stresses; (ii) a failure envelope that defines cracking in tension and crushing in compression; and (iii) a strategy to model post-cracking and crushing behavior of the material.

Because of the complexity of the material description used for the FEM model, an appropriate strategy for solving the nonlinear finite element equations was selected, specifically, the Newton-Raphson iteration scheme. In the Newton-Raphson formulation, the equilibrium conditions at time $t + \Delta t$ are satisfied by successive approximation of the form (Meyer and Bathe, 1982)

$$K^{i-1} \Delta U^{i} = R - F^{i-1} \tag{7}$$

in which K^{i-1} is the tangent stiffness matrix at the iteration $i - 1$ and time $t + \Delta t$; ΔU^{i} is the ith correction to the current displacement vector; R is the externally applied load vector; F^{i-1} is the force vector that corresponds to the current element stresses. The displacement increment correction is used to obtain the next displacement approximation (Bathe, 1981)

$$U^i = U^{i-1} + \Delta U^i. \tag{8}$$

Equations (7) and (8) constitute the Newton-Raphson solution of the equilibrium equations and subjected to the initial conditions $K(t + \Delta t)^\circ = K(t)$, $F(t + \Delta t)^\circ = F(t)$, and $U(t + \Delta t)^\circ = U(t)$. The iteration continues until appropriate convergence criteria are satisfied.

In nonlinear dynamic analysis, the solution of the governing differential equations is obtained by direct integration procedures. Of utmost concern in the selection of an appropriate time integration scheme is stability of the solution technique and accuracy of the analysis. In this study, the Newmark-Beta method of implicit time integration was selected because it is unconditionally stable, regardless of the time step.

The dynamic equilibrium equations for the structure are written as

$$M\ddot{U}(t) + C\dot{U}(t) + KU(t) = R(t) \tag{9}$$

when M, C, K are the mass, damping, and stiffness matrices; R is the external load vector; and U, $\dot{U}$, and $\ddot{U}$ are the displacement, velocity, and acceleration vectors of the finite element assemblage. In an implicit time integration scheme, equilibrium of the system (Eq. (9)) is considered at time $t + \Delta t$ to obtain the solution at time $t + \Delta t$. In nonlinear analysis, this requires that an iteration be performed. Using the Newton-Raphson iteration, the governing equilibrium equations (neglecting the effects of a damping matrix) are:

$$M\,{}^{t+\Delta t}\ddot{U}^i + {}^{t}K\Delta U^i = {}^{t+\Delta t}R - {}^{t+\Delta t}F^{i-1} \tag{10}$$

$${}^{t+\Delta t}U^i = {}^{t+\Delta t}U^{i-1} + \Delta U^i. \tag{11}$$

In the Newmark-Beta time integration scheme, the following assumptions are employed:

$$^{t+\Delta t}U = {}^{t}U + \frac{\Delta t}{2}\left({}^{t}\dot{U} + {}^{t+\Delta t}\dot{U}\right) \tag{12}$$

$$^{t+\Delta t}\dot{U} = {}^{t}\dot{U} + \frac{\Delta t}{2}\left({}^{t}\ddot{U} + {}^{t+\Delta t}\ddot{U}\right). \tag{13}$$

Using the relations in Eqs. (10) to (13) results in

$$^{t+\Delta t}\ddot{U}^{i} = \frac{4}{\Delta t^2}\left({}^{t+\Delta t}U^{i-1} - {}^{t}U + \Delta U^{i}\right) - \frac{4}{\Delta t}\,{}^{t}\dot{U} - {}^{t}\ddot{U} \tag{14}$$

and substituting into Eq. (10) yields

$$\hat{K}\Delta U^{i} = {}^{t+\Delta t}R - {}^{t+\Delta t}F^{i-1} - M\left(\frac{4}{\Delta t^2}\left({}^{t+\Delta t}U^{i-1} - {}^{t}U\right) - \frac{4}{\Delta t}\,{}^{t}\dot{U} - {}^{t}\ddot{U}\right), \tag{15}$$

where

$$\hat{K} = K + \frac{4}{\Delta t^2}M.$$

The selection of an appropriate time step, Δt, is very important to the accuracy of analysis. Since the time integration scheme employed is implicit, its stability is unaffected by the size of the time step. In an implicit, unconditionally stable time integration scheme, Δt should be small enough that the response in all modes, which significantly contribute to the total structural response, is calculated accurately. The other modal response components are not evaluated accurately, but the errors are unimportant because the response measured in those components is negligible. For the case under investigation, the major dynamic response is associated with the primary flexural mode of the fluke (see Figure 5). The period of vibration for this mode, for the linear case, was calculated to be 0.005 s. Therefore, a time step of 0.001 s was selected for use in the time integration scheme.

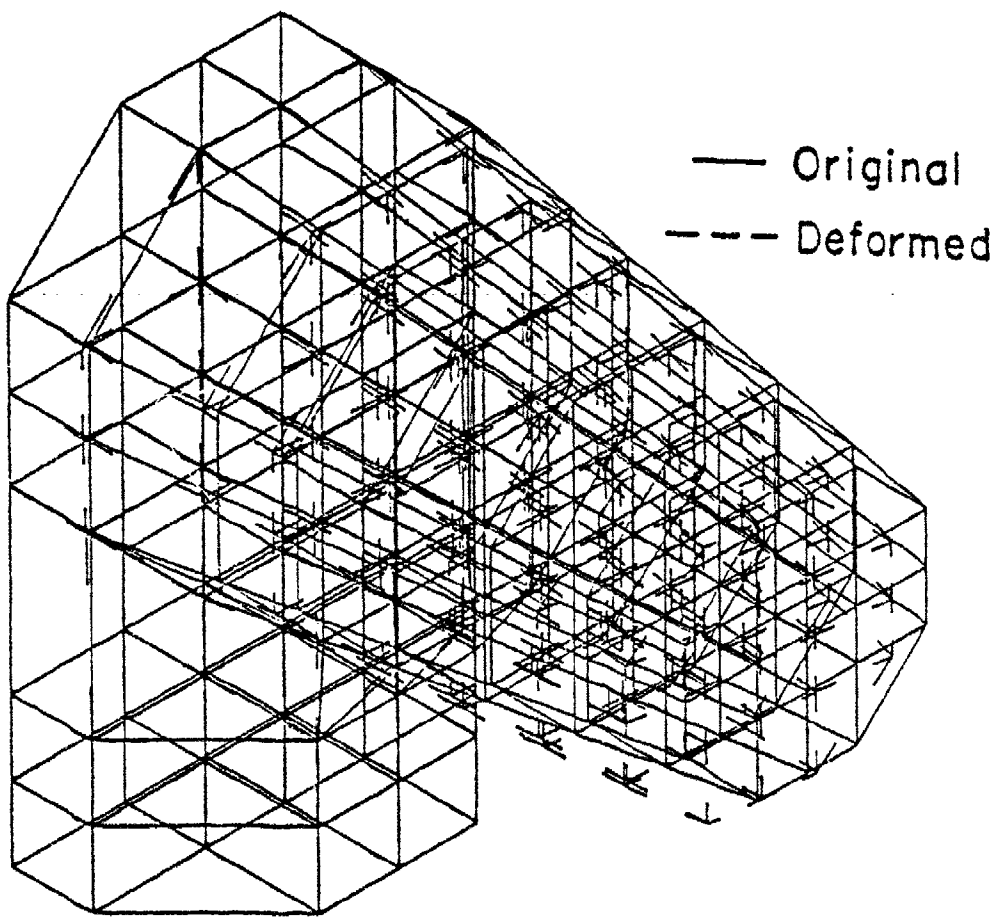


Figure 5. Fluke deflections.

5. RESULTS OF ANALYSIS

The FEM analysis was conducted for three different sizes of dolosse (see Table 1). The analyses were performed for concrete compressive strengths, f'_c, of 5000 psi and 4000 psi for each model. The dolosse were assumed to be unreinforced, and the tensile strength of the concrete was assumed to be equal to the split cylinder strength, f_{ct}, of the concrete (where $f_{ct} = 6.7\sqrt{f'_c}$). The pertinent material properties and stress limits are summarized in Table 2.

The failure criterion for the dolosse was defined as first cracking in the concrete. This stringent criterion was selected because of the repetitious nature of the wave loading. Once the structure has cracked, the effective section capable of resisting the cyclic wave forces is reduced, and complete destruction of the dolos is imminent. In all cases, first cracking developed at the juncture of the fluke and shank. A typical crack is shown in Figure 6. At this section in the dolos, high tensile stresses were induced by bending action in the fluke.

The results of the analyses for maximum tensile and compressive stresses at first cracking are summarized in Tables 3 and 4, for $f'_c = 5000$ psi and $f'_c = 4000$ psi, respectively. These results indicate that at failure, the compressive stresses in the dolosse are just a fraction (approximately 11%) of the concrete compressive stress, f'_c.

The failure of the dolosse depends on the maximum slamming load. This load, in the present analysis, is only a function of the wave conditions for a specific dolos unit. This load is calculated as discussed above and regions of structural stability are identified as a function of the wave conditions. These results are shown in Figures 7, 8, and 9 for the three dolos sizes examined. Stable and failure regions are identified for 4000 psi and 5000 psi

Table 2. Concrete properties.

Concrete Material Parameters	Compressive Strength	
	f'_c = 5000 psi	f'_c = 4000 psi
Density	.2246 x 10^{-3} lbf·s/in^4	.2246 lbf·s/in^4
Initial tangent modulus	5580 ksi	5170 ksi
Poisson's ratio	0.2	0.2
Uniaxial cut-off tensile strength	474 psi	424 psi
Uniaxial maximum compressive stress (f'_c)	5000 psi	4000 psi
Compressive strain at f'_c	.002 in/in	.002 in/in
Uniaxial ultimate compressive stress	3250 psi	3250 psi
Uniaxial ultimate compressive strain	.003 in/in	.003 in/in

Table 3. Summary of stress analyses, f'_c = 5000 psi

Dolos Size (Tons)	Maximum Tensile Stress at Failure (psi)	Maximum Compressive Stress at Failure (psi)	Failure Load (kips)
16.71	474	581	141.8
32.43	474	563	222.2
44.57	474	564	269.5

Table 4. Summary of stress analyses, f'_c = 4000 psi

Dolos Size (Tons)	Maximum Tensile Stress at Failure (psi)	Maximum Compressive Stress at Failure (psi)	Failure Load (kips)
16.71	424	549	116.4
32.43	424	547	184.6
44.57	424	546	224.0

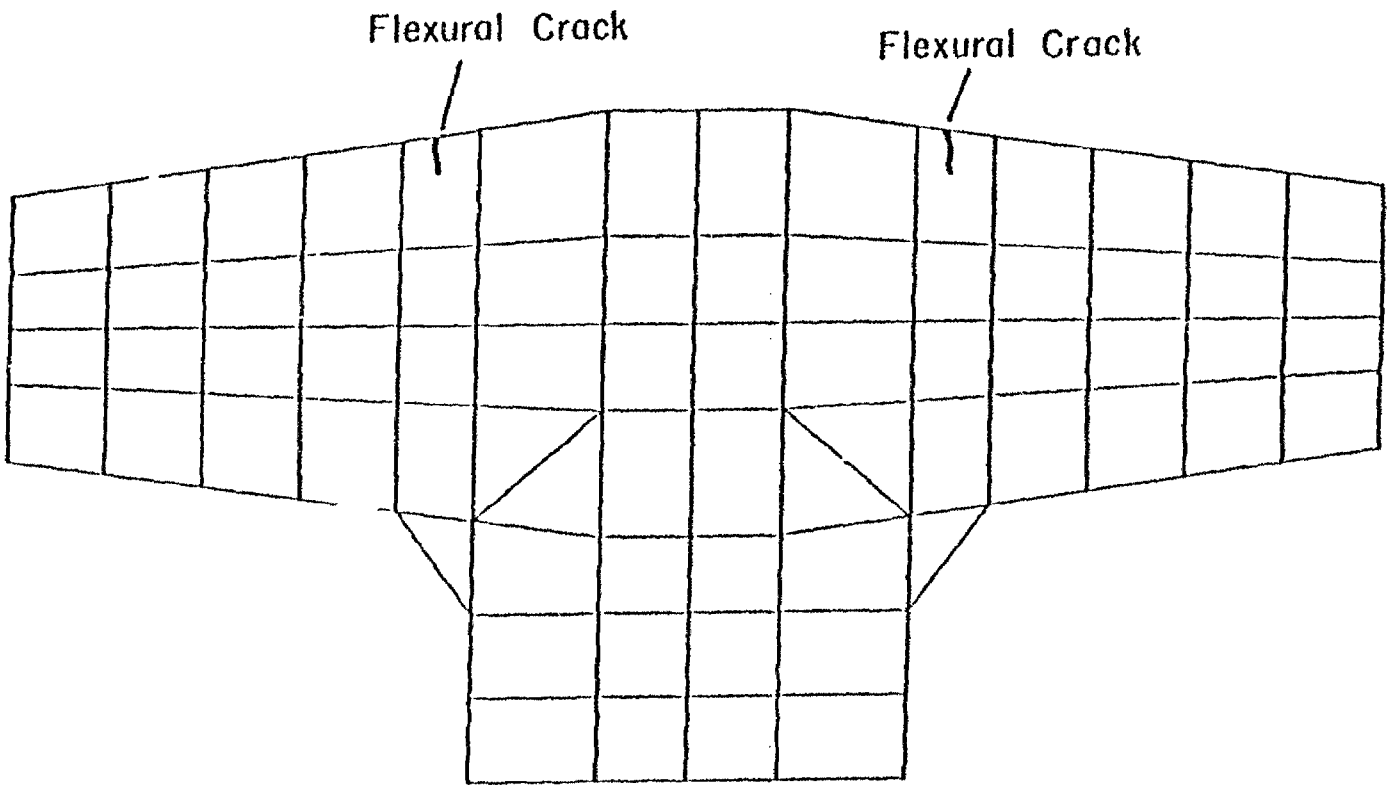


Figure 6. Location of cracks.

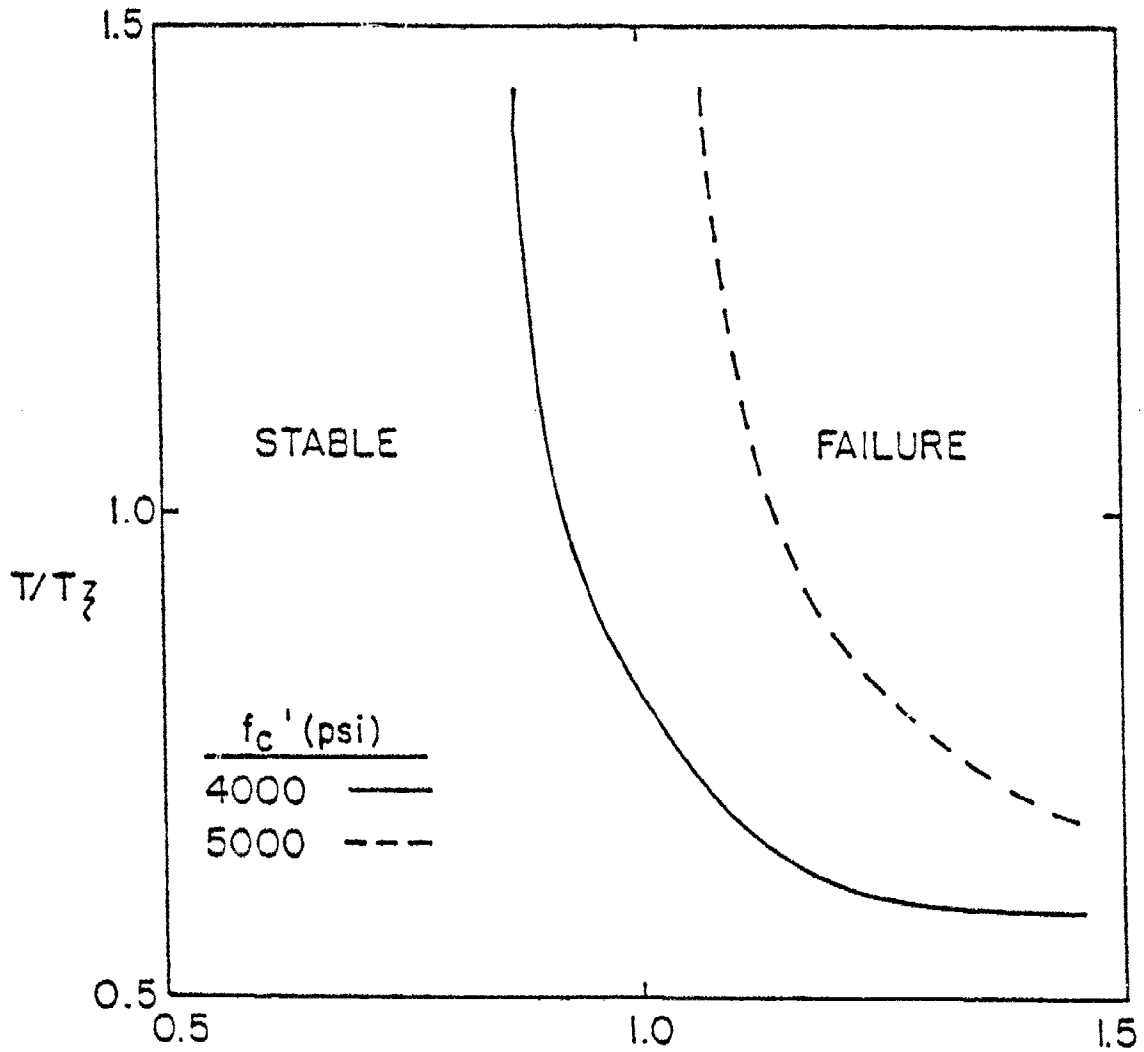


Figure 7. Failure envelope for 44.57 ton dolos unit.

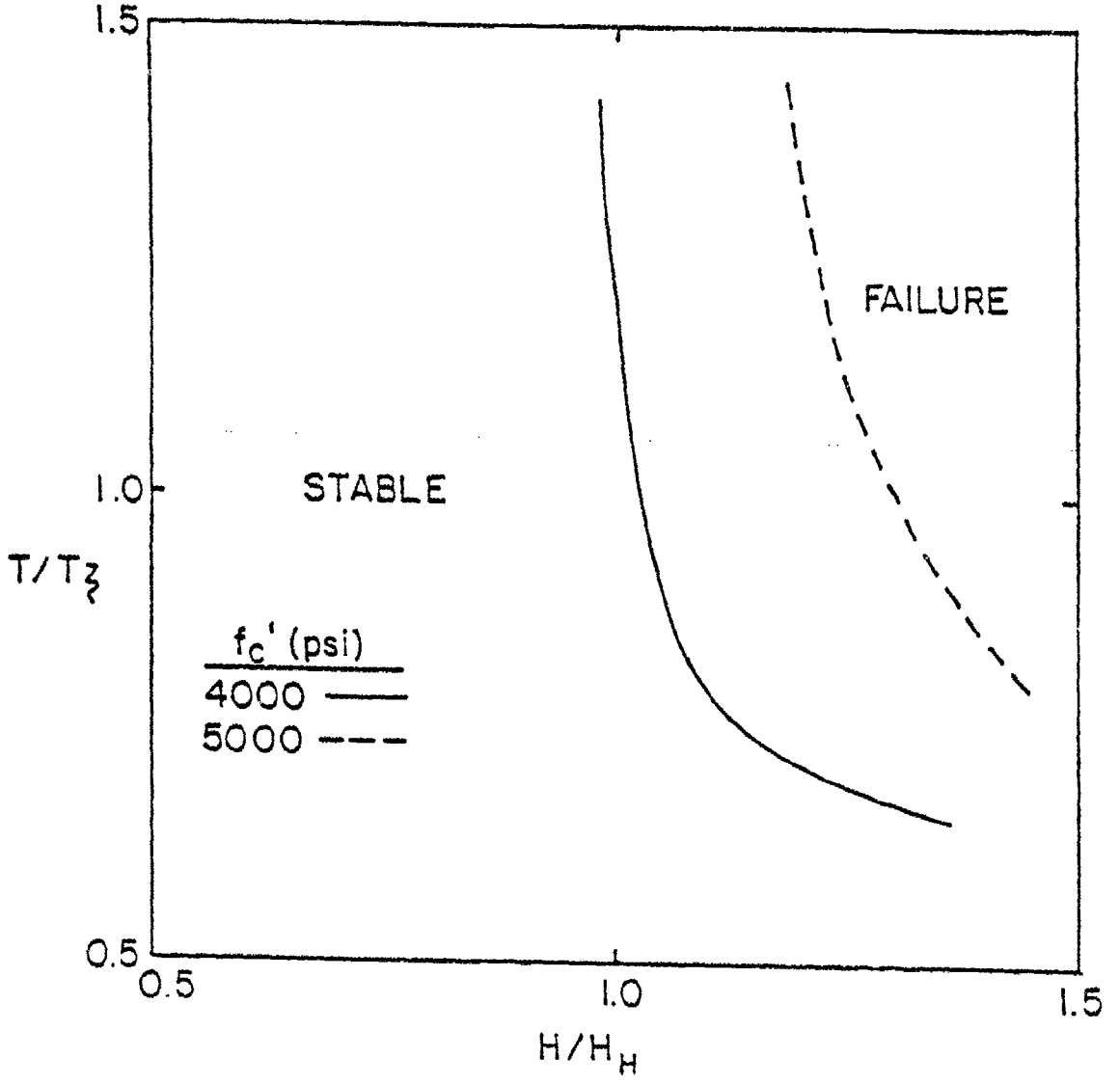


Figure 8. Failure envelope for 32.43 ton dolos unit.

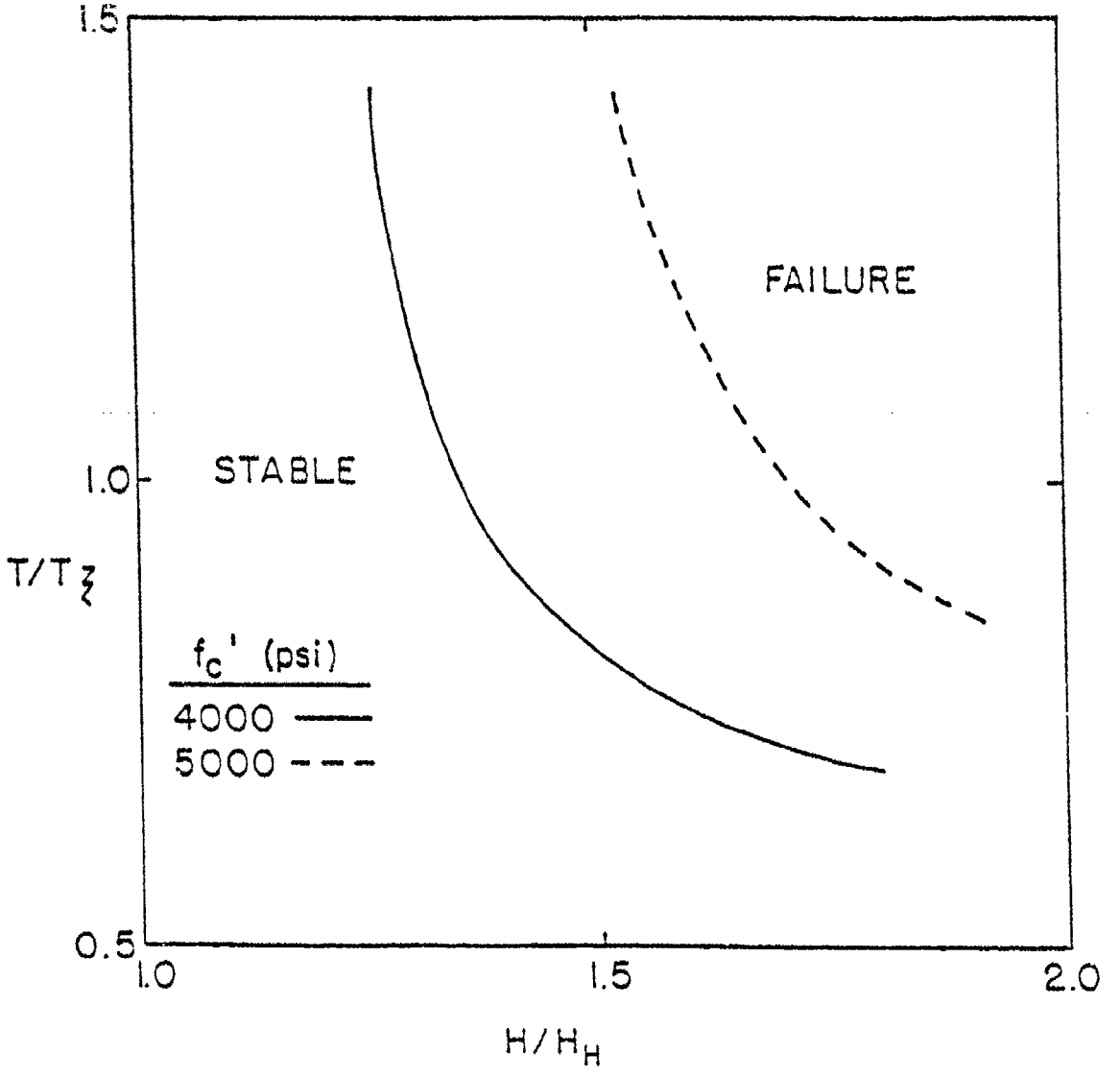


Figure 9. Failure envelope for 16.71 ton dolos unit.

concrete. For the large dolos unit (44.57 t) structural failure would occur for the 4000 psi concrete at wave conditions for which the unit is hydrodynamically stable. The higher strength concrete unit is very near failure at these wave conditions. As the size of the dolos decreases, the structural stability increases relative to the hydrodynamic stability.

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MR. LILLEVANG:

Do you have in mind trying this thing with the impact loads coming out the horizontal fluke from the other side?

DR. TEDESCO:

Yes. Again, I think one of the problems is defining what is the critical condition.

MR. LILLEVANG:

Yes. The photoelastic stuff I was talking about suggests that you get far greater stresses. The moment arm may be less but the concentration at what I refer to as the notch generates things tremendously.

DR. TEDESCO:

We have the methodology to do it, so it doesn't matter. Again, it's identifying these things.

MR. LILLEVANG:

Well, it could be useful, I think, and I've made no argument that the photoelastic analyses are definitive answers. They're not. They're reconnaissance.

Photoelastic analyses treated as a reconnaisance tool, it's useful to have a look at them and see what they suggest.

The other thing is a philosophical comment. I agree with you that you can do all kinds of things here in order to get this thing to be a surviving structure, but I referred earlier to the question, specifically to sites, specifically to breakwater, of the implications of a broken armor piece. Dolos, rock, tribar, tetrapod, you name it, any of them needs to be looked at in an engineering economics viewpoint as to whether the cost of saving it is worth the expense of replacement or an implication of mortal damage to the breakwater as a whole. It is a pimple that needs to be squeezed and gotten rid of. It's very important that we not forget the economics. If we do, we're not engineers.

DR. TEDESCO:

I agree.

DR. WALTON:

Did you look at the differences that you get with your nonlinear analysis as compared to linear?

DR. TEDESCO:

Let me say this. You look at these stresses I've shown here. You're talking about a tensile stress of 424 psi, a compressive stress of 550 psi, so for all essential purposes, this is linear behavior. Because you're only a tenth of the way up the compression curve. So obviously, if you put reinforcement in it, the behavior becomes more and more nonlinear.

What I would say is that if I took that broken unit and then continued passing waves against it, what would happen is that I would have a larger and larger cracking of the section; however, I would have greater compressive stress. Then the behavior becomes nonlinear until the time it fractured.

DR. WALTON:

The second question is not quite so related to your presentation, but maybe you've thought about it. It seems to me that post-tensioning might be less expensive than conventional reinforcing. Have you given any thought to--

DR. TEDESCO:

That was my first impression. First of all, in terms of labor, you wouldn't have to construct the cages that Professor Zwamborn showed first. All you'd have to do is, when you cast the unit, put that tendon in it. Not the tendon but just a conduit, and then, when it hardens, you take the forms off, you shove the conduit through it, and that's all you need.

You can get amazing forces from some of these handjacks that you can bring to the field, and the labor cost would be drastically reduced from having to form that cage as far as material goes, as well as just post-tensioning that thing.

MR. LILLEVANG:

If I were taking responsibility for the design, I'd be real nervous about the survivability versus quality control.

DR. LIGTERINGEN:

Yes.

MR. LILLEVANG:

Terribly nervous about it.

DR. TEDESCO:

I don't think you'd have any more problem potential with the survivability with prestressing them than you would have with conventionally reinforcing them.

DR. ZWAMBORN:

I would just tie into economics. In the case of Sines, we looked at the possibility of prestressing the 40-ton dolosse. And I know very little about prestressing. We had some experts looking at this. If you just make the design, you take the cross

sectional area of the dolos shank, multiply that by tensile strength of three or four or five, you get about 800 tons. Now, if I understand correctly, if you prestress the whole thing, you have to overcome that before you can reduce or eliminate the cracking.

DR. TEDESCO:

There are various degrees of prestressing. There is full prestressing and various degrees of partial prestressing. So you can determine just where you want to be.

DR. ZWAMBORN:

I would suggest to you that you look at the possibility of prestressing the sides rather than putting it in the center. Try and keep it nearer the outside, rather, because that's where the cracks occur.

DR. TEDESCO:

Whatever creates the best prestress field is the way you want to do it. I would say the simplest way would be to put one tendon through the center.

DR. ZWAMBORN:

I think you need to have a big tendon. It gives you effect on the side.

MR. NORMAN:

I think there's a few things that ought to be brought out when you're doing nonlinear analysis of concrete. I'm a firm believer in nonlinear analysis, but of course there's a wide variation of opinion of different types of material models for concrete.

You have chosen one of the Adena, which just shows strain softening, I believe, and there are some problems in terms of material stability with that model. You can pick some types of loading that will cause you to get fictitious answers, and not that that isn't possible.

Are you working impact type problems?

DR. TEDESCO:

Yes.

MR. NORMAN:

The tests have indicated significant rate effects for concrete material.

DR. TEDESCO:

Tensile strength, yes.

MR. NORMAN:

The bad part about that is nobody agrees on what the rate effects are and how to implement these.

DR. TEDESCO:

Yes.

MR. NORMAN:

If you do use a rate-dependent model, you can get away from the material stability problems. Again, there's a lot of discussion on that. Just as bad, and probably worse, is the cracking, the implementation of your cracking, whether it's the smeared cracking approach or if you actually try to do fracture mechanics. There's the question whether fracture mechanics is applicable. I want to point out that all of these things are very difficult to take any one or another on cracking. I don't know what in the Adena you're doing with cracking. Do you monitor opening and closing in cracking?

DR. TEDESCO:

Yes, the program monitors itself. It'll monitor the opening of the cracking. It works on the principle that once there is a crack, you no longer have a tension capacity in that.

MR. NORMAN:

Does it have a shear retention?

DR. TEDESCO:

Yes, it does. Under compression.

MR. NORMAN:

Does that shear retention change as cracks open and close?

DR. TEDESCO:

Yes, it does.

FINITE-ELEMENT ANALYSIS OF A DOLOS

Robert A. Cole and Robert E. Walker

Background

In support of the Prototype Dolos Measurement Program being conducted by the Coastal Engineering Research Center (CERC), the Structural Mechanics Division of the Structures Laboratory (SMD/SL), was tasked to perform research on the structural response of concrete dolosse. To this date, the SMD has conducted analytical and experimental work concerning the dynamic structural response of dolosse. This work supports the prototype measurement study at Crescent City, CA.

Analysis

Structural analysis can be thought of as illustrated in Figure 1. The system of transfer functions C(s) can be a simple single-degree-of-freedom (SDOF) representation or a complex approximation of the total multiple-degree-of-freedom (MDOF) system. The method selected to solve the system equations can be linear or nonlinear. Associated with this model are boundary conditions; i.e., how, where, and in what manner the structure is constrained. These constraints can range from a simply supported condition to a fully fixed condition and even involve a time varying boundary condition. The inputs to this model R(s) are the prescribed static loads, load-time histories, or initial conditions. The outputs from this model Y(s) are the stresses which have to be related to the material deformation properties and the failure law for the critical area being affected. If design information is wanted, the stress distribution throughout the structure has to be in a format suitable for the designer. Generally, the designer is fortunate that the loads and number of boundary cases are usually less than a dozen. This is the situation in a deterministic design.

Unfortunately, the dolos analysis problem does not present itself as a deterministic problem. The dolos, as it is used in a breakwater, has a whole range of orientations, a large number of support conditions, and is subjected to forces that are deterministically unknown. One problem is that the worst case may not be the case that occurs very often, and a probabilistic way of determining the likelihood of a case occurring is needed.

All computations were performed using the WES/SAPV finite-element computer code. A detailed finite-element grid was developed with brick elements. When it was realized that the detailed grid was rather extensive (i.e., it required excessive computer resources and generated large volumes of data), a simpler beam-element grid was constructed. This simpler model is to be used to investigate many simulations of boundary conditions, etc., and build a probabilistic data base.

Brick-Element Grid

The first finite-element model was a very detailed representation of the dolos structure. The structure is simplistic in one sense; but because of the octagon cross section, the detail of the chamfers, and the right angle members, the dolos presented quite a number of difficulties in the construction of the model. The model has 1513 nodes and 936 8- to 21-node brick elements for a total of 4,503 degrees of freedom (DOF). The finite-element grid is shown in Figure 2. The calibration computation consisted of: (1) considering the work done by Lillevang, (2) the static test done by Terao et al., and (3) the drop tests done by Burcharth. These test data compared favorably with that of the brick-element computation. These results are summarized in Figures 3 and 4.

Beam Element Grid

The computations using the brick-element grid proved to be a major analysis effort in postdata presentation due to the volume of output. It was soon realized that to conduct parametric studies, it would be useful to have a simpler grid; hence, the beam-element grid was formulated. The finite-element grid is shown in Figure 5. Because of the nature of the beam element, a beam element grid generator code was constructed. This code allows the users to create various grids at will.

The advantage of the beam-element grid, other than its simplicity, is that the outputs for a dynamic simulation are moments, thrusts, shears, and torques; the required information that a designer would need. The code to process the outputs from a beam-element dynamic analysis condenses the data to find the maximum envelopes of moment, thrust, torque, and shear for the fluke and shank of the dolos.

The beam grid was calibrated using the drop test of Reference 3 and the same initial conditions given in Figure 5. A summary of the computations in the form of the stress distribution at the shank-fluke intersection is shown in Figure 6. These maximum principal stresses will be used to assess elastic cracking and failure of the dolos.

Conclusion:

In summary, two finite-element models of a dolos have been constructed and calibrated for dynamic impact and static forces. A modal analysis of a 42-ton dolos has been performed, and the resulting frequencies will be used to establish instrumentation recording requirements. Further calibration calculations will be performed in the future to refine the dynamic properties of the finite-element models.

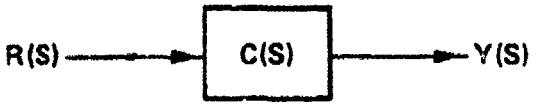
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1. STEADY STATE - FLOW
2. TRANSIENT ANALYSIS - IMPACT
3. Σ 1 & 2



$$Y(S) = R(S)\,C(S)$$

THE SYSTEM C(S):

GEOMETRY
BOUNDARY CONDITIONS
MATERIAL MODEL

THE INPUT R (S):

FORCING TIME FUNCTIONS (LOADS)
INITIAL CONDITIONS

THE OUTPUT Y(S) AT ONE OR MORE AREAS OF INTEREST

MOTION HISTORIES
FAILURE CRITERIAN = f (STRESS HISTORIES)
MOMENT, THRUST, ETC HISTORIES (DESIGN)

THE ANALYSIS PROBLEM

FIGURE 1. ANALYSIS CONCEPT

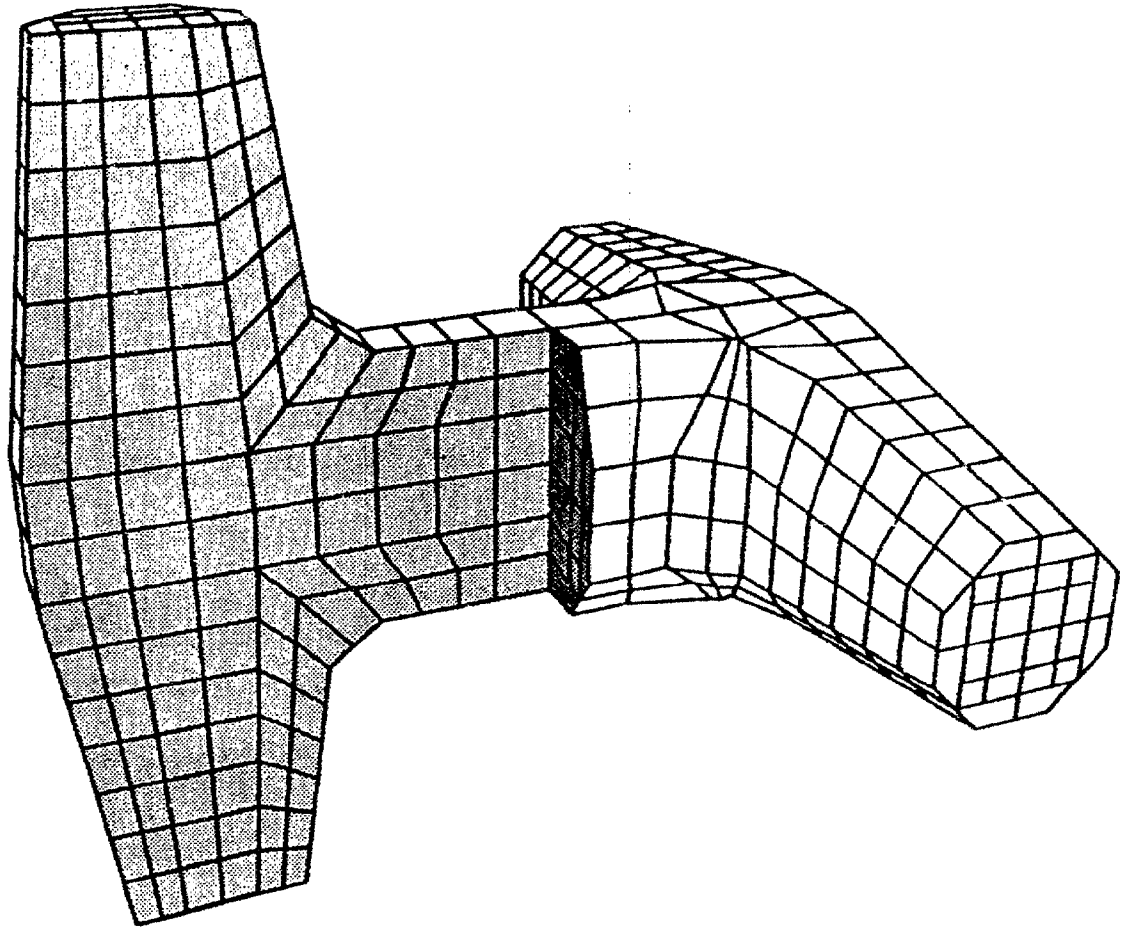
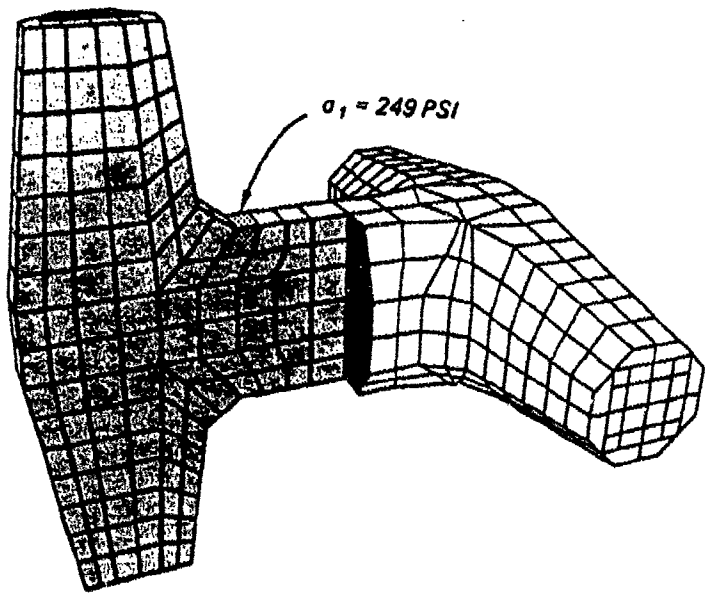


FIGURE 2. DOLOS F.E. MODEL WITH CHAMFER, SMD/WES
UNDEFORMED GRID



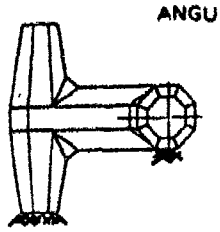
TEST PARAMETERS

L = 1650 MM = 64.9 IN.

DROP HEIGTH = 153 MM = 6.02 IN.

1.5t = 1500 kg = 3307 LB_f

ANGULAR VELOCITY AT IMPACT



$$W = \sqrt{\frac{2Mgh}{I_o}} = 1.86 \text{ RAD/SEC}$$

DYNAMIC F.E. ANALYSIS

W_1 = 116.6 htz

W_2 = 242.5

W_3 = 340.14

SCALE RELATIONSHIP

$$W_p L_p = W_m L_m$$

$$f'_c = 28.9 \frac{N}{mm^2} = 4191 \text{ PSI}$$

$$\text{MEAN STATIC STRESS} = 29.5 \frac{N}{mm^2} = 428 \text{ PSI}$$

FIGURE 3. BURCHARTH DROP TEST

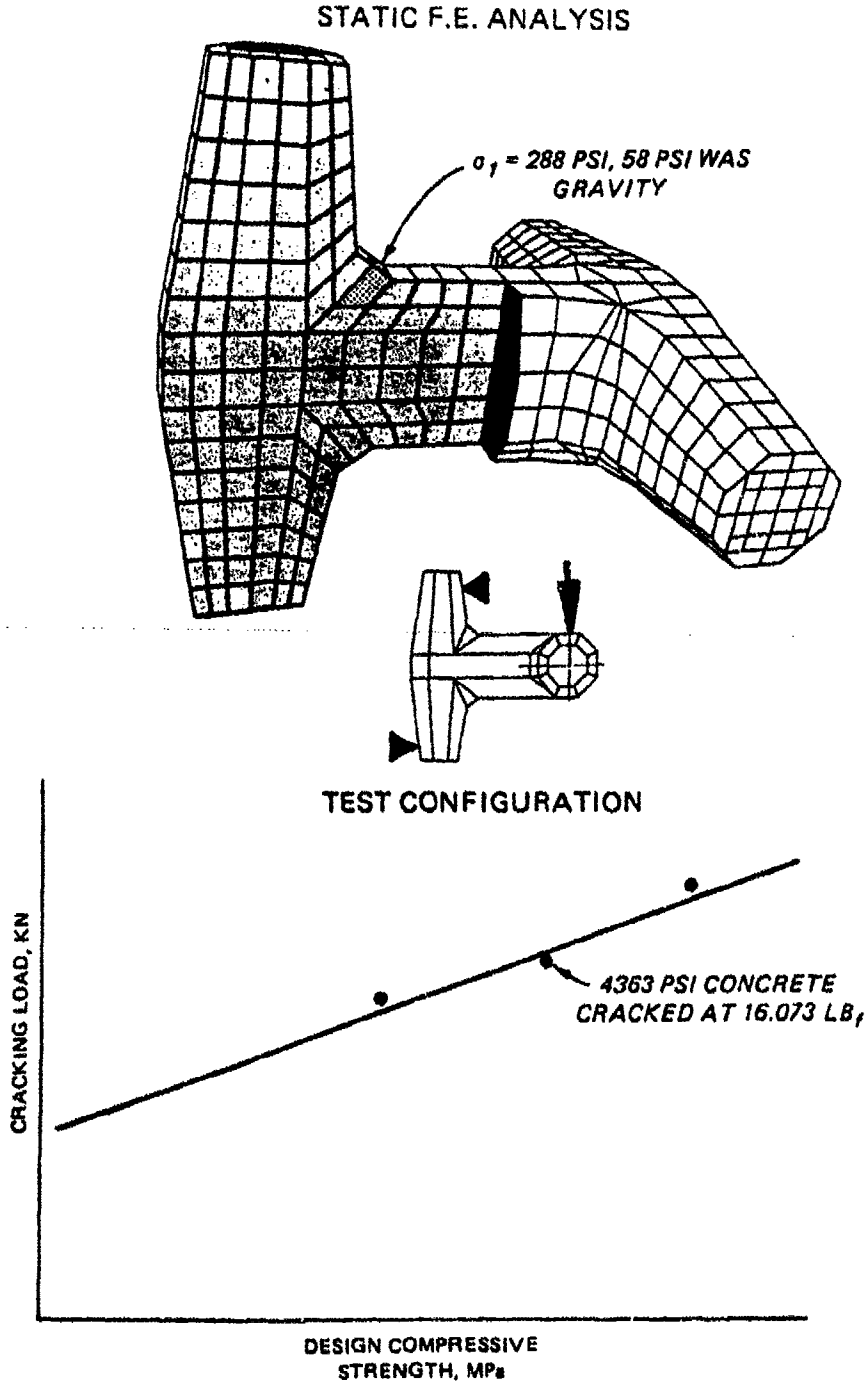


FIGURE 4. TERAO, ET AL., STATIC TESTS

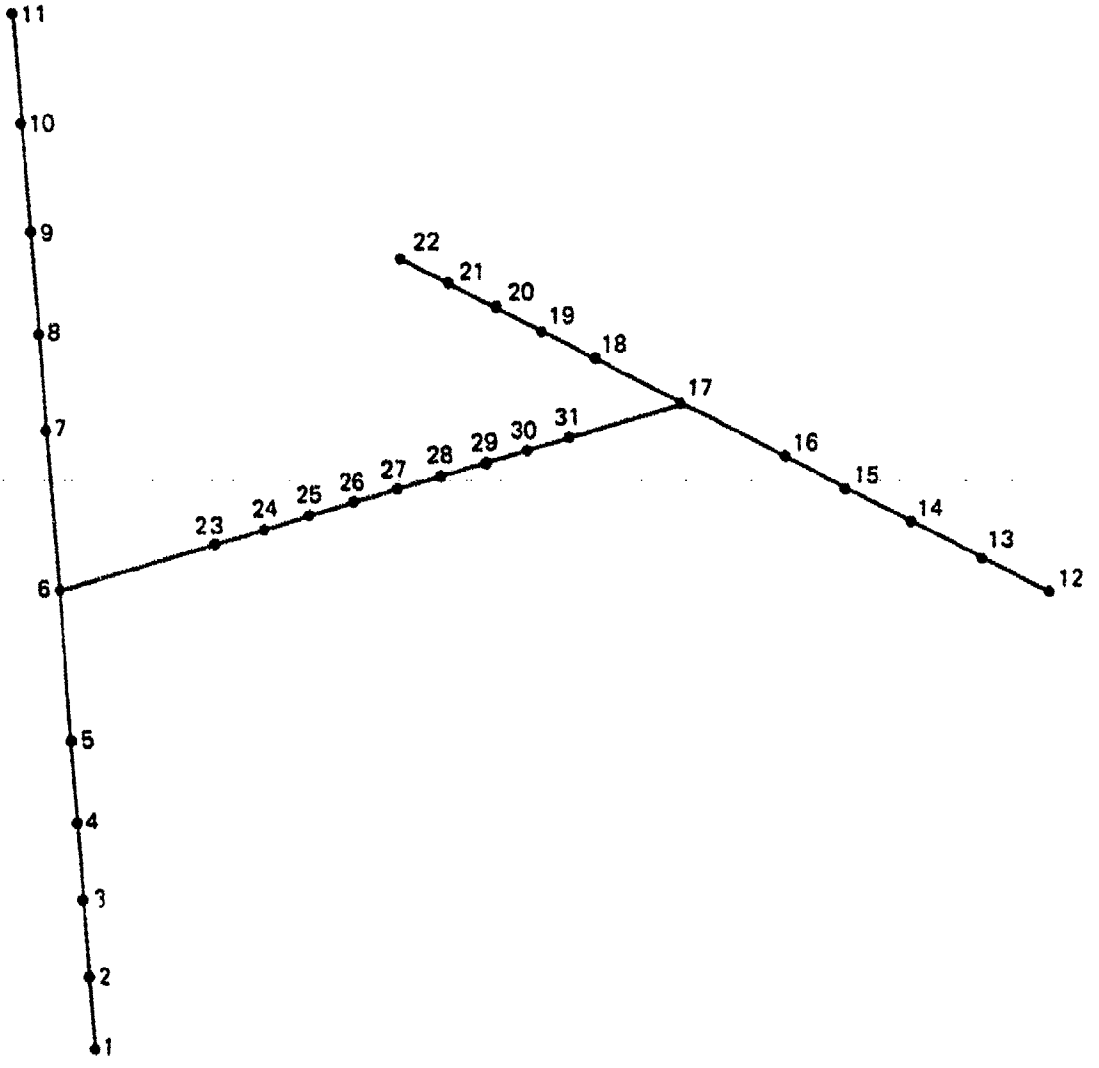
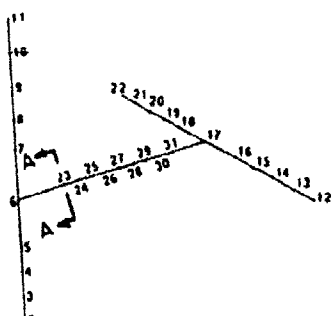


FIGURE 5. BEAM ELEMENT GRID





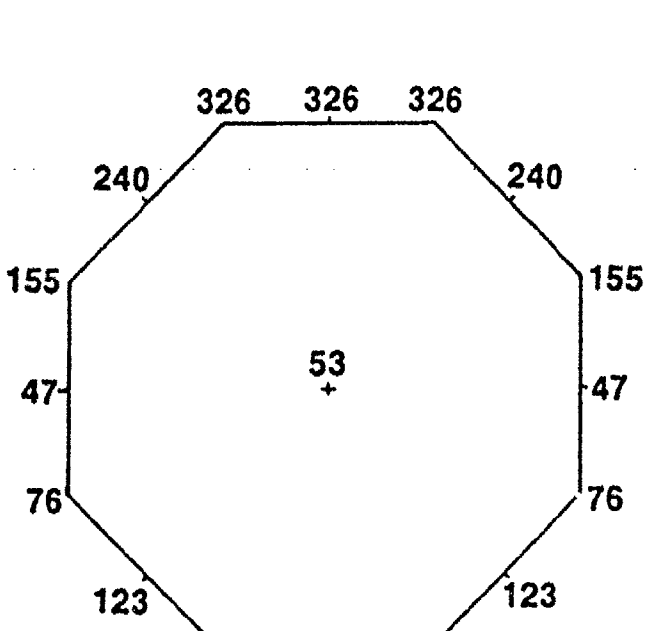


FIGURE 6. DROP TEST, SIMPLE GRID

MR. SCOTT:

You mentioned doing static test calibration with the prototype.

MR. COLE:

I have done some calibration of the FE model with static tests that have already been done.

MR. SCOTT:

Okay.

MR. COLE:

We would like to calibrate the prototype although being a very fragile unit, that size, I don't know how much we'd want to put it at risk.

MR. SCOTT:

I was wondering, if you're planning to do some calibration tests, perhaps taking the prototype unit, lifting it up slightly and--

MR. COLE:

Well, certainly, when we go to move them, I know we have the instrumentation on it that will be a calibration. When you go to move the dolos in a sling, that is a calibration recording its output when it's suspended in a sling. Its own weight would be a calibration.

MR. SCOTT:

Yes, but, to some sense, I think you could obtain some of this data. Because, taking a prototype unit and just performing a small drop test on it, just not to the point where the unit's damaged, would be useful calibrating the model. It would also be useful for evaluating instrumentation on it.

MR. COLE:

I would like to take one and do some sort of fundamental static test where we can actually put a load on the shank and on the fluke and one at the shank to actually measure those loads and moments and torque.

MR. HOWELL:

I think we're going to try to do that. What we had planned to do is instrument 30 dolosse of which we hope to have 20 and use them. Assuming that we get more than 20, whatever more than 20 we have, we can devote--

UNIDENTIFIED VOICE:

So, to answer your question, we realize the value of that, but it's a matter of your bullets again.

MR. MAGOON:

Could I ask a question? I understand WES has some sort of geotechnical model for movement of blocks in a quarry or something like that. Would it be possible to take that model--the question of the orientation of the blocks now and how many combinations do you have, can you modify the geotechnic block quarry model, whatever that's called, to use a simple block like this and then get the possible combinations, given some boundary condition of slope and layers and what not?

DR. WALTON:

I'm only familiar with the two-dimensional model of that at the WES. If there is a three, I'm not aware of it right at this moment.

MR. MAGOON:

I saw a proposal from a Dames and Moore. Was that only a two-dimensional or was that--

DR. WALTON:

That was two-dimensional. We have the early work here and then it was kind of a geotech model we have used with some of our weapons work. It is a two-dimensional model.

MR. MAGOON:

Could that be modified? If that could be modified in a very simple way just for blocks, you would be able to then answer the question of the orientation of the percent of loadings in any one type of scheme. Then you could answer the first part of this question.

MR. COLE:

We've talked about it in generalities, and we'd like to have that. If anybody's done any very tedious work, just somebody's taken a photograph of a bed and saying okay, here's a dolosse and here's the way I see it lying in the bed, supported like so, with another dolos on top, like so, and then you go dolos by dolos and get a summary of orientations and conditions and get a statistical basis to say, okay, now this occurs so much of the time. Like even a dynamic situation, too, there are conditions which are just mirror images of the other. So once you have a particular pattern, you go look at another one, you realize that it's just a mirror image of the other because you can just flip it around. Hey, that's what I just looked at a moment ago.

If we had that statistical base of those summaries, and this beam element model is, although discrete, not as awesome as that other one. I mean, not only is my argon (phonetic) value extraction three or four hours on our machine here at WES, to do a modal survey of five minutes at the most, but I need a hundred eighty core of the computer because it needs a lot of memory. Then you have to process all the data

afterwards. Every single six components per element, convert that to some sort of material, to principal stress, and then extract moments and shears and torques out because that's what I think you really want ultimately.

Now, the beam element model, I can run the whole thing in the middle of the day. It costs about $25. I can run a whole bunch of those and build up a statistical basis of--

MR. HOWELL:

I might comment that this, looking at how they're supported statistically, is not as unreasonable as it sounds. We were deciding on where to bring our cable out of instruments and we just went down to D.D.'s lab, and I spent several hours just staring at piles of dolosse to determine the particular site we were going to bring the cable out, how to rest it on another dolos a relatively small percentage of the time. The dolos, even though it does look very random, in piles, it seems to have a relatively small selection of the ways that it ends up being supported.

DR. DEAN:

The test dolos units will have a cavity to accommodate the instrumentation. Have you or do you plan to incorporate that cavity into something, a more complex model, to determine any degradation of the strength? That's one question. Then, secondly, will that supposed degradation in strength be compensated for by putting in some reinforcing? It seems to me that that cavity reduces the strength just by--

MR. COLE:

That cavity's going to act like a--it's a steel cylinder. It's going to be more steel.

MR. NORMAN:

I thought the intent on that was just to measure the forces in the field, and they are reinforced around that. Didn't we talk about that?

MR. HOWELL:

I think we talked about doing the modal analysis on the instrumented and uninstrumented dolosse and comparing that.

MR. WALKER:

There's a detail there in actual placement of that block and how we're going to glue all that in the casting process. That's not detailed sufficiently at this time, but I envision that with a rebar in there. That's the other part of the problem. Maybe even strengthen the section.

COMMENTS

BY

DR. ROD SOBEY

I would like to try to synthesize, from a personal viewpoint, what I think everybody has been talking about for the last day and a half. The topic of the workshop is measurements and analysis of structural response and the crux of that, I think, is the analysis of structural response. I would like to concentrate on some assumptions that perhaps we all agree on.

From a structural engineer's point of view, given the loading, the stress field can be computed. There's been a number of presentations of finite element model studies and established technology although, recently, there's quite an amount of computation involved in that.

The problem then comes back to what is the loading, and that's the real crux of the question. I think it's reasonably understood, certainly implicit in this meeting, that loading is essentially stochastic. It's certainly not deterministic.

What I would like to present, very briefly, then, is what I call a stochastic design methodology for armor units. The solution here appears to be available from the dynamics of projectiles, and I don't claim any originality from this. At least one person has beaten me to this by three hundred years. Newton, in about 1650, in the simplicity of what I'm about to describe, and I'm sure a number of other people, have presented it or at least thought of it. It's certainly, I think, intuitive.

What I would like to consider then is the motion of a single unit surrounded or partially surrounded by other units. The dynamics of projectiles, then, which are described by Newton's Second Law, in the Cartesian sense, there's three force components, Newton's Second Law of three moment components, so you have six equations they're describing, three displacement and three rotations.

The problem comes back to describing the force components and the moment components. Just in terms of whether the total forces in one of these "I" directions are three. Of course, hydrodynamically there's a drag force. It's a question mark there to indicate the other hydrodynamic forces. As Bill McDougal said this morning, there's a first cut, there's a slamming load, there is perhaps an added mass effect. There's a weight force that's buoyancy force because these units are only partially submerged, and the last one, the important one, the impact force from interactions with other units.

That particular problem up here is an initial value problem. In a mathematical sense, there are six second-order ordered differential equations. It's a relatively simple matter to solve those given the forces and the moments.

What I would like to present, then, is a schematic of a methodology we could use from the Newton's Second Law approach. The first thing we need is the hydrodynamics. That's to be able to predict the wave forces. That's not a complicated problem as we all know, this wave breaking involved. It's, I think, a porous media flow through the structure itself. It is a definable problem. Given that, we can go on to

predict the displacements, velocities, accelerations, components which will, of course, be random. That provides the input into the hydrodynamic loading part of the problem.

The next part is the solution with the Newton's Second Law equation, which will give us displacements, velocities, and accelerations, which gives us the movement of the dolos units within a particular confinement. If I had some more dolosse, perhaps we could envision a single dolos that's surrounded by lots of others. We're looking at motion of this one dolos surrounded by others. The displacement velocities and accelerations will predict how it moves. If there is another dolos in the road, there will be an interaction, and that's indicated here by the collision. That's a relatively standard problem in dynamics of projectiles. It's perhaps the "billiard ball problem."

Given the collisions, we can represent the impact forces and the impact moments. From that, we have, then, the complete description of the force field using any of the structural analysis techniques to get the complete stress field. We get time histories, then, of the complete forces on the structure, the complete stress field, and we can go to the probabilities of the occurrence.

If there's no collision, it doesn't move far enough. Of course it just goes back into Newton's cycle, which is, for example, one that doesn't move far enough to collide or one that's in the free surface and moves out into the wave field a little.

Just summarizing, I think what Newton's Second Law does provide, and I'm sure you'll realize, is a rational primer to proceed. It puts in perspective the information that we don't know and points out where we need to collect further information. Specifically, the hydrodynamics, perhaps the impact problems here. It's also almost identical with the field experiment that Gary has designed. It is the level of the mix of the individual particles. You can accommodate things like packing density with how close the surrounding dolosse are. Those surrounding dolosse can also be moving down so you can accommodate most of the behavior.

I think that's about all I'd like to say. I'm rather embarrassed by the simplicity of it. It's Newton's Second Law, and I'm sure it's what most of you have been thinking about for some time.

CLOSING DISCUSSION

DR. WOOD:

At this point I guess I would like to open the floor for discussion of papers from this morning's presenters. Perhaps we can begin with general comments or remarks on Mr. Lillevang's presentation.

DR. LIGTERINGEN:

I tend to agree with him that the definition of the wave climate in front of the particular breakwater which will be instrumented is very important. I have the feeling that staff gages or single wave height just in front of the breakwater does not give the appropriate information. You have the incoming and reflecting pattern and although the reflection coefficient may only be 30 percent, the actual incoming wave height is still confusing. I would like to indicate that we have developed an instrument which measures the wave height in such a way that incoming and reflected spectra can be differentiated in between, by measuring two velocities and the wave height. The technique for it is available and if we can be of any assistance in adding to that single wave gage, the instrumentation for velocity measurement and analysis, then we will be most happy.

DR. WOOD:

You're really talking about directional wave gaging, then, at the point--

DR. LIGTERINGEN:

Not directional wave gage. It doesn't give directions, it only makes different shades between total incoming and total reflected spectra.

MR. LILLEVANG:

Dr. Raichlen used techniques suggested by Professor Goda for separating the incident from the reflected, and though it was in a three-dimensional model and Goda was talking back about it in a flume, the characteristics of that site were such that it worked very well. I think whether or not the Goda technique for separating incident reflected waves would work well at Crescent City will depend on that site as to whether or not the reflections are fairly consistent and virtually unidirectional, no matter what the incident direction may be. Things of that sort are very helpful.

Now, besides the question of saying what the incident wave is, which is certainly a basic parameter that we need to know something about, tied in with stresses that are observed, the other aspect of that is that, with the reflection, there's such a translation of volumes of water down the slope of the breakwater during the recession that stability of armor pieces, particularly at the toe, is, more often than not, a major jeopardy to the stability of the whole armor zone and could have something to do with it. You need to know what it is. To what extent any of these techniques would help with that, I don't know. That problem, not just what was the incident wave but what are the phenomena that are actually working in a budget of water that needs to be transferred from somewhere it ran up, where it's got to get back to the sea again, and its heavy downward flow at the place where the dolosse or any other armor piece of the toe is most vulnerable, and under the shallow water

conditions, the shallow toe conditions we've got here, those bottom dolosse are going to be where the velocities are maximum during the reflection phenomena.

DR. LIGTERINGEN:

Well, in that respect I agree with Orville when he says that the section with instrumented dolosse give a very good indication of the actual water movement on the slope. Just by looking to the time difference between consecutive units being attacked, you get an idea of the forward speed of propagation upward and downward of the surging wave.

DR. ZWAMBORN:

Did I understand correctly that you are going to measure anyhow pressures underneath the armor?

DR. WOOD:

Yes, that is correct.

DR. ZWAMBORN:

I would like to emphasize that the input wave conditions are most important, and I believe we can look at the topography later on. I think this is the crux of the matter. If you don't know that, you're talking about 30 percent difference. That's a big difference in terms of a third power type of thing.

MR. MAGOON:

I prepared rather extensive comments, which I'll probably hand in instead of reading, but I just wanted to comment on the complex bathymetry at Crescent City. Of all of the sites you could have picked, that's one of the more difficult sites, as to nature, because of the bathymetry. So if in fact that site must be selected and is selected, I think you should, one, have an excellent bathymetric measurement and, secondly, be able to put that in a very good hydraulic model, appropriate model, to relate that to what's going on in the breakwater. So I would feel that perhaps a sophisticated wave tank facility should be funded along with this to make sense out of what's gone on. It's going to be a complex problem anyway and I would think that might give some help in this regard.

DR. WOOD:

Thank you. Let's move then to Dr. Zwamborn's presentation.

DR. GALVIN:

I have some questions concerning the three sites, the Gans Bay, the Richards Bay and the Koeberg site.

The wave heights weren't clear to me. Were they maximum waves achieved in deep water offshore or at the site, or could you say something about the waves that the dolosse actually experienced?

DR. ZWAMBORN:

The wave heights that I referred to in the talk were all wave heights at the structure itself. I then converted from the wave rider record to the actual site and--

DR. GALVIN:

So they were individual waves, not--

DR. ZWAMBORN:

You're talking about maximum waves. Of course, the Koeberg case and the Gans Bay case are both different. On the Koeberg, we had twenty-four cases where we had that limiting wave height on the breakwater over the period of two years. On the Gans Bay, I think it was four occasions. In the Richards Bay case and the Gans Bay case, we used the measured wave heights from the model to convert from the wave rider position to the breakwater position where we're talking about.

In the Koeberg case, the refraction is very simple. It's a very straight cross line, straight contours, and there was no refraction. In fact, the wave rider is sitting just off the inlet basin, in deeper, 23-meter water depths just about right in front of the structure.

DR. GALVIN:

After you gave the general details on those methods, you then mentioned what I think you said was a hundred-year storm in May 1984?

DR. ZWAMBORN:

Yes

DR. GALVIN:

It wasn't clear to me whether that hundred-year storm had been included in the previous discussion or if it was something that came after that analysis.

DR. ZWAMBORN:

No, it came afterwards, but I did mention the effect of that storm as well. Again, in the case of both Gans Bay and Koeberg, which are in the Cape area, the wave heights were, again, the limiting wave heights, six to eight meters. Eight and a half meters at Gans Bay, and Koeberg, normally, six to seven meters, limited. In this particular storm, the wave set-up was such and the water level set-up was such that we could have had waves up to about eight meters at the peak of the storm.

DR. GALVIN:

That's maximum height?

DR. ZWAMBORN:

Yes.

DR. GALVIN:

Do we have an information on the compressive strength of the concrete used in the dolosse for those three sites?

DR. ZWAMBORN:

Richards Bay is in the order of 40 meters, possibly. Koeberg is more. It's 50 to 60. Gans Bay, I don't know. I would have to check on that. It was a recent contract and very much attention was given to the quality of the concrete, so I think it would be.

MR. DAVIDSON:

Was the data plot that you presented in generalized data, for breaking conditions as you did in the prototype or was that developed for basically deep water waves?

DR. ZWAMBORN:

No, those were the basic test series, what we've done over the years on this. Well, not particularly breaking waves conditions. So that is why, as I mentioned, these comparisons are very, very rough. Also, comparing the flume test with the head, just in order of magnitude to proceed, we found a direct comparison like what was done at Gioia Tauro is much more useful. That's where you can draw some direct conclusions from, which we did, about the comparison. In my case, I just sort of indicated the orders. These curves are used only as a first feeling.

In presenting the different methods of motion, we tried to get as much information as possible out of these tests presented, and we know the next step is to use that. Are you going to use the displacement curve or are you going to use the displacement properties or are you going to be dropping? We have to find all these different types.

MR. DAVIDSON:

If that's the best we have, that's a good valuable piece of information. I'd like to add that to this prototype study of Crescent City. In the past there has been design for nonbreaking waves, which offshore they don't break, immediately offshore, but as they come into that structure, due to that old mound in the dolosse section, I think we're going to have to be careful as to whether we call it breaking or nonbreaking, if those are the terms. We're going to have to be careful how we compare the form of the wave that goes with these forces.

MR. MAGOON:

I believe that when the severe waves come, they're all breaking considerably seaward so that only those waves that are small enough will break directly on the structure. In the large storms, they break considerably seaward.

I wanted to ask a question about the wave, and I think maybe Mr. Davidson could answer. As I understand, the design wave that I believe we're still using is in our design calculations, is that wave which would have been there without the structure in place. Is that what you test in the model?

MR. DAVIDSON:

Yes.

MR. MAGOON:

If we're trying to correlate with that, or correlate to our existing type of analysis, we have to be sure to somehow be able to get that represented value.

MR. DAVIDSON:

Right. I would assume so because the 35-foot design wave, as previously used, (I looked back in my calculations and that's what they gave us), was based on a hind cast, bringing it in from a deep water point to that structure with a hind cast so no structure's there. That's the way it usually goes in Hudson's formula.

MR. MAGOON:

I believe it comes from the model here at WES. There was a similar model of the Crescent City, and at that time some waves were run and the waves we got were furnished by WES. Of course, now, perhaps we have a little better refinement of that, but I think they came from here to start with, on your very early model of Crescent City.

MR. DAVIDSON:

Along with those decisions, we're measuring the wave without the structure in place even on those or the model. So that's another consideration as to that construction.

DR. WOOD:

Any comments?

MR. LILLEVANG:

I guess it's important to point out that what I'm thinking about when I talk about the incident wave, it's that wave. I hope that everybody is hearing that the way I'm saying it and that I'm hearing you the way I understand it. The incident wave to me is the one that would be there if the structure were not there to create disturbances or reflection or whatever else.

DR. WOOD:

All right.

UNIDENTIFIED SPEAKER:

Did we hear yesterday that there was going to be directional wave? I'm kind of confused by what I thought I heard yesterday and what I'm hearing today.

DR. WOOD:

Yes.

MR. HOWELL:

There is going to be a directional wave gage used. What's not decided is where it ought to be. There are two philosophies. One is to put the directional gage out in some moderate deep water site where it could have good bathymetry and then use a model technique, whether it be physically numerical procedures, to bring the incident wave into the breakwater. The other possibility would be to place the gage closer to the breakwater and just accept the fact that the bathymetry there--we know just by looking at Crescent City that we do not have long crested waves breaking on the breakwater. The wave energy is low, so I think that's something I would like to see more specific recommendations rather than just a lot of philosophizing about.

MR. MAGOON:

I believe there are aerial photographs, taken during the largest storm occurrence that I know of, February 1960, and it shows these very long crested waves. So there--

MR. HOWELL:

Aerial photos always show long crested waves.

MR. MAGOON:

Well, they're continuous breaking at the structure so there are--

MR. HOWELL:

I think it unfortunate that people went up in airplanes and looked at waves. It always shows these long crested waves. The guy down there doesn't see the same long crested waves. I agree with you, but it doesn't necessarily mean that if you're out standing on a breakwater--

MR. MAGOON:

I don't think there's any question of seeing them. I would like to suggest you look at the 1960 storm photos which were furnished to the Chief's office, and other files, and I think you'd be quite convinced if you looked at those.

MR. HOWELL:

I'm not arguing. I'm just saying that if I'm standing out on the breakwater, when I dodge the waves, it's quite obvious that I wouldn't be able to get out there the times that I do if they all broke at once.

MR. MAGOON:

It's almost a thousand foot section or something, it's almost continuous over that length when it's quite large. I think you can see that. I'm not talking about way inside, but it's a pretty long crested phenomena and it's quite well defined. Of course, in that case that the wave would start to break in deep water, they'll break continuously from round rock into shore. So they're really quite a major wave system I would think.

MR. HOWELL:

I'm not disagreeing. I'm just saying the aerial photographs can be quite deceiving.

MR. DAVIDSON:

That's getting into a problem in which I've got concern, and everybody else has too.

You're saying they're long crested, so we should assume that over that 300 feet that wave hits all of that structure at once?

MR. MAGOON:

I'm not assuming anything. I'm just saying the wave is observed. It's up to you to interpret how.

MR. DAVIDSON:

Well, that's where Gary got us, and this is what a lot of you brought out. Our concern, depending on where this section of instrumented dolosse are, or if they're simply instrumented, or in two areas, I'm an advocate for knowing what the wave is in front of this section and this section if they were separated. With the present technology, I don't know if he can get that, because, to start with, they are going to put a gage offshore and by refraction, you'd bring it in. It is one wave. They're going to call it one wave at that point. If anybody's got any suggestion of how you get it over this three hundred feet and segregate it--topography has its effect--we'd like to know that.

UNIDENTIFIED SPEAKER:

D. D., personally, I'd like to say that, from my point of view, it's very clear to me that whether you have long crested waves or don't depends on how far the storm is away. If the storm is a couple thousand miles away, you have long crested waves, and if you're in a local generating area, you don't. I think it's that simple.

MR. DAVIDSON:

You're going to have both at Crescent City. We can show you the waves where, as Orville says, they look like they break over the whole thing, and then there's instances where you'll have concentrated breaking of the wave.

DR. WALTON:

I'd like to make two comments. I think one is, with the directional gage that you have, it would be good to look at the result, directional resolving power to see whether you can pick out the crested waves--some better than others, I suppose. I don't claim to be an expert on that, but I think that would be useful to do.

I think another question is whether you plan to do wave-by-wave analysis, and then I think this question of whether the waves are long crested or not becomes very important. Because, if you intend to do wave-by-wave analysis, and I guess you do from

what I heard yesterday, then I think you could probably analyze the long crested conditions better than you can short crested.

DR. WHALIN:

I think, for the discussion, nothing should be assumed about any limitations on the instrumentation. It's been stated we're going to have one directional wave gage. Well, we're going to have at least one directional wave gage. Obviously, spending this much money, we're not going to worry about throwing in an extra two or three gages.

MR. MAGOON:

If I could add to the waves--

DR. WHALIN:

Any suggestions that folks might want to make about numbers of observations or locations or density of gages or different types of gages would be very much appreciated.

DR. GALVIN:

D. D., I think you told me that you consider the monochromatic wave condition as a conservative condition, if anything. If it will stand up on a monochromatic, it will do all right with spectral waves?

MR. DAVIDSON:

I feel like that's my personal opinion over a range of wave conditions.

DR. GALVIN:

I sort of agree with you, and I think maybe you could apply that to the field case of whether you need many gages or fewer.

MR. MAGOON:

Yes. I just wanted to add one thing about this wave measurement.

One of the things that was suggested years ago by the SERB Board, I believe it was Professor Wiegle and Professor Dean, was to place a time lapse camera on top of the Battery Point Lighthouse and actually, although it looks obliquely into this area and only works during daylight hours and when it's not raining and so forth, it would probably be a very inexpensive and fairly simple to do. And it's a fairly small angle. I think it's only about twenty degrees, but it's a 150 feet high and maybe as high as you can get it. I don't know how high you'd get there. But I think you could get a rough picture of what's going on here and at least know whether they were swell that was coming that day or we had seas or what's what. So I would think it's very simple.

MR. DAVIDSON:

Well, I was interested in size. I'm an advocate of defining the wave form across the dolosse section, if that's what we're looking at. You heard Gary say that he can

look at data and determine whether it is static, which is very low time series, or pulsating, which is a long thing, and impact, which is peaked. Now he's looking at a record, he's seeing those responses, and he can place them in those categories. What I'd like to see is also, on the hydraulic end, somebody standing there and say the waves are breaking and impacting. Because the impact load could be obtained from a surging action of two units hitting together.

I'm trying to get this into the detail now that goes into defining this wave along this reach. It may be breaking in one set of instrumented units. It might be sort of a running up in another set or in another time. So his analysis won't always give us all I want from the hydraulic end. I just point that out. Do we need to go to that detail in order to have something in the end?

DR. WALTON:

Just one reinforcement to what Orville said. That is that a camera, I think, has great qualities in the sense that you can tell whether a wave has broken or not and where it's broken, and have it in your field of view. You could labor over a wave trace for a long time and never know whether it's breaking or broken or whatever, but just one look at it.

DR. McDOUGAL:

I had a comment I was saving for later until we're talking about the experiment. It was a comment on the field program. I think they should routinely take still photographs of each armor unit and show it contacts with its neighbors. The motivation I was thinking of is you need to know what those boundary conditions are for the stress analysis. Go out there, take a picture, and that way you don't have to guess.

MR. DAVIDSON:

Use it on the instrumented units.

DR. McDOUGAL:

Yes, and how they interact, where their contact points are.

MR. LILLEVANG:

How about stereo pairing?

DR. McDOUGAL:

That's a possibility. I was going to take it one more. Under the worst conditions, when they're moving, it's the most difficult time to get out there to take a picture. But under those circumstances it might be useful to have moving pictures. Anyway, that just follows along.

MR. LILLEVANG:

There is great value in understanding what's going on if you can put them under a stereoscope. I find myself, with a split frame thirty-five millimeter camera, taking a picture of something, stepping over about four feet and taking another, and when I get back I find greater recall of what I've seen or discover things that I didn't see, and

stereo is very helpful. You don't need to be precise about how much you lose because you're not going to take measurements from it.

MR. MAGOON:

I would just like to add that I have seen some of Dr. Endo's facilities. Perhaps some of you didn't hear the paper in South Africa, but it's actually a very careful and well done work. It's a number of dynamic tests, very large instrument station, instrumentation packages, and equipment, and I believe it would be perhaps sharing or cooperating somehow in doing things. It would be mutually beneficial to the parties involved.

DR. WOOD:

Dr. Endo, I noticed in your pictures you had some of the miniature dolosse instrumented. Did you have strain gages? There were wires coming off of it.

DR. ENDO:

That tetrapod. Yes.

DR. WOOD:

There have been a number of comments about the making of measurements on models. Could you comment on how you regard your results?

DR. ENDO:

So, you measure with the instrument to tetrapod the size of approximately fifty kilograms. We pretty generally study in increment so there are no problems to get the movement of the rocks. We use the one kilogram models so we will use the same wire to measure the fifty kilogram tetrapod.

MR. DAVIDSON:

I guess, following that line, though, is the material that you were using the larger test concrete? The large scale-model units, are they concrete?

DR. ENDO:

Yes.

MR. DAVIDSON:

And there will be strain gages on the concrete?

DR. ENDO:

Yes.

DR. WOOD:

Are there any other comments at this time? That is one of the directions, certainly, that has been suggested a number of times.

Are there any other thoughts on the problems of materials for replicating scale models or instrumentation or such?

MR. MAGOON:

Yes. I'd just like to add that there was some discussion that the use of instrumented units in the model revealed some problems with the impact of those units, or the deceleration. It may be, instead of it being a problem, that ought to be a research effort. Because I think if they're that close, that that's all that's left, that might be a very good place to perhaps conditionally accept the approach and try to solve the problem of that deceleration.

DR. da SILVA:

One ought to regard the material behavior. I think there should be emphasis on what the importance of the repeated loading has on the behavior or fatigue type of phenomenon because I think we have seen that it has relevance to the strength of the armor unit.

MR. DAVIDSON:

Within the model material.

DR. WOOD:

It has on the prototype.

DR. da SILVA:

Well, it has on the prototype and, therefore, there should be some way of taking that into account.

In the matter of the strain gages, I was waiting for Dr. Ligteringen to say something because we talked about it at noon. I did not know of the problem with strain gages, and my understanding, from what we discussed, is that the contact time is important in defining what the size of the magnitude of the force or the peak will be. Once we have very much of a long linear problem, the effect that the stresses should--the material properties should be reduced by the same scale as the geometry, and they are not, will create, in my opinion, a local phenomenon that is very much different from what takes place in reality. At the same time, it distorts, let's say, the pulse, so what we read from strain gages for the particular single type of load may be difficult to correlate to the prototype.

I don't know if I said something that is terribly wrong, but that is my understanding from what I heard.

DR. LIGTERINGEN:

That is correct. In addition to that, I agree with the idea to continue along these lines. I wasn't intending yesterday to say that we shouldn't do so, only in the actual practical measurements which we are doing at the moment, where we haven't come up with the material, say a model concrete material, which reproduces all these properties and characteristics. We have chosen the line of deceleration because we have more confidence in them. Of course, at the same time, we should continue efforts

in this field of developing model material which does give a proper prediction of the collision and the corresponding forces. At that moment, I will have confidence in strain gages.

UNIDENTIFIED VOICE (Bob):

On that same point, I would like to ask whether we need a model material that will simulate concrete in those respects. Because it seems to me that if I understand what I heard this morning about the capabilities in nonlinear finite element modeling, then perhaps there may be something as simple as a coefficient for impact that could be applied to model material. That coefficient of impact could be determined by simulating both the model materials and concrete units to get a coefficient that would be applicable.

Yesterday you replied that this test with forces which are large, which I think is even valuable in itself, but if you can get that transferred, you know, something as simple as maybe coefficient or maybe depend on the square of the impulse or something. But if we know the characteristics of the material and can understand those well enough to finite element model, seems to me there's the hope of doing model tests with the best material for that and then transferring those results to the prototype.

DR. LIGTERINGEN:

Yes. But, for the time being, we do not have sufficient tools to determine the actual impact times as they are occurring in the prototype.

UNIDENTIFIED VOICE:

Couldn't that be simulated with knowing what we know about concrete and nonlinear behavior or--

DR. LIGTERINGEN:

For the time being, apparently not. We have been looking into it, but, for instance, in the case of dolosse, the tests which Hans Burcharth did, he could only correlate the full impact time because he has been measuring it with a structural characteristic of that particular dolos, by taking into account the factual model, the mode of vibration of the dolos.

Now, for a cube or for a tetrapod, it would be a completely different phenomenon and therefore we can't tell, for a specific concrete, for a specific time, what the actual impact time is in prototype. I think there the first research effort should be put in because, if we have that, we might come up with the transfer coefficient or something like that.

UNIDENTIFIED VOICE:

There may be simple bench tests, as Bill McDougal put it this morning, that would allow you to determine those characteristics. I may be wrong, but isn't the finite element model a good way to establish this correlation, or is that expecting too much?

DR. da SILVA:

May I present something in that respect? From what we have seen, I think that there are still many questions even relating to the actual behavior of those nonprismatic forms of concrete. There are so many doubts. Some other effects, like thermal cracking, which are not so important in dolosse but may be in other units--I think that placing very great expectations on what can be done with a finite element model, speaking as myself, because I'm more of an analytical man than implementist, would not be justified.

We have to work with the tools we have, so it may be we have to do some approximate guessing that will estimate, with some judgment that we like to think is good judgment, what has been done. But I don't believe that at this stage we can really put too much trust on the finite element method to solve such a complicated problem.

MR. LILLEVANG:

I have a slightly different reason for being interested in the development of good model concretes. That is that if I approach a definite project and model the structure I have developed on paper and want to see its behavior, the question has been raised, more than one time, as to what are the concealed factors that cause my thermal setting--plastic model pieces are much, much too strong to respond to the possibilities of breaking and what that breaking signifies in terms of the continuing life of the structure, as to whether it is a pimple or a cancer. I think those questions need answering even if they're viewed as being, to some degree, qualitative. They are, nevertheless, guides to judgment, and I don't believe that I will be all finished with my work on the day we can all feed it into something and it comes out and nobody applies judgment. The judgment aspect is the thrill and the reward.

So to dispose of this question of whether or not an overly strong model piece conceals a fatal flaw, I think we need positive answers to that instead of speculative ones. For that reason, a model armor piece with all the appropriate compliances with scale, I recited a few this morning and there probably are others--I don't think I mentioned Poisson's ratio, but all of these things, to find the optimum. It won't be the "most best" but maybe the "least worst" can be found and, thereby, we can improve the confidence of our conclusion-drawing as we go on.

Now, if we then find out this is a very important thing to the structure as a whole, uniquely to that structure and its site, to the approach of waves and how they are concentrated and how they attack, then we certainly have to have a very close examination of the things we're talking about, which is the unique characteristic. Whether that then is done by finite element or by a faithful scale material or whatever else, I'm going to be amazed and greatful for what those who know how to do these things can tell me. I will ask the questions before I decide whether or not I go along with their judgment.

The first item I want to know is if this breakwater is going to survive whether or not there's been some breakage.

DR. ENDO:

My company has done the full scale dropping test using the proto tetrapod, and we have done many sizes of tetrapods for the dropping test so that now we have a conclusion that if we use the large 50 kilogram to get their strength, that scaled, now, but I try to confirm this to get to the conclusion.

MR. MAGOON:

I would be interested in knowing how he compares the information he's acquired with the stuff that Danel did many years ago and reported.

Are your results consistent with his on drop heights and the critical heights at which fracture occurs? He said it didn't matter what the size was, if the material was identical and the geometry was proportionate, that the tribar broke at something like three meters' drop if it all went onto the same kind of a platform. That was very interesting. I'm wondering whether or not you say things in your data. Do you?

DR. ENDO:

(Nodding head)

UNIDENTIFIED VOICE:

Were you referring to tetrapods or tribars?

MR. LILLEVANG:

Those were tetrapods.

DR. LIGTERINGEN:

I think the article written in 1960 was slightly different from the actual results I got from some. The actual conclusion was that the drop height wasn't different from the actual weight of the units. But when you see the actual results, you see indeed some difference on the weight of the unit.

MR. MAGOON:

He never gave a plot showing scattering data.

DR. HEIJDRA:

But gives the actual results of about one meter twenty. The actual results differ from about thirty centimeters up to one meter twenty. I thought that the larger ones was for the thirty centimeters and smaller units was for the one meter twenty.

DR. LIGTERINGEN:

Yes.

DR. WOOD:

I think we'll move on to Kiyomiya's presentation, a unique concept in breakwater design.

DR. GALVIN:

I wonder if he can comment on the total overall force on the unit as far as sliding over the bed. Is that likely to be a problem?

DR. KIYOMIYA:

We have big concern with the sliding problem of the breakwater. We make some device to stop the sliding. One way is under the breakwater to arrange the small blocks to stop the movement. Also, any devices proposal?

DR. LIGTERINGEN:

I understand in the model test you have prevented the sliding. So you looked upon the idealized case where you only had the hydraulic input and analyzed the stress and strains.

Another matter is whether a structure like that can stand up on a certain foundation. My question was whether you have looked into that. Because having the lateral force from a structure like that must be gigantic.

MR. MAGOON:

I believe they investigated that in a different program. There were a number of programs. If I can remember, this one addressed the structure itself.

DR. KIYOMIYA:

We put some measure in the mount, some kind of measure gaging, but these measures did not work during the entire observation, so I cannot determine whether this is a problem.

DR. WOOD:

All right. Comments on Dr. McDougal's and Dr. Tedesco's presentations?

DR. McDOUGAL:

I'd like to start out with one. If anyone is interested in that information, I think it's going to come out in press in a journal called Computers and Structures this June.

MR. WALKER:

This goes, I guess, with regard to the structures. Did you attempt to use any of the current data base to calibrate your model, as you envisioned it, for the dynamics versus any of the drop tests?

DR. McDOUGAL:

No, we didn't do that.

MR. WALKER:

The reason for the question is that, based on the grid size, we, in our linear analysis, cannot reproduce the focusing of the stresses at the corners. We have some concern about the magnitudes that we "calibrated to in linear analysis," and so we were interested in whether you--

DR. TEDESCO:

Yes, I think that's a good idea to calibrate the numerical model because to run a wave with a finite element analysis like this, you put in the wrong direction, I wouldn't give you a plugged nickel for it. I mean, I've had enough experience in finite element models that you have to calibrate your model. It's a function as you go along.

MR. NORMAN:

We were talking earlier about the modeling. Was this part of your presentation? Did you do some modeling testing?

DR. McDOUGAL:

We did no physical modeling. This was a first-run-through problem.

DR. TEDESCO:

Strictly numerical.

DR. McDOUGAL:

Physical modeling is in the back of our minds.

DR. TEDESCO:

We need something to be able to really--we hope this will be testing.

MR. CHIARITO:

I thought it was a good point that was brought out about the utilization of material. I think it's a good point that point concrete is very weak in tension. One way to try to utilize the total strength of the material, in a way, is post-tensioning. Prestressing or post-tensioning is really the same thing in this matter. I think that's one way to make full use of the concrete if that's the material that is chosen. Otherwise, you might have to choose a more efficient material.

DR. LIGTERINGEN:

Like what?

MR. CHIARITO:

That's the question.

MR. LILLEVANG:

Wrought iron never rusts according to the industry.

MR. CHIARITO:

I think the key concerning material is that materials is not the biggest problem but possibly utilizing the full strength, as was proposed. A post-tensioning manner is a viable solution, I think.

DR. WOOD:

Are there any comments on Bob Cole's presentation?

DR. HEIJDRA:

I think the main point is that it's important that you know the load scheme. What we want to do in the Netherlands for the second phase of the study for the concrete strength of armor units is to collect data from the same hydraulic model of the load acting on several positions and the impulse acting on the unit itself so we have the information of the load scheme and the impact itself. I think that can give some information to calculate afterwards what kind of method you have to use, if you have to use the linear model or maybe you have to use a non-linear model--one like what you have seen this morning. But, I think it can help that you have a hydraulic modeling and that you watch by high speed film or by video, at the same time, the positioning of the load and measure together the impulse acting on the units. That can give information for calculating afterwards.

MR. CHIARITO:

I have another question that I'd like to propose to Bob, Mr. Lillevang, and Doctors Tedesco and McDougal.

Possibly in a model sense and a prototype sense, you mentioned photoelastic methods, experimental stress methods, for analyzing maximum value of stresses that occur. Here is something I propose to you. Does it sound feasible to possibly brittle coat some prototype units--this would not involve any kind of instrumentation except the brittle coating itself--and possibly brittle coat some models and run some flume tests on these to see if there's any comparison? This would give you a measure of maximum stresses that would occur at some threshold. Granted, the brittle coating is limited. It would be qualitative in nature, but it still might give some idea what is the correlation between model type responses and prototype responses.

DR. WOOD:

Re[illegible]se?

MR. LILLEVANG:

Sounds useful to me. I don't know enough about the materials that are available and how they'll behave emersed in water or pasted onto plastic or whatever you're going to use for your model pieces, but if we have the luxury of assuming that that's no problem at all, then I can't see but what that could be a very inexpensive and useful device with which, again, to do reconnaissance and begin to look at it and see what's happening here. We don't really know.

Photography is fine if you can stay all under water or all above water, but refraction, if nothing else, messes you up as you try to see what's going on in the area that you are most interested in. All I can say is blessings on thee, little man. Try it.

DR. ZWAMBORN:

This ties in with what Dr. Heijdra just mentioned.

Whenever the operation has been completed at Crescent City I very strongly recommend that you do survey the whole structure very, very carefully. I don't know what you can do under water in this area. It's probably also a problem like it is in so much of ours. But I think if this photography is perpendicular to the slope there is a very good possibility. I don't think you only want to monitor that area where you have your instrumented dolosse. I think you want the whole area, see whether this was unique or whether the reaction is average over the whole section.

I think if you use that technique plus the profiling, which is basically based on WES's own technique, but I think it's very effective straight after construction. We use it all over the breakwater's bases. You always can come, one or two years afterwards, and there's always some questions which you can't answer anymore. The reason for that, of course, is this Sines only happened in 1978, and some of these structures were built before that.

I think a full monitoring of what is there after construction of this is extremely important.

DR. HEIJDRA:

I think in this respect it is worthwhile to mention again that they have prototype measurements and they hope to do it in 1986. They construct a bridge perpendicular to the breakwater. The breakwater is water under the cubes, and they want to instrument several cubes with accelerometers on the basis of the results of hydraulic model tests. So I want at first to test in a model at the worst location, where we actually find the largest accelerations or the largest movements, and locate there the instrumented units.

I think it's constructive then to see on the bridge, to have the possibility to instrument several things like the wave height, the water level, They can install there a camera for taking film or a video. All these kinds of things you can get are in one section.

I think that's more or less the same comment Professor Burcharth has made, that you have to concentrate it in a certain area, the instrumentation part. Then you have, for a certain cross section, all the information. Especially for the situation at Crescent City is such that you only have for a very small--a two dimensional situation, you are just near-by the elbow. I think that is quite another phenomenon occurring on the breakwater and that's just different from what happens in the, say, cross section that is more or less a two-dimensional situation. For that purpose, it may be interesting for you to contact the people in Belgium who are performing some prototype tests on breakwater that are about the same as what you have here at Crescent City. But they also do some efforts in that.

MR. HOWELL:

That's an interesting idea. What group is that?

DR. HEIJDRA:

The best way to contact them is to call the consultant firm Haecon.

DR. LIGTERINGEN:

It's the breakwater of Shaebourg (phonetic) which will be instrumented in that way.

DR HEIJDRA:

They have performed model tests before, and what they intend to do is to have a model that is a prototype on the breakwater that can perform the test immediately, to see what happens.

DR. WOOD:

I invite general comments. Let me set a bit of a ground rule in that if you would present a comment allow some response to it to get an exchange around the group. Would someone like to start off with a general comment?

DR. GALVIN:

This is not too general. It's somewhat specific. I haven't heard anybody mention the Manasquan Inlet jetty which is the only east coast U.S. installation of a dolosse structure that I'm aware of, and Mr. Gabbert (phonetic), at the Philadelphia District, has made a very nice study using rather simple survey techniques in photogrammetry of tne movement and placement of those dolosse. I think much of what he's done, of course, is applicable to what you will be doing at Crescent City.

MR. HOWELL:

That work was sponsored by our group here at WES and there was a paper on it at the Coastal Engineering Conference in Houston. There also was a WES report on it.

I will mention just briefly we intend to use the same photogrammetric type units at Crescent City. A lot of these suggestions, which I think are excellent suggestions, are things that we planned to do anyway. I think we have a tendency to focus more on high tech strain gages and computers. We also intend, as part of this program, to do the best monitoring job that's ever been done so far during construction of a breakwater, including all of those techniques.

MR. MAGOON:

I'd like to make a comment, and there may be quite a bit of disagreement on this. In the meeting we've had here, in sections on breakwaters and particularly using large concrete armor units, I believe that it has not been demonstrated that the movement of the armor unit is required for breakage of the unit. I think this is a very important point that for practical design in a long section, just the cumulative loads near the bottom of the section may be enough, almost, with just the pulsating forces,

as Burcharth's term, to cause breakage without that. If that's the case, then, in that particular case, a lot of simple finite assumptions could be used.

DR. WOOD:

Good Point.

DR. DEAN:

I would like to recommend that consideration be given to saving about six or so of these bullets until after you get experience with the first complement, fourteen or so, because I think this may justify it on a couple of grounds particularly. One of them is some reservations that Omar mentioned this morning: if you have cable problems, you don't want to have them on all twenty. And I can see that also, even if things go beautifully with the, say, fourteen or so, then you may want to explore other placement locations or configurations. You may learn something from the first batch, and you then can more specifically place the remainder.

MR. DAVIDSON:

Professor Burcharth's opinion was we should put them all in one place and more or less have them nested in, and we want to have in that matrix nesting units. He didn't seem to think that we needed rocking or anything necessarily. I know that consolidation and maybe some small rocking will occur anyway, but we don't design them for that reason. Maybe we do, maybe we don't. But should we purposely put one or two of the bullets in a position to get those movement forces?

I've argued this point with Gary, and he's instrumentation. He doesn't want to lose that unit. But I would like to see what the group says, because that would be valuable information.

DR. LIGTERINGEN:

Even if you place them nested in the matrix you are building, the chance that one to twenty is moving is about 100%. So I wouldn't worry about any--

MR. DAVIDSON:

I'm talking about obvious movement, something we can see on camera. I want to see it flip-flop.

DR. LIGTERINGEN:

That is a technique we have applied to the model where we just have the hydraulic without instrument units and looked to which units were really in unbalance and replaced those with the instrumented ones. Of course in the model that is much easier than prototype, but I agree. It would be good to have indicated one of which you are sure and of which you know which mode it moves.

MR. DAVIDSON:

Right, that's what I'm getting at. What is the consensus?

MR. MAGOON:

I would agree that you should certainly have something that you'd get a fairly large excursion, fairly quickly, on in the experiment so that if in fact it has to sit there for six months before the major storm comes that the slow movement of the change, hour in and hour out, hasn't chafed the cable in half so that you can get some result, as poor as it is, fairly--or as unrepresentative as it might be, fairly quickly.

In response to Davidson's questions, I would like to suggest that you could find this selection of the pattern of where to put the armor units in a hydraulic model. We've talked about the techniques in Canada. I would suggest that you just ask them to come down here and plug in their instumentation in your model and actually try to determine the question of where the unit should go in the prototype in the hydraulic model.

If we have any faith in the hydraulic model at all, we ought to be able to tell where the places to put the unit are better in the model. Just let the model tell us the answer.

So I would like to suggest that you invite the Canadians to bring their instruments down. You might have a little interface problem, but I'm sure technicians could figure that out. Stick the dolos in your model here, certainly drum up something with the right scale, plug it in, and see what motions you get. You ought to be able to get a good answer yourself, I think better than we could guesstimate, without using the analog model.

DR. LIGTERINGEN:

What instrumentation are you talking about now?

MR. MAGOON:

I'm talking about using the simple instrumentation that the Canadians, Baird (phonetic) and Turcke (phonetic), and these folks--are using. Taking those instrumented model dolosse and simply running them in an appropriate model of Crescent City and trying to determine, instead of just all of us guessing in our mind where to put them, put them in the model as best they can in arrangement and let the model tell us the answer to that.

MR. HOWELL:

What is better? That is something I haven't heard yet. What is it? What are we looking for? What's better? The model's going to tell us something.

DR. LIGTERINGEN:

I don't understand.

MR. HOWELL:

What is better? High stress, low stress?

DR. HEIJDRA:

I think, first, it's important to know where the movement occurred. I think that you have two direction on the forces acting on the unit itself. But I think that it's important to know where you have the largest movements. I think from that point of view the overlay technique will give you a good idea of in which part of the cross section above, also below, the water level will have the largest movement.

MR. HOWELL:

Is that the consensus of the workshop? That we should put instrumented units where we have the largest movements? I think we already know that if we have an instability, that we will have broken units. I guess one of the questions is, if you have a correctly designed breakwater which is subjected to forces within its design range, should you or do you get stresses which exceed the limits?

What Burcharth's suggested is that you do not want to put the units where there is necessarily a lot of movement. He says you should treat the design problem separate from the structural problem. We don't need to go out to the prototype to realize that we had the units on the head section. We know that already.

So what is good?

DR. LIGTERINGEN:

In my opinion, what you're looking for is an average picture of what happens in that breakwater cross section. I agree that it's better to have them in one section because then you have somewhat higher and somewhat lower. As I set some of them, or one or two of them, in movement, and some might fail, okay, but that's what we're looking for.

MR. HOWELL:

So you're saying not to try to put them especially in an area where we can guarantee large movements or potential instability?

DR. LIGTERINGEN:

I would propose you do have an overall picture of what happens even if that sacrifices one of the instrument units.

Coming back to Orville's suggestion to have it first find out in the model, if you try to build a model with twenty particular units, and you repeat that twenty times, you'll find that your matrix is twenty times different. It's no use, I think, to try this out in the model, because what you view in the model is most certainly not the same as what you would achieve in the prototype.

DR. WOOD:

Dr. Zwamborn?

DR. ZWAMBORN:

Yes, I think that was a remark that I wanted to make. I would agree with it except if the model would represent the entire picture. In other words, if you could see the difference between the particular concentration area and so on. Your present model, I don't think, can do that. Only what you can do there is sort of decide that the central area is probably the best in terms of being away from your boundary conditions.

Now I've got a big problem with putting one of the dolosse in a position where it will move around a lot. I would then suggest what Bob Dean (phonetic) had done. Put one out now and find out whether you've got technical problems. Don't use sixteen. Put one to use now. Just put it on top of the bin in an un-interlock position and it'll move as much as you like. I think that would be a very good practice.

But then, when you do your actual test, I think you should place the dolosse any way you would do it in reality, not make it weak or better, than you do it under the normal circumstances, because you want to try and find out the statistics. This is why Hans would like to get them fairly bolstered. I would like to agree with him that you shouldn't spread them along the breakwater. I think you should have a reasonable spread along the face, not all at low water line or something. Have them reasonably spread along the face.

But if you would have one outside, it's like a hundred years' storm. You get one point setting right up; whereas, you're looking for a matrix, the actual stresses in the dolos which are put in the normal sort of matrix forms which you would do under well designed structural condition.

So I've got a problem about putting--because it's only the one. That one will probably break a few others also in the motion. I think you should have a well constructed use of breakwater if you put the dolos in. But I would go with Bob. I believe I have some experience with monitoring of prototype with these mooring forces and I would say put one in as soon as possible and find out what your practical problems are.

MR. MAGOON:

I would like to respond back that the actual comment I had was if we're trying to look for some idea of movement in the under layer, its upper layer, whether they should be at the toe or where they should be, I think the model won't represent exactly the situation placed, but I would hope that the model could at least tell you where the extreme conditions were. So it would be my thought at least you would try to glean what you could from the model although it's not going to represent the individual placements.

I think that maybe you're trying to decide whether to put this way down in the toe or put it up on the top of the slope. Obviously, I think the model would be good enough to give you some kind of response like that. So I feel like some testing in the appropriate model would be very helpful.

DR. WOOD:

I want to ask Mr. Lillevang if he might want to make a comment or two on his mapping for those of you who would like to get with him and look over some of these preparations for the map.

MR. LILLEVANG:

We've been talking about Crescent City, and I think a pertinent point can be made here that the published chart of the Diablo Canyon site showed affirmable detail. This is too sophisticated an instrument for my experience.

We see similar things here. Here's a mound of some kind. Here's something that is awash at half tide, bare at low water, awash at three-quarters' tide. These are like a thousand yards, maybe fifteen hundred yards, out from the breakwater.

We've got Steamboat Rock here. We can expect azimuths of waves to come this way as well as swellings here and here and the others. I don't know what the farther south azimuth is. I think it was point something like this the other day.

Here's another feature in here. Rocking in here. An eight-fathom depth right there, something that breaks surface here. Rocks in here. The remnant of the aborted attempt to get out to Round Rock can be seen here.

This is a contorted terrain situation. Established wisdom currently has been that if something isn't some proportion of wave length of what's coming in, you ignore it in terms of effects of waves. Not so. You can take a mock of a very contorted sea floor like this and propagate waves into it. Whether they're monochromatic spectrum makes no difference. You see major concentration of effects.

At Diablo Canyon, we had a spot right comparable with here, if that were the end, and a submerged--or it came awash--rock with a hundred fifty-five thousand cubic yards of material in it standing on minus seventy. It came up and broke the surface at low tide. And in the lee of it, where you might expect to see some reduced energy, we've had the worst problem with constructing. We couldn't go through that area.

The effects of things that we have thought of as subtle or merely contributory in a texture sort of a way can be extremely important.

Now, with that independent mind, when you've got a severe problem of design in a site where your contours are not regular, it appears to me that it is very important to spend money on good mapping. I was able to convince my client of that at Diablo Canyon and I have, along with me, some reduced-scale prints of what we got from the surveyors. There is a title page that shows the layout. You'll see four sheets indicated on this title page that were surveyed at twenty feet to the inch for two-foot contour intervals with a precision that the surveyor was willing to sign them as being the contours that were within one interval at all places one might want to take a check.

In order to do that on things like the mound rock, he put rod-men out there, divers in scuba gear, and they routed those things where it was too dangerous to take a boat or where you fell off a slope too rapidly and they had to do shots. The slopes were so steep there that at twenty feet to the inch it was hard to draw the two-foot contours. They were too close together.

The rest of the map is a hundred feet to the inch original scale except those parts that we had at twenty which we blew up and fitted into it. We have twenty feet to the inch stuff of detail of the terrain at the breakwater site itself, we have the balance reaching a mile away west at a hundred feet to the inch, all of it two-foot contour intervals.

But you'll see a set of sheets there with the subscript "S" meaning soundings. Those show the plotted points that the surveyor wrote on his hard copy from his sounding charts. He ran his sounding chart at the scale of the map, and he had an on-board plotter that plotted his position constantly as they went. It plotted it to scale of the map. If his runs were too close or not close enough together to give him confidence that he could draw contours to the limitations that he had asked that it be some contract for, then he came back to run it again immediately. There was very close control on this.

So you'll see the map of the new numerical values of soundings, and that will indicate to you the spacing between runs. It doesn't indicate the detail that was available to him along the run. He could have taken them off more frequently if he'd needed to.

Then the contours that were interpolated from the third set of drawings are the positions--and we call it a map inventory--of every tribar that remained in the position on the breakwater after the storm that created 150 feet of damage. Those maps of the tribars were drawn by two instruments set up at the end of the baseline, shooting a rod, one diver going down to the end of the tribar cylinder. Each leg of each tribar was surveyed. He held the rod. When he was ready, by telephone, he told one of the instrumentmen that he had it set on center and a companion, swimming in the water above, was given the signal. With a bull's eye level, he swam a two-by-two redwood pole, with a band mark on it, to plumb position. This was reversed from the usual survey condition where the rodmen sit and wait until the instrumentman says he's ready to take a shot.

In this case the control was the man in the water, and the instrumentman had to learn to be nimble. So the two instruments caught the high point of the rod as he swam to vertical. Because the diver can't stay there forever, he got the vertical angle, horizontal angle, plugged it in immediately, from instrument, to a computer and immediately on the spot, we had X, Y, and Z of the end of each leg of each tribar. The map shows those positions.

Another one then takes that map, with the Z value of each tribar, and calculates the representative surface of the filter stone (Phonetic) on which the tribars rest. You'll see a map of that one which will show you the inferred surface of the breakwater less the armor.

If there's another set there right now, I don't think of it. They're all reduced to one-half the size of the tracings.

I will spread them out here. You'll see that they map terrain with remarkable clarity and with remarkable impact to anybody who's preparing to build a three-dimensional model. We took these then and blew them up to one forty-fifth of full size, in sheets about one meter by a hundred and twenty-five centimeters, and we pasted them onto the floor of our model and, once they were pasted down, along each contour, at about eight-inch intervals, a small angle iron clip was fastened, it's standing leg tangent to the contour, to the concrete floor. Then it was bent or bowed

around these standing legs, and they put in galvanized iron ribbons and checked the upper edge for exact comparison with the scaled elevation represented by that contour. Once that was verified by a surveyor, they were spot welded. Then it was filled in with sand up to two inches and molded at the top.

So we built, at one forty-fifth of full size, a model of the map you see. From that we proceeded with some very interesting model studies. It's an example of mapping that Orville wanted people to see. I'm pleased with it. It wasn't inexpensive, but I think it was not costly.

DR. WHALIN:

Would you mind telling us what it did cost?

MR LILLEVANG:

The surveying of the tribars, of course, were the very tedious, time-consuming and money-consuming operation, and I can't separate between the inventory of tribars and the bathymetry. Our total bill was on the order of $450,000.

DR. WOOD:

Doesn't seem that bad to me.

UNIDENTIFIED SPEAKER:

What horizontal and vertical accuracy would you specify for the offshore survey at Crescent City?

MR. LILLEVANG:

I don't think that's an answer off the top of my head. Probably something like this.

MR. DAVIDSON:

Well, obviously you thought it was necessary to do that or you wouldn't have done it, but would you do that on every job from now on?

MR. LILLEVANG:

If there were terrain like...

DR. WOOD:

I would like to focus attention in the latter portion of the workshop on comments directed specifically at the prototype study. There have been a number of very useful suggestions. We're just having the discussion on the placement and ways one may determine that. We would like to pin down some specific objectives you feel we could achieve in this prototype study, perhaps suggesting with those the hypothesis that would justify it, a particular way that we might consider the measurement or the way we might consider the placement.

At this stage, we're wide open to suggestion. One of the things that clearly has come up is the question as to the monitoring program for the forcing parameters. How

many gages do we want? We've talked about attempting to measure velocities within the structure. That's been a topic that has come up. Is there any use to having current meters in or around any of the structural units?

I welcome anything that might come to your mind that you think would be useful input. I might lead that off by a question that I had penciled down the other day in noting three of the presentations here and also Doctor Burcharth's visit. Doctor Burcharth, Dr. Ligteringen, and Dr. da Silva all made the comment in their presentations about having "a major question related to the stress or problem of curing effects related either to concrete mix, the curing condition, or the size." We have the initial presentation of our results in the first stage of the prototype study on dolosse.

Do the results that Dean Norman presented with respect to a lack of a great deal of concern for weakening of the structures in the curing process concern you? For the dolosse, specifically. Is that question set to rest, or is it still a major question with dolosse? I realize some of you addressed other geometries. That may be another question.

MR. MAGOON:

In the prototype dolosse that are existing at Humboldt, there appeared a crack in the ones that were cast on sunny days, I believe, and the crack extends down to the steel. I call it a Poisson crack for lack of anything else. Once the crack started to form, it propagated down until it hit the steel. You can see those when you go out there because the water enters in there and it will actually come out through the steel. You'll see there are rust streaks down the side of the unit.

That crack appears at other locations in the world besides Humboldt. Obviously at times it seemed to occur on sunny days and not foggy days. I can't tell you it seems it was warmer on one than the other. I just wanted to make a comment because I believe Doctor Zwamborn is going to visit the prototype.

When you look at the photos you'll see where the blocks that held the reinforcing in place were displaced, adobe blocks, displaced the cage, shifted over to the side of the unit so there's color, and you'll see the rust streaks. You'll see the exposed steel on the side. You have to be careful when you interpret that. It seemed in the case where there's actual bars placed, in case of your experimental units, I was going to say you have to be very careful that they aren't displaced, however you decided to hold them in there. They certainly were in some of our units and obviously because the cage is out touching the side of the form.

DR. LIGTERINGEN:

Of course the curing affecting reinforced dolosse is different from unreinforced. In fact, in that respect, relaxation has come to be helpful of the unreinforced unit where the strains might be reduced. I don't think you can generally state that dolosse won't have any problems with temperature cracks. We haven't brought the computations up to a level that can analyze a dolos completely at this moment, but we hope to be at that stage quite soon.

We are almost sure that, for instance, in San Ciprian, the 50-ton dolosse have experienced temperature cracks during curing. That is almost the only explanation you can give for the fact that so many dolosse have been cracked even before a major storm hits the structure.

As far as tetrapod and cubes above certain sized and dimensions are concerned, you have to take microcracks in curing stress into account. We can't say in general all cubes above 60 ton have temperature cracks because it very much depends on the type of concrete mix used, curing conditions, etc. I think it's a problem which should be looked into, and we need to sharpen our tools in that respect.

MR. NORMAN:

I think I agree with you. I would just simply say that it appears to me that the dolosse that are being constructed at Eureka with the fiber-reinforced concrete that is being used, and the construction procedures that they are using, there seems to be no problem with curing-related cracks or stresses for those structures. No restraint there. There's no rebars to give internal restraint, and so there's no indication that that would be a problem for those structures.

MR. MAGOON:

I'd like to add, since San Ciprian was mentioned, I believe, through your experience, that's extraneous because I believe they used steam curing. When I was there, they were. It's a different type of curing.

Actually, at Pornask (phonetic) Nuclear Power Plant in Scotland, they were draped and cured indoors. So there is a great diversity of opinion on this point. Although for this particular project it might not be too important, I think what we're saying is, on the long range, job to job, maybe someone not using fibers is something important for you to look at.

MR. LILLEVANG:

I visited a project in Iceland while they were still casting. They had most of them in place, but they were still making them. Because of the short season and cold weather, they were casting under shelter. They had a floor with steam delivered down below that, and they were getting a very large number of broken dolosse. Some in the form, some in handling for transport, and some breaking in transport. If the dolosse survived the trip to the breakwater, they seemed to go in with very little loss.

I discussed it with the engineer-contractor who had the job, and suggested to him that it looked to me like form restraint was resisting this change in dimension and inducing planes of weakness, if not cracks.

He examined that later and wrote to me and said that he thought that my inference was absolutely correct. As he analyzed and looked at it and played with it a little bit, it appeared that clearing up his form restraints, by early stripping, and then controlling his steam process better, he reduced his losses dramatically.

There is a restraining in the form. The dimensional change factor may be small, but if you get a very stiff form and these sharp angles to restrain, why, there's no place for it to go except to weaken itself.

MR. MAGOON:

You can also put an O-ring. Professor Berwick (phonetic) suggested, in that situation, that you slice the forms and put in a compressible gasket type. I didn't know is in this case you have enough reinforcing. I believe putting a compressible gasket was done in Sines. It was not done at San Ciprian.

DR. LIGTERINGEN:

Yes.

DR. WOOD:

All right, any specific topic suggested for the study? One we should initiate?

DR. SOBEY:

I would like to come back to some of the comments made earlier about placement of the fourteen bullets to follow up Professor Dean's recommendation. Six to be kept and one out in the beginning.

If you follow the argument back to what the important loading is in the structural design, I think it's the extreme load which seems to impact loads. If you follow that back through what we can calculate it, I think we can calculate the impact load from the dynamics of projectiles. The forcing for that comes back to the hydrodynamic loading and the particular elements that get it moving in the first place. Then it starts to interact with the dynamics.

I think that the important thing that will come out of the field experiment will be some idea of the dynamic loading. In that sense, I refer briefly back to the experiment. We do have material problems in regard to the chances of getting reasonable hydrodynamic loading on the unit in the lab. It's a little remote from us, but perhaps it can be improved. We have to get some very good information from the field from hydrodynamic loading, which I perceive as an important part of the problem.

In that sense I think it's important to put these elements in typical location, not in extreme location. I would start with the rocking problem and impact load where they're essentially measuring the hydrodynamics.

DR. WOOD:

On the placement questions, when we look at our thirteen bullets, if we go along with that suggestions and we consider that we may be looking at an under layer and over layer, that really is not a large number of units to place anywhere. Is there any consensus as to whether one looks at the interface, the proposed still-water level?

DR. LIGTERINGEN:

In the test we did at the Delta Flume for Sines, we had those units with a model concrete, in brackets, placed in an area. You saw them on the slides. Those were the white blocks, and they were placed in an area from minus the wave height, below the still-water level, to about plus half the wave height above it, if I recall correctly. That was based on the measurements we did in the small-scale model where

we found that most of the action was taking place in that region anyway. So that would limit, to some extent, the area in which you are looking.

I tend to disagree with Bob when he says keep six apart because it would be very difficult afterwards to place those six anywhere in the breakwater. I think I'm very much in favor of the idea of the experiment with one. After that, I think you might as well put all the bullets in a layer, some in the under layer and some, the majority, in the upper layer.

Although of course the hydrodynamic loads are very important, referring to the remark by Doctor Sobey, I think we look to the whole range of loading, and that includes dead load, hydrodynamic loading, and loading due to colliding blocks. I think we should try to cover that whole range.

DR. ZWAMBORN:

I support that. I think it ties in with what I said before. The plus half and the minus half also agree with, and unless one feels you would like to have one or two units further down, I personally haven't got a big problem with that on a slope like what you're going to have here. One and two or two and a half.

DR. WOOD:

One and three.

DR. ZWAMBORN:

One in three. I think the concept, if you've got a slope, the bottom of the units are supporting all the rest. I think that is a completely wrong example. Then you have built a wrong breakwater. The forces should be transferred through your underlayer, as soon as possible, to the body of the breakwater. He even suggests you put sort of an underlayer of tentacles out so that you transfer that force quicker. I think some people here have expressed some doubts about the carrying capacities of the low units, and maybe if you want to go outside the half, plus half and minus half, I would feel that you would perhaps put one or two units lower down. I personally wouldn't feel strong about that. I don't know.

MR. MAGOON:

I'd like to add that if you look at the literature at a well known failure such as Sines, and you compare the various hypotheses for that failure, there were a large number of scenarios that were suggested. You might think if a failure occurs and you're forced to try to decide the reason for that failure, then would you have the information to decide whether it was a pretty flat slope. We assume it's not going to slump down in one big clump, but I don't know that right now. I would think the first thing that I'd try to answer, when you pose the question, is what method of failure it will be. If you can determine that in advance, then that would be quite convenient. I don't think you can do that. What you have to have is the placement of units so that you can describe the method of failure.

I would think that in the event that they ever did slump down at once, let's say you had placed them in the area where you have the problem in the breakwater right now and they'd all gone out at once, that would be the end of your experiment.

Although you could have enough there to make a measurement, if you had something somewhere else you'll always get some data.

My thought would be to have it in a place so that even if the experiment lasts for three months you could get some data out of it. If it lasts ten years, you'd get tremendous amounts of data. If it lasts for one year, you would get something.

So I would say the majority in maybe a clump, but a goodly sprinkling of units in other locations and one in a place where it would surely get high loadings on it and maybe roll around and however it's displaced or whatever that comes up.

MR. LILLEVANG:

I'm persuaded that there's great value in the observer, of whatever level of sophistication, standing over one of these structures during a major event, in a helicopter or slow flying airplane if the wind and weather will permit, and watching what's going on. The highest level of sophistication in design concepts and so forth that is available, whoever that may be, and how soon he can get there. That may mean that he's the Corps' man in the District, who may not know an awful lot about what the designer's really getting at, but he knows a lot about how these pieces behave during construction and over the years, while he's watching them, or if it may be somebody can hop into a chartered plane and get up from San Francisco to take a look.

I value, beyond measure, the opportunity I had at Diablo, before the storm had completely died away, of being in a helicopter over it and watching what was going on. It was immensely valuable in those kinds of observations to help with interpretation of what you see at that particular place, to observe how the water is moving and how pieces are responding, and, if they're moving by what mechanism they appear to be snatched away. All of these things are immensely valuable and shouldn't be overlooked as something that is an authorized procedure on quick notice.

DR. WHALIN:

Good Idea.

DR. McDOUGAL:

In regard to scattering out, there are not that many measurements, in the first place, and I think that Baird's group made the comment from their model tests that the results were very specific to individual units where they nested, even if they were reasonably close to each other. So with that anticipated variability, even the ones that are close to each other, if they start scattering them out, it will be difficult to put any type of statistical reliability on the numbers you get. At least, if you keep them close together, you would increase the faith that you can put in the numbers.

DR. ZWAMBORN:

I would just like to come back to support what Hans Ligteringen has said. I didn't mean we use one first and then thirteen and then leave the rest. It's very, very important that you place the majority of the instruments, let's say nineteen or whatever, during normal construction as part of normal construction. Put them in the same way as you would be doing the normal units. I think that is extremely important. Otherwise we get some fancy results, but we don't know what they mean.

DR. WHALIN:

Right.

DR. WOOD:

Is it correct, realizing there's been contrary views expressed, that the consensus seems to be that a nesting of these is the best way to optimize these nineteen bullets? As you described, plus, minus.

MR. MAGOON:

I would say that you could have a greater number in one place, but I want to be sure that whatever the failure scenarios you expect are included in that, so I wouldn't put them all in one place.

DR. WOOD:

What is one place?

MR. MAGOON:

One little section where the units are all touching each other. One method would be to put all units in contact, and you have a zone where you're going to put the whole entire experiment. If that zone fails, you have lost the whole experiment in one swoop. Fair enough. You might get good data while it's doing that. So I would tend to suggest that you could put more than half in one location, but I would still feel that you need some in some other places just in the event something catastrophic happens or just the fact of the site you selected not being representative.

DR. WOOD:

Perhaps to clarify that better, from Dr. Zwamborn's and Dr. Ligteringen's comments, we have this kind of vertical scaling of plus or minus a half a wave height. What then would be your recommendation on that horizontal limit? Is there a comparable ground you'd choose there?

DR. ZWAMBORN:

Well, I wouldn't mind if they were all more or less in one row, but you don't get placed like that. Mathematically, you get sort of left and right, but whether you have to say two and two, sort of two rows, I can't say just like that.

UNIDENTIFIED SPEAKER:

If you have two rows, fifteen to a fluke, you're talking about a band width of 30 feet, to answer your question, Bill, but because they don't always go in 30 feet, there's going to be probably over a 40-foot range, just guessing.

DR. ZWAMBORN:

You're just placing them in that area.

UNIDENTIFIED SPEAKER:

Maybe over a 50-foot section.

MR. LILLEVANG:

I was thinking of the area occupied by these if they're clustered. Although I like the cluster concept, suddenly I begin to see things that are more complicated than the strings that hold a marionette with these chains. I'm wondering what kind of interaction there might be with a massive bunch of chains like that around these dolosse that are in a cluster. We've got these umbilicals here to be concerned about. I think they're going to move around. We're talking about them maybe kicking and tangling but interacting with one another, and one chain on another tribar that is also instrumented. This is going to be very interesting.

DR. WHALIN:

Let me throw one more comment out that the group may question. We have said that we are going to instrument twenty of these things. Okay, that's true. Twenty of them are going to be instrumented: six of them have accelerometers in them also. I don't know if you remember that or not. All twenty have strain gages and six of the twenty also have accelerometers in them. The original proposal was for forty, okay, of which about a dozen or so, I've forgotten the number, would have accelerometers.

In our negotiations with those people that provide the money and so on, the limit in which we, CERC, refused to back down, said don't do the experiment if you have fewer than twenty. That was our bottom-line figure.

So now the thing is going now with the twenty. However, you may wish to comment on whether that is adequate. We're receptive to any comments that you might want to make regarding that number.

Obviously, the more we all know, the better.

DR. HEIJDRA:

That would be a pity if fewer than all of the instrumented units were in one location, say, all the section triangle units on one spot, because then you lose quite a lot of information, when you remember the information shown by Mr. da Silva and you saw a figure, say, the forces for acceleration on the cross section. I think it's more important to get some information of the total cross section than only to have more detailed information about one spot.

I think when you are interested in some measurement in one spot, I think it's valid then to combine the measurements with some kinds of tracing the movements. That might be by photographs or might be some movies. Because then you can get information not only of the actual impact forces, but you could also get the information of the load schemes. When you are interested in one specific spot, I think then you have to combine that because then you have not only information of the impact force itself but also the load scheme.

I think from the actual information of total cross section, it's more valuable. To get information of total cross section, not to spot them on one location but to spread them on the total cross section, is safer. That is more based on what we have seen in

our laboratory for one of the breakwaters. There is some agreement among the measurements of instrumented units in one location. When you have one location, I think you get only one information.

DR. WOOD:

Yes.

UNIDENTIFIED VOICE:

Maybe I'm introducing something that's already resolved, but what about other dimension layers? Now, are we talking about spread laterally? But, what about the outer layer versus under layer of dolosse?

DR. LIGTERINGEN:

As I have said, they're one-third underlayer and two-thirds in the outer layer, and I think that would be good.

Just to be a bit more specific, how many units we find in a row from the toe to the crest?

UNIDENTIFIED VOICE:

I don't have the number.

MR. HOWELL:

It figures out numerically. I think we were talking about four by five, four across and five up. I think it was five units.

DR. LIGTERINGEN:

Yes.

MR. HOWELL:

Five units vertically, we could pretty well cover that. Four units, that is assuming all four on top. Of course, if you put some underneath, then it may be two by five or whatever you want.

DR. LIGTERINGEN:

Yes.

MR. LILLEVANG:

Let me ask about layers. I'm always troubled by layers in breakwaters because I feel that you should never build them in layers. If you want to talk about two units thick, well, I accept that, but not two layers. What is the intention of construction procedures here? To put down one layer and then come back and put another on top, or to build the full section as you go?

MR. DAVIDSON:

I think it would be to build the full section as you go.

MR. LILLEVANG:

That's my feeling on it. So it's not layers, it's--

MR. DAVIDSON:

That's the District's prerogative. I mean they don't often get built as we so specify.

DR. ZWAMBORN:

I would strongly support that. I think it's wrong to do that another way. It should be done in one, bottom up. I don't know what your height would be of the breakwater. What the length--height of the slope?

DR. DAVIDSON:

It's 160 feet to the toe horizontally, so it's part of this is on a one-to-four, and then it compounds to a one-to-three. Two hundred feet, maybe, a maybe along the slope. I don't really know, but you're talking about a hundred and--

DR. ZWAMBORN:

You virtually have them running along in one row, I suppose, if you want to cover from plus half to minus half, but--

MR. DAVIDSON:

Mr. Markel, when we did the test with prebreakage of units in clusters and things like that, we segregated them by staggering them up the slope. We had them below water halfway down to the toe, at the water line, and above water line. Would that be a suggestion here? This gets back to Omar's problem or if you put them all in one section, you've got all the chains coming to you. I disagree with the group, but a spread above water in this lane, some at the water in this lane, and right now I don't know what that dimension is, and some at the toe in this lane, or back over here, so if any one fails, you don't lose the others either.

DR. LIGTERINGEN:

Sort of herringbone?

MR. DAVIDSON:

I throw that out because Dennis brought that up. That's sort of the way we did it then.

MR. HOWELL:

It's a question complicated a little further. We haven't mentioned the fact that, I guess, just for cost reasons we're proposing to instrument only one shank fluke

interface. I was wondering if some numerical modelers would comment to whether that would cause them unsolvable problems in trying to calculate to our results.

UNIDENTIFIED SPEAKER:

I know exactly what your concern is with, but it would seem to me if you have 20 dolosse instrumented with strain gages and you're assuming that in your random distribution of these dolosse that you're going to--, at least there would be no strain distribution or stress distribution that you would miss by your symmetric structure. So when you picked it off, I'm assuming you're not going to refer the location of every one of them so if one or two of them, you missed a strain distribution, so it would seem to me half instrumented, just half of 20 dolosse, would be fine.

MR. SCOTT:

I would think it would be better instrumented to mid-shank as opposed to one or the other. I also have a little reservation about placing strain gages on regions that could have high stress concentrations which could affect your results. I would prefer to see them mid-shank where distribution is a little cleaner.

MR. HOWELL:

I guess that brings--I don't understand how it's reasonable to have them mid-shank. Admittedly I'm not a structural guy, but seems to me what we're really trying to get at is the moments and torque. We know there's a problem that we don't see too many dolosse broken in the fluke.

MR. SCOTT:

What I'm thinking is in terms of calibration for your numerical model.

MR. HOWELL:

But your strain gages don't know whether they're on cylinders or whether they're on dolosse. You could have a bunch of dog bones in your model and strain gage at mid-shank and you would really have adverse scattering. You would have multiple torques and moments that would give you the same strains at mid-shank, so I don't think I could agree with that.

MR. NORMAN:

You're worried about losing the strain gages, damage in the high stress.

MR. SCOTT:

Not so much damage as getting good clean information from them.

MR. NORMAN:

I think Gary's right. I don't think there's just a unique way you can, with the data today, you could get at one location. You wouldn't know what really caused it. You have too many combinations of torques, thrusts, and moments that might have combined in the units.

MR. HOWELL:

We have certain advantages in the prototype. I have to remind myself that I have some advantages over the physical model. One of them is for the 42-ton dolosse, is big, so I think the main reason you're down on the trunk is because your model is so small you can't get up where you really need to be. We initially looked at doing the small dolosse and, after doing cost estimates, it turned out it probably would be more difficult to do smaller dolosse than larger units.

UNIDENTIFIED SPEAKER:

The gaging on the fluke shank interface is done with the three rebars that you're trying to model with (inaudible)--

MR. HOWELL:

What you do essentially is, large type Carlson strain gages. You're using the rebar to be the strain material, and it's like a calibrated load cell.

UNIDENTIFIED SPEAKER:

You're really talking about a resistance of bond in the bar.

UNIDENTIFIED SPEAKER:

That's true.

UNIDENTIFIED SPEAKER:

That's compared to the overall area of the structure that's probably negligible compared to the overall displacement of the structure. So, it's basically like a big strain gage.

MR. HOWELL:

Is it reasonable to assume for the prototype study that we cannot answer and solve every problem related to structural aspects of dolosse, not even concrete armor units, and we certainly cannot solve all the problems of hydraulic stability in breakwaters. Let me propose that we drive on to the prototype experiment. Should the prototype data which could be reproduced, we hope both in a physical model and in the numerical, structure the way the prototype experiment is designed such as to make that agreement as good as possible? In other words, if we can't pick a section where we understand what's going on and calibrate to the data in a physical model for a simple section, we certainly can't expect to do it for a complicated situation. So is that some kind of reasonable basic assumption that we should make for the prototype study or not?

I think we need to reduce it to some high enough level to have some consensus before we can--

DR. WOOD:

Comments?

DR. LIGTERINGEN:

I think it's a basic assumption that I would say it's the minimum and that I would agree to that. It's even important to have the opportunity afterwards to reproduce what happened in the prototype in the model set-up, possibly reconstructing the prototype as well as if you have documented it. As far as I'm concerned, if you put 20 dolosse in the prototype, if only four of them were going to give valuable results, that would be already a valuable result.

MR. HOWELL:

Does anyone disagree with that? There's a basic philosophy. I feel if we can agree on a basic philosophy of the experiment, then we will have some guidelines of evaluating the decisions.

MR. MAGOON:

I think I disagree a little bit. I feel you have to decide really what it is you're going to do with it when you get through with it. If you're just looking at representative measurements in the prototype, that's really one thing. If you're trying to verify what's going on in a hydraulic model, I feel that perhaps that's entirely different. I would think that would be one part of it that I would say I would defer to someone who is a hydraulic modelist for that.

As far as gathering information, I think any successful unit would be a help. I have some comments regarding that on a slightly different subject when we get to it. I would say that my own feeling is that you would really want to look at a range of wave conditions and so that at some point you need to get at or near the bottom of the section and some point at or near the top of the section. I don't really have a feeling whether they're in a line, or a row, or across, or what they are, but that the basic thing is to get a wide range of conditions on the units to be sure to gather some information, regardless of the condition of that wave and the section as to which it's exposed.

DR. SOBEY:

I think it would be a mistake to attempt to design experiments to verify the hydraulic model. The hydraulic model is, after all, an approximation, and I think it would be comparing one approximation with another. I think we have to look at the experiment as trying to find some measurements of what's really happening there.

DR. ZWAMBORN:

Having the possibility to go back after to the hydraulic. I think that's what Hans is saying.

DR. LIGTERINGEN:

Yes.

DR. ZWAMBORN:

Data first. You can do it the other way around as well, but that doesn't seem to be sensible in this case. Get the data and, once you've got the data, then you reproduce it in your model and see if you can calibrate the model in that way.

I think what Gary mentioned as well, I don't know whether we really discussed that, the placing of the units with this accelerometer.

I feel that we're talking about plus and minus half wave factors spread around. In the center half of that, I would suggest you put the ones with the accelerometers. Anybody have a comment on that? You have to measure the stresses in all of the units sufficiently to have a comparison. I think most of the motions that we know happen beyond the water line. Although you don't want them at the water line, I think you could concentrate them in the center half of your measuring area. That's my suggestion.

DR. WOOD:

Do you have any problems with--

DR. SOBEY:

Yes, I think the accelerometers' measurements are the most valuable of the lot and I would much rather see them spread than concentrated together.

DR. LIGTERINGEN:

Yes, I agree with Dr. Sobey, particularly on a minimum slope like this one. You see in the model, that the dolosse, from the lower part of the breakwater, will tend to be moved upwards in a heavy wave attack of the up-rush. Now that is something which you would like to monitor as well and so, therefore, I think you should spread the accelerometer units evenly over the whole range.

MR. HOWELL:

Is it a correct assumption that the accelerometer should always be on top?

DR. LIGTERINGEN:

Yes.

MR. HOWELL:

It doesn't make any sense to have it somewhere it doesn't fit.

DR. ZWAMBORN:

I agree with that.

DR. LIGTERINGEN:

I agree.

MR. HOWELL:

Everybody agrees on that?

DR. LIGTERINGEN:

Yes.

DR. SOBEY:

I think it's representative data you're looking for and I think the accelerometers have been as representative as the rest of them.

DR. LIGTERINGEN:

No, no.

DR. ZWAMBORN:

No.

MR. HOWELL:

It seems to me the thing of it is it's a difference in the philosophy of the experiment. I don't see how you can design specifics of the experiment until you have an agreement on the philosophy. We're all scientists and engineers, and we should be following the scientific method, if I can remember exactly what it is. I think we're supposed to have something in the hypothesis testing.

MR. MAGOON:

I'd like to throw out something I thought of. It's something I would really call a very simple strong-motion gage. Let's say you have some of these units that you're able to get to at low tide when it's fairly calm, and you simply put a writer between the two units and you would get continuity through the unit. When the unit first moved, it will break a piece of copper wire, and you would at least know you had a very simple motion. Perhaps tie the unit to the bottom or whatever you wanted, and

you'd have a very simple strong motion gage even if it wasn't hooked electrically. If the two of them moved, you could almost just tie any small cable that would break. You'd be able to tell that.

If the experiments fail, it's really nice to have some sort of strong-motion type gage. Maybe there's some very simple things in addition to all this sophisticated material you're talking about that you might consider.

MR. HOWELL:

(Nodding head.) We'll consider that. I guess I don't know how you'd tell the difference between motion and other things breaking. Just having gone through all the work we've gone through in making the cables survey, I can tell you there's a lot of things out there that will break wires besides motion.

DR. LIGTERINGEN:

Yes, floating debris.

MR. MAGOON:

Well, you certainly see some things like scratch marks on a unit, which you see on a lot of units. You obviously started off with just a plain dab of paint between the two units. You can take a spray can--I'm thinking of a simple test that you do in conjunction with the sophisticated tests so that when you go back and look at it and all else fails, you say this thing moved three feet or something like that.

MR. HOWELL:

I think it's very important we get motion, but I think we have a number of ways of doing that. There's Mannasquan and then the techniques we've talked about here today, so I guess I'm more interested in getting my money's worth from this group of experiments on the question of what are we going to do as a group and not so much the nitty-gritty.

I think, you can't do an experiment unless you have well-defined goals of what you want to accomplish. How will we define success with this experiment? What are we trying to do?

MR. MAGOON:

Well, ultimately you'll know whether you got your money's worth for your client, which is the public.

MR. HOWELL:

Is the experiment even necessary? Maybe we don't need the experiment.

MR. LILLEVANG:

That's another can of worms. Do you really want to take a shot at that?

DR. WOOD:

You probably have a better concept than any of the rest of us here, WES, as to what other experimental questions you have not gotten feedback. Do you have any questions you would wish to pose at this time to our guests?

MR. HOWELL:

I don't think I got an answer to the question of the instrument on one end, one shank length. That's very important.

DR. WOOD:

The question again is posed as to whether or not just simply having the strain gage measurements in one end of the unit was sufficient for the objective of the experiment. We haven't arrived at it yet.

MR. COLE:

I have a suggestion. Maybe on the layout plan, since this is an open-ended question I think we can talk about for months, why not, in a day or two after everybody is on the way home and thinking, just map out a plan as you would envision it, where you'd place what, five of them, six of them, with accelerometers and send it back to Gary. He would have maybe a concensus of opinion out of twenty-five or so plans that--

DR. WALTON:

Could I make an alternate suggestion? I think what you say is true. That we really cannot design the experiment; we can provide input, and I think Gary now has what benefit there is of our thoughts. I would like to modify your suggestion, if it's agreeable to you, that he take what he's heard from us and work up a plan and send it out and we can provide comments on it because otherwise it's going to be kind of a shotgun report.

DR. WHALIN:

That's a good suggestion.

DR. SOBEY:

Just a comment on that. If the options are having the instrumented placed up or instrumented down, that's saying you're putting this special placement of these units or can you do it without saying that it's special placement. Your standard construction procedure.

MR. HOWELL:

I think we can put it either way if we wanted to. Would you agree?

MR. DAVIDSON:

(Nods head.)

MR. MAGOON:

For most cranes and most ways you pick it up, it turns out that a contractor often has a certain kind of a device that grabs onto it, a certain grapple that he's made, and you can place it in any orientation. He likes to--he swings this out and releases it a certain way as a conventional thing he does. If you specify, probably he could rig something to place it almost any way you wanted. Conventionally, he grabs onto it off a truck that always comes the same way, he swings it out. There's various lines to shore. He normally doesn't have a device to rotate the unit. There's a tag line or device back to the trunk line or whatever you call it, but there's several lines, and it could be arranged on some kind of a device that could rotate the unit in any position.

MR. NORMAN:

You wouldn't want to do that, though, would you? You would then have a preferred placement of measurements.

MR. MAGOON:

I think you'd put it like he normally would build a section.

MR. NORMAN:

Like he does it, right.

MR. MAGOON:

It is prejudged because that crane can only, if he grabbed it in the middle, place in certain ways, normally, unless you do something special.

MR. NORMAN:

Just make sure you don't have all the instrumentation packages in one direction. Now, you don't do anything to make him place it in any particular geometry, but you do make sure he doesn't place it so all of your instrumented connections or shank fluke interfaces are all up.

MR. MAGOON:

I think you recognize the way he's going to place it and you put the instrumentation to take into account whatever method this gentleman has to put his units out.

DR. WHALIN:

You're saying put the package on the same end every time, every way it comes out on the truck, and they'll probably end up in some preferred position.

DR. WHALIN:

You'd have to have one one way, one turned the other way.

MR. MAGOON:

You think of that before it goes to the site if possible.

DR. WHALIN:

It ought to end up that way. That's a good point.

MR. DAVIDSON:

Don't get us wrong, now. Orville's right. In construction, as you well know, it's force of habit that he picks it up and puts it (indicating), but we have specified that they're randomly placed except along the toe. We like to work from the leg out and build that toe or scotch it. Above that, we specified to the District that it be randomly placed. Whether they'll do that or not, I don't know. That's the whole matrix now. I'm not talking about just the instrumenting. Very purposely don't want, as Omar's pictures show, all of them down slope necessarily. By force of habit, he'll do that to you if you don't mind.

UNIDENTIFIED SPEAKER:

As I understand the casting process, they're cast like this, and the instrumented section will always be this end. When the contractor goes to grab them, of course, six days after the casting, the form's taken off the first day, and six days later the soffit, and it's lowered to the ground. That's how it is in the yard.

When they go to lay it, they lift it up and place it in a random fashion. They don't like to turn it. It's very difficult, but they do place them in a random fashion. Murphy's Law: The best laid plans that we may have, the very last day of putting these out there, maybe a storm coming the next day, we've got to get the crane off and somebody says "Drop it right there." That's possible, too.

MR. MAGOON:

Though when you place it, it of course moves as it's actually nesting. I'm just saying normal procedure is it goes on the truck certain ways. You just have to recognize that in however you put that in, whatever the contractor's doing out there.

DR. WOOD:

Do you have a comment?

DR. LIGTERINGEN:

The question which Gary asked, if we had to choose between 20 units with only instruments at one interface or ten units with instruments on both interface, I would choose the first.

DR. WOOD:

Dr. Zwamborn?

DR. ZWAMBORN:

I was going to come back to the same point. Aren't you addressing yourself to the finite element people. If you measure on one, can you then infer what the stresses are on the load? Is that not possible?

MR. HOWELL:

That's what I'm asking.

DR. ZWAMBORN:

That's what I thought you were asking.

DR. McDOUGAL:

The analogy is that with groundwater, the inverse possibly, you put in a couple of wells so you don't have measurements, couple of specified locations, and you solve the groundwater backwards to solving it--that's called the inverse problem. Unless you do some tricky mathematics, it's not unique.

Well, this problem's no different. You have a couple of specific measurements inside, and, unless you have the rest of the problem perfectly defined or do some tricky mathematics, it's not a unique problem.

DR. ZWAMBORN:

Then I do some tricky mathematics.

DR. McDOUGAL:

I'm working on those tricky mathematics, and to incorporate that with an Adena type model is a major, major undertaking.

DR. ZWAMBORN:

But, I would agree with what Hans has said. Even if you couldn't--

DR. McDOUGAL:

Sure, I agree with you also. It turns out the more measurements you have, the greater probabilities, the more unique it becomes.

MR. NORMAN:

It determines on how precise you want to be in determining all of these. If you are willing to accept trends in data or bounds in the loads and stresses, then you can do that. I think that's what our group here is planning on doing.

DR. McDOUGAL:

I have one comment on the question on placement. Reading through some of the information the Corps put on monitoring, some of the concrete armor units on the prototype test, it seemed like a reasonable number of them were failed during placements. Three, four percent, something on that order. Do you plan to monitor during placement?

MR. HOWELL:

Yes.

DR. LIGTERINGEN:

In addition to this, do you plan to have a full test of the units, in the first place, the units which are not instrumented, on site? I understood yesterday that you have, in any case, static tests of the units planned. Would it be sufficient to have correlation between a measurement on instrumented units, like you plan not to fail, and a measurement on non-instrumented dolosse which might go to failure?

MR. HOWELL:

I don't think we've ruled anything like that out, but we haven't committed to that. I think, as we mentioned once before, we're going to cast 30 instrumented dolosse, and if all those are successfully instrumented, then we will do things like that.

In any event, we plan to do the whole test on those instrumented and uninstrumented units. So we haven't ruled that out, but we haven't made the firm committment either.

DR. LIGTERINGEN:

I would recommend to make some kinds of correlation in that respect in order to eliminate, in any case, suggestion that the instrumented units were in any way weaker or stronger than the non-instrumented units and also to have indication from the known instrumented units, when it is brought to failure in a fall test, what the resistance in that respect is.

MR. MAGOON:

That is a very good point.

MR. HOWELL:

We will take a good look at that.

MR. MAGOON:

I would like to perhaps discuss a slightly different aspect of this. I feel there's a lot of methods of testing of these units not in the ocean. On bench tests or whatever you want to call them. I think that, as I understand, some of Burcharth's work is he tried to come up with a series of standard tests that could be in fact applied fairly simply at other locations. I would like some society of testing materials to would come up with standard tests.

I would like to suggest it would be important with your appropriately instrumented units to try to conduct some of the tests that either Burcharth or others have done and try with these very sophisticated units to do something with that. It might even be worth taking a dolos out of the water and in fact doing a test on it in the dry. Two parts of it, either not to failure, and I would suggest lift it one millimeter or something like that, that you know it won't break, or something you feel happy with, or drop a one-ounce feather on it. But, whatever you do, it would seem like there would be a number of tests that would be very valuable. At least, I think Burcharth has submitted a plea in other papers for some kind of testing of concrete armor units.

This might be a very good place to try to come up with a standard test. You've got a little time to do that, and whether you use this test or some others, I think a lot of good information could be obtained by testing.

DR. LIGTERINGEN:

Yes.

MR. NORMAN:

I have a generalized comment to make, and it may not really be appropriate. It seems to me that we've talked a lot about determining the loads on the dolosse, which

is a problem, but it seems to me that in talking about an effective structural design of the dolosse, you can at least break it down into the areas of load determination, structural analysis, developing a material failure criteria, and then a structural or global failure of criteria of the dolosse.

Finally, a structural design to evaluate the design in terms of effectiveness, safety factor and cost, something which we have to look at.

The main point I want to make is that none of these things is easy. Structural analysis, we've got a finite element method. That's a quick way of saying you can solve any problem that you can envision. There's linear and non-linear finite element methods. I think both should be used here.

There's geometric and material nonlinear. I think both should be used here. From material failure criteria, we don't have a good handle on this. There's tension and compression in general failure criteria. There are things that affect that very strongly like rate of an impact, for the impact problem, fatigue and other things that you are going to effect in the prototype study.

The structural failure criteria, from a global sense, you've got to ask, generally, whether you are looking for a brittle failure criteria or a ductile failure criteria. What I mean by that is, are you going to say my design is inadequate if tensile stresses at any point in a structure come back as brittle failure? Then I think I mentioned the structure's design concept.

I just wanted to mention all of these things. I guess you've got to prioritize them. You kind of think about where you put your money, what you know about something, but none of these points could get a consensus on any particular aspect of this problem. I just wanted to bring that out.

I know you are addressing that, but it's an important thing, I think.

MR. MAGOON:

I have one more point. I'm not clear on this, and maybe someone can explain. It's probably a very simple question. How will the instrumentation you have tell you what the loads are and where they're applied on the unit? We have the prototype units in the field, they wiggle around a bit, and how will we determine where these loads are in fact on the prototype unit?

UNIDENTIFIED VOICE:

One comment made earlier was that's one of the reasons I recommended doing some detailed photographs so you can at least get the support. That's a big thing. You can at least get our loads out. If you can narrow those down, you have one foot in the door. Then you're going to have to come up with some estimates of the hydrodynamics.

MR. MAGOON:

Well, let's assume that you saw the waves that impact on the structure and the units move around a bit before something happened, before they really start to break. Let's say they can take a little motion, which I think we assume they can. You'll have to have some way to determine where these loads are, I assume. I'm assuming that from

the measurements you have internally in the unit and the appropriate models you'll be able to determine where the loads were applied? Is that correct?

DR. McDOUGAL:

I don't think so. Again, that problem is not unique.

MR. NORMAN:

It's as if you put an accelerometer on the ground and you have an earthquake in a fault down several hundreds of thousands of feet, what was the acceleration down there? They do the same process with a deconvolution process.

MR. MAGOON:

Then they're going to have a very serious problem with the experiment.

UNIDENTIFIED SPEAKER:

It's your best shot.

MR. NORMAN:

The earthquake people have a problem, too.

UNIDENTIFIED SPEAKER:

You know, it's your best shot, and it's better than doing nothing.

MR. HOWELL:

It also raises the fundamental questions of a lot of people assuming we do a wave-by-wave analysis. It's almost impossible to look at results completely statistically. Takes total analysis.

MR. NORMAN:

What earthquake people do, instead of doing complete acceleration-type history in designing the structures, they get some idea of what the forces are, and they do some type of spectral analysis. They could come up with a design response spectral. You could come up with a design wave loading spectral, too.

DR. WHALIN:

One thing that Bill suggested is that we will probably try to map out, as best as possible and as often as possible, where the units are supported. The ones that are measured. We'll try to do our best to get a very good definition of that when we put them in above water, and we'll send Bob down to check out the ones below wataer.

MR. MAGOON:

Describing these units under water, you think of what Mr. Lillevang did with the tribars and all these efforts he went to. It's in fact possible at Humboldt that in 1890 they had a huge cane and catwalk and located the units. You're either going to

know where they are and you're going to go through some real verification, or that will always be open to doubt. I think that however you're going to do it, certainly some data are better than none. I would agree with that. Perhaps you can't determine where they are under water. I think you would have to think very carefully do you still want them under water, not knowing where the unit's are going to be.

UNIDENTIFIED VOICE:

I'd say one response to that is all the finite element and all that, that doesn't eliminate the possibility of doing empirical analysis. That in itself is a useful result.

DR. LIGTERINGEN:

From our experience in the modelway, we had the exact same problem in the units with accelerometers in several places and having to analyze the data which came from it. We found that once we knew what the position of the unit was, it became a fairly easy one, and I think we came up with fairly accurate results as far as that actual loading pattern is concerned. You have to make assumptions, either as far as the hydraulic load is concerned or as far as the actual moment of motion of that particular unit is concerned. So you are not 100 percent sure, but you come within very reasonable limits in the model.

MR. MAGOON:

Then perhaps could you extend that? In the model, how did you measure the location and the loading conditions in the particular units you looked at?

DR. LIGTERINGEN:

We knew how the unit was supported, where it was located. We had the correlation between the wave motion, what stages of the actual wave we were looking at, so we knew about where the force factor was working, and, from that, we could analyze the actual acceleration. I think it was fairly accurate.

The final recommendation I have, which is general, is that we start to differentiate between wave impact and collision impact because that gives a lot of confusion, even these two days.

MR. MAGOON:

Good point.

DR. WOOD:

We're drawing near the time where we had intended to end.

In summarizing, I think the suggestion is, Gary, for you to try a short document, to feed back to the participants and ask the participants if they would comment on that document and return it to us. It would be very helpful in the preparation of this experiment.

MR. MAGOON:

Could you add in that the other environmental measurements like wind, pressure, temperature, and those sorts of things. Maybe, if you didn't have time to present that, that could be included. The whole experiment could be included so we could review that on paper.

DR. WOOD:

I think we're not only saying we're presenting the experiment as it is designed here or we're presenting what seemed to be the consensus from the transcript, the papers that are submitted, and so on, and bring that together.

MR. MAGOON:

I was thinking something as simple as if you had a small earthquake. I assume somewhere around there's a seismographer. You'd know well enough, and you don't have to have a seismograph right at the site for example. If it occurred, you'd certainly want to take advantage of the events or whatever those little events were that could tell whether it's a sea or swell or whatever these things are. A few bits of information would be very helpful--visual observations, whatever you do.

I'd like to commend the Waterways Experiment Station on the effort of getting the people together. Realizing that perhaps all these opinions aren't all exactly what you wanted to hear, the openness with which you received it is extraordinary. I think it epitomizes what a scientific effort should be, and I certainly appreciate being here, and I think all of us do. I think that should really go into the formal transcript of the record.

DR. WHALIN:

Thank You.

APPENDICES

Appendix A

Delft Hydraulics Laboratory
Delft University of Technology - Department of Civil Engineering
Institute TNO for Building Materials and Building Structures

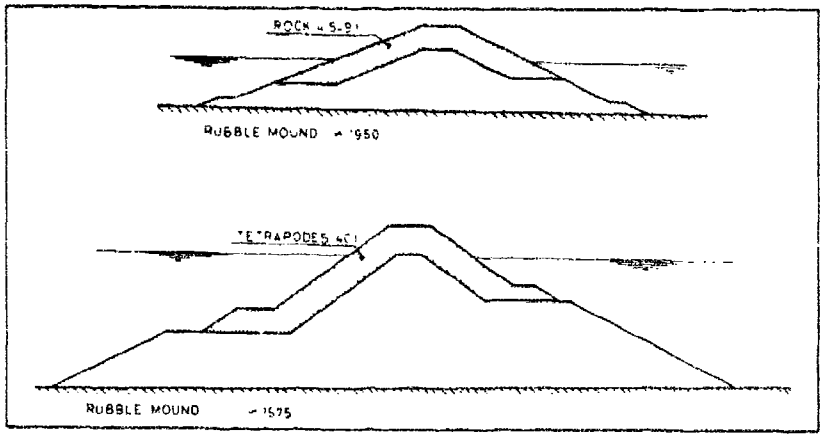
STRENGTH OF CONCRETE ARMOUR UNITS FOR BREAKWATERS

Assignment: Up to the present day, the most popular design for breakwaters is the rubble-mound type covered with large armour stones or concrete armour units. Since the last thirty years the design for this type of breakwaters has only been modified by the introduction of relatively slender unreinforced concrete units. The hydraulic stability of these slender armour units is increased by the interlocking forces due to the shape of the units necessitated by the increase in breakwater design conditions.

Recently doubts are developing about the effectiveness of the present design methods in view of the serious damage done to newly constructed breakwaters, such as the breakwaters of Sines, Portugal in 1978; Arzew, Algeria in 1980; Tripoli, Libya one month later; Gioia Tauro, Italy in 1979 during construction, and San Ciprian, Spain in 1980.

The study has been performed by above Dutch institutes in close conjunction with the following Dutch companies:
- Frederic R. Harris Holland B.V.
- F. C. de Weger International B.V.
- Royal Volker Stevin
- Hollandsche Beton Group N.V.

Period: The study was carried out in 1983.



Development in breakwater dimensions



Seriously damaged breakwater with concrete armour units

Mailing address: P.O. Box 152, 2600 MH DELFT, The Netherlands

Close analysis has indicated that substantial portions of the concrete armour units had been broken. In four of the five cases it could be established that the mechanical failure of the armour units had initiated the breakwaters final collapse.

Problems are becoming apparent relating to the increasing dimensions of slender units such as Tetrapods and Dolos that were not known or underestimated at the time. But also bulk elements like Cubes have similar problems as witnessed by loading tests with prototype Antifer Cubes for Sines. For large-scale breakwaters with large armour units, the mechanical stability of the armour units can be the limiting factor for overall stability prior to hydraulic stability.

At present better insight can be obtained through hydraulic model tests since the introduction of new model techniques. This however increases the need to obtain more generally applicable data from other fields, such as loading conditions and mechanical failure resistance of individual armour units in an armour layer.

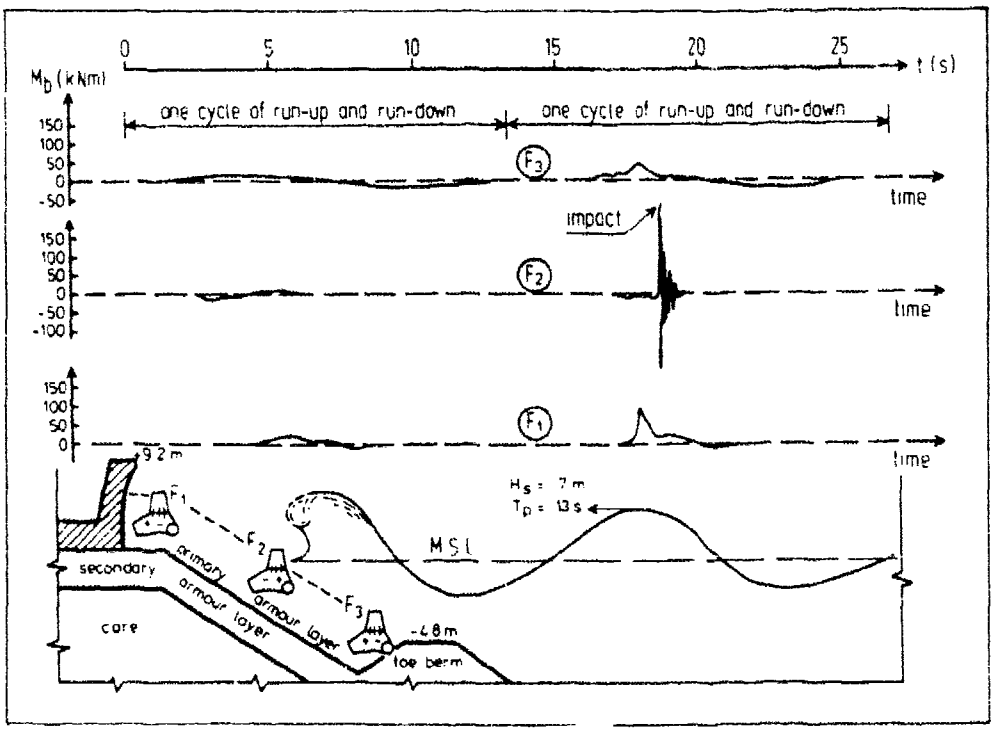


Mechanical failure



Strain-gauge technique small scale model

A study has just been completed, that brings together all existing knowledge with regard to observed breakwater failures, design problem aspects, hydraulic and mechanical loading conditions of individual armour units, mechanical strength and effective block strength in view of residual stresses and cracks, aspects of construction practice, experience with physical model investigation, measuring techniques and scaling laws for strength and motion.



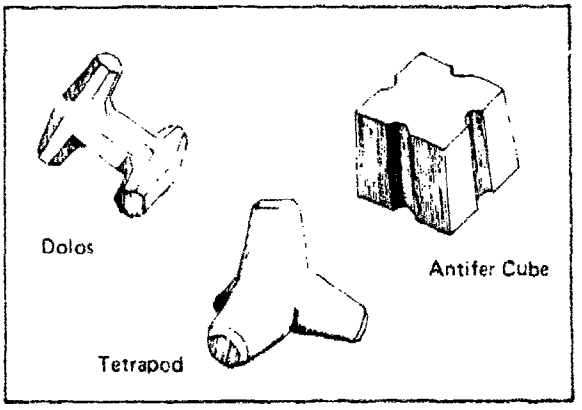
Hydraulic and mechanical loading conditions of individual armour units

The study was divided into three areas of concern:
1. Loadings
2. The strength of concrete armour units
3. Concrete technology.

With regard to the shapes of armour units the study has been limited to Dolos, Tetrapod and Cube, mainly because of the wide use of these shapes for very large units, with Dolos and Cube as extreme examples in relation to slenderness and Tetrapod in the middle.

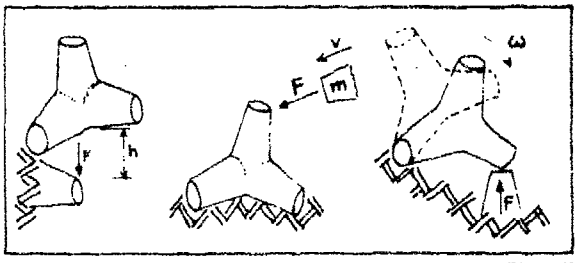
The study on loadings covers a review of the existing procedure for armour unit selection, observation and analysis of the recent breakwater failures, a review of prototype loading tests, the loading conditions during and after placing, that may affect the mechanical stability. A survey is given of present day measuring techniques for physical model investigation and damage observation in prototype. Results are presented with regard to the scaling of concrete strength.

The strength of the concrete armour units reflects the use of present day criteria in determining the mechanical resistance of these units to the various loading conditions over a lengthy period of time.
Critical values for pressure, drop height, collision velocity and rotation velocity are presented for elastic behaviour and according to concrete fracture mechanics theory. These parameters are revaluated with respect to block size, critical stress intensity, tensile strength and impact time. Scaling laws for physical model tests are derived and reference is made to known test results.



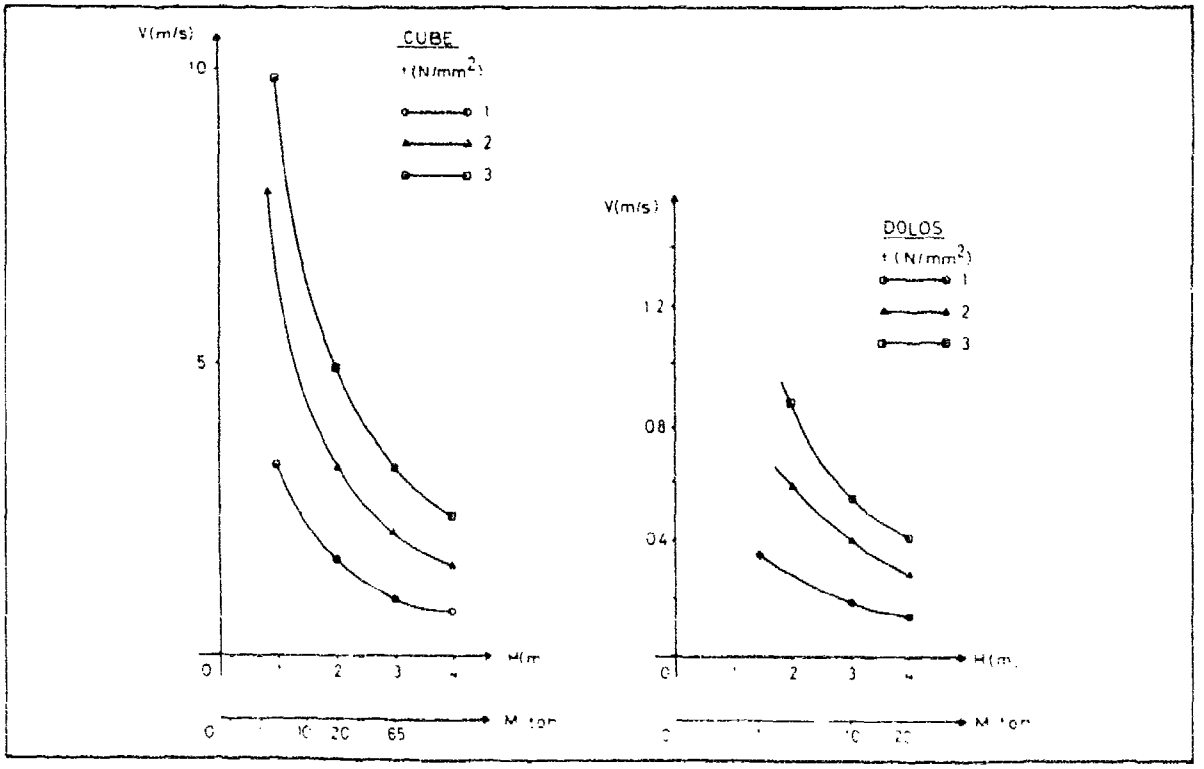


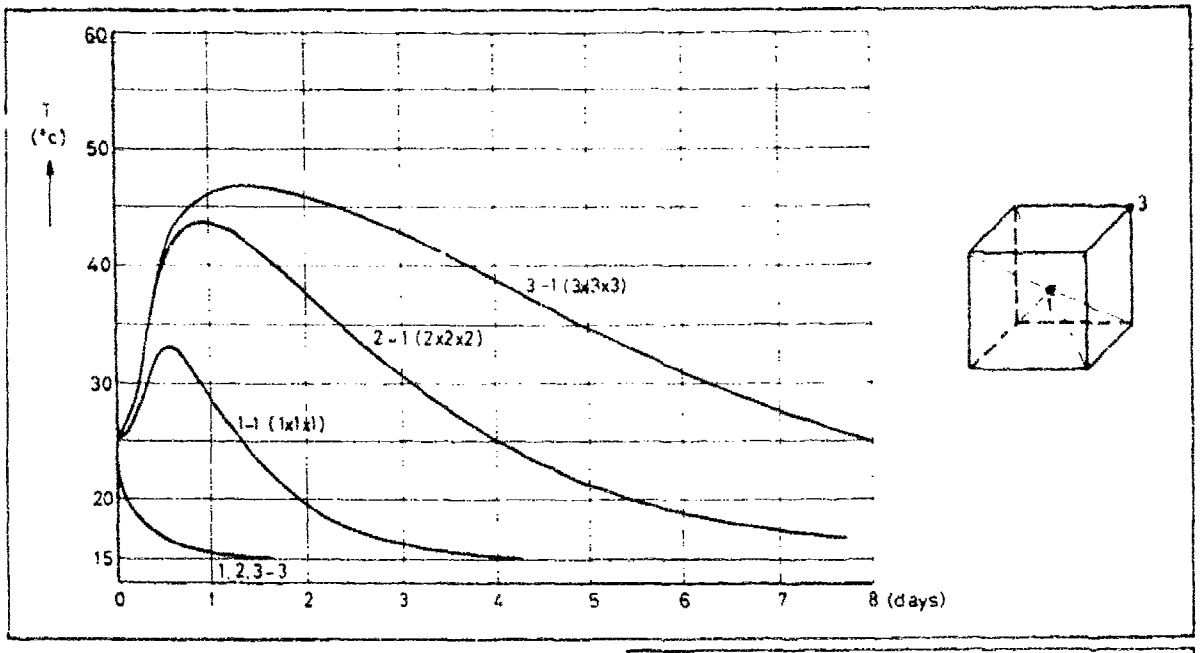
Construction of armour layer



Loading conditions during and after placing units

Calculated values for collision velocity





Hydratation heat development

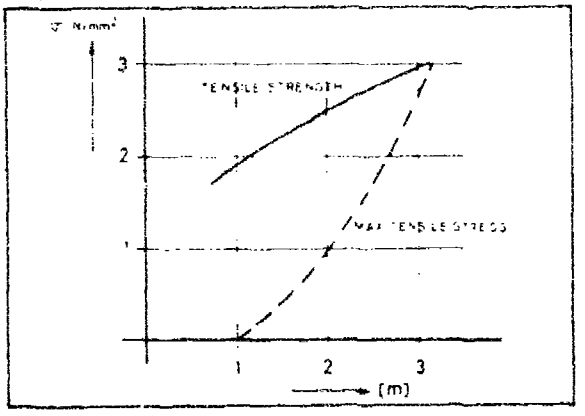
Various aspects of concrete technology focussed on the relationship between actual block strength and design concrete strength in view of block size and residual stresses from cracks due to hydratation heat development and drying out shrinkage. The study mainly refers to Cube units for which a suitable numerical computer code could be used. An indication is given for units of different shapes. Due account is given of the construction practice aspects.

The confrontation of the results from these different fields of expertise and their integration into the overall conclusions, proved to be the most valuable part of the study.

However, many of the findings must be seen as indicative, because insufficient data are available from tests and publications on the subject.

Some conclusions are as follows:
- Above 15 tons damage can occur to slender armour units from the effects of rocking behaviour.
- Slender armour units of more than 40-50 tons can already break under static loading such as caused by breakwater-mound settlement.
- The present-day design methodology is insufficient for the use of armour units above 10-15 tons. More fundamental research and full scale observation is required to establish proper design guidelines, not only for the armour configuration, but also for the effective resistance of armour units.
- Probability density functions would have to be developed to cover the complex of factors involved.

This study will be continued with fundamental research into how impacts cause elements to rock and which loading diagram acts on concrete units. Also design rules are to be drawn up governing the composition of concrete for both slender and bulky armour units.

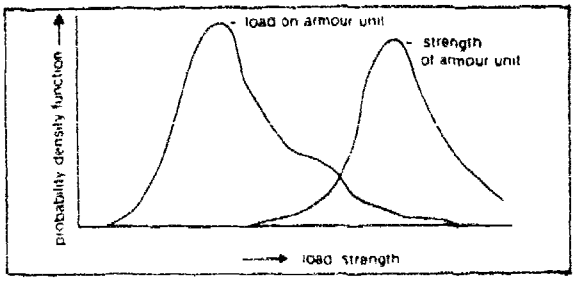


Actual block stength



Damage due to rocking

Probability density functions of load and strength



Appendix B

Pre-Print of a Paper Prepared for
THE FIFTEENTH INTERNATIONAL CONFERENCE
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Honolulu, July 976

EXPERIMENTAL STUDIES OF STRESSES WITHIN THE BREAKWATER ARMOR PIECE 'DOLOS'

Omar J. Lillevang,* F.ASCE and Wayne E. Nickola**

Introduction

Stresses induced within the breakwater armor piece "Dolos", when it is subjected to loads, are not reliably inferred from the two-dimensional techniques of analysis used in conventional design of structures. Other methods that take the solid geometry of the dolos into consideration are available, and comprehensive application of one of them, three-dimensional photoelastic stress analysis, is reported here.

Breakage of Dolosse

Breakage experienced at 15 projects throughout the world, where nearly 150,000 dolosse ranging from 3 to 42 short tons (2,000 pounds or 907 Kg.) in weight are in use on breakwaters, is digested for the reader by Table I. It has been impressively low. With the exception of one project, Humboldt Bay, none of those dolosse are reinforced.

It is well known that most of the breakage of pre-cast armor pieces takes place during the manufacture and storing and during the construction of the breakwater. Table I illustrates it. Several details within the table that stand out are commented upon in the following notes, each note being identified by the line number from the table:

Line 3. According to the owner's report, the relative high breakage during manufacture stems from 100°F air temperatures during casting, which contributed to development of shrinkage stresses and minute cracking.

A wave storm during construction rolled numerous dolosse that were not yet nested in the armor matrix and they suffered impact fractures.

Breakage in service is attributed by the owner's report to battering during severe storms by loose large quarrystones.

Line 4. Most of the in-service breakage occurred when a localized area of the foundation eroded, and an abrupt subsidence into the pit caused 10 dolosse to break.

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TABLE I

DIGEST OF BREAKAGE OF DOLOSSE AT 15 LOCATIONS WORLDWIDE

Line	Project	Year	Number	Short Tons	Per Cent Broken		
					Making	Building	In Service*
1	Cap Aux Meules	'70	7,418	4	1.0	1.0	"Limited Number"
2	" " "	'70	3,934	6			
3	Crescent City	'74	246	40	2.4	6.5	2.8
4	East London	'64	2,000	19.75	0.15	0.5	0.25
5	Gaansbaai	'70	2,786	5	←---4.0---→		0.5
6	"	'70	865	13.5			
7	"	'70	928	18.5			
8	High Island	'73	6,619	27.5	"Insig."	0.8	0 ?
9	Hirtshals	'72	2,600	9.6	0.3	0.3	0.04
10	Honolulu	'74	13,692	4	0	1.4	0.2
11	"	'74	4,317	6	0	1.5	
12	Humboldt Bay	'71	4,794	42	0.02	0.02	0.75
13	Mossel Bay	'69	3,423	3	1.0	1.2	1.0
14	" "	'69	2,634	6			2.0
15	Port Elizabeth	'67	14,266	3	4.0	"Neglig."	"Neglig."
16	" "	'67	13,790	3	0.15		
17	Richards Bay	'72	28,084	5.5	0.3	0.8	"Insig."
18	" "	'72		16.5			
19	" "	'72		22.1			
20	" "	'72		33.2			
21	Riviere au Renard	'72	7,921	5	3.3	0.6	"A Few"
22	" " "	'72	1,552	14	2.5	0.5	
23	Sandy Point	'69	500	7	←-------- 1.0 ----------→		
24	Table Bay	'69	5,900	3.4	0.6	0.14	0.01
25	" "	'69	11,700	6.7	0.3	0.07	
26	" "	'71	3,000	3.4	←-------- 1.0 ----------→		
27	" "	'71	1,500	6.7			
28	" "	'74	400	11.2	0	0	0
29	Thorlakshöfn	'75	2,600	9.9	0.4	1.2	1.0

* Includes breakage during consolidation of the structures, e.g. during first storms.

Lines 5, 6, 7. Extensive litigation over the project has made factual discussion of breakage unavailable, but subsequent to consolidation of the structure it appears breakage has been nominal.

Line 12. All but 26 of the dolosse contained steel bar reinforcement. The owner's representative reports the unreinforced versions were placed in low areas after completion of the armoring, and were thus "not integrated with other units". Of the 36 dolosse broken, 9 were not reinforced.

Lines 13, 14. Owner's representative reports manufacturing breakage was reduced by halting cold-weather pours and changing the cementing ingredients, from 50/50 Portland cement/"slagment" to all Portland cement, during cold-weather pours.

All the dolosse were made with "all-in", pit run sandstone aggregate, resulting in concrete that was sometimes over-sanded and sometimes under-sanded. Cement was 50% Portland and 50% "Slagment", blast furnace slag.

Line 14. In-service breakage can be separated into 1.7% initial consolidation fractures caused by insufficient fitting, and by some abrupt localized downslope adjustments, and 0.3% breakage since the consolidations during the first storms.

Line 15. Contractor tried to use a form made of concrete for the lower half of these dolosse. Those unyielding form surfaces proved to be warped and bulged, and tended to lock in the newly cast dolosse. Excessive force was needed to remove them. When the problem was diagnosed, new fabricated steel forms were substituted and breakage immediately was all but eliminated.

Lines 21, 22. Heavy breakage during manufacture was attributed to removal of forms when castings were only 4 hours old.

Line 29. Contractor surmises breakage relates to steam heating of the new castings while still in the forms.

The various projects listed in Table I include many variations in materials, in quality control during manufacture, in strength of concrete and in dimensional proportions. Thus it would be surprizing if there were systematic relationships between breakage intensity and size, and there is none to be seen in the table.

Popov (4) and Danel, et al (1) have shown that armor pieces of different sizes but identically similar shape, and made of the same material, will break on impact with an unyielding surface from the same height of fall. The theory and Danel's experiments with dropping 295 tetrapods, of from 20 ounces to 25 tons weight, suggest design

loads for impulsive forces on the dolos would be the same for any size piece. With other types of loads, however, steady ones or dynamic forces less severe than impacts, stresses sustained by a dolos under design conditions may be larger or smaller as the size and weight of the piece is larger or smaller. In that case there would be reason to modify the larger pieces with stress reducing measures that smaller dolosse would not require.

Three approaches toward stress management which could be combined, or be mutually exclusive, are discussed in this paper. They are:

1. Incorporating steel bars to take the tensile stress;
2. Reduction of force moments by thickening the shank, and consequently shortening the flukes;
3. Reduction of stress concentrations at critical parts of the dolos by minor geometric modifications, viz. at the intersections of the flukes with the shank, to keep stresses below the modulus of rupture of the concrete.

The photoelastic tests indicate that, to have any useful effect toward preventing fracture of a dolos at its intersections of flukes with the shank, steel bar reinforcement would have to be placed so close to the skin of the piece that loss of the steel by corrosion in a short time would be unavoidable; the cost of such steel thus would be wasted and its effect would be ephemeral.

The tests suggest some stress reduction does result from thickening of the shank, but not significantly when the extent of thickening is kept below sensible limits that are proposed by the dolos' conceptor, Eric M. Merrifield. It is his view, expressed in personal communications with the senior author, that thickness of shanks in excess of 36 per cent of the height of the dolos would cause undesirable losses of interlocking characteristics.

Modifying the intersections between flukes and shank showed significant reductions of stress concentrations in the test pieces. Making dolosse of all sizes in future with a small curved fillet at those corners is proposed, not only to reduce stress concentrations but also to minimize concrete imperfections that often occur at sharp corners during pouring of concrete.

Photoelastic Stress Analysis

Among methods of experimental stress analysis that have been developed and proved is the photoelastic study of loaded two-dimensional plastic models. Either reflection or refraction of polarized light from or through various plastic materials yields light interference patterns that are rationally related to stresses within the plastic. Knowledge of this phenomenon is not new. It has been available with use of two-dimensional models for a century, and has been in widening use since the 1930's. Beginning with discoveries made in 1936, three-dimensional model pieces of complicated geometry have been loaded in

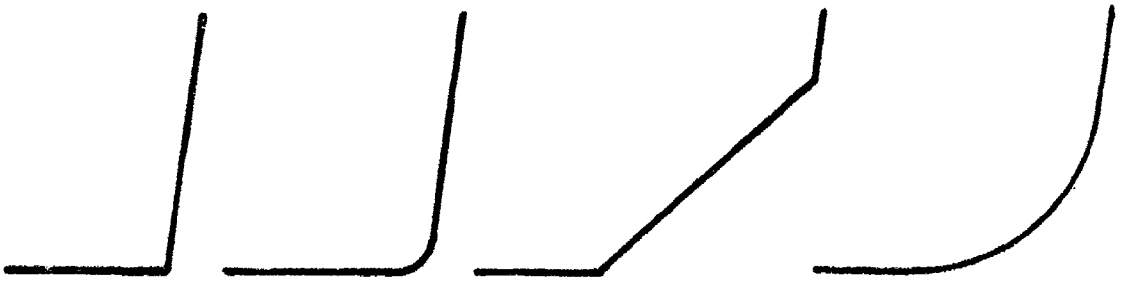
laboratory ovens, at temperatures elevated very near but not to the level where the material loses its elastic properties. After being held under load at carefully controlled specific elevated temperatures, the oven heat is systematically and very slowly withdrawn until the model piece has reached room temperature. During this cooling phase of the "stress-freeze" process, which may take several days, the loads on the model are sustained. When the model has reached room temperature the loads can be removed, but the stress patterns persist within the plastic. The model can then be cut into thin slices at any planes of interest, and polished. When polarized light is transmitted through those slices and examined with appropriate optical equipment each one shows the stress patterns that were induced along that plane by the test loads. Calibrations and rational computation procedures relate the patterns to definitive stress values. These techniques are commonly relied upon by industry. Complicated forgings, castings, fabrications, pressure vessels, struts, bearing housings and a host of other shapes and devices have been evaluated by this method. Except for some work that may have been done in England, it apparently has not been used before on a breakwater armor piece.

Description of Models

Sixteen three-dimensional models of dolosse for the present tests were cast from photoelastic thermal-setting plastic. All were made with the dimension h equal to 6 inches, which is 15.24 centimeters. As will be seen in several figures in this paper, the dimension h is present in two ways in the dolos. It is the overall height from tip to tip of two adjoining flukes, and is also the overall length of the piece, measured parallel to the axis of the shank. Half the models were made with shank thickness 32 per cent of h and the other half with 35 per cent. For each thickness ratio, four different versions were cast that varied the geometric details at the intersection of the fluke with the shank. The traditional dolos, with sharp intersections between fluke planes and the planes forming the shank, was tested at both thickness ratios. Those specimens were identified by codes 32HS and 35HS, the letter S identifying the sharp intersection characteristic for pieces with shanks 32 per cent as wide and 35 per cent as wide, respectively, as the height. Other versions of corner geometry, also tested for both 32 and 35 per cent shank thickness, had chamfers created by planes (32HC and 35HC), a small-radius circular fillet (32HF and 35HF), and a larger-radius circular fillet (32Hϕ and 35 Hϕ).

In the stress-freeze tests, stresses frozen into the model dolosse were produced with two different loading patterns. In the "Tension" series, equal forces that were all directionally parallel with the shank's axis were applied at the ends of the four flukes. Pairs of forces were oppositely directed, in a normal sense placing the shank in tension; therefrom the "Tension" term for describing those tests. In the "Torsion" series, equal forces were again applied at the ends of the four flukes, but acting in planes perpendicular to the axis of the shank and directed to twist the shank; therefrom the "Torsion" term for that series. Sixteen models were made and stressed. Fifty-eight sections were sliced from various parallel or intersecting planes, to find the stress characteristics within the dolosse resulting with

all the variations that were involved. Figure 1 shows and compares the four details of the corner that were studied, being cross-sections in



Sharp	Small Fillet R=.0078h	Chamfer .04h	Large Fillet R=.04h

Figure 1

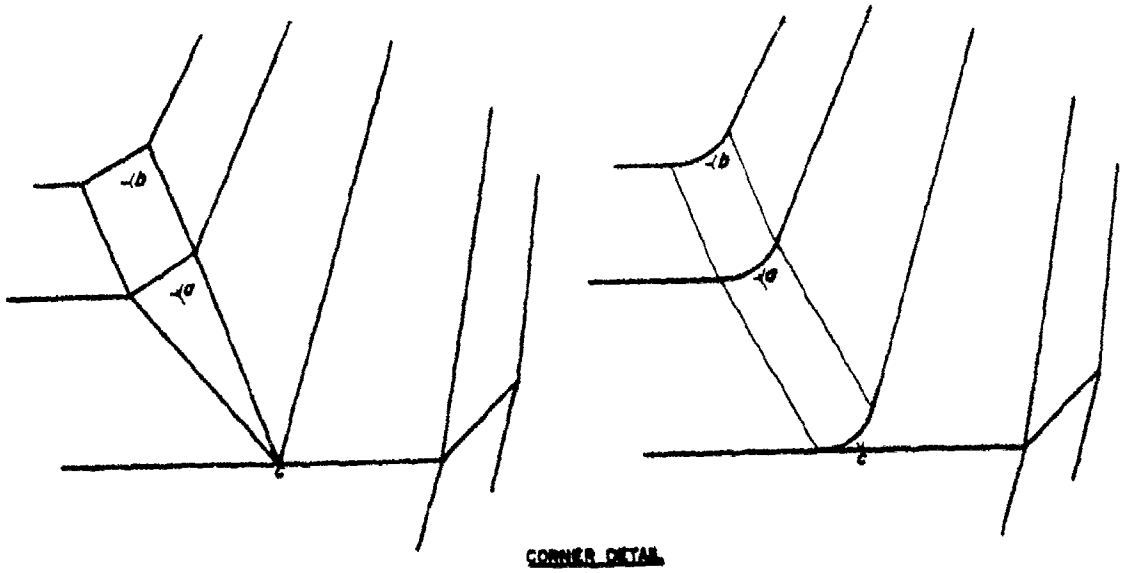
the plane common to both the fluke axis and the shank axis. As illustrated in Figure 2, the chamfer tapered to a point in the corners between planes either side of the ones cut for the profiles that are shown in Figure 1. The circular fillets in those same two flanking corners had to have radii 1.5 times as long as those in the central corner, because the angle of intersection of planes adjoining ac is larger than the angle of intersection along ab. The circular fillets did not taper to a conical point.

All the models were made with flukes whose cross-sections normal to their axes were not symmetrical octagons. The left side of Figure 3 shows the awkward geometry that develops at the intersections of flukes with the shank if a symmetrically octagonal fluke is made. To eliminate the intersection problem a slab of constant thickness, S, should be taken off the whole octagonal side of the fluke, above the shank at the intersection ab. If the maximum breadth of the octagon at the tip of the fluke is the commonly used .20h, and t is the thickness or breadth of the shank octagon, then the thickness of the slab to be removed, shown at the right side of Figure 3, is:

$$S = \frac{0.12132(.2t/h - t^2/h^2)}{(t/h)\text{Tan } 22.5° - 1}(h)$$

The models with chamfered and filleted corners, consistently for comparison values, also where made with flukes of asymmetrical cross-sections.

Rubber moulds for casting all the photoelastic models were formed by pouring a thermal-setting silicone rubber compound around a precisely machined acrylic resin master dolos that had sharp corners. To modify the master in order to make moulds for the chamfer and fillet versions, pattern maker's beeswax was hand-tooled into the corners of the master pattern. At the scale of these models, the corners of the hand-molded chamfer planes were not as sharp as one would expect intersections to be in the prototype, where structural steel plate is the likely material from which such forms would be fabricated. Because the

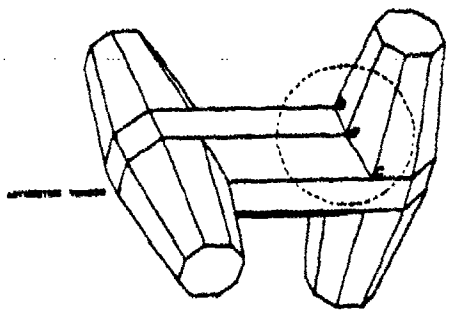


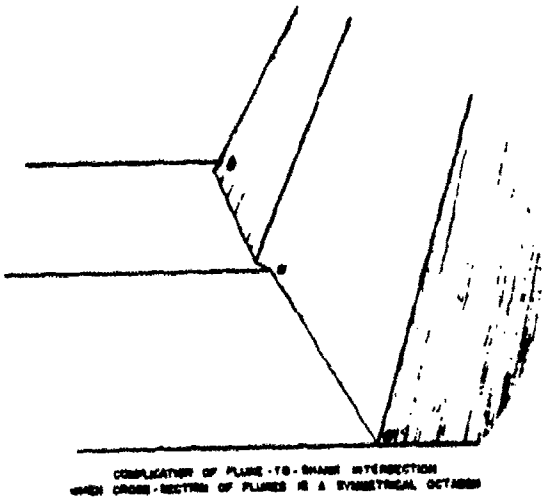
CORNER DETAIL

CHAMFERED

FILLETED

Figure 2





COMPLICATION OF PLANE-TO-SHANK INTERSECTION WHEN CROSS-SECTION OF FLUTES IS A SYMMETRICAL OCTAGON

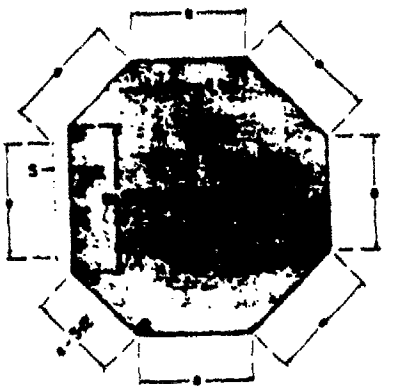


Figure 3

chamfer intersections probably were slightly rounded, the stress concentrations shown by the photoelastic models with the chamfered corners probably are less severe than they would be at prototype size, or than they would have been had the model chamfers been precisely machined instead of being hand formed of beeswax on the master pattern.

The theory of photoelastic stress analysis is well covered in reference works that are readily available in technical libraries, so there is no development of the theory in this paper. However, the laboratory techniques employed in the present tests and the analytical procedures that were followed deserve description.

A reconnaissance test program was carried out before the stress freeze models were made, to compare at least qualitatively the effects of six different arrangements of loading. For the reconnaissance, a specially compounded aluminum-filled epoxy resin was selected, and model dolosse cast from it were clad with a thoroughly bonded bi-refringent coating material. The bond was made with an epoxy-based reflective cement. A reflection polariscope was used to view the model while it was under each loading arrangement. Elastic deformations induced by the loads were transferred from the dolos to the cladding by shear forces developed at their interface. When polarized light was reflected from the surface of the model through the bonded layer, the coating exhibited patterns of birefringence which were quantitatively analyzed. Out of those preliminary tests it was concluded that the "Tension" and "Torsion" loading patterns previously described would best develop the internal stress information that was wanted from the stress-freeze photoelastic procedures.

Internal Stress Data, Quasi-Dimensionless Form

Stresses that were shown in the photoelastic models were reported in quasi-dimensionless form, to enable easy calculation of stresses in prototype dolosse of any size and for any selected value of the moment and shear producing forces. Stresses in the prototype were related to stresses in the model by:

$$\sigma_p/\sigma_m = (F_p/F_m)\ (h_m/h_p)^2$$

Where σ_p, σ_m = Stress in prototype and model, respectively

F_p, F_m = Load on prototype and model, respectively

h_p, h_m = Corresponding dimensions of prototype and model, respectively.

If F_p is expressed in units of the total dead weight of the prototype dolos, and
ρ is the unit weight of the concrete from which the prototype dolos is made, and
V is the volume of the dolos, then

$$\sigma_p/\sigma_m = (n\ V\ \rho/F_m)\ (h_m/h_p)^2$$

where n is the number of units of the dolos' weight, a convenient way to express the design load.
All models were six inches high, i.e. h = 0.5 feet
Thus, for models where the shank thickness t is 32 per cent of h, and Volume consequently equals $.1550h^3$,

$$\sigma_p = .03875\ h_p\ \rho\ n\ (\sigma_m/F_m)$$

For the models with (t/h) = 0.35 the volume is

$$V = .1739h^3,$$

So for those models it can similarly be shown that

$$\sigma_p = .043475\ h_p\ \rho\ n\ (\sigma_m/F_m)$$

The test results for surface stresses then could be presented as the parameter $\sigma/\rho h$, and for internal stress components on the plane of each slice as $(\sigma_1-\sigma_2)/\rho h$. In all cases, the numerical values of the parameter were calculated with n equal to 0.5, that is to say they are stresses induced by two forces whose sum is the dead weight of the dolos. Multiplying the numerical values of the stress parameters by the unit weight of concrete intended for a prototype dolos, in pounds per cubic foot, and the product by the height of that dolos in feet, yields stress in the prototype in pounds per square inch.

Presentation of Results

Figures 4, 5 and 6 are examples of the forms in which stress analyses from most of the 58 slices in the complete test program were reported. Figures 4 and 5 are examples of reports on those slices that presented "Tension" test results and Figure 6 is for a "Torsion" test.

At upper right on Figures 4 and 6 the dimensions of the dolos are shown and the loading patterns of the forces F are displayed.

At top center of all sheets like Figures 4 and 6 is shown where the reported-upon slice was cut from the stress-freeze three-dimensional model.

In the photoelastic examination of each slice the optical system presented the lines of constant stress, the isochromatic fringe patterns $(\sigma_1-\sigma_2)$, at full model size. It also projected the slices at ten times model size. The enlarged projections were examined to identify the locations of maximum stress concentrations at the surface and the direction of steepest gradient of stress variation within the dolos. That part of each slice was reproduced as a line drawing at the lower left of all the sheets like Figures 4 and 6, showing the surface lines of the fluke and of the shank and the intersection profile and the "contours" of the stress parameter $(\sigma_1-\sigma_2/\rho h)$. These slice displays were oriented to place the direction of the visually determined transect of steepest gradient of stresses parallel with the horizontal direction of the data sheet.

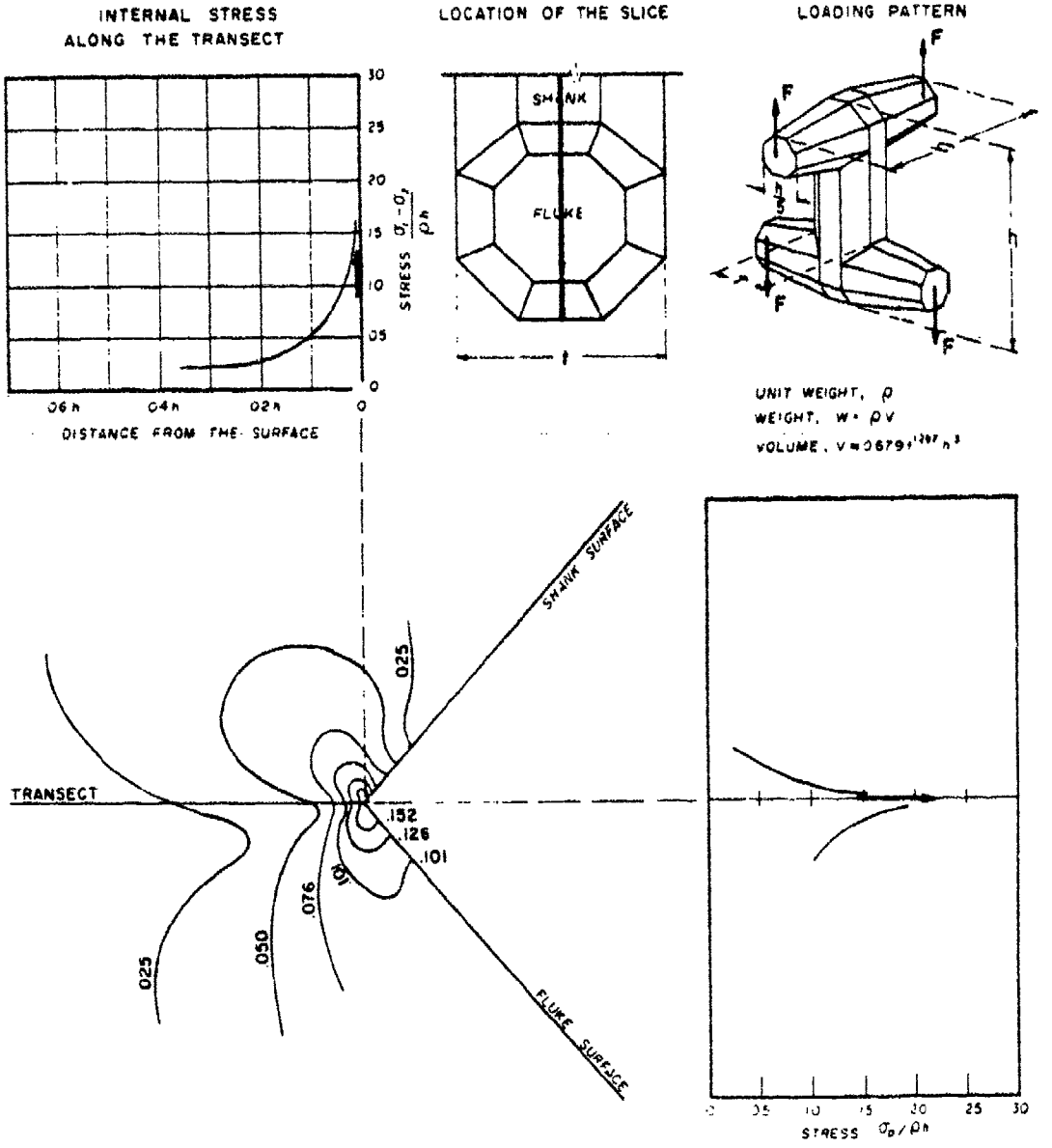
The diagram at lower right on all the sheets like Figures 4 and 6 is a direct projection, from the left, of the stress values at the surface of the dolos, and illustrate the rate of stress increase toward

SURFACE AND INTERNAL STRESS PATTERNS
IN THE BREAKWATER ARMOR PIECE "DOLOS"

SHARP CORNER ; $t/h = 0.32$

ANALYSIS OF SLICE A

TENSION TEST, NO. 32 HSIA



CONTOURS OF EQUAL STRESS
WITHIN THE DOLOS
NEAR THE JUNCTION OF SHANK & FLUKE
$\left(\frac{\sigma_1 - \sigma_2}{\rho h}, \quad F = \frac{W}{2} \right)$

STRESS AT THE SURFACE
OF THE DOLOS

Figure 4

CONTOURS OF INTERNAL STRESSES, $\sigma_1 - \sigma_2$

IN THE BREAKWATER ARMOR PIECE "DOLOS"

"TENSION" LOADING

SHARP CORNERS

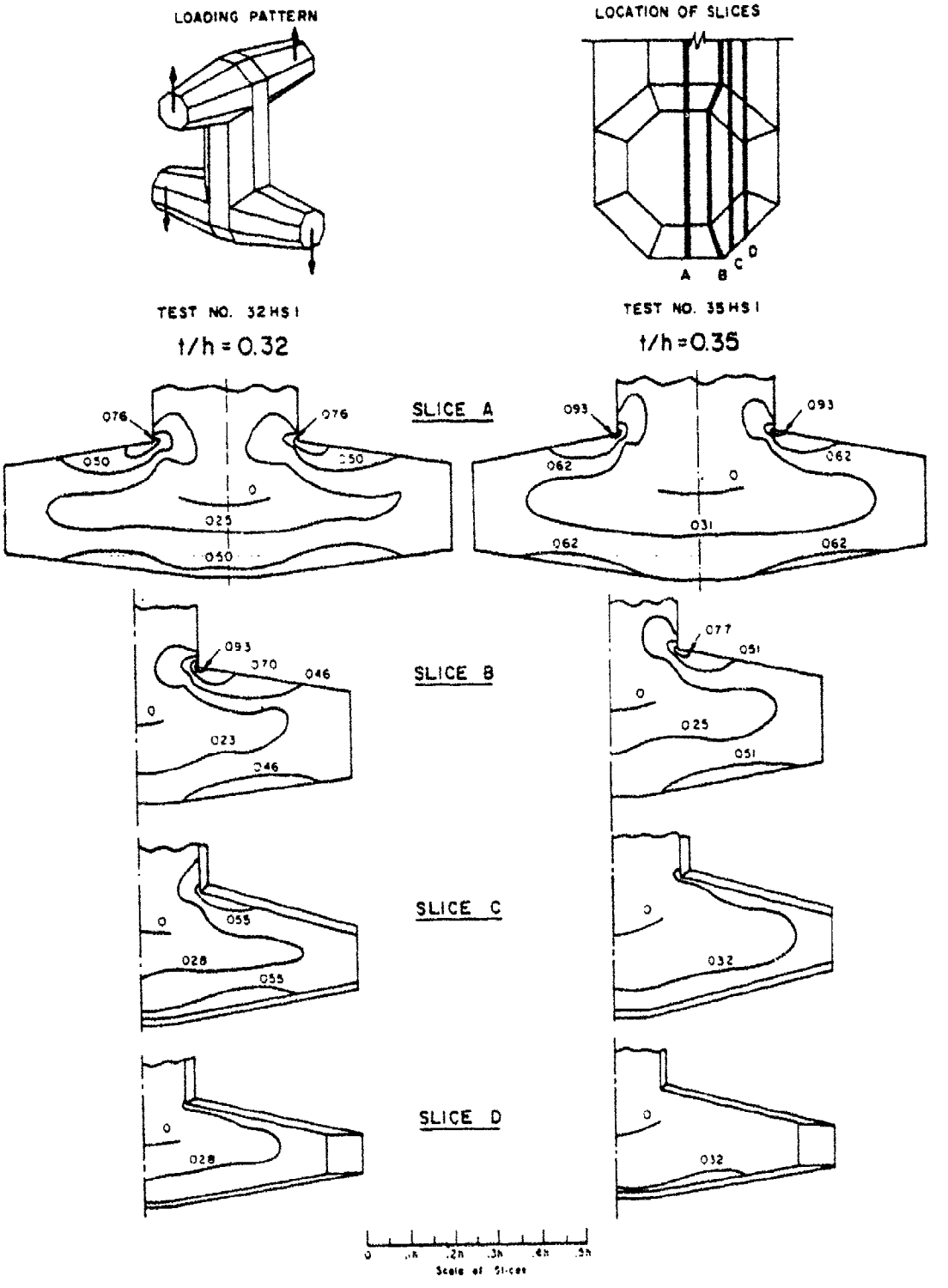


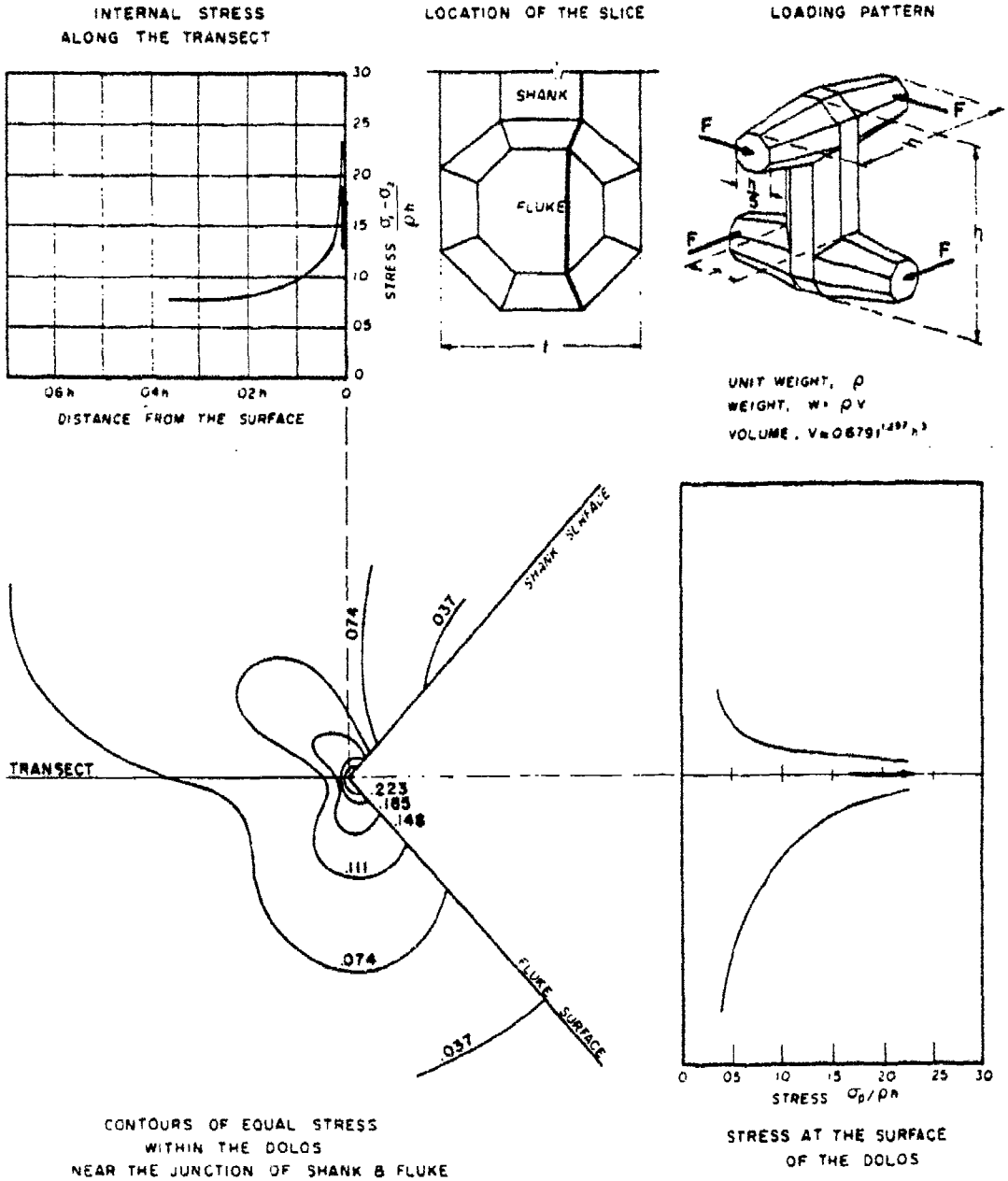
Figure 5

SURFACE AND INTERNAL STRESS PATTERNS
IN THE BREAKWATER ARMOR PIECE "DOLOS"

SHARP CORNER ; t/h = 0.32

ANALYSIS OF SLICE B

TORSION TEST, NO 32 HSB



CONTOURS OF EQUAL STRESS
WITHIN THE DOLOS
NEAR THE JUNCTION OF SHANK & FLUKE

STRESS AT THE SURFACE
OF THE DOLOS

Figure 6

the maximum concentration at the corner. These values at the surface are the tensile stress, because σ_2 must be zero at a free surface and the load pattern is such that σ_1 in this case must be tensile.

At top left, the internal stress variation along the transect of steepest gradient was projected upward from the cross-section at lower left.

Stress gradients in the sharp cornered dolosse were extremely steep at the surface, and absolute determination of values at the corners was not possible. This is indicated on all plots of surface stress and of internal stress for sharp cornered pieces by small arrows, emphasizing that the value of $\sigma/\rho h$ was not determined, and the curves stop short of joining on the Surface Stress graph and of reaching the $h = 0$ abscissa on the Internal Stress graph. In fact, if it were possible to make an absolutely sharp corner, the $h = 0$ line would be the vertical asymptote of the internal stress curve.

Four slices were taken from each "Tension" model for analysis. In models with sharp corners and with chamfered corners and with small radius filleted corners the slices were all taken at the planes described at the upper right of Figure 5. The lower parts of Figure 5 show the equal stress contours for each of the whole slices of the Tension Tests for Sharp Corners, not just the enlarged detail close to the corners that were reproduced on Figures 4 and 6. As before, numerical values for the maximum concentration of stress in the corners could not be shown, but the analysts estimated they would be on the order of 0.30 to 0.32. The rapidity with which stresses reduce, as one considers planes removed from the shank's shoulder at Slice B, is apparent when one examines the stress patterns on Figure 5 of slices C and D at successively greater offsets from the axis of the piece.

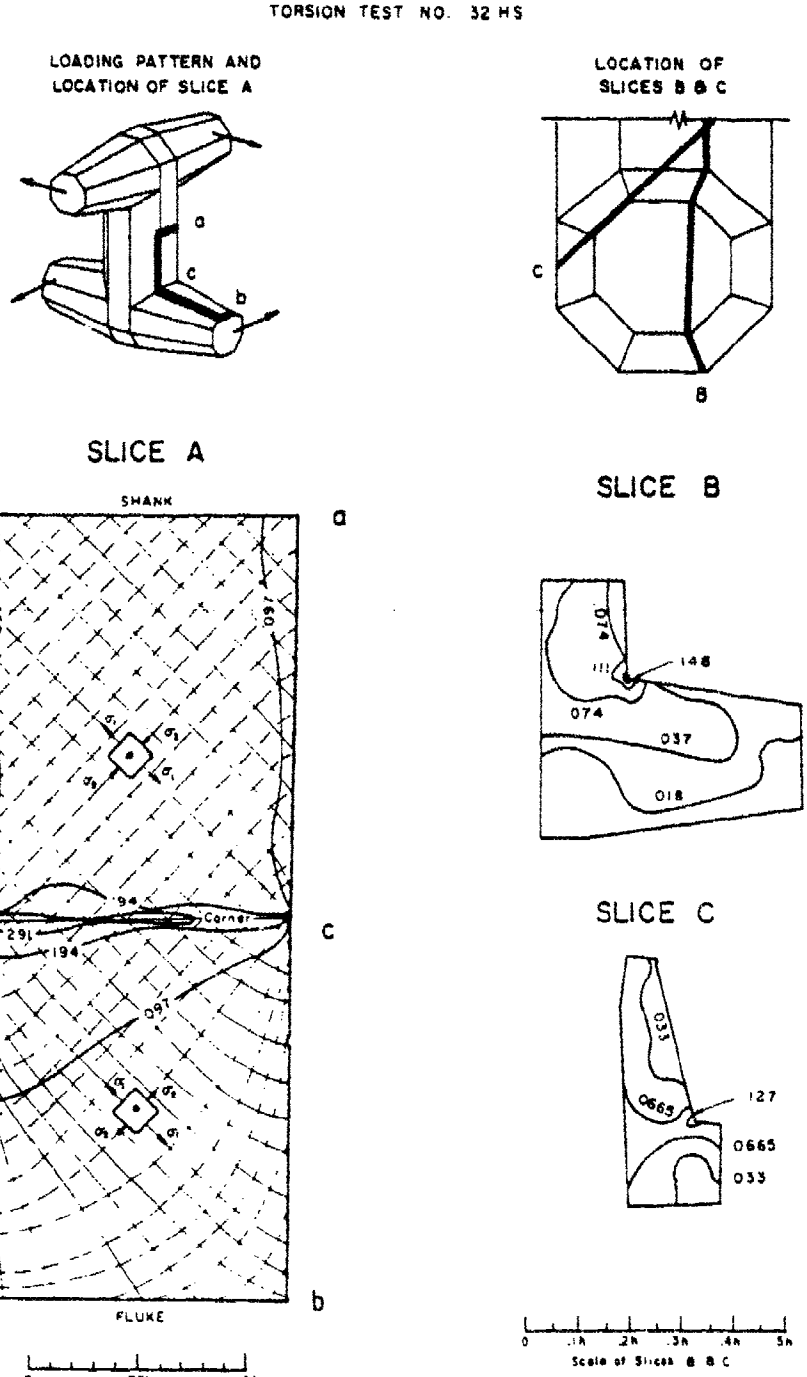
There were three slices removed from each "Torsion" test model that had sharp or chamfered or short-radius filleted corners. Two of the locations are shown at the upper right on Figure 7 and the third, a surface slab containing the corner and called Slice A, is shown on the sketch at upper left. Stress contours for all three slices are shown as before, and orthogonal trajectories of the principle stresses in Slice A are also shown. The solid orthogonals indicate the direction of the maximum tensile stress, and the dashed ones indicate the direction of the maximum compressive stress. These stress trajectories are not to be confused with the lines of constant stress, isochromatic fringe patterns (σ_1-σ_2), referred to as "stress contours" before in this paper. The stress trajectories represent the force flow lines of principal stress direction and are of variable stress intensity along the trajectory. Data sheets similar to Figure 7 were prepared for the Torsion tests of the chamfered and of the small radius filleted corners, but are not reproduced here.

When all data were available from testing the first three variants, with sharp, chamfered and small-radius filleted corners, comparisons were made that suggested yet another corner variation should be investigated. Table II shows maximum stress values at the surface in the corners of the three variants, all expressed as fractions of the stress

STRESS TRAJECTORIES AND CONTOURS OF TORSION INDUCED INTERNAL STRESSES, $\sigma_1 - \sigma_2$ IN THE BREAKWATER ARMOR PIECE DOLOS

SHARP CORNER; t/h=0.32

TORSION TEST NO. 32 HS



DEVELOPED VIEWS OF THE SLICES

Figure 7

analysts' best estimate of the corner concentration stresses in the sharp cornered models, which is 0.30 σh.

TABLE II

RELATIVE MAGNITUDES OF SURFACE STRESS CONCENTRATIONS

	Sharp Corners	Small Fillet Corners	Chamfered Corners
t/h = 0.32 "Tension"	1.00	0.73	0.63
t/h = 0.32 "Torsion"	1.00	0.80	0.63
t/h = 0.35 "Tension"	1.00	0.67	0.67
t/h = 0.35 "Torsion"	1.00	0.87	0.49

Considering the analysts' view, that imprecise moulding of intersections of the chamfer planes with the fluke and shank planes tended toward understatement of stresses by the chamferred models, it appeared that the minute-radius fillet reduced stresses essentially as well as did the chamfer.

Guided by published stress reduction factors for details of common structures, a judgment was made that a dolos with circular fillets of a radius equal to 4 per cent of the dimension h should be tested. With larger radii, the published factors suggested, incremental reduction of stress concentrations became less significant. There was a concern, lest the fillet become so large that the very desirable nesting or tangling characteristic of dolosse in an armor matrix should be impaired. New stress-freeze model dolosse were made,with circular fillets of .04h radius on the corner labelled ab in Figures 2 and 3 and of 1.5 x .04h on the side corners, ac of Figures 2 and 3. As before their h dimension was 6 inches and the same Tension and Torsion loads were applied.

There were models, also as before, where the shank thickness was 32 per cent of h (t/h = .32) and where it was 35 per cent. Figures 8, 9, 10 and 11 show the results, and are directly comparable with Figures 4-7 inclusive. The pattern of slices taken from "Tensi odels in these experiments was modified, as can be seen by comparing the descriptive drawing at the upper right of Figures 5 and 9. By cutting Slice C from the large fillet models on a plane radial to the fluke axis it became possible to identify the inclination and direction of the plane through the crotch of the dolos where all stress gradients were of maximum steepness. The new slice E_A was then cut along that plane. As can be seen on the slice E_A stress contour plot, at the bottom of Figure 9, the stress magnitudes are negligible along that plane. Thus, recognizing that the E_A plane is perpendicular to the A slice, and nominally coincident with the maximum gradient transect lines of the A and B slices, it is practical and conservative to use the internal stress values from slices A and B as indicative of the maximum principal stress σ_1 (tension) rather than the principal stress difference $\sigma_1-\sigma_2$.

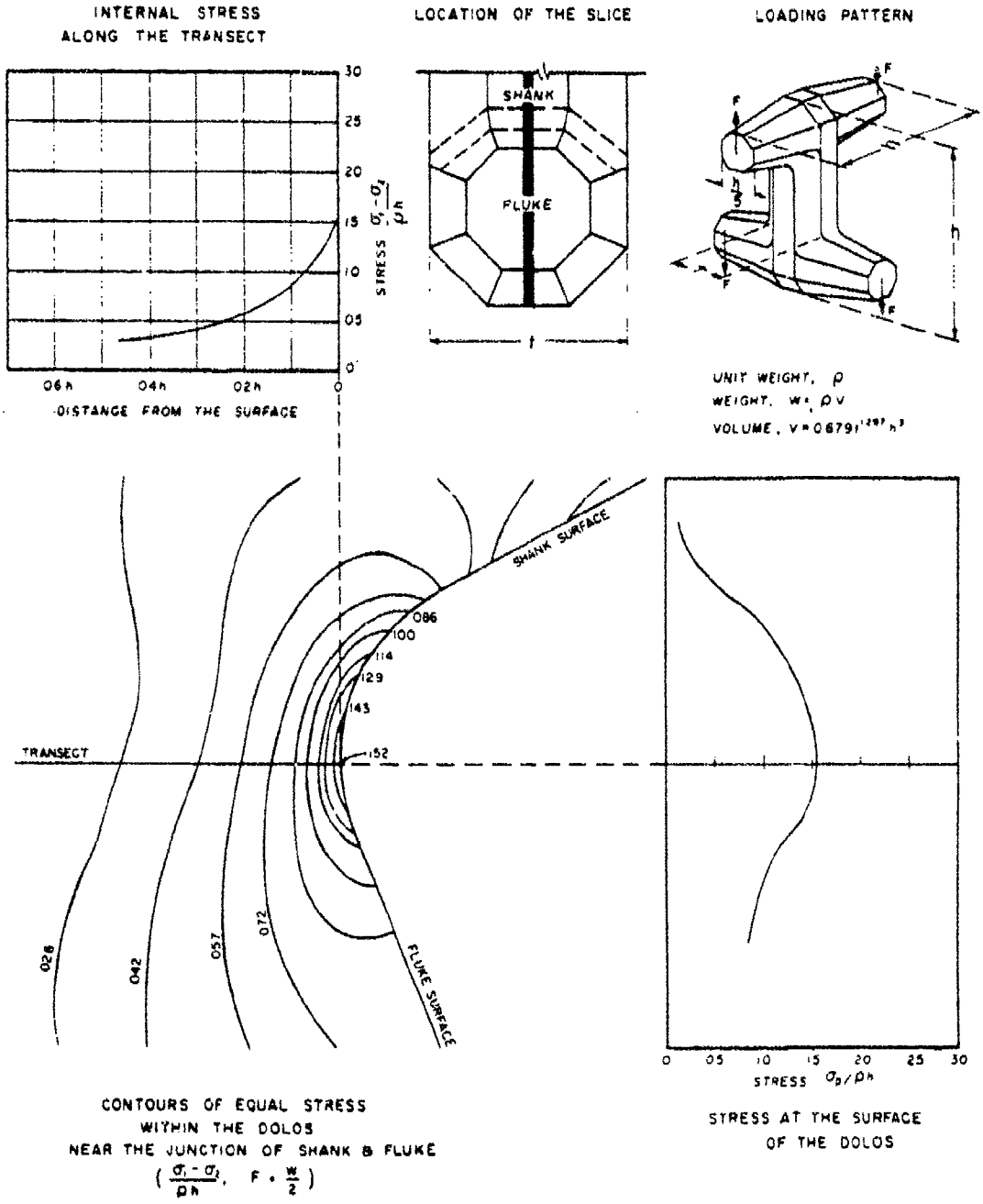
In certain instances it is desirable to know the shear stress magnitude in concrete. This information is readily available from the

SURFACE AND INTERNAL STRESS PATTERNS
IN THE BREAKWATER ARMOR PIECE "DOLOS"

ENLARGED FILLET CORNER; $t/h = 0.32$

ANALYSIS OF SLICE A

TENSION TEST, NO. 32HΦIA



CONTOURS OF EQUAL STRESS
WITHIN THE DOLOS
NEAR THE JUNCTION OF SHANK & FLUKE
$\left(\frac{\sigma_1-\sigma_2}{\rho h},\ F=\frac{W}{2}\right)$

STRESS AT THE SURFACE
OF THE DOLOS

Figure 8

CONTOURS OF INTERNAL STRESSES, $\sigma_1 - \sigma_2$

IN THE BREAKWATER ARMOR PIECE "DOLOS"

"TENSION" LOADING

ENLARGED FILLET CORNER; t/h = 0.32 & 0.35

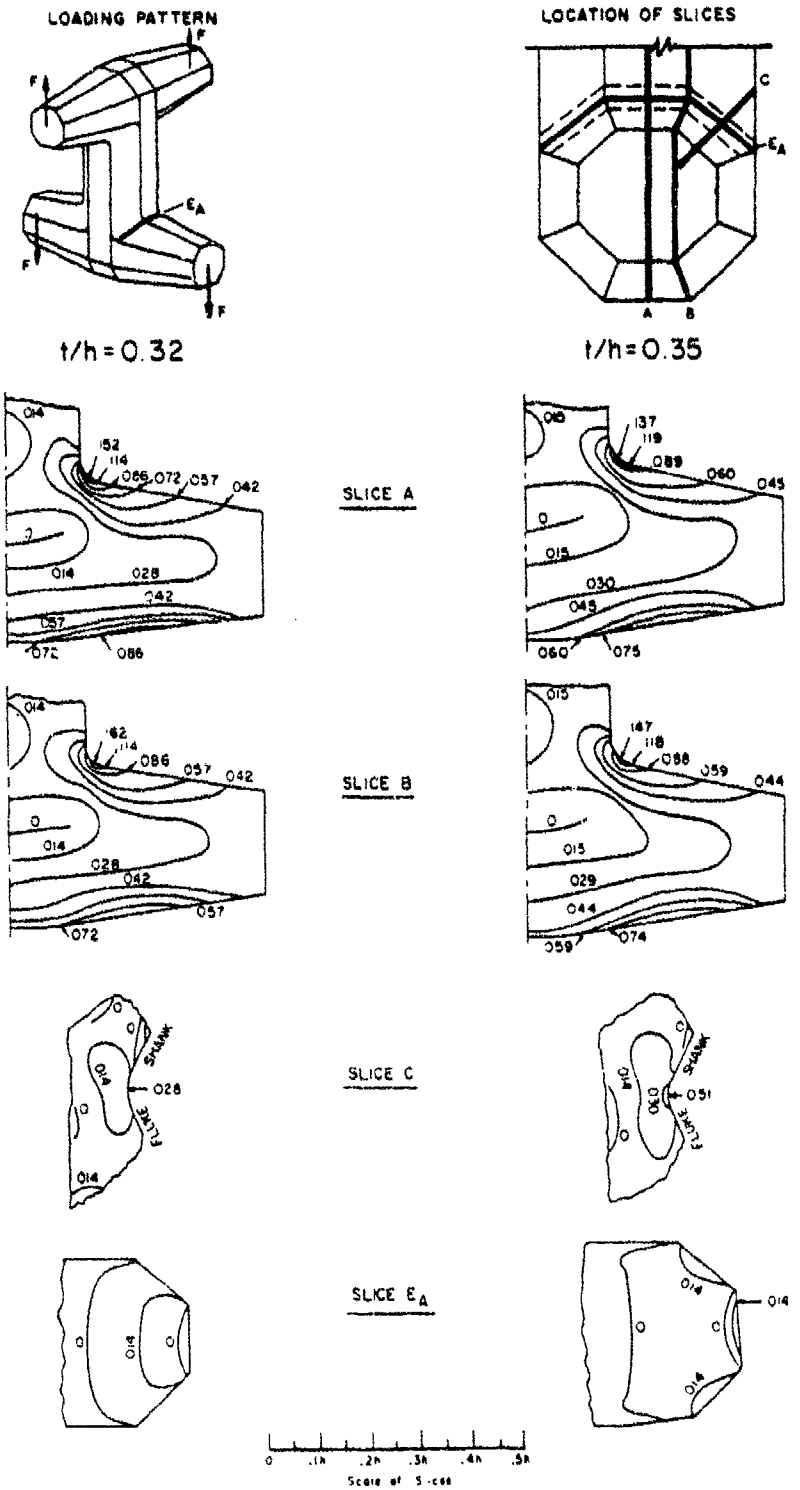


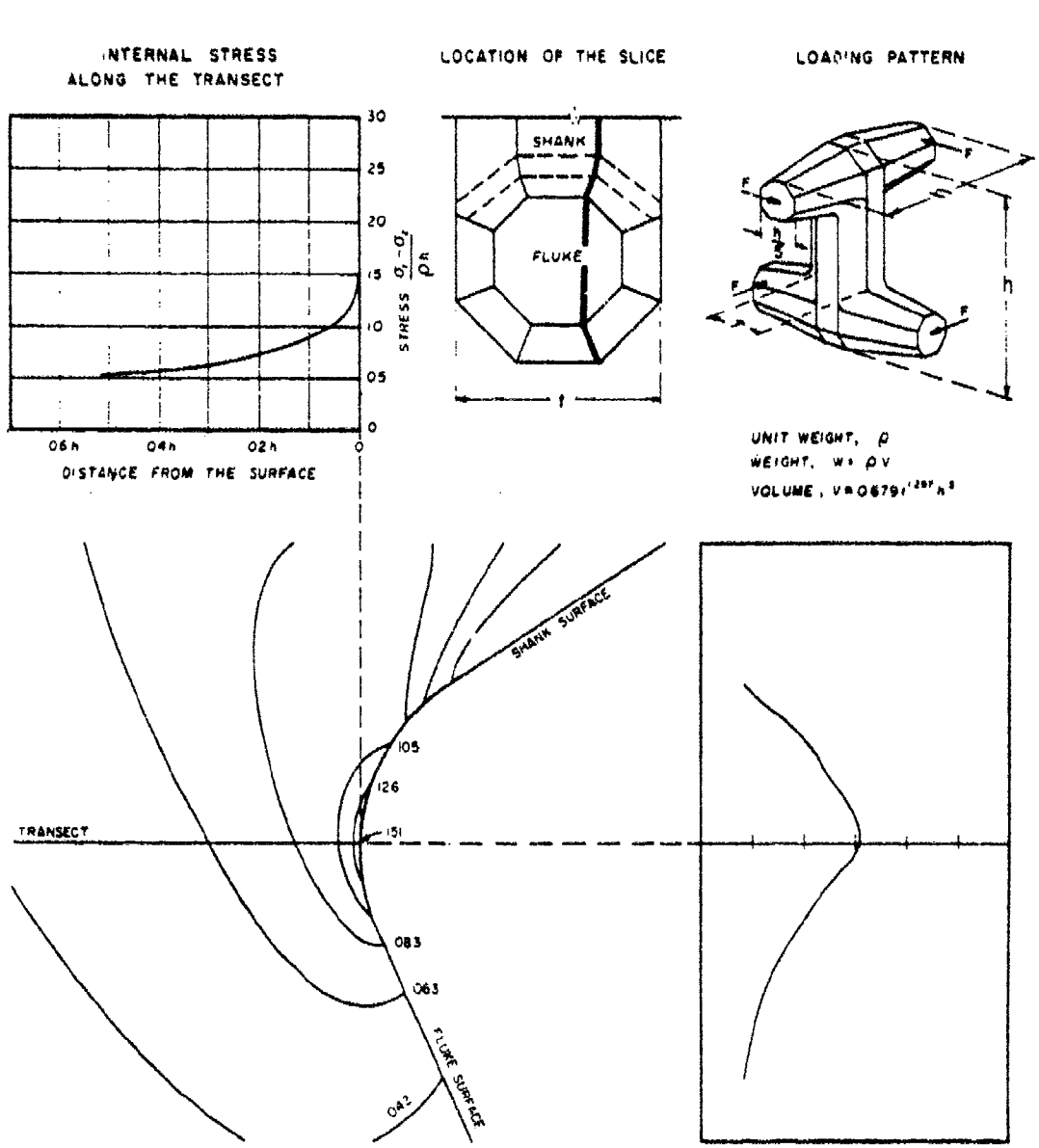
Figure 9

SURFACE AND INTERNAL STRESS PATTERNS
IN THE BREAKWATER ARMOR PIECE "DOLOS"

ENLARGED FILLET CORNER; $t/h = 0.32$

ANALYSIS OF SLICE B

TORSION TEST, NO 32H6B



CONTOURS OF EQUAL STRESS
WITHIN THE DOLOS
NEAR THE JUNCTION OF SHANK & FLUKE
$\left(\frac{\sigma_1 - \sigma_2}{\rho h},\ F = \frac{W}{2}\right)$

STRESS AT THE SURFACE
OF THE DOLOS

Figure 10

CONTOURS OF INTERNAL STRESSES, $\sigma_1 - \sigma_2$

IN THE BREAKWATER ARMOR PIECE "DOLOS"

TORSION LOADING

ENLARGED FILLET CORNER; t/h = 0.32 & 0.35

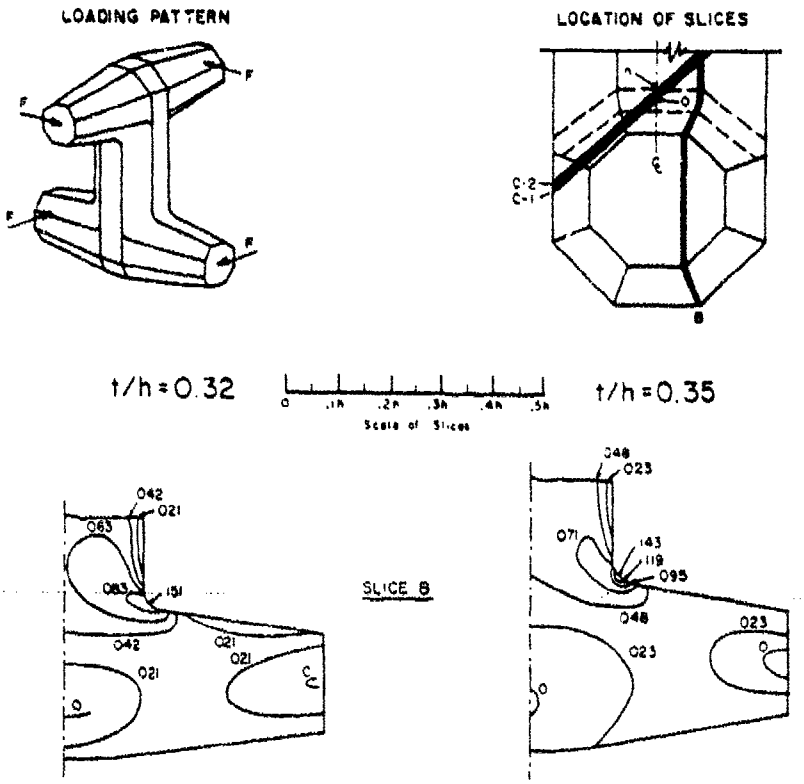


Figure 11

stress contours. The contours as shown are for the difference of principal stresses ($\sigma_1-\sigma_2$). From Mohr's Circle the maximum shear stress τ_{max} is:

$$\tau_{max} = (\sigma_1-\sigma_2)/2$$

and the contours truly display 2 τ_{max} as shown.

Figures 12 and 13 each summarize and provide direct visual comparison of the internal stress gradients for all sixteen of the models that were tested. Figure 12 compares the effects of corner details and shows, to the immediate right from each graph, the numerical value of the stress parameter at the surface for each of the three corners that were not square. Recalling the analysts' estimate, that the sharp cornered dolosse had surface stress values between .30 and .32 ph, it appears the circular fillet with radius of .04h would reduce the critical stress in the corners by 50 per cent, at least, of the sharp cornered stress value.

Figure 13 presents the same curves as Figure 12, but arranged to evaluate the effect of varying the ratio of shank thickness to dolos height, t/h.

EFFECTS ON STRESS OF THREE FLUKE-TO-SHANK CORNER DETAILS

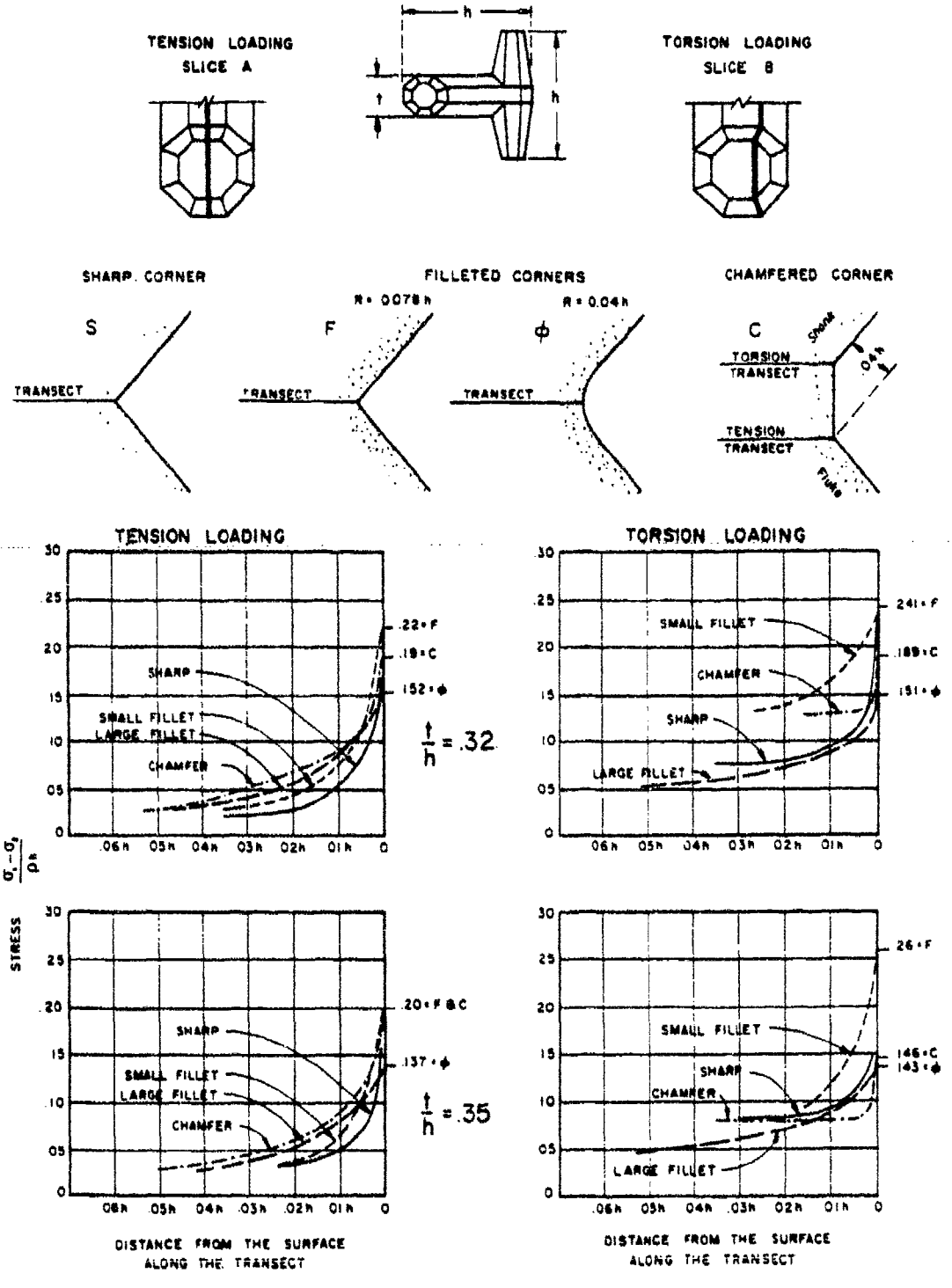


Figure 12

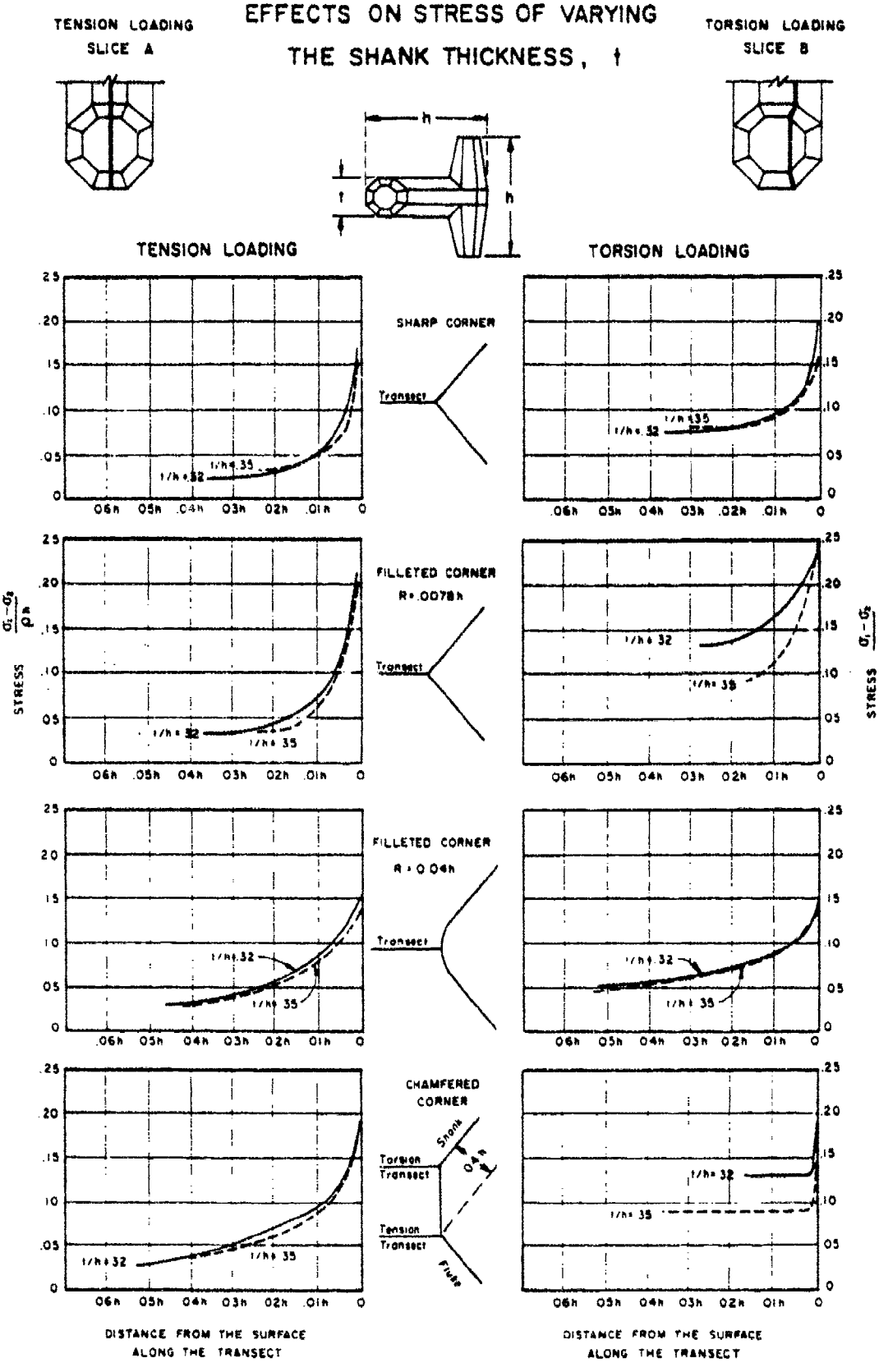

EFFECTS ON STRESS OF VARYING THE SHANK THICKNESS, t
TENSION LOADING SLICE A
TORSION LOADING SLICE B
h
t
h
TENSION LOADING
TORSION LOADING
SHARP CORNER
Transect
FILLETED CORNER R=.007Bh
Transect
FILLETED CORNER R=0.04h
Transect
CHAMFERED CORNER
Shank
0.4h
Torsion Transect
Tension Transect
Fillet
t/h=.32
t/h=.35
STRESS (σ1 − σ2)/ρh
.25
.20
.15
.10
.05
0
.06h
.05h
.04h
.03h
.02h
.01h
0
DISTANCE FROM THE SURFACE ALONG THE TRANSECT

Figure 13

Flexural Strength of Concrete

In 1957 Paul Klieger (3) presented data from comprehensive flexural tests of concretes that are remarkably systematic and are useful in considering fracture stresses in the dolos.

Beams 6 by 6 inches and supported over 18-inch spans were loaded to failure, and the tensile strengths of their concretes were calculated as the Modulus of Rupture. Strengths were determined for ages varying from one day to one year. Six-inch cubes were prepared from the ends of the broken beams and tested as compression specimens. A sufficient number of standard 6-inch diameter by 12-inch length standard cylinders also were cast from the same batches of concrete made for the beams, and they also were broken in compression. That permitted developing a reliable factor for converting cube compressive strengths to standard cylinder compressive strengths. It was determined that, for the aggregate being used, the ratio of 6" by 12" cylinder compressive strength at all ages of test specimens to the compressive strength of 6-inch modified cubes was 0.93.

One of the experimental series dealt with a concrete typical of marine structures, made with ASTM Type II cement and with air entrainment of 4.5%. One hundred forty-three test beams of ages ranging from one day to one year were tested in flexure. There is very little scatter of data from the straight line $R = 1.1 (f_c)^{.75}$, where

R = Modulus of Rupture, the flexural tensile stress in pounds per square inch at the beams' extreme fibre at loads producing failure;

f_c = compressive strength of a standard cylinder at the age of the flexural test specimen.

Figure 14 comes from another arrangement of Klieger's data (points not plotted), to show how the Modulus of Rupture in his tests was found to vary with age of the concrete. It can be put to good practical use when making judgments as to how soon after casting a dolos one can handle it, with acceptable risks of damage to its structural integrity.

Concrete for dolosse has commonly been specified with a minimum acceptable compressive strength at 28 days of 5,000 pounds per square inch, roughly 350 Kg/cm^2. The Modulus of Rupture at age 28 days for that specified compressive strength could be estimated from the Krieger experiments at 650 pounds per square inch, or $46 Kg/cm^2$. At one day, according to Figure 14, that same concrete might have a flexural strength of 200 pounds or more per square inch, or 15 Kg/cm^2.

A project being planned in the United States will use dolosse of the unprecedented weight of 62 short tons, which is just over 56 metric tons. If made from concrete with a specific gravity of approximately 2.4, their h dimension will be 17.5 feet, or 5.33 meters. On the same project smaller dolosse also will be placed, weighing 40 and 11 short tons. Respectively, their h dimensions would be 15.1 and 9.8 feet. Table III has been calculated from the transect curves of Figures 12

VARIATION IN MODULUS OF RUPTURE WITH TIME

TYPE II CEMENT CONCRETE WITH 4.5% AIR ENTRAINMENT
FABRICATED AT VARIOUS TEMPERATURES, 73°F TO 105°F
(Journal ACI, Vol 29 No 12, June 1958)

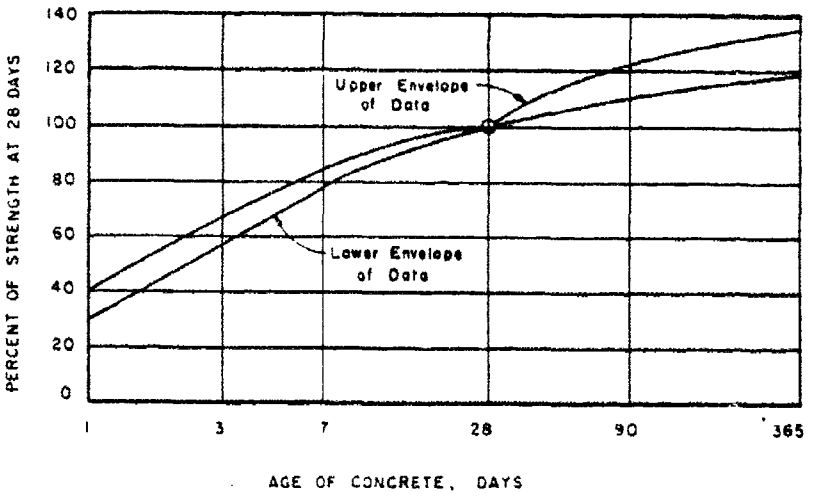


Figure 14

and 13 to illustrate the stress magnitudes, under the loads F of the photoelastic experiments, at various depths in the three sizes of dolosse for the planned project.

Good practice in reinforced concrete design for hydraulic structures, particularly for a sea water environment, requires that there be substantial thickness of dense, hard, sound concrete between embedded steel and the water that surrounds or splashes the concrete. A 3-inch cover, which is 7.5 cm, commonly is required and 4 inches, nominally 10 cm, or more is required by some.

The stresses displayed in Table III suggest the reason for the low breakage experience with dolosse at existing projects that achieved uniformly high quality concrete and that were faithfully built in compliance with appropriate breakwater designs. The largest existing dolosse with sharp corners are those at East London, South Africa, and weigh just under 20 tons each. Larger ones at Hong Kong's High Island East Cofferdam (27.5 tons), Richards Bay, South Africa (33 tons), Crescent City, California (40 tons), Humboldt Bay, California (42 and 43 tons) and Sines, Portugal (44 tons) all are chamfered. Under the loading conditions used in calculating the Table III stresses, all would have surface maximum stresses of less than 650 pounds per square inch. At 4 inches depth, the closest to the surface many experienced

TABLE III

FLEXURAL STRESS AT THE PEAK SURFACE CONCENTRATION POINT AND $\sigma_1-\sigma_2$ AT DEPTHS, NEAR THE PEAK POINT

t/h = 0.32; ρ = 150#/cu.ft.; F = 0.5W.

(Pounds Per Square Inch)

Position	Corner Treatment	Dolos Size & Mode of Loading: 11 Tons h=9.8' Tension	11 Tons h=9.8' Torsion	40 Tons h=15.1' Tension	40 Tons h=15.1' Torsion	62 Tons h=17.5' Tension	62 Tons h=17.5' Torsion
Surface Peak Stress	Sharp	440?	440?	680?	680?	790?	790?
	Sml.Fillet	325	355	500	550	580	635
	Chamfered	280	280	430	430	500	495
	Lge.Fillet	235	220	360	340	420	395
2" From Surface	Sharp	50	125	105	215	140	255
	Sml.Fillet	75	210	155	360	200	440
	Chamfered	115	195	205	300	250	350
	Lge.Fillet	95	115	185	195	230	235
4" From Surface	Sharp	30	110	65	180	80	215
	Sml.Fillet	40	195	95	310	125	370
	Chamfered	55	195	150	300	190	350
	Lge.Fillet	55	90	120	160	155	190

engineers want reinforcing steel placed in concrete immersed in or splashed by sea water, the stresses shown in Table III are so low that the effect of steel bar reinforcement on crack prevention might not be discernible. Such steel then would be redundant at best, because it could only act if some mortal blow striking the dolos opened a crack the bars would be incapable of preventing. For some time, the bars in such a cracked dolos might stop separation into fragments, but probably only so long as oxygenated sea water seeping to the bars through the crack had not yet completed corroding the bars to the point of severing or to exerting swelling stresses on the surrounding concrete that typically makes it spall away and create failure by that condition. It appears that reinforcing steel in dolosse must be an economic waste. However, a possibly stronger reason for not burying them within the piece is a concern such bars could induce shrinkage cracks, during hydration of cement in the freshly poured concrete.

The authors are pursuaded that large dolosse, say 30 tons and heavier, need to have eased corners between flukes and shank to reduce stress concentrations to prudent maximum levels. The use of central fillets of radius .04h and side fillets of .06h radius is seen as the best means for easing those corners, because important collateral benefits in concreting derive. Other easing geometries that are almost as effective in stress reduction do not provide as clear a concreting

advantage. The relative simplicity of incorporating the fillet details when fabricating steel forms suggests it will be useful to have filleted corners in dolosse of all sizes, the concrete placing advantages being the justification.

The viewpoints expressed in the foregoing paper and conclusions reached are those of the authors. The photoelastic testing that made the paper possible were performed for Public Service Electric and Gas Company, of Newark, New Jersey. Their permission to publish the test data for the benefit of practicing engineers is acknowledged, with appreciation.

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Ultimate Strength and Toughness of ARC Shaped Wave Dissipating Members

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(VI－9－B)

ULTIMATE STRENGTH AND TOUGHNESS OF ARC SHAPED WAVE DISSIPATING MEMBERS

Osamu KIYOMIYA*, Hiroshi YOKOTA* and Hideo NISHIZAWA*

ABSTRACT

Arc shaped prestressed concrete members will be applied to offshore structures in deep sea. These members are subjected to large wave forces from outside and inside alternately. Mechanical properties of them under alternate loads are estimated by experimental works.

INTRODUCTION

Offshore structures in deep sea are subjected to large wave forces. Several new types of structures have been considered so as to reduce wave forces and energies, and to keep proper safety. The arc shaped slit caisson breakwater is one of these structures. Several arc shaped members are attached in front of breakwaters and form the front wall of the wave dissipating chambers as shown in Fig. 1. The arc shaped member is a quarter circle in the side view. The prototype breakwater of this type has been placed off Akita in Japan Sea for performance tests and observations.

Arc shaped wave dissipating members of the structure are subjected to large wave forces from both sides alternately. The values and distributions of wave forces were estimated by hydraulic model experiments. There are several problems regarding the mechanical properties of the members such as ultimate strength, toughness and so on. This report provides the experimental results of the arc shaped wave dissipating members made of reinforced concrete or prestressed concrete under alternate loads.

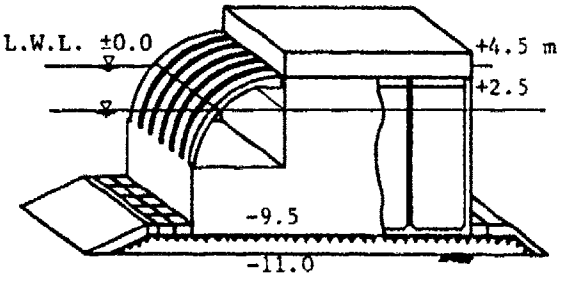


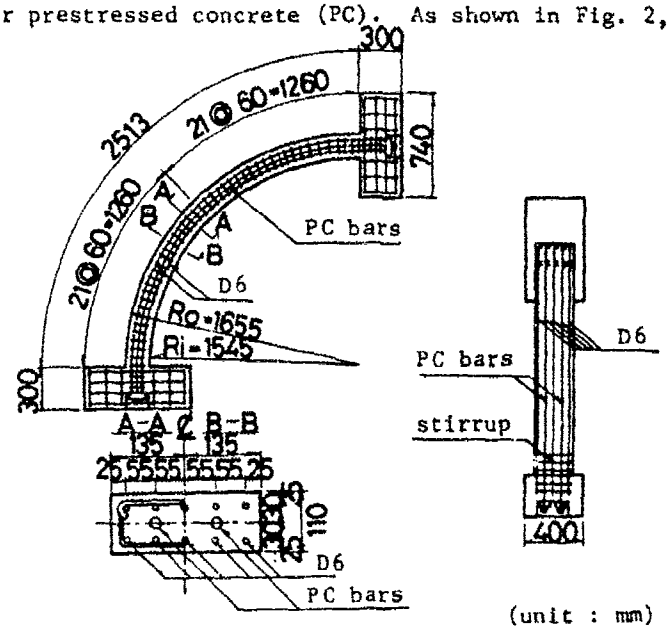
Fig. 1. The prototype caisson breakwater with arc shaped wave dissipating members

* Port and Harbour Research Institute, Ministry of Transport

The test program was carried out as a part of a specific research works, "Study on Analysis of Gravity Type Structures in Deep Waters."

1. TESTING PRCEDURE

Arc shaped beam specimens employed for the experiments are made of reinforced concrete (RC) or prestressed concrete (PC). As shown in Fig. 2, the beam specimen is 27 cm wide, 11 cm high and 160 cm of centroid radius. Two blocks dimensioned 74 (L) × 40 (W) × 30 cm (H) are fixed at both ends of the beam specimen in order to satisfy the fixed end condition. Effective prestress at the midspan section in the prestressed concrete beam specimen is approximately 20 kgf/cm^2 introduced by two prestressing bars with the diameter of 11 mm.



(unit : mm)

Fig. 2. Details of the beam specimen

Mix proportion of concrete is shown in Table 1. The maximum size of aggregate is 10 mm. The mechanical properties of reinforcing and prestressing bars are presented in Table 2.

Table 1. Mix proportion of concrete

Slump	Maximum size of aggregate	Air content	Water cement ratio	Fine agg. ratio	Weights of materials				Air entraining agent
					Water	Cement	Fine agg.	Coarse agg.	
(cm)	(mm)	(%)	(%)	(%)	(kgf/m^3)				(ℓ/m^3)
8±2	10	5±1	58	50	169	293	884	913	2.93

Table 2. Properties of reinforcements

Type	Diameter (mm)	Yield stress (kgf/mm^2)	Ultimate stress (kgf/mm^2)	Elongation (%)	Modulus of elasticity (kgf/mm^2)
Deformed bar	6	37.2	52.8	27.5	2.10×10^4
PC bar	11	143	152	9	2.06×10^4

Load is applied at midspan of the beam specimen repeatedly from outside to inside (designated as O-load), from inside to outside (I-load) or from both sides alternately (A-load) by a 50 tf hydraulic jack. The load is increased by a increment of 1 tf up to first yield. After first yield, the magnitude of the increments depends on the midspan deflection at first yield.

A load cell is used so as to measure the applied load in each direction of loading. The deformations of the specimens are measured by eight displacement transducers. The strains in the reinforcement and concrete are measured by electrical resistant strain gauges attached to the surfaces, respectively. Contact points are attached on the concrete surface at interval of 10 cm, and widths of cracks are measured by the contact type gauges at load steps.

2. TEST RESULTS AND CONSIDERATIONS

The results of the experimental work are summarized in Table 3. The ductility factor at ultimate (μ_u) is obtained by $\mu_u = \delta_u / \delta_y$, where δ_u is the midspan deflection of the beam at ultimate, and δ_y is that at first yield.

2.1 LOAD-DEFLECTION CURVE

The load-deflection curves of the beams under each direction of load-

Table 3. Summary of test results

Beam No.	σ_c	σ_b	E_c	P_u	P_y	δ_y	μ_u	P_c	$P_{0.2}$
	(kgf/cm^2)			(tf)	(tf)	(mm)		(tf)	(tf)
PC-O	426	58.7	2.85×10^5	20.5	9.5	2.7	3.2	2	12.0
PC-I	412	50.0	2.96	15.2	4.5	3.6	12.8	2	4.0
PC-A	422	63.4	2.86	16.0*	10.0*	3.0*	3.2*	2	-
RC-O	438	51.7	2.86	14.7	5.9	4.6	3.0	1	8.9
RC-I	395	54.4	2.96	7.4	2.6	3.2	14.0	<1	1.8
RC-A	420	61.4	2.94	15.1*	5.0*	2.0*	4.6*	1	3.0

σ_c : compressive strength of concrete

σ_b : flexural strength of concrete

E_c : modulus of elasticity of concrete

P_u : load at ultimate P_y : load at first yield

δ_y : midspan deflection at first yield

μ_u : ductility factor P_c : load at first crack

$P_{0.2}$: load at maximum crack width of 0.2 mm

*) obtained when load is applied from outside to inside

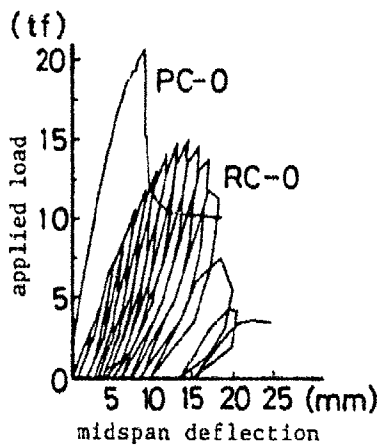


Fig. 3. Load-deflection curve for O-load

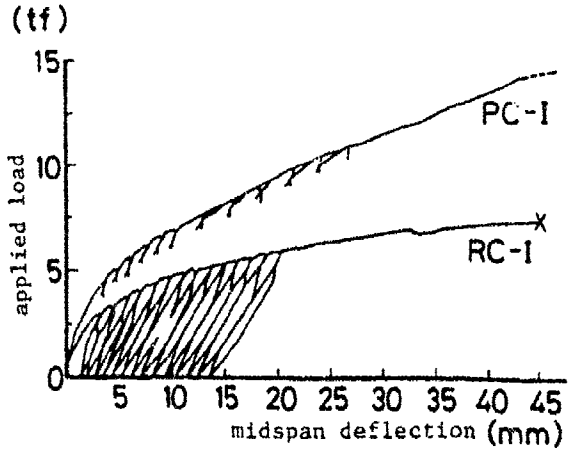


Fig. 4. Load-deflection curve for I-load

ing are shown in Figs. 3, 4 and 5. The first yield load and ultimate load of the PC and RC beams subjected to O-load are considerably larger than those subjected to I-load. When O-load is applied, the slopes of deflection envelopes are roughly constant up to the ultimate loads. On the other hand, the slopes gradually become gentler when I-load is applied. The ductility factor of the beam specimen at ultimate subjected to O-load is about 3, and that subjected to I-load is about 13. Therefore, the beams subjected to O-load provide less deformations after first yields and then collapse. The stiffness of the beam subjected to O-load descends immediately after the ultimate load, and the peak of the applied load is observed clearly. The stiffness of the beam subjected to I-load keeps almost the constant value and the peak of the applied load is not found clearly in the load-deflection relationship. The load-deflection relationship for A-load is almost similar to the load-deflection relations obtained by a compound of O-load and I-load.

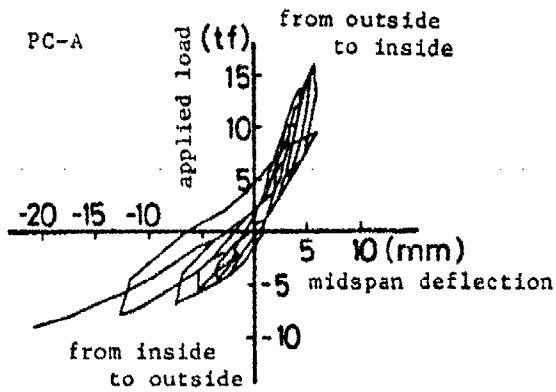


Fig. 5. Load-deflection curve for A-load

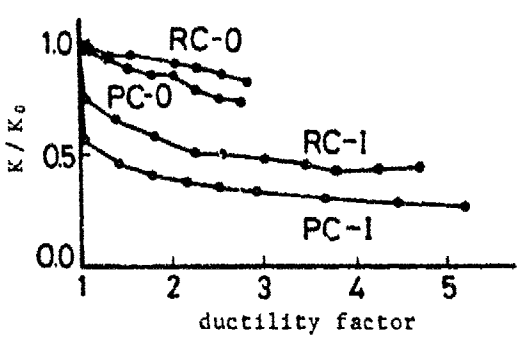


Fig. 6. Stiffness deterioration

2.2 STIFFNESS DETERIORATION

Fig. 6 shows the ratio of average stiffness at each loading cycle (K) to initial stiffness (K_0) plotted against the ductility factor. The average stiffness is obtained by $K = dP / d\delta$ of the load (P) - deflection (δ) curve at each loading cycle. The stiffnesses of the RC and PC beams descend with the increasing ductility factor. The stiffness deterioration of the beams subjected to I-load is greater than those subjected to O-load.

2.3 ENERGY DISSIPATION

Fig. 7 shows the cumulative energy dissipated and the equivalent viscous damping coefficient of the RC and PC beams plotted against loading cycles. The equivalent viscous damping coefficients decrease slowly to 2 to 4 cycles of loading, and subsequently, become almost constant values of 4 to 6 %. The cumulative energy dissipated in the RC beams subjected to O-load is about twice as large as that subjected to I-load. However, the difference of directions of loadings does not affect the energy dissipation in the PC beams.

2.4 DEFORMATION

Fig. 8 shows the deflected shapes of the RC beams at certain load steps. Subjected to I-load, the deflections at midspan obtained from the experiment is larger than that from elastic analyses, but the deflections at a quarter of the span from the experiment is smaller than that from elastic analyses. The reason is considered that the beams turn into two cantilevers after cracks have passed through some sections nearby the midspan. Subjected to O-load, plastic hinges form at point A shown in Fig. 8 as the load is increased, and the deformations of the beam become different from those in early load stages. Therefore, in case of O-load, the beam does not turn into two cantilevers evidently.

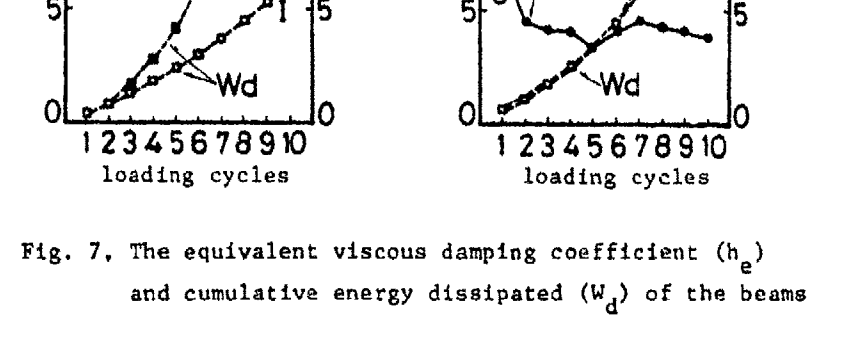


Fig. 7, The equivalent viscous damping coefficient (h_e) and cumulative energy dissipated (W_d) of the beams

2.5 CRACK FORMATIONS AND MODES OF FAILURES

The appearances of the beams at the ultimate loads are shown in Fig. 9. Cracks are initiated in some sections nearby the midspan at first when I-load is applied, and tend to pass through the sections before other sections have cracks. After that, cracks are formed on inner surface, widened and gradually developed towards the outside fiber as the load is increased, and the beams turn into two cantilevers as mentioned before. Just before the ultimate load, crushing of concrete occurred at point B shown in Fig. 9. The mode of failure of the beam subjected to I-load is what is called the flexural failure.

In case of O-load, cracks are formed not only on the inner surface of concrete nearby the midspan section, but also at points A and B at early loading stages as shown in Fig. 9. Cracks formed in these sections are widened and extended as the load is increased. The mode of failure of the beam subjected to O-load is crushing of concrete at a quarter points of the span and shear failure of concrete at the midspan. Bending moments at the midspan and at the ends of the beams are almost the same as those obtained from elastic analyses. However, apparent cracks are not formed actually at the ends of the beams. The reason is considered that the bending moment is redistributed between points A and B after the plastic hinge is developed at the midspan section.

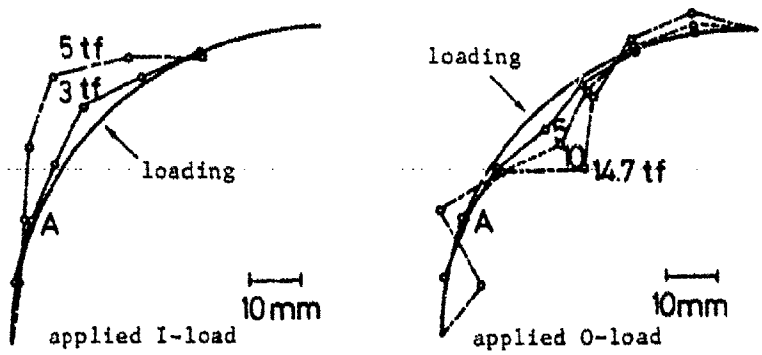


Fig. 8. Deflected shapes of the RC beams

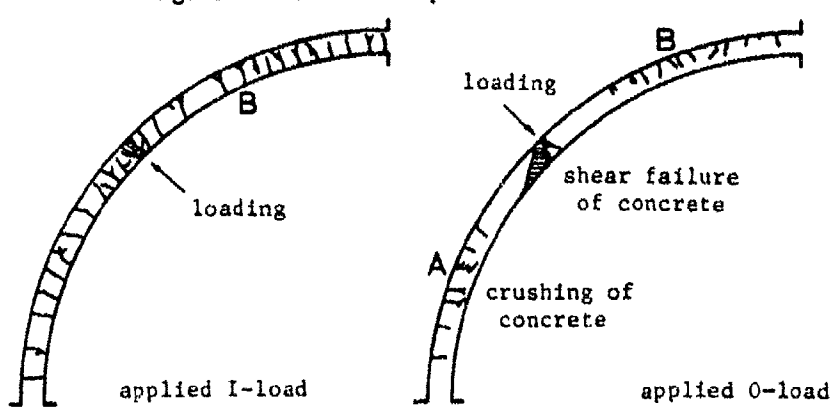


Fig. 9. Crack formations in the RC beams at the ultimate loads

Fig. 10 shows the relationship between the applied load and the maximum width of crack at a quarter section of the span in the RC beams. The maximum width of crack is observed at the loading section, that is, the midspan section. An allowable width of crack in offshore structures is 0.2 mm from the durability point of view. The applied loads at the maximum widths of cracks of 0.2 mm are shown in Table 3. At the same applied loads, the maximum width of crack in the beams subjected to O-load is apparently larger than that subjected to I-load.

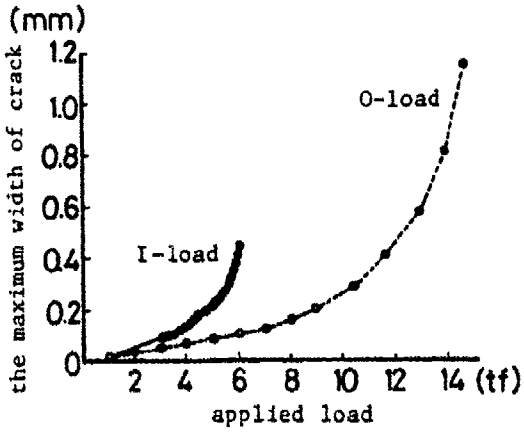


Fig. 10. The maximum width of crack vs. applied load

Relationship between the maximum width of crack and strain in the tensile reinforcing bar is shown in Fig. 11. The maximum width of crack is in proportion to strain in the tensile reinforcing bar up to the strain of 1000×10^{-6} subjected to O-load. The maximum width of crack is in proportion to strain in the tensile reinforcing bar within the limitation in Fig. 11 when I-load is applied. Those are mainly due to the result of modes of failures. In case of O-load, plastic hinges are formed at a quarter sections of the span derived from the redistribution of bending moment, and larger width of cracks are formed as the result of rotation in these sections. Differences between the maximum widths of cracks may be the result that strain distribution in these sections are not linear owing to the curvature of the beams.

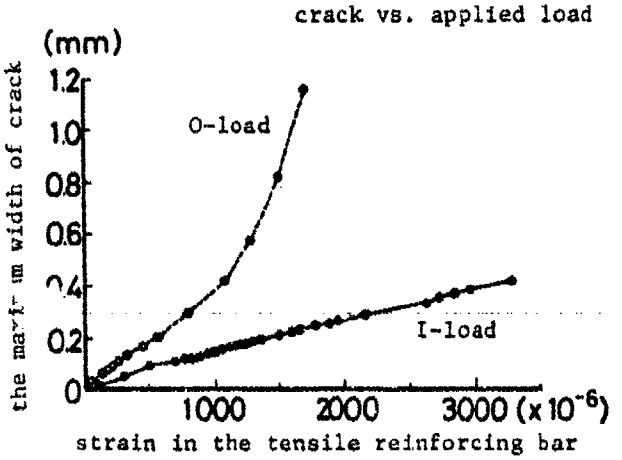


Fig. 11. The maximum width of crack vs. strain in the tensile reinforcing bar

CONCLUSIONS

The following conclusions may be drawn from the results of experimental works :

1) The first yield and ultimate load of the beam when load is applied from outside to inside are larger than those when load is applied from inside to outside. However, the ductility factor and deflection of the beam at the ultimate load which is applied from outside to inside are smaller than those applied from inside to outside. When load is alternately applied from both sides, the properties of the beams are almost similar to those obtained by a compound of the loadings from outside to inside and

from inside to outside.

2) The average stiffness of the beam at the ultimate load is almost the same as that at initial state when load is applied from outside to inside. On the other hand, the average stiffness of the beam when load is applied from inside to outside descends slowly to 30 to 40 % of the initial stiffness at the ultimate load.

3) The difference of directions of loads does not affect the equivalent viscous damping coefficients. The values are 4 to 6 %.

4) When load is applied from outside to inside, the mode of failure of the beam is crushing and shear failure of concrete at the midspan section, and crushing of concrete at a quarter sections of the span. When load is applied from inside to outside, cracks pass through some sections nearby the midspan and the beam turns into two cantilevers.

5) The applied load at the maximum crack width of 0.2 mm in the RC beam are about 9 tf and 2 tf when load is applied from outside to inside and from inside to outside, respectively. The maximum width of crack is in proportion to the strain in the tensile reinforcing bar, when load is applied from inside to outside. However, when load is applied from outside to inside, the increment of the maximum width of crack increases greater than that of the strain in the tensile reinforcing bar.

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2) KIYOMIYA, O., H. YOKOTA and H. NISHIZAWA, "Loading Test of PC Curved Beam," Proc. of 10th Annual Research Meeting of the JSCE Kanto Branch, pp. 157 - 158, Jan. 1983.

Appendix D

Workshop on
MEASUREMENT AND ANALYSIS OF STRUCTURAL RESPONSE
IN CONCRETE ARMOR UNITS
January 23-24, 1985

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