Guidelines for single layer hollow cube armour systems for breakwaters and related marine structures

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Report SR 482
March 1995, revised November 1996

HR Wallingford

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Summary

Guidelines for single layer hollow cube armour systems for breakwaters and related marine structures

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This report gives guidance on the design and use of a particular type of single layer armour systems for rubble mound breakwaters. Hollow cube armour units such as the Cob or Shed units offer very high stability against wave action relative to the armour unit size. Their use in a single rather than double layer further increases their effectiveness.

The Guidelines have been compiled from research by the Single Layer Armour Club, supported by members of this research club, and by the Department of Environment Construction Sponsorship Directorate under research contract PECD 7/6/230. Additional support was given in the compilation of these guidelines by the University of Sheffield, members of the Single Layer Armour Club, and Mr J.E. Clifford, co-ordinator of the research club.

For any further information on these and related studies, please contact Professor N.W.H. Allsop, in the Coastal Group at HR Wallingford, and at University of Sheffield.

Contents

Figures

Appendices

1 Introduction

1.1 General

These Guidelines have been prepared following an extensive programme of research by the Single Layer Armour Club (SLAC). The Club members taking part in the studies were:

Coode Blizard	G Maunsell & Partners
Kirk McClure & Morton	Posford Duvivier
Shephard Hill & Co	Soil Structures (to 9/90)
HR Wallingford	University of Bristol
Plymouth University	Harbourmaster, States of Jersey

Funding support was provided both from the resources of the UK government through the Department of the Environment's Construction Policy Directorate, and from the industrial and research members' contributions. Additional research and publications by Professor Waldron, Professor Allsop and Dr Toner was supported by the University of Sheffield.

This chapter introduces the basis and objectives of the Guidelines in the context of current engineering practice. Chapter 2 describes the design process and discusses the design parameters relevant to single layer hollow cube armour units. Chapter 3 gives detailed conclusions and recommendations for the design of a structure with hollow cube armour. Hydraulic performance and stability, and structural strength and integrity of the units are included. Advice is given on specification requirements and on construction in Chapters 4 and 5. Appendix A outlines an fault tree analysis to identify all significant causes of cracking to these units. Appendix B lists papers and reports produced by the research club and its members relevant to the design / performance of hollow cube armour units.

The research included studies of hydraulic performance and structural loads and stresses of hollow cube concrete armour units on rubble mound breakwaters or revetments. Field measurements were made on existing structures, involving the necessary development of instruments and data recovery systems.

Full scale tests were made on stresses in units under loads likely to occur in practice, and numerical modelling was developed to study structural behaviour.

Physical hydraulic modelling was carried out to study hydraulic performance, and co-ordinated with numerical model studies of run up and reflection of waves. Measurements were also made in such models of wave impact loads and stresses.

The results of the field studies, full scale tests, hydraulic and numerical modelling were compared to give an improved understanding of the most significant aspects of structure performance, and thus to permit preparation of these Guidelines.

1.2 Objectives of Guidelines

The primary objective of these Guidelines is to promote and encourage best engineering practice in the use of single layer hollow cube armour units as armouring to marine structures such as breakwaters, seawalls and shoreline reclamation.

The Guidelines are intended for use only by engineers experienced in the design of marine works, and familiarity with the overall design and construction aspects of breakwaters and coastal revetments is essential.

HR Wallingford and the other members of SLAC who have prepared these Guidelines have based the document on the research studies carried out up to the time of publication, and on their own interpretation and their particular experience of the subject. Users of the Guidelines must make their own evaluation of the suitability, for their own cases, of the information and opinions presented, and the authors of this document decline any liability whatsoever for any consequences of such use.

It is important to note that these guidelines deal only with rubble mounds protected by hollow cube armour units, and not with other single layer units. A number of other single layer armour units have been reviewed by Toner & Allsop (1994), but are not discussed here, except where needed for comparison.

1.3 Essential Literature

There is an extensive selection of publications dealing with the overall design of breakwaters, seawalls and coastal revetments, many of which will be of relevance in the design of such structures armoured with single layer hollow cube units. The principal publications considered to be most useful are:-

a) British Standards Institution (1984) "Maritime Structures - Part 1 General Criteria: BS 6349: Part 1: 1984, BSI, London.

b) British Standards Institution (1991) "Maritime Structures - Part 7 Guide to the Design and Construction of Breakwaters: BS 6349: Part 7: 1991, BSI, London.

c) Simm J.D. Editor (1991) "Manual on the Use of Rock in Coastal and Shoreline Structures" CIRIA Special Publication 83 and CUR Report 154, CIRIA, London.

d) Thomas R.S. & Hall B. (1992) "Seawall Design" CIRIA / Butterworth Heinemann, London.

e) Toner W. & Allsop N.W.H. (1994) "Single Layer Concrete Armour Units for Breakwaters: an up-dating review" Report IT 416, HR Wallingford, Wallingford.

1.4 Structural Elements

Each rubble structure is composed of a number of different elements, particularly core, underlayers, and armour; crown wall; toe armour. Typical rubble structures armoured with single layer hollow cube units are illustrated in Figure 1.1. It must be emphasised that the elements shown cannot be considered in isolation from other parts of the whole structure, even though specific reference may not be included in these Guidelines.

The two main hollow cube armour unit types studied in this research, Sheds and Cobs, are depicted in Figure 1.2. For the purposes of the research studies, an additional and simplified form of hollow cube unit, the Frame Unit, was used in some physical and numerical modelling. This idealised unit, shown in Fig 1.3, permitted the porosity to be varied by simply altering the limb thickness.

1.5 Project Organisation

The *Single Layer Armour Club* did not have a formal constitution, but its research work was co-ordinated by Research Club meeting held about every six months. These were chaired by Mr J.E. Clifford (Consultant) who also acted as overall co-ordinator of the project.

Work by HR Wallingford and the industrial members of the club was funded by their own contribution, and by the Department of the Environment. Some funding support at the universities was forthcoming from SERC, later EPSRC, but this funding was often out of step with the work of other parts of the club.

2 The Design Process

2.1 Introduction

The design process requires the systematic development of a structure to meet the defined purpose in a given environment. BS 6349 Part 7 (1991) indicates a typical sequence of design from concept to completed structure.

Single layer hollow cube armour units have shown, in physical model tests and in prototype, impressive stability under wave action with a very economical volume of concrete in the armour layer compared to many other artificial armour units. As with all concrete units a balance is needed between hydraulic performance and unit structure strength. High porosity, giving good hydraulics, is associated with a tendency to unit fragility, and it is this latter uncertainty which has been the factor of greatest concern and which the current research has been investigating. It should however still be noted that the use of this class of units can permit a rubble revetment or breakwater to be armoured with much smaller units, and hence less concrete, than is possible with most other types of armour.

The interaction of various factors requires an iterative design process, which benefits from the use of a Fault Tree, so that potential causes and effects can be identified and the structure designed to the required levels of safety and performance. A Fault Tree to examine potential causes leading to cracking or breaking of single layer armour units is attached as Appendix A, and serves as a guide to considering the various factors described below.

2.2 Design Considerations

2.2.1 Data Collection

The collection of oceanographic data on tides, winds, waves and currents requires careful attention for all marine structures, and is described fully in BS 6349 Part 1 (1984), the CIRIA rock manual by Simm (1991) and elsewhere. In the case of slender concrete units such as hollow cubes, it is important also to collect data on temperatures, particularly those caused by solar radiation, the effects of which are described in 2.2.7 below.

2.2.2 Purpose and Function of Structure

The function of the structure needs to be defined with care, paying particular attention to the location and the environment. For example, concrete armour units with slender limbs on a revetment may be at risk of excessive abrasion damage due to beach material movement. Potential toe erosion and loss of support to the units (see 2.2.10 below) may also give peculiar problems. A further consideration, not only for hollow cube units, is the risk of injury to persons

attempting to climb over or through them, particularly as their uniform size may make them appear inviting.

2.2.3 Design Life

As for many marine structures, a design life of 50 to 100 years may be appropriate, but the selection of design wave conditions will depend on the acceptable probability of exceedance of the design conditions and associated structure response. BS 6349 Part 7 and the CIRIA rock manual discuss this aspect and the way in which risk may be analysed.

2.2.4 Run-up and Overtopping

Acceptable wave run-up levels, and/or the degree of wave overtopping, will depend on the purpose and location of the structure. The stability of single layer hollow cube units is such that relatively small units are generally used for given wave heights. Wave run-up levels for a given wave climate are therefore higher than for a thicker layer of random placed concrete armour units. In practice the overtopping, usually expressed as mean overtopping rate per unit length of structure, is a more useful measurable parameter, and the recommendations in the CIRIA rock manual edited by Simm (1991) are probably the best current guide to acceptable values for various circumstances.

2.2.5 Wave Reflection

For certain structures wave reflection may need to be limited, such as where navigation may be impeded or where other structures or beaches may be exposed to unwanted wave action.

The reflection performance depends principally on the structure slope angle, and the incident wave steepness. Reflections also tend to decrease where overtopping increases. The choice of reflection coefficient may be limited, and a compromise may be needed.

2.2.6 Armour Stability

A significant feature of single layer hollow cube armour placed closely on a rock slope is the very good stability of the armour layer under wave attack. Extensive measurement of forces on units has confirmed the field experience that lift-out forces are seldom sufficient to extract a unit from a properly constrained armour layer. It is however important to note that these systems rely on placement as a close array to generate stability. Any un-restrained units, being relatively light in relation to other armour types, may be more easily moved. This is particularly important at the top of an armoured slope, see 2.2.10 below.

2.2.7 Armour Integrity

Stresses in individual armour units can arise from the following:

> Casting and handling;
> Loading due to static weight of units;
> Wave impact pressures;
> Settlement of the mound / foundation;
> Thermal strains / stresses;
> Accidental (mechanical) impacts.

Of these causes, stresses from casting and handling have been found to be relatively small providing good control is exercised during construction.

Self-weight loads can be significant if stresses are induced where point loads from up-slope units meet the centre rather than the corner of a lower unit. The risk of these higher stresses can be reduced if the units are placed in columns up the slope. In the special case of roundheads or bends, consideration might be given to placing filler blocks at intervals to retain the columnar pattern as far as possible. It should be noted that the units are structurally highly redundant and stresses cannot be assessed accurately for all possible boundary conditions of load and support.

Significant wave impact pressures have been recorded both in the field and the laboratory, and related to the incident wave height. Both high frequency transient peak pressures and lower frequency more persistent pressure rises have been detected.

Settlement of the mound could lead to an increase of load on units, but at present no damage has been observed in the field which could be attributed to settlement.

Stresses due to differential thermal strain can be the largest component in total stress. Slow variations in ambient temperature do not induce significant stress, but unequal heating by the sun's rays can be important for these structurally redundant units. Rapid cooling of units as the tide rises will compound this problem.

Stress modelling calculations using finite element models have shown that mid-summer sunlight in mid-latitude locations can induce stresses of the same order as the tensile strength of concrete. Rapid cooling induces similar orders of stress.

Should the tensile strength be exceeded causing cracking, the stress distribution of the unit would be altered and peak stress values would probably decrease. This has not however been included in the studies, but it has been observed that many cracked units in service remain whole and appear fully effective.

Research to date thus indicates that the significant stresses for design are those due to self-weight, wave impact and thermal strain.

2.2.8   Concrete Quality

Normal precautions should be taken for concrete in the marine environment, such as described in BS 6349: Part 1 (1984). The selection of suitable aggregates and cement, a sufficiently high cement content and a reasonably low water/cement ratio are all factors to ensure strength and durability. Temperature control of the process and good curing are also important. There is no evidence to indicate that concrete used for hollow cube armour units requires any unusual attention.

2.2.9   Reinforcement

In cases where it is predicted that stresses in the concrete in service can exceed the tensile strength, consideration has been given to providing reinforcement to the otherwise plain concrete. Resistance to the complex and varying stresses around the hollow cube would normally require reinforcement with small cover at virtually all faces. Even if non-corrodible materials of suitable elasticity were to be used instead of steel, the complexity and cost of casting have generally been prohibitive. It may therefore be concluded that the use of conventional reinforcement in these units is unlikely to offer any significant advantage.

If however high stresses can occur, then some cracking of units must be expected. One solution which has been adopted in such a case has been to provide connected hoops of steel in the centre of each limb in the cube. This solution is not effective as conventional stress reinforcement, but if cracking should occur, the hoops will hold the unit together, even if the cracks become complete breaks in the unit.

Accidental impacts from vessels or floating debris could cause cracking, fracture and possibly displacement of units. No general advice can be given, but the immediate effects of impacts would be reduced by the provision of hoop reinforcement, even if later replacement of the damaged armour were needed, and this approach has been used on a number of structures with hollow cube armour.

2.2.10   Crown Wall and Toe Support
It is important to ensure that units at the top of the slope are restrained from moving up the slope under wave forces. Where a high freeboard is required, the weight of units above the highest run-up level may be enough to give adequate restraint, but a low freeboard will almost certainly require other measures. This could take the form of a heavy cap, with up-lift forceson the cap reduced by providing holes for easy venting of water travelling up the slope through the hollow units.

Firm toe support to the armour slope is also essential to prevent any settlement of the lower unit and risk of the slope opening and units being plucked out where they are no longer close together. Such a toe support is normally provided by a rigid mass concrete beam or heavy blocks well founded on a firm support, but the particular configurationwill depend on the soil conditions.

2.2.11   Summary
The foregoing paragraphs describe in general terms considerations of special importance in the design of a single layer hollow cube armoured structure. The following sections give detailed recommendations for design taking account of the results of recent research into hydraulic performance, stability, and structural strength / integrity of the units.

3   Structure Design

3.1 Introduction
This Chapter should be read in conjunction with the appropriate sections of BS 6349 (1984, 1991), the CIRIA rock and sea wall manuals (1991, 1992) and those documents should be taken into consideration when interpreting these Guidelines.

The elements to be studied in the design are listed in order of those concerned with hydraulic performance and stability, followed by considerations of structural strength and integrity. This listing has some relevance to the order of design development, but the elements interact, and adjustment and reappraisal will be needed before the design is completed to ensure that all criteria are satisfied.

In these Guidelines the recommendations assume that the location and concept have been determined; the necessary design data have been collected; and design criteria have been establiShed.

There will in practice be alternative designs and economic comparisons to be made, but these are not within the scope of these Guidelines.

3.2 Hydraulic Performance and Stability

3.2.1 Selection of Armour Unit

Starting with the decision to utilise hollow cube units in the primary armour layer, the size of unit, its particular shape and its porosity require consideration.

Field experience and research has been with 2 tonne nominal hollow cubes with 1.3 m side dimensions, symmetrical in 3 directions and with a porosity of about 61%.

Experience to date suggests that Cob or Shed units of 1.3 m side may be used for many situations where wave conditions fall between H_s = 2 to 4m. Below the lower end of this range of wave heights, it may be more economical to use smaller units, although some benefits may accrue by using large units in relation to the wave height, and thus by the reduction in the number of plant operations required to cover the given area. For wave conditions larger than H_s = 4m, the unit size may be increased by multiplying the linear dimension by the ratio of the design significant wave height to H_s = 4m.

For armour units scaled for wave conditions larger than H_s = 4m, it is expected that the relative structural strength will decrease with increasing unit size, a phenomenon now well known for all concrete armour units, and methods to determine these effects are discussed in section 3.3.

During the research studies for SLAC, the porosity of simplified hollow cube units (Frame Units) was progressively reduced from n_v≈65% to n_v=50%. These experiments confirmed that reductions of porosity increase wave reflections and run-up levels / overtopping. Calculations of stresses for example loading cases confirmed that stresses due to wave slam and self weight decrease for units with lower porosity, and hence with relatively thicker limbs. The same is not however the case for stresses due to solar radiation, where the thicker limbs do not reduce stresses in the unit.

3.2.2 Slope Angle

The selection of a slope angle, as for all rubble mound structures, is mainly determined by site conditions, such as foundation pressure, and construction economics. Slopes between 1 : 1.333 and 1 : 2.0 have been used successfully with hollow cube units, which probably covers the range which would be considered for a given design case. Over this range of slopes, it is unlikely that changes to the slope angle will greatly alter the hydraulic performance. No information is available on the performance of these armour systems at slopes flatter than 1:2.0.

3.2.3 Crest Level

The crest level of a rock mound structure is determined by two primary considerations, the level required to ensure that the overtopping by wave action does not exceed the selected criteria, and the level which is needed for economical construction. In the case of a free standing breakwater, the crest width is also a function of the plant to be used in construction.

In many cases a wave wall can be used to minimise the structure height, affecting particularly the volume of core, for a given amount of overtopping. Such a wave wall is often associated with a crest berm of underlayer supporting the sea side

armour. In the case of hollow cube armour the need for close placing constrains the manner in which a berm can be provided, as restraint to units against moving up the slope must be retained.

Wave overtopping

Wave overtopping may be described by the number or percentage of waves passing over the crest expressed as N_{wo}; or by the mean overtopping discharge per unit length, Q. The data available seldom identifies both responses, so analysis has generally been concentrated on the prediction of the mean overtopping discharge Q, addressed here.

Wave overtopping depends on freeboard R_c, and incident wave conditions, usually described by H_s and T_m. The prediction method developed by Owen (1980) for simple slopes relates the dimensionless discharge Q^* to the dimensionless freeboard R^* by an exponential equation with a relative run-up or roughness coefficient, r, and coefficients A and B for each slope angle:

$$Q^* = A \exp (-B R^* / r) \tag{3.1}$$

where $\qquad Q^*=Q/(gT_m H_s)$ $\qquad\qquad\qquad$ (3.2)

and $\qquad R^*=R_c/T_m(gH_s)^{0.5}$ $\qquad\qquad\qquad$ (3.3)

For smooth slopes, r = 1.0, and values of A & B have been derived for slopes from 1:1.0 to 1:2.0:

Slope	A	B
1:1.0	0.0079	20.1
1:1.5	0.0102	20.1
1:2.0	0.0125	22.1

Table 3.1 Values of A and B for smooth slopes, r=1

The form of eqn (3.1) is shown in Figure 3.1 by plotting Q^* against R^* and using coefficients A and B from Table 3.1. For structures with small relative freeboards and/or large wave heights, the regression lines come together at one point, indicating that the slope angle, and relative roughness are no longer effective in controlling the overtopping discharge at these low (relative) freeboards. The discharge characteristics for slopes 1:1, 1:1.15 and 1:2 are very similar, but overtopping reduces significantly for slope angles less than 1:2.

Owen's method was developed initially from laboratory measurements for smooth slopes only, but the use of the roughness factor, r, allowed its extrapolation to study the overtopping performance of rough, and even armoured slopes. Since 1980, various researchers have explored alternative prediction methods for armoured slopes, see Bradbury & Allsop (1988) and Aminti & Franco (1988), but no new method has proved any more reliable. The advantage of Owen's method is its simplicity, and the ready availability of data to support particular coefficient values. Three alternative approaches have therefore been developed:

a)   Use Owen's method and coefficients A and B with r derived from tests with the correct slope geometry;

b)   Use Owen's general equation, but with new values of A and B derived for similar cross section, and r = 1.0;

c)   Develop alternative equation, with new coefficients.

For armoured slopes, it is suggested that the original Owen equation may be used for overtopping, but that the coefficients A and B should be changed depending on the armour type and structure slope. The original Owen method using values of the roughness coefficient is not as accurate as using regression lines for site specific data. The simple Owen method is however very quick and easy to use where little site specific data is available.

Structure	Slope	A	B
Cob units	1:1.33	0.00839	46.5
Shed units	1:1.33	0.00268	29.9
Antifer Cubes	1:1.5	0.49600	82.7
Tetrapod	1:1.5	0.0075	71.0

Table 3.2 Values of A and B for armoured structures, r=1

The overtopping performance of a single layer and a double layer hollow cube armour system are compared in Figure 3.1. Both armour units had a porosity of about 60%, and were placed to a tight pattern on a slope of 1:1.333.

Where values of A and B cannot be calculated using site specific data, the original Owen formula with values of A and B for various slopes can be used with a roughness coefficient r appropriate for the armour concerned. Values of the coefficient r for various armour units derived by Besley et al (1993) are given in Table 3.3.

Armour type	r
Rock	0.5-0.6
Hollow cubes	0.5
Dolos	0.4
Stabits	0.35
Tetrapods	0.3

Table 3.3 Recommended values of r for armoured structures using A and B values given in Table 3.1

3.2.4   Wave reflection performance

Wave reflections from structures armoured with single layer armour units were studied by HR Wallingford on a number of separate occasions. The final series of model tests were designed to investigate the reflective properties of a single layer armoured structures, see Besley et al (1993), the observations made during the course of that test series therefore form the core of the reflection analysis presented in this report.

Investigations by Allsop (1990) confirmed the use of a relationship originally developed by Seelig & Ahrens between the reflection coefficient C_r and the mean Iribarren number ξ_m as:

$$C_r = (a\xi_m^2) / (b+\xi_m^2) \tag{3.4}$$

where $\quad \xi_m = \tan \alpha / (2\pi H_s / gT_m^2)^{0.5}$ \hfill (3.5)

and a and b are empirically derived coefficients. Allsop suggests values of a=0.64 and b=8.85 for random waves on 2 layer rock armoured slopes, and a=0.96 and b=4.8 for smooth slopes.

Values of the empirical coefficients for a and b were determined for different single layer armour units constructed with a slope angle of 1:1.33 having 16 rows of armour units laid on their slopes:

Armour type	a	b
Cob	1.00	12.0
Sheds / Frame units	1.04	14.2
Frame units (2 layers)	1.07	21.5

Generally reflections from a single layer armoured structure fall between those predicted for an equivalent smooth slope and those for the equivalent 2 layered rock slope (Fig 3.2). At low Iribarren numbers, reflections from Cob and Shed units compare favourably with the equivalent rock slope structure. As the Iribarren number increases, there is a loss of efficiency at about $\xi_m=3.8$. Thereafter the efficiency continues to fall off, approaching the reflection performance of equivalent smooth slopes.

The reflection performance of Shed units were shown to be marginally better than that of Cob units (Fig 3.2). Armour unit placement pattern had no effect on the reflection coefficient of the structure (Fig 3.3). The reflection performance of the structure armoured with different porosity Frame units improved marginally with increased unit porosity (Fig 3.4).

3.2.5   Underlayer, Core, Foundation and Toe

These elements of the design of a single layer hollow cube armoured rock mound require conventional considerations which are largely concerned with overall geometry and materials to be used, and are referred to in 3.2.7 below with regard to overall stability.

The size of underlayer is an important factor for the relatively small primary armour thickness provided by single layer units.   The usual relationships between armour size and underlayer size, as referred to in the references, need to be modified.   It is important that underlayer does not pass through the holes in the units, but it is not necessary for all the rock particles to be greater than the hole size.   Model tests and experience in practice have shown that a narrow graded underlayer of D_{50} equal to the size of the hole, which for a 1.3 m Cob or Shed is about 0.5 m, is satisfactory to avoid extraction.

A particular breakwater which was model tested had prototype rock underlayer of size $D_{15}=0.37m$; $D_{50}=0.57m$; $D_{85}=0.77m$, which at model scales became 12.5mm; 19mm; and 26mm respectively. The model indicated no extraction of underlayer through the armour. Similar model tests with underlayer about 40% of the size given above, i.e. with D_{50} prototype about 0.23m, showed some displacement of finer underlayer near the lower face of the single layer armour, but no extraction of the rock through the armour layer. This may imply that D_{15} for the underlayer should not be allowed to fall below 0.15m.

It should however be noted that during construction any exposed underlayer is vulnerable to erosion by direct wave action. Where smaller than normally used under random placed concrete armour for the same wave action, the underlayer for hollow cube armour might more easily be displaced. For this reason it will often be beneficial to restrict underlayer gradings to the envelope given by $D_{85}=0.8m$ and $D_{15}=0.3m$.   With these sizes of underlayer it is still possible to prepare a reasonably smooth surface to permit regular placing of the armour layer.

10

The core requires to meet normal filter criteria with relation to the underlayer. With relatively small underlayer the maximum size of core material is likely to be similar to that of the underlayer, but with less need for close grading limits.

The foundation of the mound and particularly the toe support must ensure the retention of the armour units in close contact, as described in 2.2.10 above, so significant differential settlements should be avoided. The design of foundation adopted will depend critically on the quality of the underlying soils, and generalisations cannot be made, but shallow water effects and possible ground replacement should be considered where relevant. In one case toe stability on relatively poor ground was ensured by using horizontal restraining tie bars buried in the mound, and in others, additional rock was placed to bolster the toe support units.

3.2.6   Armour Stability
The general relationship between armour unit size and wave height for hydraulic stability of the armour layer has been indicated in 3.2.1 above.

Jones and Allsop (1996) reported on a series of comprehensive laboratory studies and field measurements made during this project. These studies have identified new data on the hydraulic performance of rubble slopes armoured with hollow cube armour units. Wave pressure and wave force observations made on a Cob armoured structure were similar to those made on a similar Shed armoured structure.

Maximum impact pressures measured in the field were observed on the upper limbs of the armour units and were generally equivalent to $p_{max} = 3$ to $3.5\rho g\ H_s$ (the incident significant wave height), but instances of $p_{max}=10\rho gH_s$ were occasionally observed. Pressure impacts were seen to be concentrated on the armour units located around the static water level. No relationship could be found linking wave pressures and sea steepness. Observations of pressures made on the trunk of both 2- and 3-dimensional Shed structures were similar, implying pressures are reasonably independent of incident wave direction over the ranges studied (β = 0-45° to the structure face).

Measurements were made of whole-body forces under both 2-d and 3-d wave attack. For normal wave attack, forces were shown to be independent of slope angle over the ranges studied. Variations of underlayer size (within the ranges discussed in 3.2.5 had negligible effect upon measured wave forces. Wave forces were however strongly dependent upon the relative position of the armour unit, with the greatest forces acting on armour units at or close to the static water level. Wave forces were dominated by the up-slope impact / drag component (approximately twice the in-slope component). The down-slope and out lope or lift forces were not significantly changed by unit elevation or incident wave condition, within the ranges tested.

Up-slope wave forces were smaller under oblique attack or on roundheads, except for armour units in line with the wave attack. Transverse wave forces were larger than up-slope forces for armour units positioned on the structure trunk and at the rear of the roundhead. Down-slope wave forces were independent of the armour unit's relative position and the incident wave condition (as the 2-d structures). In-slope forces experienced by armour units on the trunk and the front of the roundhead were similar in magnitude to the 2-d tests. The out-of-slope forces were greatest on the armour units positioned on the trunk and the rear of the roundhead (about twice the value of those for normal 2-d wave attack).

Wave loading analysis was completed using hodographs produced by plotting the resultant of up-slope and out- slope components for each time interval during (a typical) wave cycle. These allowed the resultant force, direction and magnitude, to be assessed at any point in a typical wave. The hodographs were generally repeatable, and both Shed and Frame units of similar porosities produced similar wave force responses. No significant difference in wave force response could be detected between different porosity Frame units (52 - 65% porosity).

The maximum wave loads were generally the up-slope slam forces as the up-rushing wave hit the (usually upper) limbs of the unit. These often exceeded the magnitude of the dry weight for a unit, but acted in a different direction. Total out-slope forces (including buoyancy) seldom reached or exceeded the dry weight of a unit. Down-slope drag forces were relatively small.

The largest wave force hodographs were produced for armour units at or close to SWL. Wave forces reduced as the measurement unit was lifted above SWL.

An increase in the incident wave height produced a corresponding increase in the forces experienced at any given armour unit. Incident wave period was seen to re-distribute the forces observed on the structure.

The failure of an armour unit at the bottom of a column of units due to the addition of run-down wave forces combining with the self weight of the units was considered in the analysis of the model test data. The tests suggested that wave run-down forces add a further 10% to the total loading of the column of armour units, which further enforces the suggestion that point loading onto mid-limb on any hollow cube armour unit should be avoided.

A simple model was developed to study the failure of an armour slope by the extraction of a loose unit from the slope. The model ignored the effects of friction and the weight of any units above the test unit. The model assumed that a unit could only be extracted when the unit's self weight was exceeded by the out-of-slope lift force. The results from this modelling were not conclusive, but it was noted that there were no extractions of armour units from any of the structures tested during the course of the project.

3.2.7   Overall Structure Stability

In analysis of conventional rubble mound structures, overall stability of the mound is seldom as important as stability of the armour against wave action. These lighter armour units may however transfer the limiting case to the (geotechnic) stability of mound or outer layers, with potential slip or sliding failure driven by wave shear forces and/or elevated phreatic pressures. The overall stability of the structure should therefore be evaluated by analysis of geotechnical stability under extreme phreatic surfaces caused by design levels of wave action.

Geotechnical failures are usually assumed to be by sliding parallel to the slope or by circular slips.   Conventional analyses use 2-dimensional slip surface calculations to determine critical centres of rotation giving minimum safety factors. It is necessary to assign appropriate geotechnical parameters to the elements of the mound and to account for wave-induced pore pressures and buoyancy. The underlayer and armour layer of a hollow cube armoured structure are thinner than for the equivalent structure armoured by rock or random concrete units, and there may thus be somewhat lower shear resistance provided by the outer slope. The relatively thin armour layer may also cause steeper phreatic surface gradients through the underlayer and core. The phreatic surfaces, or pore

pressures, may be assessed using physical modelling, or may perhaps use appropriate numerical models, is suitably validated.

A factor of safety of at least 1.25 should be the target under the selected (extreme) design wave condition.

3.2.8   Use of Physical and Numerical Modelling

Hydraulic model studies remain useful tools supporting the analysis / design of any structure subjected to direct wave action. In particular, the performance of breakwaters and seawalls remains most reliably assessed using physical model tests of a section or sections of the structure. The key objectives of such model studies may be to:

a)   Confirm the performance of the structure armouring, crown and toe details at a particular design condition;

b)   Provide information on the hydraulic performance, and stability of the main elements for a range of wave conditions and water levels, thus generating a performance map for each main response.

The choice of studying the proposed structure in either a 2-dimensional (2-d) wave flume or 3-dimensional (3-d) wave basin will be dictated by site conditions, particularly by wave obliquity. If incident waves are near normal to the structure, a 2-d study may be undertaken on the structure trunk. If incident waves are oblique and/or the outer roundhead requires detailed investigation, a 3-d model may be needed.

Physical model tests of conventional rubble mounds armoured by rock or massive concrete units are primarily used to identify the magnitude and extent of armour displacement, as well as the standard hydraulic responses of overtopping and reflection. For hollow cube armour, extraction or significant displacement of armour should seldom occur at normal probability levels. It is much more important to identify incipient motion of armour units in the more vulnerable positions, possibly supported by measurements of wavae forces on selected armour units, and pore pressures or phreatic surface excursions.

The design, operation and interpretation of such model tests is complex, and the laboratory selected will require considerable experience in the field of physical modelling to ensure that the correct quality is achieved.

Definition of wave conditions and water levels

Any design study requires well-defined wave conditions and water levels. A joint probability study may be needed to identify the most damaging combinations of wave heights and water levels. Typically up to 4 wave conditions may be considered, perhaps maximising either water level or wave condition. These conditions might relate to:

a)   a frequent condition, (say 1:1 or 1:10 year return period);

b)   an infrequent storm condition, (1:50);

c)   the design condition, (1:100);

d)   design condition + say 20% overload, to represent an extreme return period, perhaps 1:250 to 1:1000 year.

13

Data requirements

The following information will be required before the start of a model study:

a) Bathymetric survey of the area;

b) Design details of the proposed structure including grading curves of the mound, sizes and type of armour units and of the crown / toe details;

c) Wave and water level conditions at agreed locations in front of the structure, and assumptions used in their derivation;

d) Description of output parameters required, measurement and modelling methods to be used, and analysis approach.

Choice of scale

The size of the model will be set to avoid any unnecessary scale effects, and to fit the facilities available, including model armour units as well as the wave flume or basin. Owen & Briggs (1985) reviewed studies of armour stability in laboratories in the USA, Denmark, and UK, and concluded that scale effects in the flow in the primary armour on rubble breakwaters are insignificant provided that the Reynolds number, defined by the nominal armour diameter, is kept above $Re = 3 \times 10^4$. It is important therefore to note that the scale ratio itself is of little relevance in the avoidance of scale effects. Most scale effects in breakwater models may be minimised by ensuring that flow conditions are in the same regime in model and prototype, and this is usually achieved by ensuring that the test wave heights do not fall below $H_{s(model)} = 0.15m$.

The scale selected to study practical hollow cube armoured structures will typically be close to 1:20 to 1:40. The tests should use random waves.

Model Construction

The local sea bed bathymetry should be formed within the experimental facility over a representative area. Moulding accuracies should be agreed at the start of the study taking note of the accuracies to which the source data are known, equivalent to say ± 5mm (model). Calibration tests to measure waves (spectra and statistics) at the position of the model should be completed before construction of the test section. An absorbing beach at the end of the flume or around the walls of the basin will ensure that wave measurements made during calibrations are not distorted by reflections.

The model armour, crown and toe details should be scaled to reproduce the correct stability characteristics. The test fluid in the model will be fresh water, at a lower density than the prototype sea water. This variation would render model elements more stable than in the prototype if elements were simply scaled geometrically using prototype densities. The density of model concrete armour units and crown wall elements must therefore be adjusted to give correct stability, probably using a relationship based upon Hudson's formula. Close tolerances must be applied to the physical dimensions and density of model units scaled for stability.

Within the inner layers of the model, scaling corrections may be needed to ensure that viscous flow effects do not distort wave induced flows. Where appropriate, underlayer and core materials will be slightly distortion to ensure flows within those layers conforms to the correct Reynold number regime.

Tests may be conducted in a number of parts, with each consecutive part using waves of increasing severity. The most extreme condition will give an indication of the partial safety factor on the hydraulic and armour responses at the design condition. In regions of significant tidal variation, each test part may be limited to say 3 hours (prototype). Where tides are much smaller, an assessment must be made of the typical storm profile, and hence of durations at each step in wave condition. Test sections are not generally re-built between test parts.

Observations of the hydraulic performance of the structure, and any movements of elements will be recorded during each test. Measurements may be made of incident and reflected wave conditions using 3 wave probes placed about 2 wavelengths seaward of the structure, thus calculating the reflection coefficient C_r for each test condition. Water overtopping the crest of the structure may be collected in tanks behind the crest allowing mean overtopping discharges to be calculated. The number of waves overtopping the crest may also be determined.

Movement of armour or any other element may be quantified using overlay photographs taken from fixed camera positions before and after each test part. Successive prints are compared to identify displacements. Force measuring devices may be attached to individual armour units to quantify whole body loads.

3.3 Structural Strength and Integrity

3.3.1 General points

Design of concrete armour units for rubble breakwaters up to the late 1970s paid relatively little attention to loads / stresses in the armour units. One of the major activities of this research club was to quantify where possible all major sources of load / stress in the armour units. A detailed fault tree for cracking or breakage of single layer armour units is given in Appendix A to act as a guide to the design of units for structural integrity.

Research studies have confirmed that the applied loading and support conditions on the highly redundant units have a marked effect on the stresses, but reasonable estimates of these stresses can be made by the methods suggested in these guidelines for the major loads considered.

Much of the research has been accomplished by applying measured loads from field and model studies to finite element stress models of the units and so to determine stress distributions. Influence lines have been prepared for simplified load and support cases which are likely to be representative of conditions to be found in practice.

The stresses for the various loadings considered in design and described below are based on these studies, referring to the critical condition of tensile stress in the concrete of the unit.

For a more detailed study of unit design under specific loading conditions, a finite element model can be constructed for stresses within the range prior to cracking. The linear model used would not apply to any redistribution of stress after cracking.

Loads applied to the armour units during normal construction operations have been found to be relatively small, and although these may result in some locked-in stresses, these have not been taken to be of any significance at the level of calculation used.

Investigations have been made into the effects of repeated loads and rate of loading on plain concrete in tension, and these appear to be of relatively minor importance in the units considered here, although the studies have indicated that fatigue may have explained a few of the instances of cracked units found in site inspections.

Air entrainment has been studied in the laboratory and in the field, including scaling effects between fresh water models and sea water prototype. Results of studies by Howarth et al (1996) and Allsop et al (1996) indicate that pulsating wave loads generally scale directly, but that short duration wave impact pressures or forces measured in fresh water tests in hydraulic model studies may be over-estimated by around 40%.

Research carried out by SLAC has been extensive, but it must be accepted that there remain many uncertainties. Because of the uncertainties of individual forms of loading on a particular unit, and consequently the inexactitude in considering stresses due to combined loadings, it is acknowledged that calculations cannot be precise.

3.3.2   Dead Loads

The dead loads imposed on an individual unit are p'rincipally those due to the weight of units transferred from further up the slope. These can be calculated on the simple assumption that there is no friction between the underlayer and the armour layer, which is justified because the action of this friction to partly support the units cannot be guaranteed under wave action. Thus the weight force on a unit is proportional to the vertical height of the column of units above it.

The stresses in a unit caused by self weight loads depend critically on the load and reaction conditions. Ideally if the units are cast with perfectly flat faces and are stacked accurately in line, such forces would be distributed evenly over the faces. In practice, some point load contacts are likely.

A number of load cases were analysed during the research, applied to an unrestrained column of units on a frictionless slope. The load cases and the principal tensile stresses for each case considered for a Shed unit are shown in Figure 3.5 for the location A (the inside of a mid upper limb) and in Figure 3.6 for the location B (the outside of a limb perpendicular to the slope). Also indicated are the total loads in each case and the corresponding stresses which would occur for a column of units 10m high.

The most severe stress is caused inside the centre of an upper limb by the whole weight load applied there. All other stresses are significantly lower and would not cause cracking if the ultimate tensile strength (UTS) of the concrete equals or exceeds $3N/mm^2$, a typical value. In the most severe case, a point load on the lowest unit could be tolerated for a slope height of only about 3m. If a higher slope is needed, and cracking of units must be avoided, then point load conditions should be avoided to reduce the probability of cracking.

Similar computations were done for a Cob unit which has a thicker section at mid limb positions, and the highest tensile stress was about $5.7 \ N/mm^2$, also at mid limb, compared with $9.8 \ N/mm^2$ for the Shed. In this case the limiting slope height could be about 5m if cracking were to be avoided under the most severe loading case.

SR 482 24/01/97

For other load conditions the maximum stress in the Cob units could occur inside the corners of the unit, whereas for Shed units the maximum was always at mid limb.

Influence lines for components of stress at two apparently critical locations are shown in Figures 3.7 to 3.10 for both Shed and Cob from which the designer can assess the likely self weight tensile stresses for the load conditions assumed.

3.3.3   Settlement

Any settlement of the mound would be expected to modify the inter-unit loads in the armour layer. In practice, there has been no evidence of units cracking due to settlement of the mound beneath the armour units. The use of the extreme non-friction case in 3.3.2 above suggests that there is no need to include a further element of loading due to this cause of settlement modifying the support of the unit.

There is however a particular loading case that should be avoided. This is where a heavy crown wall element, which could itself be subject to settlement, bears directly onto a column of armour units. If close contact is required, for instance to limit the possibility of units being lifted under severe up-rush, it will be important to include a compressible element, such as small sections of fender to limit load transfer.

3.3.4   Live Loads

The principal live loads are those due to wave action. Wave action in/on the slope cause lift and drag tending to move the units (see 3.2.6 above), but on their own these cause relatively small stresses. The largest wavae-induced stress are those arising from direct wave slam on the face of the unit most directly exposed to the advancing wave front.

Wave slam pressures were measured in the field on a Cob armoured structure, and the laboratory on Cobs and Sheds. The pressure impacts were concentrated on the units around still water level and typical pressure heads recorded were $3.5\text{-}4H_s$ occasionally as high as $10H_s$. Wave slam forces may therefore depend on the wave heights, although sea steepness and incident wave direction appeared to have no significant influence on pressures, and pressures were similar on Cob and Shed armoured structures.

It may be assumed in design that the wave impact pressure acts only on the top horizontal limb as a uniform force over the limb length acting perpendicular to the limb. The wave slam related to the central cross sections of a Shed, Cob or Frame unit is demonstrated in Figure 3.11, from Belhadj (1993). The maximum pressure is considered to act with a constant value over one side of the limb.

3.3.5   Temperature Stresses

Stresses arise in hollow cube armour units due to exposure to solar radiation in service. The evaluation of these stresses is complex and for a given situation a large finite element model is required to encompass the diurnal change in radiation exposure, the changes / gradients of temperature, and the development of stresses in the unit. This needs to take account of the exposure or shading of parts of the unit and the development of non-uniform temperature distributions in the structure over time.

Such stresses will also need to be combined with dead load stresses due to self weight, so that a different situation will apply for each unit on the breakwater slope.

A further instance requiring analysis is for units located between high and low water levels, where solar heating of an exposed unit is followed by water cooling of the same unit, causing a further redistribution of stresses.

The variation of the day/night solar radiation cycle, the tide cycle and seasonal effects will produce a complex stress history in a unit and make simplification of analysis essential. Even so the modelling remains time consuming and expensive.

During the initial research studies a linear elastic stress model of solar radiation was made of a 2 tonne Shed unit with heat applied uniformly to the exposed top face of the unit only, with no variation in sun direction. Stresses were evaluated on a quarter of the unit for the assumed symmetrical case, in both unrestrained and restrained conditions, and the effect of varying unit size was studied.

Somewhat higher stresses were found in the restrained case, and the importance of the assumptions on the location of the restraining force was evident. This restraint, being due to the self weight of units, is referred to in 3.3.2 above.

There was very little change in stresses for change in unit size to 1 tonne or 4 tonne.

During the detailed analysis of combined stresses, referred to in 3.3.7 below, a comparison was made between the thermally induced stresses computed by the simplified method and those computed using a more detailed general method. The general method allowed for the movement of the sun and the effects of shading or exposure of different parts of the unit, utilising a time-step approach. This method required more complex modelling of the whole unit, and the results showed very little difference in stresses from the simplified approach, which is therefore considered satisfactory for design purposes.

3.3.6   Casting, Handling and Placing Loads

Many potential damaging situations may arise during the construction phase. Armour units are generally cast in steel moulds often some distance from the breakwater in which they are to be placed. A number of load cases arise during handling which might therefore cause damage to the immature concrete units. These include dynamic loads which occur when the moulds are stripped and when the units are transported and placed.

A field study was carried out to determine whether these various dynamic loadings were of significant magnitude. Tests were conducted on six separate units during the de-moulding operation approximately 24 hours after casting, and also during transport of the units by truck some weeks later. Similitude tests were then conducted on a full scale unit in the laboratory, using hammers of different hardness to simulate different types of impact. From these tests and the corresponding analyses it was concluded that the accelerations / impacts measured during the field trials were not of sufficient magnitude to cause damage to the armour units.

3.3.7 Combined Stresses

In considering combined stresses it can be reasonably assumed that the self weight dead load is constant, and that the important applied loadings of wave slam and thermal effects of solar radiation do not occur together. This is a simplification justified by the fact that wind that neccessarily accompanies major wave action will itself have a cooling effect on heat caused by solar radiation, even if not obscured by clouds, so that the predicted maximum effects are mutually exclusive.

Bearing in mind the simplification which must be made in the overall analysis, the study of combined stresses was made by using the idealised frame units with square section limbs, the thickness of which could be varied to alter unit porosity. It was found by Belhadj & Waldron (1993) that increased limb thickness caused a significant reduction in stresses due to self weight and wave slam, but caused no reduction to the thermally induced stresses.

The stress analysis was based on an existing Cob armoured Breakwater for which records of cracking had been kept so that conclusions might be drawn on the possible reasons for cracking.

The breakwater had 19 rows of units in a high tidal range, with moderate wave action and solar exposure in N Europe. The general geometry and damage recorded are shown in Fig 3.12.

The combined load cases were computed using different assumptions of self weight load and reaction applications, and different support conditions from the underlayer. Typical combinations evaluated are shown in Fig 3.13.

Maximum tensile stresses for typical load cases for each unit on the slope and selected combinations are plotted on Fig 3.14.

It can be seen, referring also to 3.3.2 above, that self weight stresses are negligible providing corner loading only is applied. Wave slam stresses are relatively small, and combined with the corner loading of self weight also produce negligible stresses.

With thermal effects, however, stresses are higher. Combined with self weight with corner loading only, the stresses over the whole slope are near or above the tensile strength of the concrete. The upper units have a longer solar exposure and higher temperature stresses, in contrast to the self weight stress which increases down the slope. There is thus little difference in the maximum stress levels over the whole slope for the load combination calculated.

The distribution of the slightly smaller maximum cooling stresses is more concentrated in the centre of the slope, i.e within the tidal range.

These more detailed evaluations confirm that it is likely that the load and support conditions between units up and down the slope, which would result in self weight stresses, are the most important factor in this element of the design. For a slope of more than limited height it is recommended that serious consideration is given to avoiding any mid-limb contact, even though the construction may be more difficult, by possibly forming projections at the four down slope corners of each unit to ensure corner bearing only.

It is apparent also that thermal effects are a major cause of higher stresses, and that the effects cannot be avoided by thickening the unit limbs.

The importance of thermal stresses also appeared to be relevant in studies of another breakwater in the Mediterranean, although resources did not permit a full study of this project. The slope was about half the height of the previous case, tide range is negligible, wave height moderate, but solar exposure would have been considerably higher than in the N European location. Percentages of cracked units, in this case Sheds, were significantly higher than in the previous case, but located only in rows above water level.

4 Specifications

4.1 General

The specification of materials and workmanship for a rock mound armoured with hollow cube armour units should include all those items usual for such a structure, so these are not included here. Considerable advice on rock quality, particle shape and grading, and on placement, are given in the CIRIA rock manual (1991), with appropriate testing procedures.

The following sections may be found useful in preparing specifications for particular aspects of hollow cube armoured mounds.

4.2 Manufacture of Units

Units must be cast in strongly made moulds so that the final surface is smooth and within close limits of the theoretical dimensions. The designer should define the acceptable tolerances, particularly considering the avoidance of convex outer limbs to limit the possibility of point contacts at mid-limb.

Concrete quality should be designed and tested with particular reference to tensile strength, and routine testing of materials and finished concrete should be defined in accordance with recognised standard procedures. Normally concrete should be of compressive strength 30 or $40N/mm^2$, with an evaluated relationship to its tensile strength depending on the materials used, which should be monitored throughout the manufacturing process.

Where hoop reinforcement is used, steel should be epoxy coated or stainless steel, made by an approved manufacturer as finished cages to the required dimensions. The fixing of reinforcement before casting should follow normal standards of workmanship for pre-cast reinforced concrete.

Good control of placing and compacting concrete is needed, and water / cement ratio and workability should be designed appropriately and monitored. The requirements of temperature should be considered, and where needed shading of moulds should be specified.

Depending on the design of the unit, appropriate parts of the moulds should be released as soon as tests show that it is safe to do so, to minimise stresses during curing and shrinkage. Similar trials should be carried out to determine the increase in concrete strength before units are lifted from the base plates of the moulds. No unit should be transported or placed until the concrete has reached a specified strength. The manner of lifting and transporting the units should be defined to minimise stresses in the concrete.

Curing should commence as soon as the upper mould elements have been removed, as specified to suit the ambient conditions.

4.3 Placing of Units

Units should be placed in close contact in a defined continuous sequence from the toe beam up the slope and working in one direction along the slope. The core profile and underlayer profile should be sequentially checked to be within defined tolerances before unit placing commences. Before re-starting placing after an interval the previously placed units should be inspected and any movements corrected, so that the layer remains closely placed.

Tolerances of units should be checked by line and level after placing. The values of tolerances will depend on practical site considerations and also include the size of the unit used. As a guide, a placing tolerance of 25mm perpendicular to the armour plane for 2 tonne units would be considered suitable, defined as the level difference between adjacent units. At the same time the acceptable variation in the armour layer profile over a longer distance, say 100m, should be defined, for which 40mm would be appropriate for 2 tonne units.

As the accuracy of placing the armour units is greatly influenced by the line of the toe beam, the manner of forming the toe beam requires careful consideration in achieving suitable tolerances. A toe beam above or below water may be precast or cast in-situ, and an appropriate specification should be drawn up to suit the particular case to ensure satisfactory tolerances to receive the lower row of the hollow cube armour units.

5   Construction Aspects

As with all structures, and particularly with marine structures, construction aspects require consideration during design. The access for construction equipment, the environmental conditions during which construction and checking can proceed and the vulnerability of the partly completed structure to damage are primary factors to be studied. These factors are discussed in BS 6349:Part7 (1991) and in the CIRIA rock manual (1991).

A further consideration involves the potential settlement of the mound and underlying ground during construction, which may influence the sequence and time scale of carrying out the work.

It is evident that no generalisations can be made for such site specific factors, but the quality of the work to be achieved on completion requires careful evaluation of whether it can be practically and economically accomplished.

A simple rock mound can be regarded as reasonably flexible, to adapt to settlement and wave induced movements, which can be restored to a certain extent by adding further rock. This is only partly true for a mound armoured with randomly placed artificial units, but is not so for a structure with a closely formed armour layer such as hollow cube units. For this reason the design should permit construction to a final state which does not require substantial correction to deal with the effects of waves or ground deformation during the design life

6 Acknowledgements

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SR 482 24/01/97

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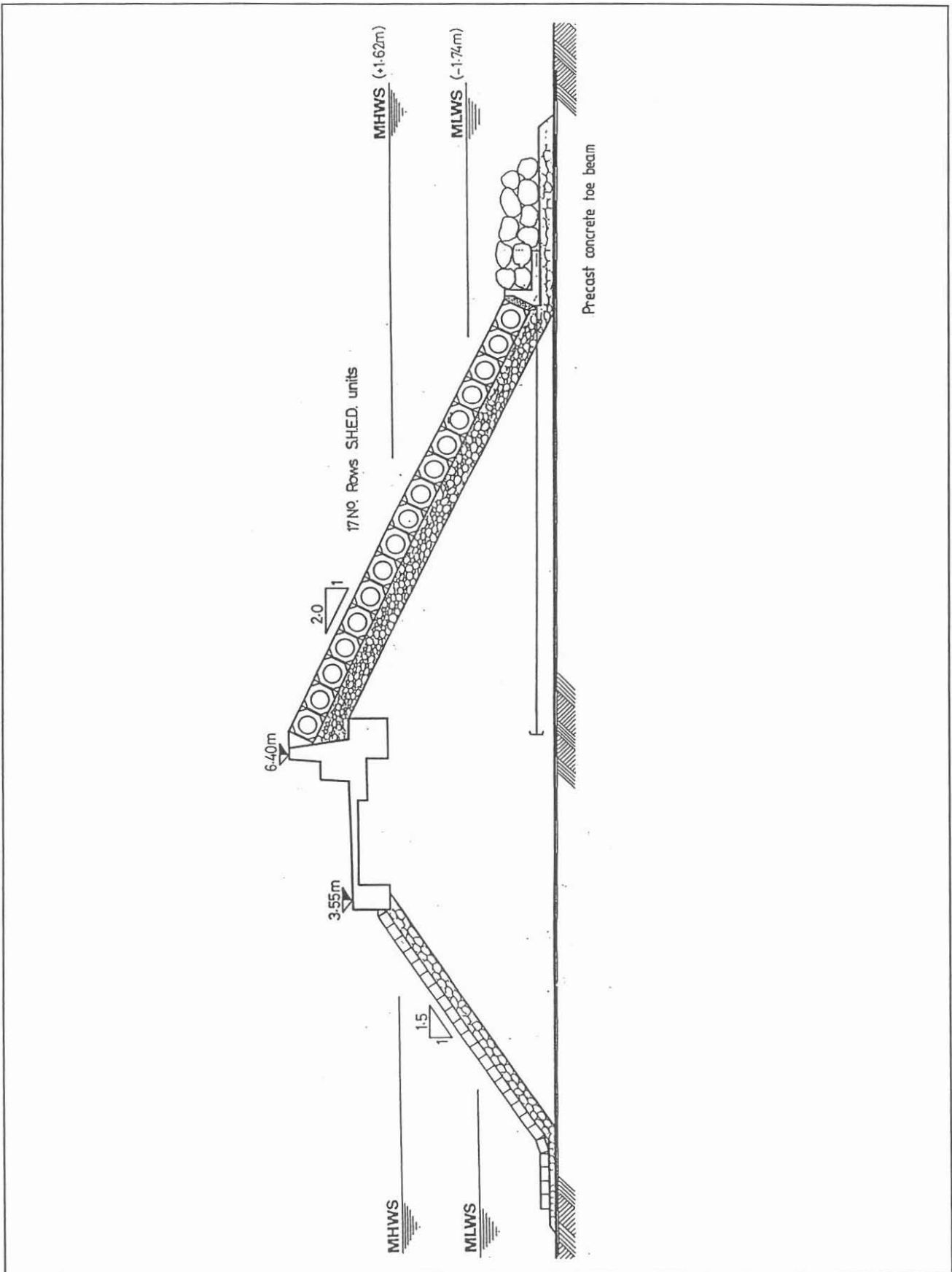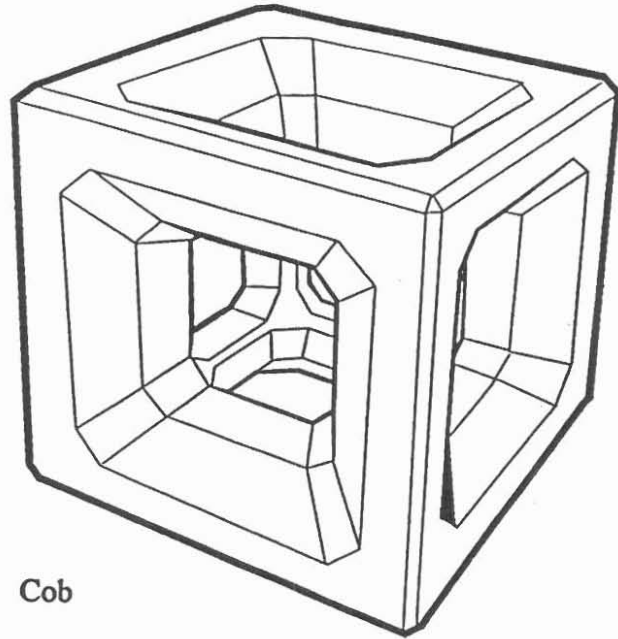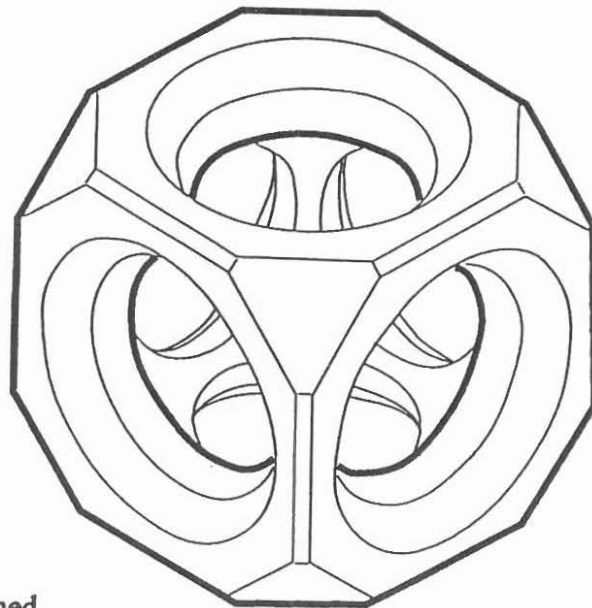
Figures

Figure 1.1 Typical section through breakwater
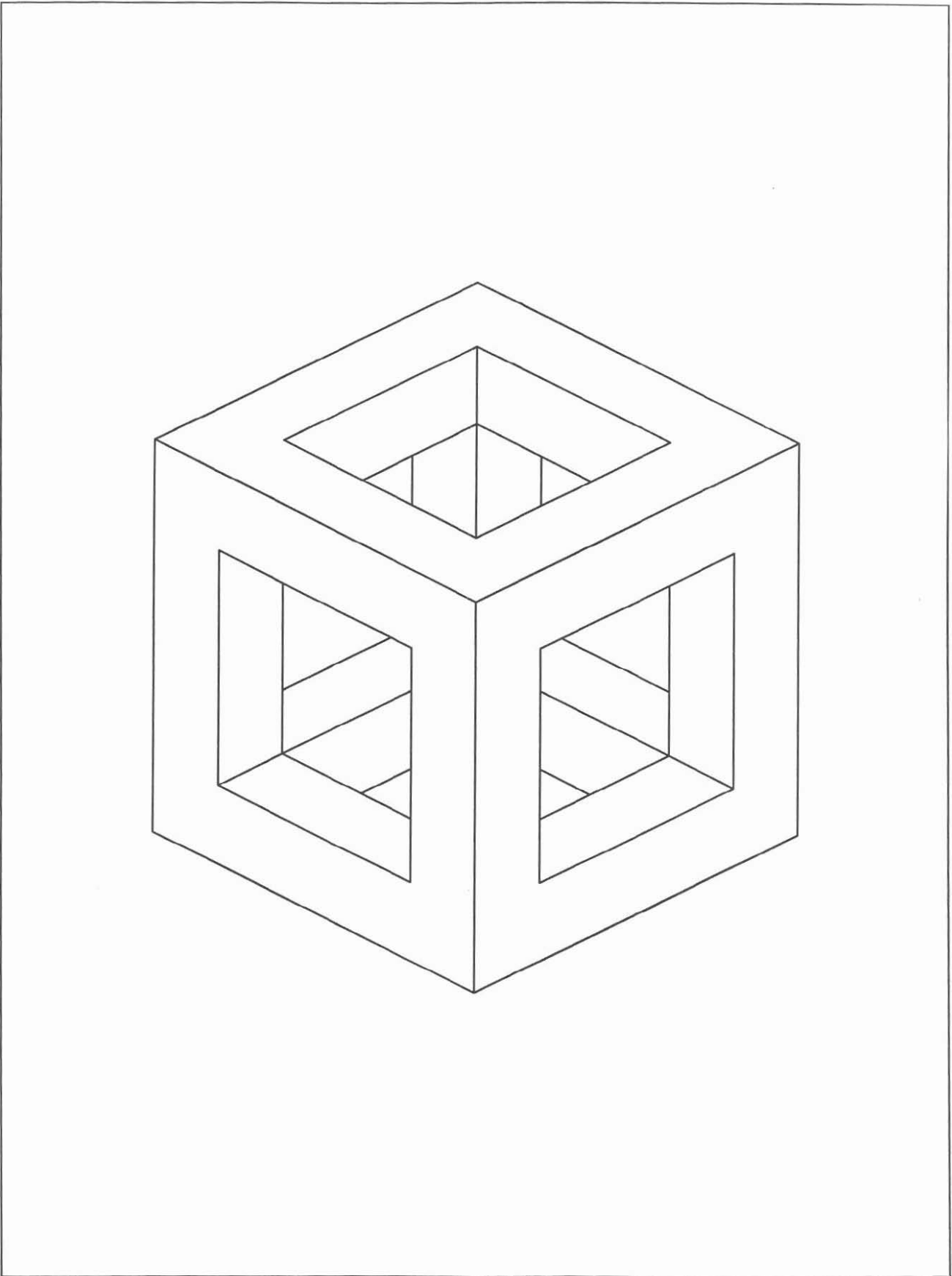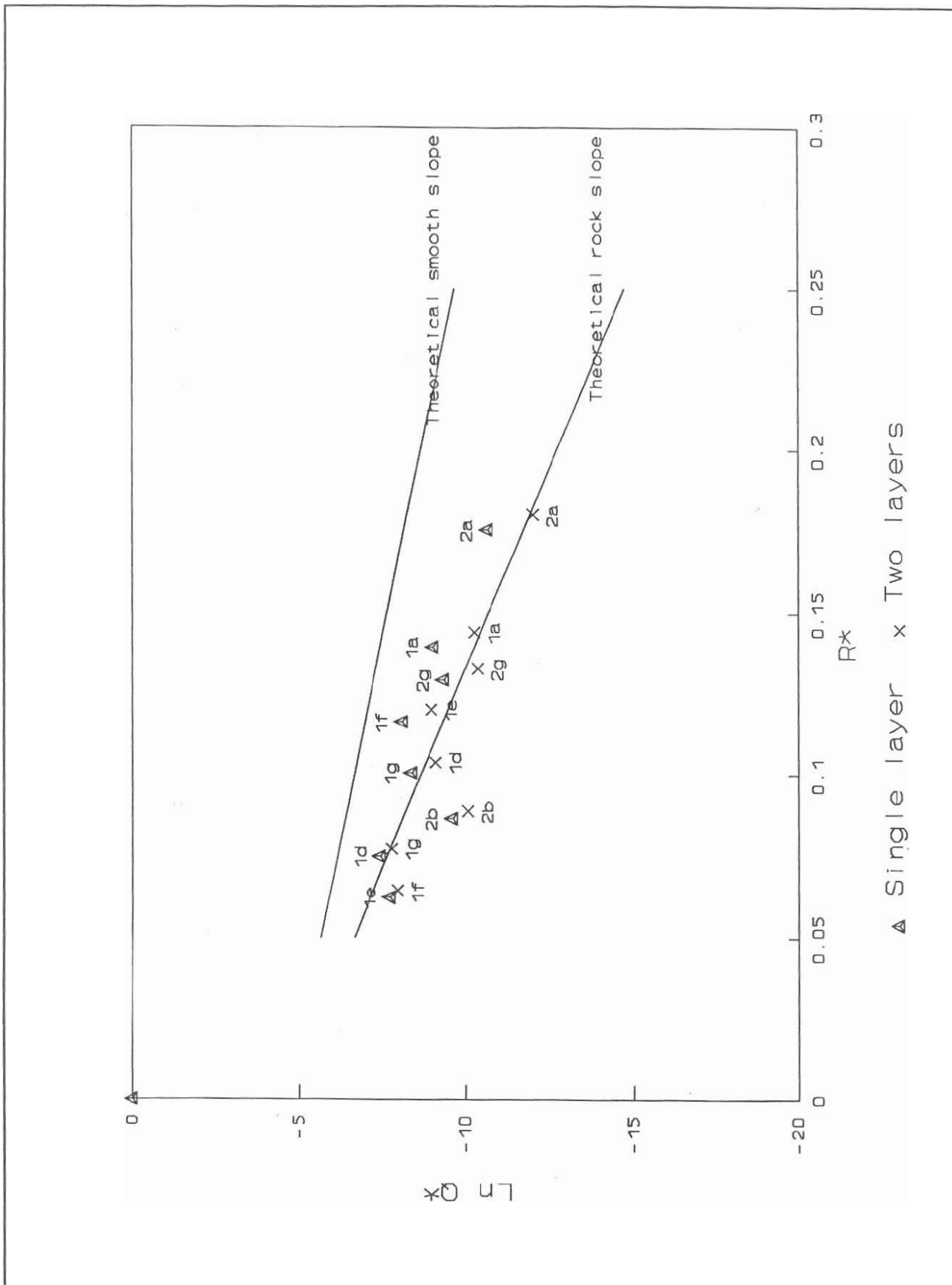
Cob

Shed

Figure 1.2 Cob and Shed units

Figure 1.3 Frame unit

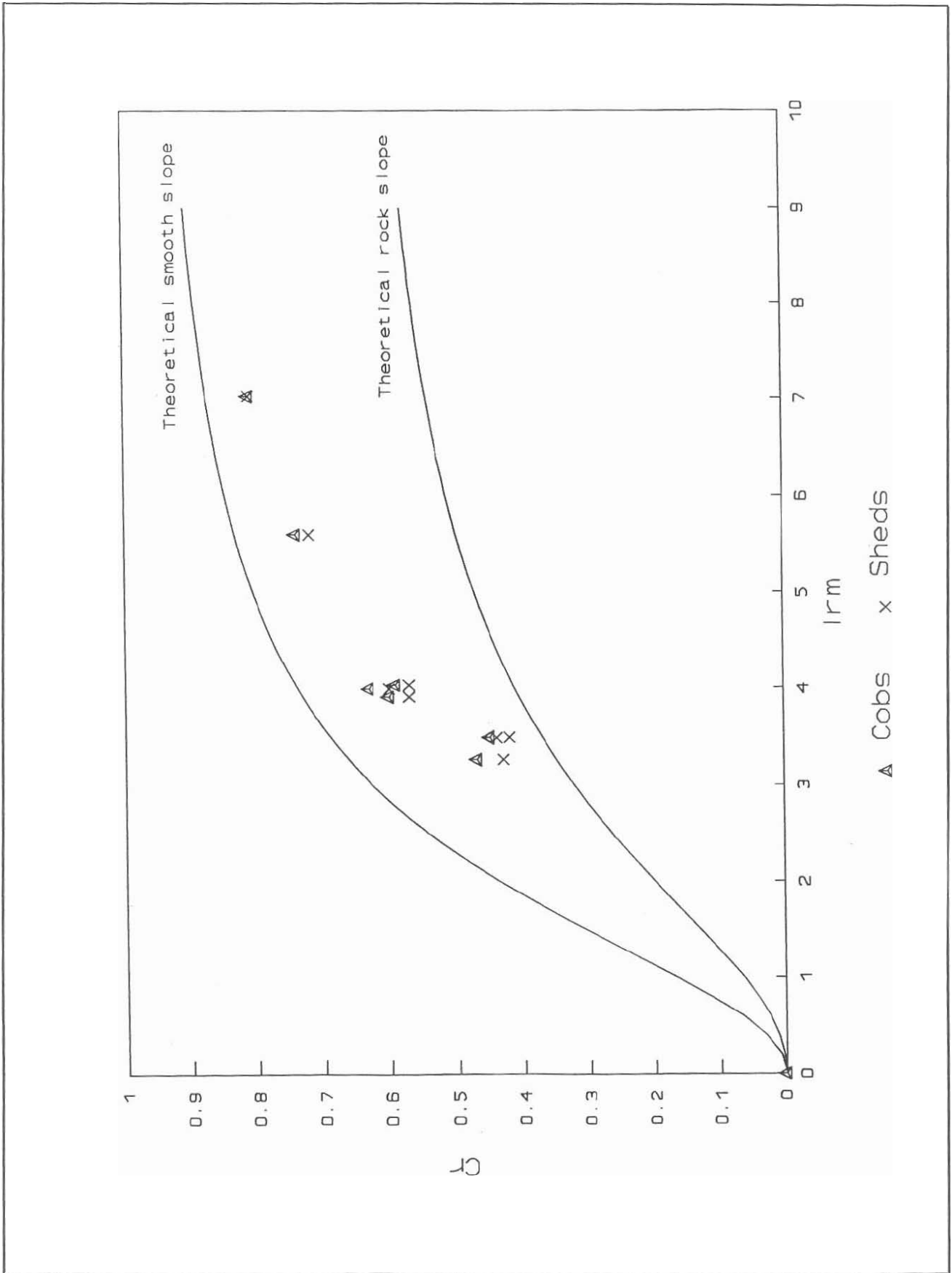**Figure 3.1 Comparison of overtopping results for one and two layers of Frame units with 60% porosity; slope 1:1.33**

Figure 3.2 Comparison of reflection results for Cob and Shed units under similar physical and environmental conditions
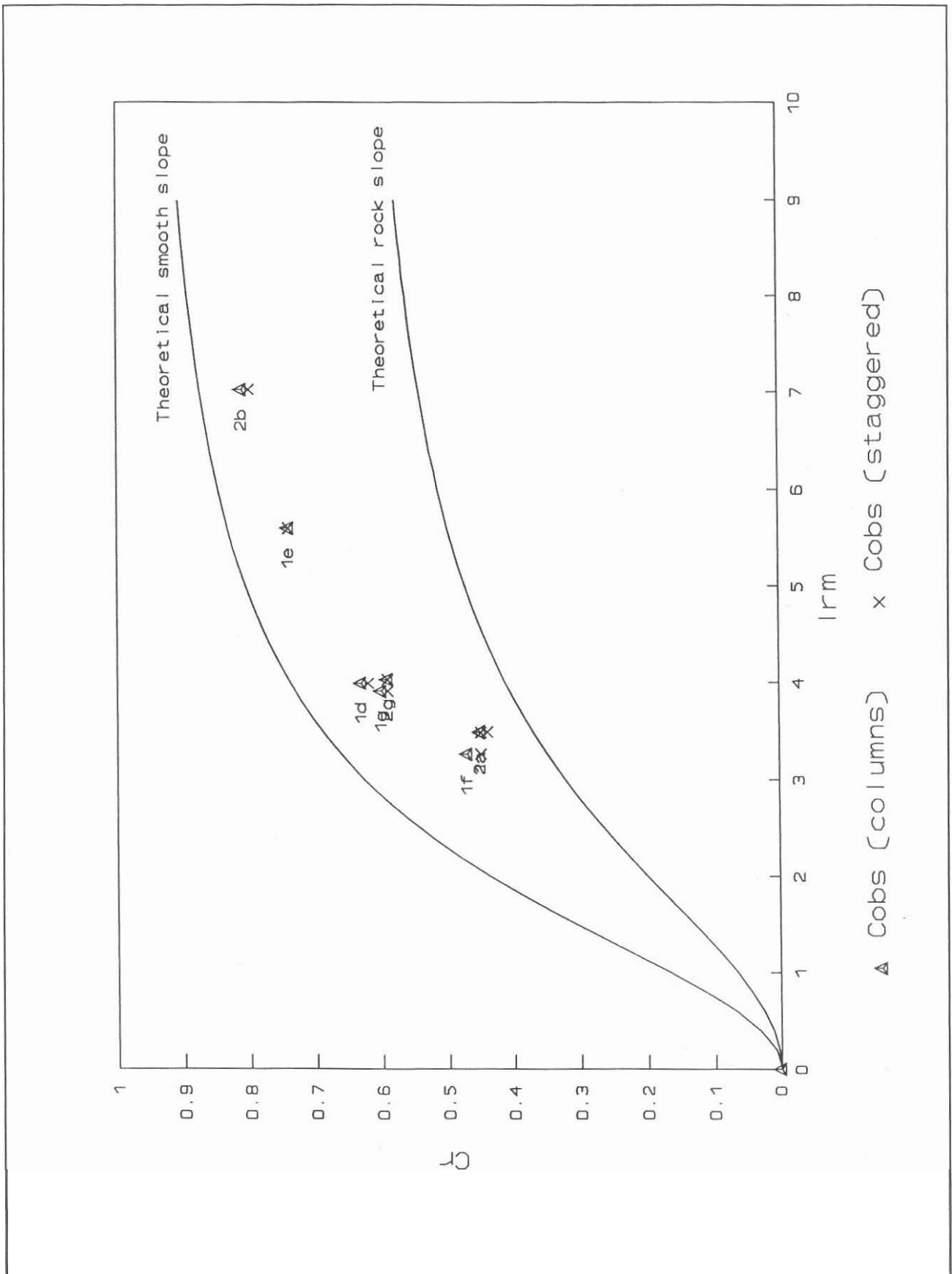
Figure 3.3 Comparison of reflection results for Cob units placed in columns and staggered formation
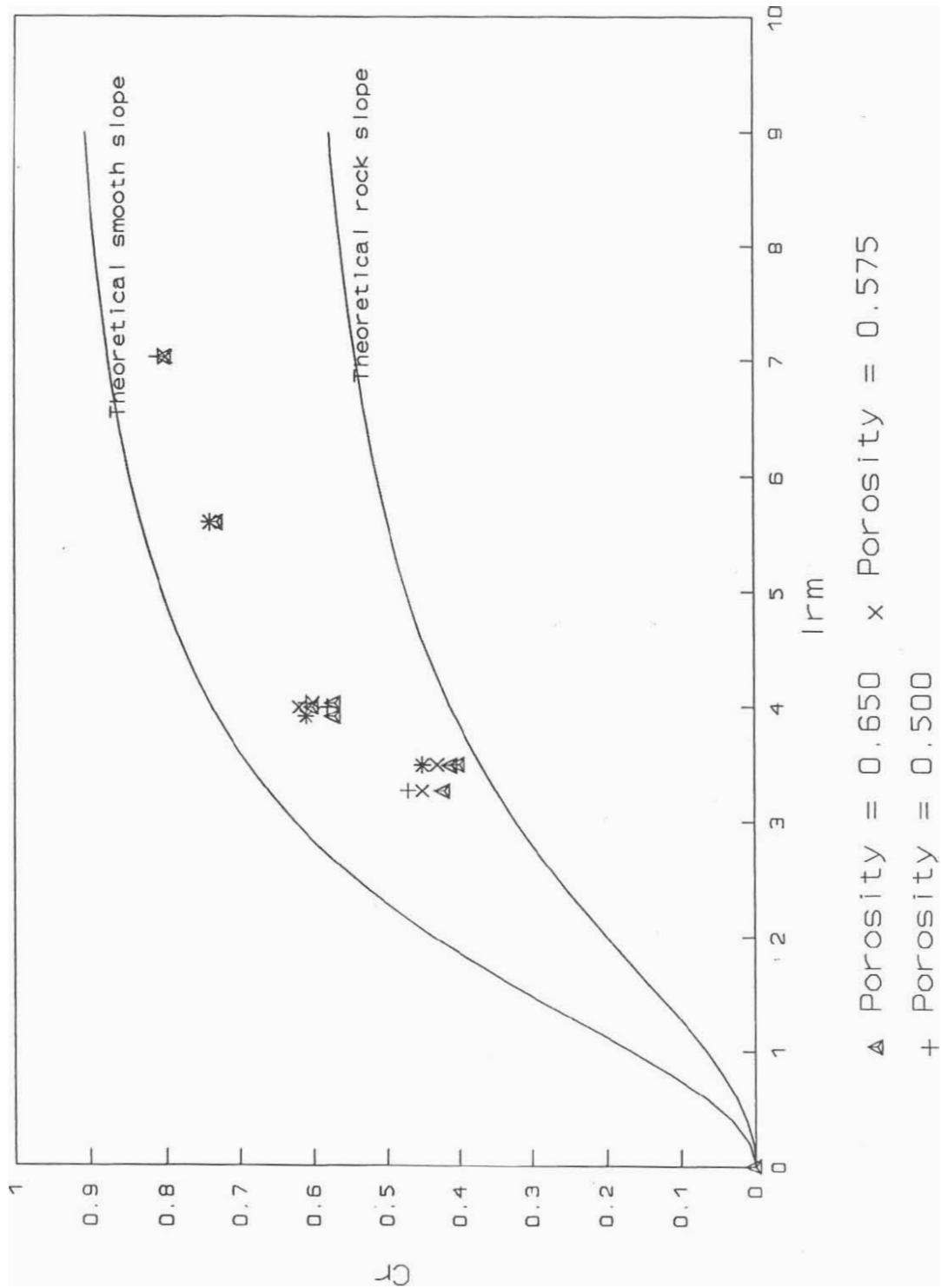
Figure 3.4 Comparison of reflection results for Frame units of differing porosity

	Stress At (A) N/mm^2	Total Load kN	Stress Per 150kN
a)	$0.023 + 0.0001 + 0.023 + 0.01 = 0.048$	40	$0.18\ N/mm^2$
b)	$0.15 + 0.01 + 0.15 + 0.01 = 0.32$	40	$1.2\ N/mm^2$
c)	$0.15 + 0.15 = 0.3$	20	$2.25\ N/mm^2$
d)	$0.65 - 0.13 + 0.07 - 0.13 = 0.46$	40	$1.73\ N/mm^2$
e)	0.65	10	$9.75\ N/mm^2$
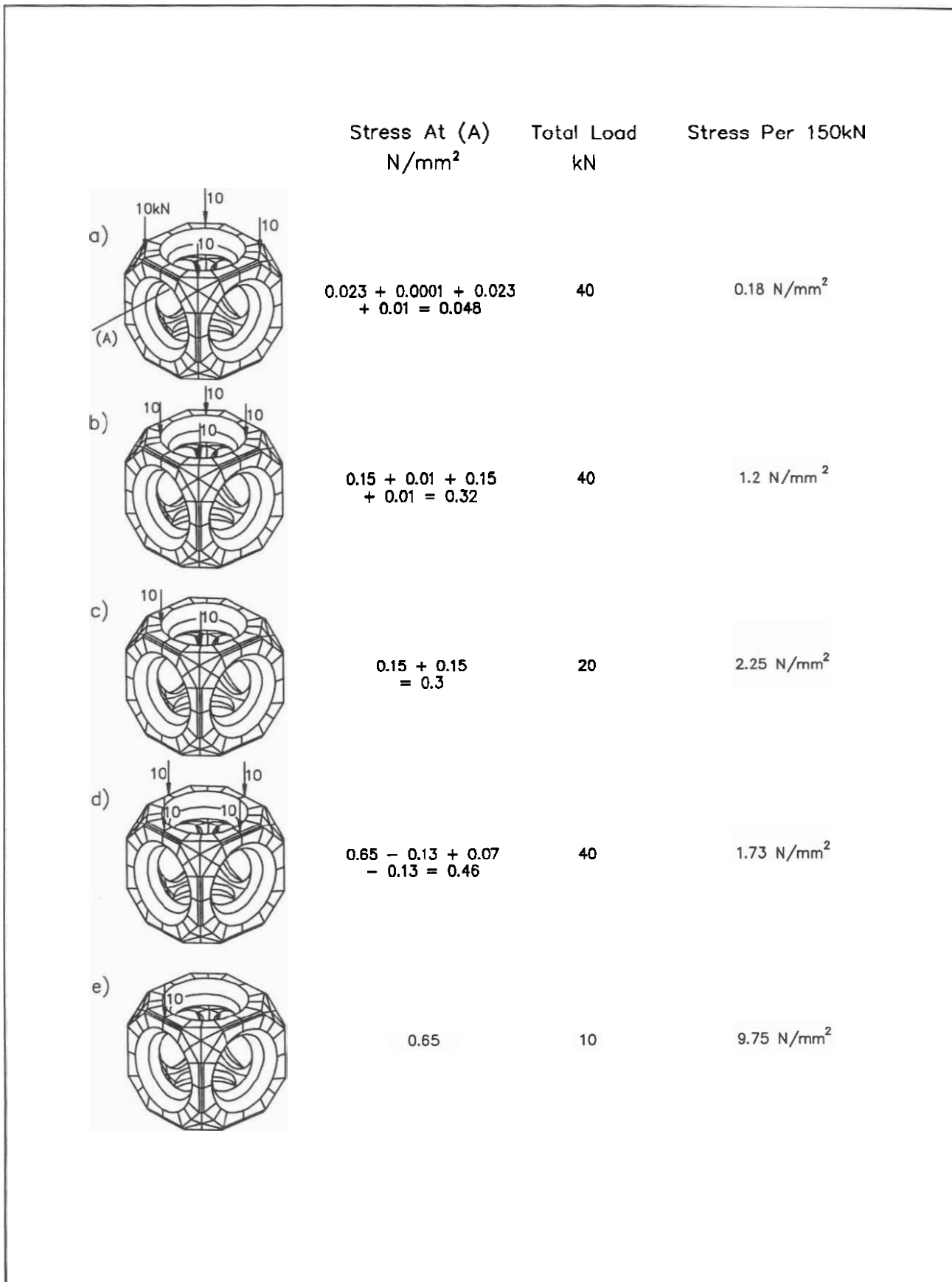
Figure 3.5 Load combinations for point (A)

	Stress At (B) N/mm²	Total Load kN	Stress Per 150kN
a)	$-0.055 + 0.024 + 0.005$ $+ 0.024 = -0.002$	40	0.01 N/mm²
b)	$0.125 + 0.064 + 0.019$ $+ 0.064 = 0.272$	40	1.02 N/mm²
c)	$0.10 + 0.006 + 0.1$ $+ 0.006 = 0.2$	40	0.75 N/mm²
d)	$0.1 + 0.1 = 0.2$	20	1.5 N/mm²
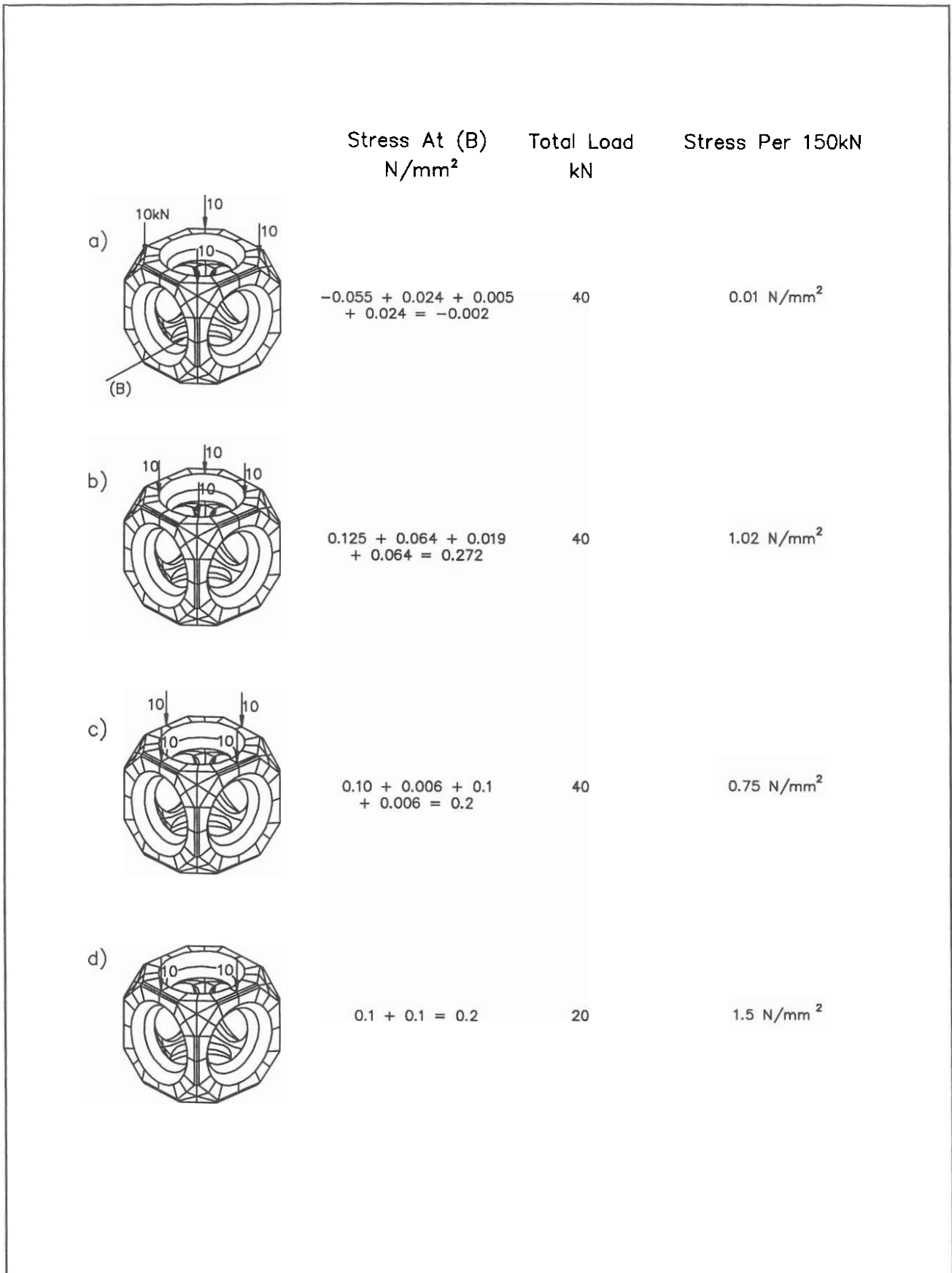
Figure 3.6 Load combinations for point (B)

Stress Per 10kN
(N/mm²)

- 0.5
- 0.4
- 0.3
- 0.2
- 0.1
- 0.0

Point (A)
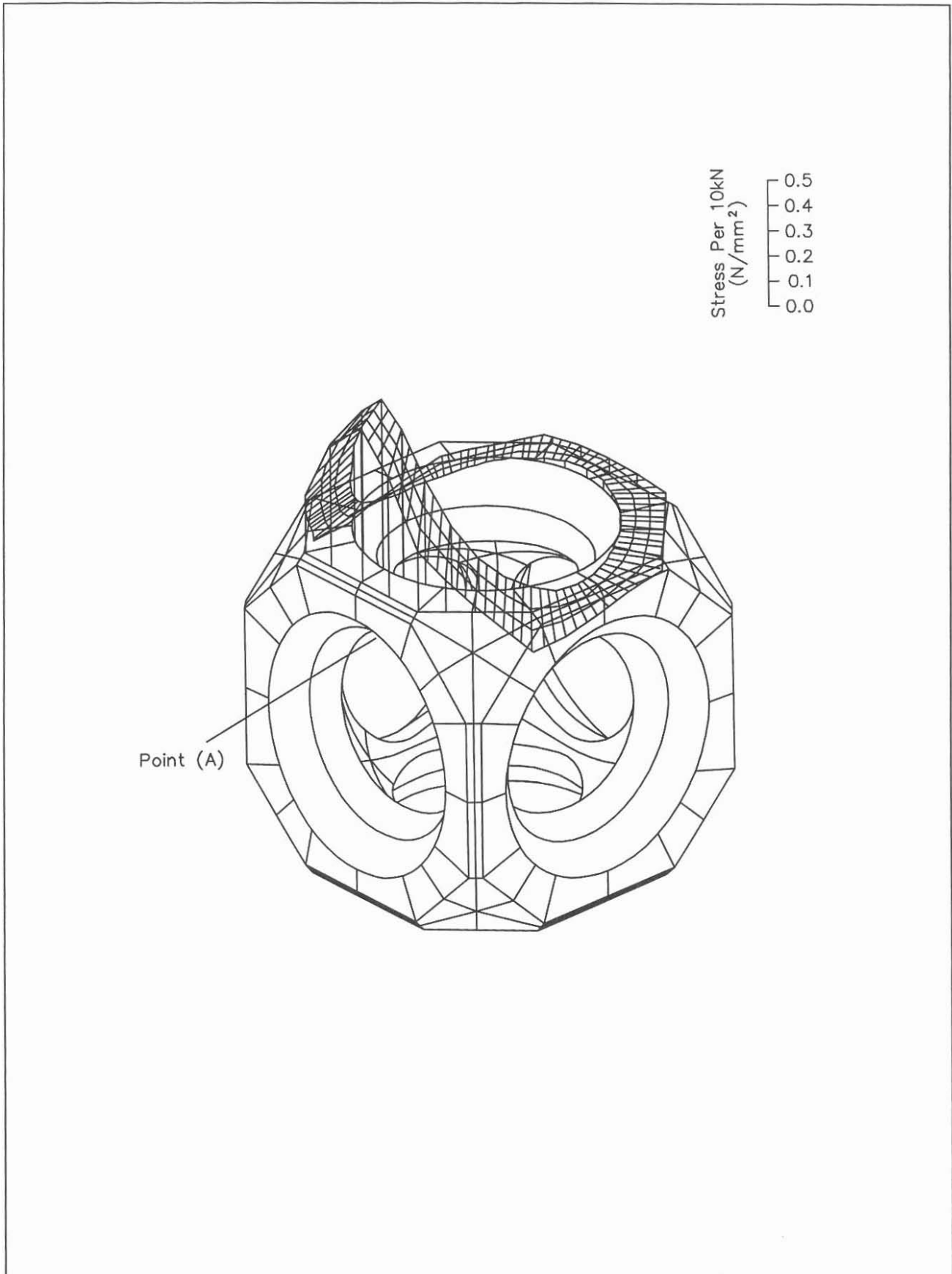
Figure 3.7 Influence surface for horizontal component of stress at (A), Shed unit

Stress Per 10kN
(N/mm²)

- 0.5
- 0.4
- 0.3
- 0.2
- 0.1
- 0.0

Point (B)
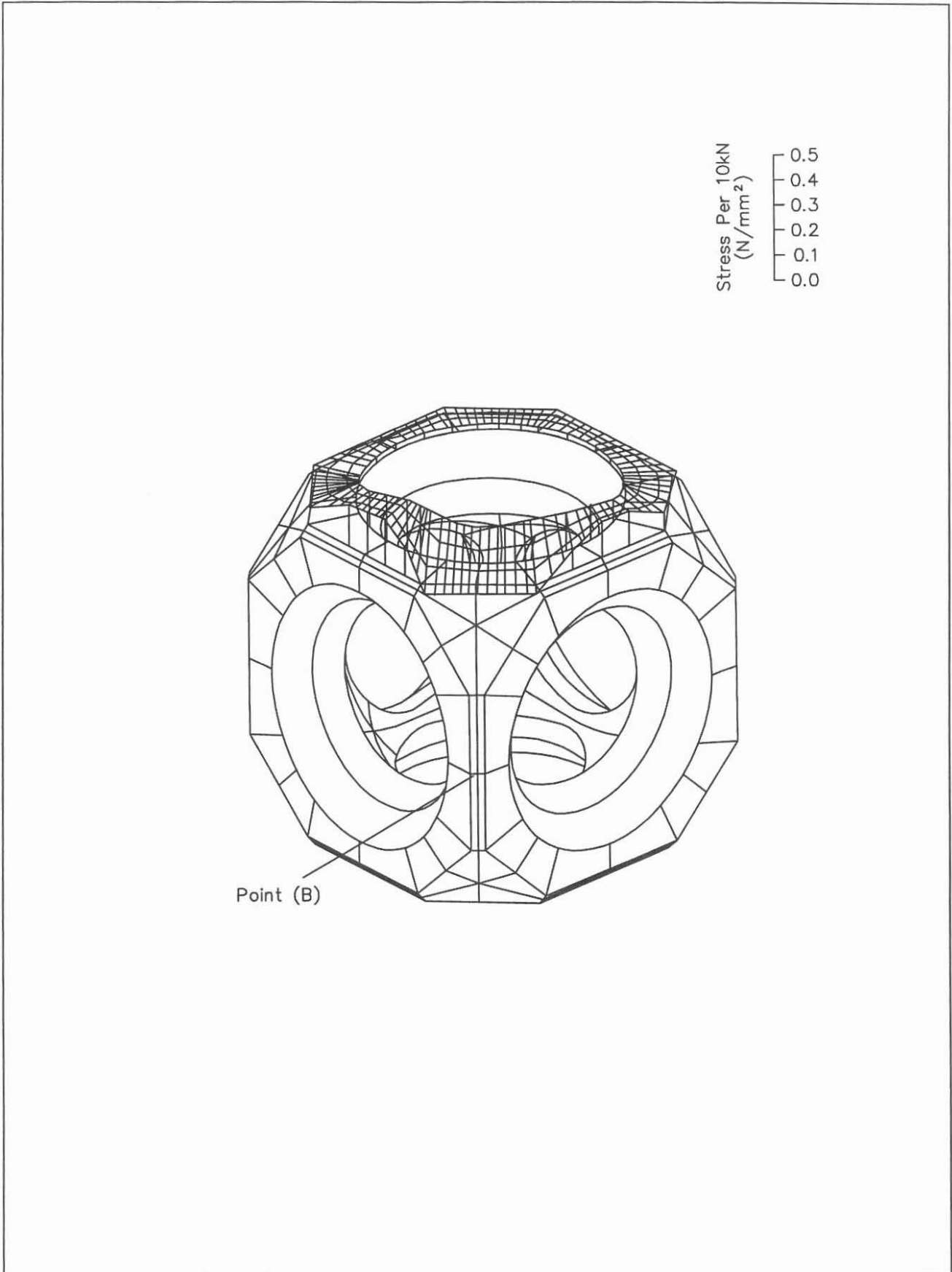
Figure 3.8 Influence surface for horizontal component of stress at (B), Shed unit
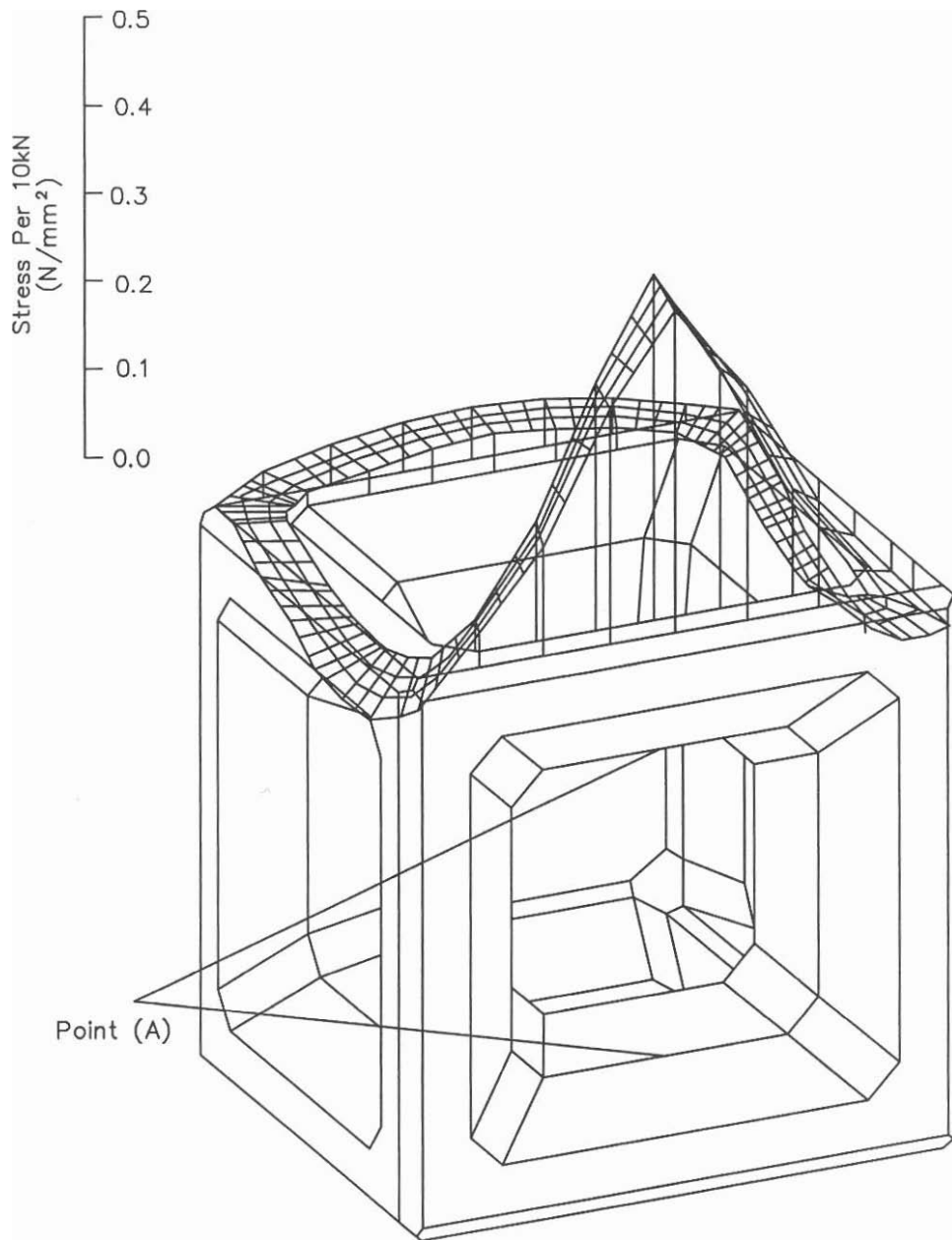
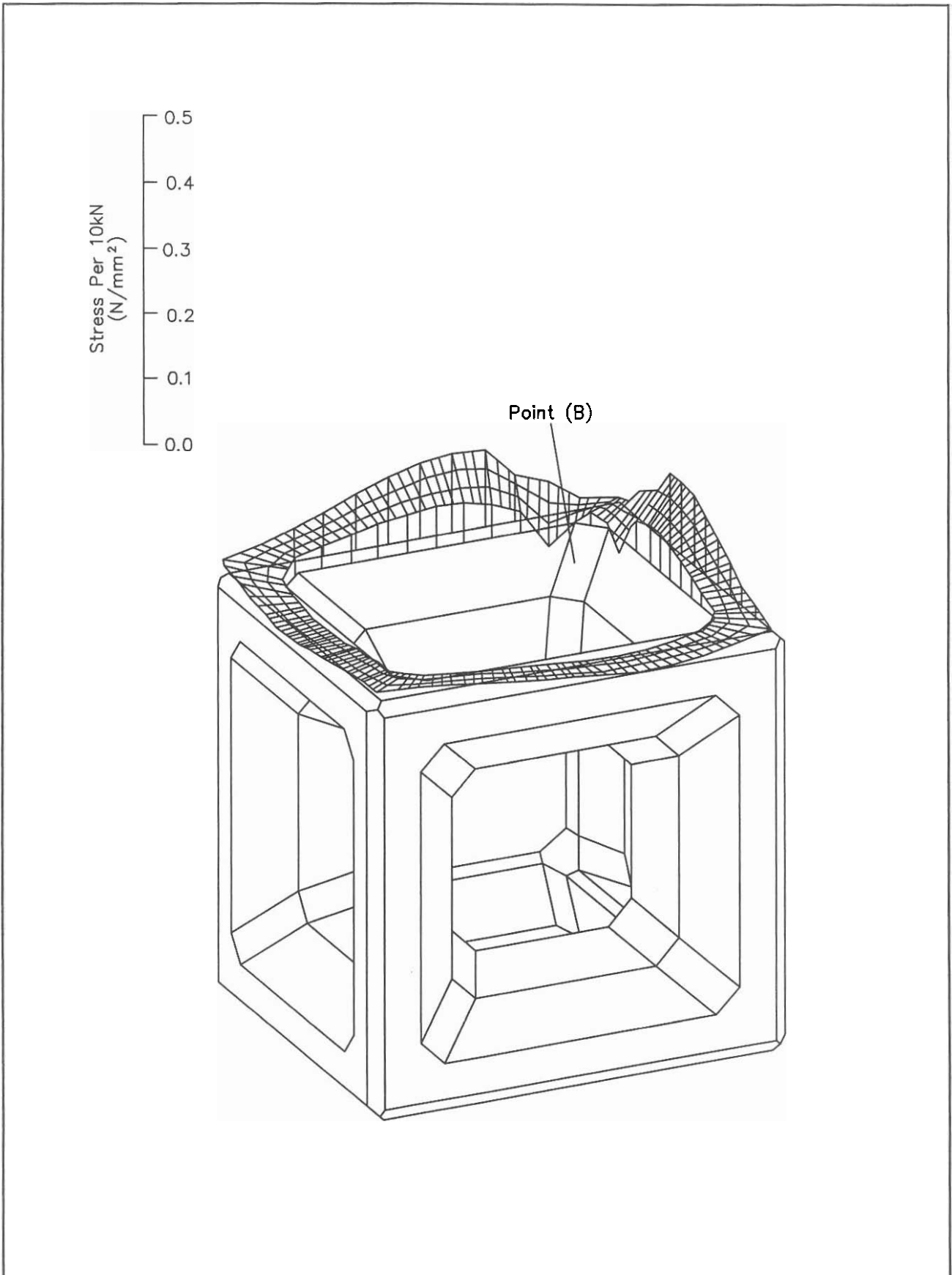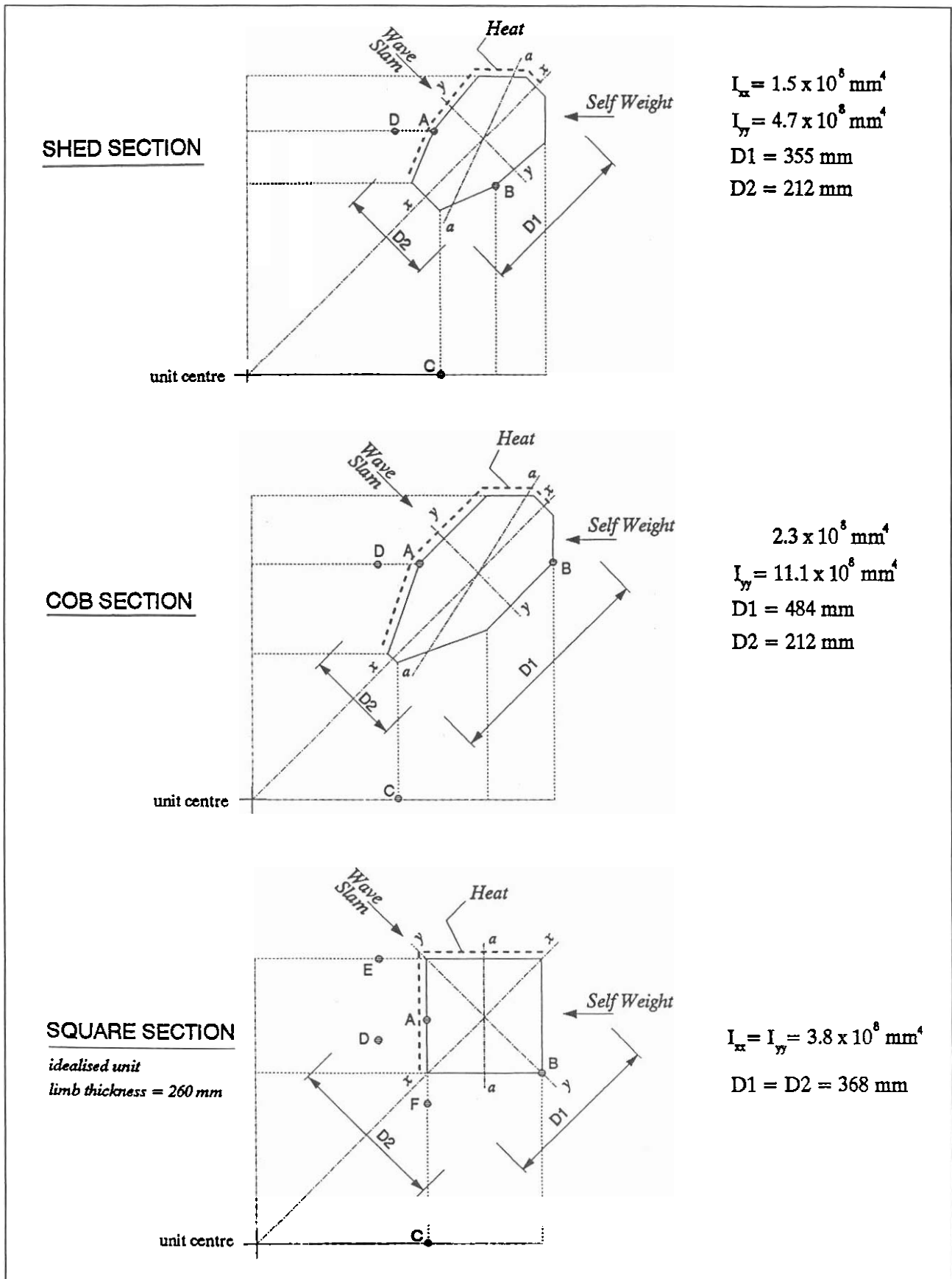**Figure 3.9 Influence surface for horizontal component of stress at (A), Cob unit**

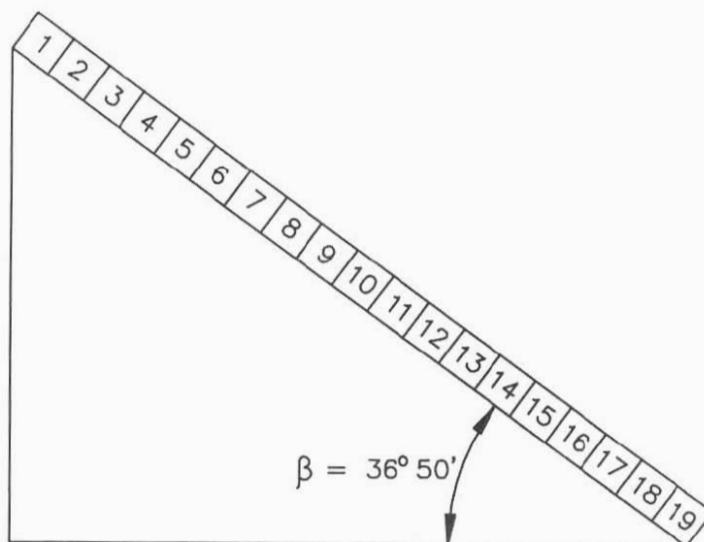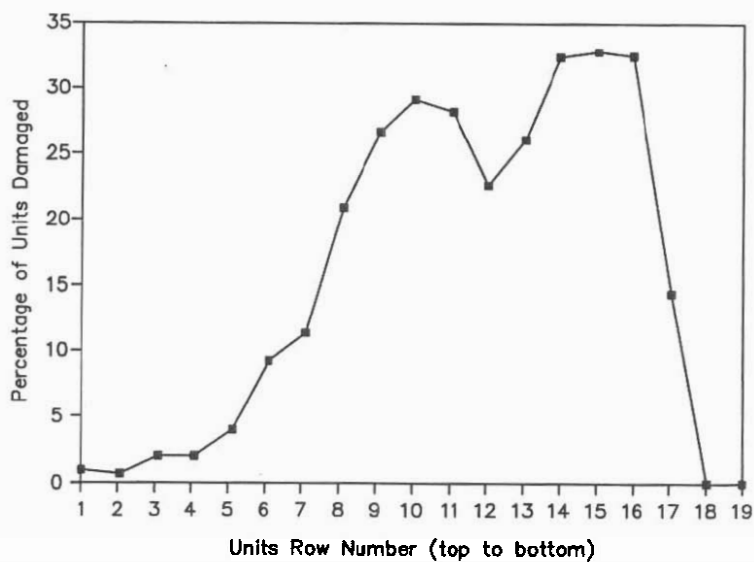**Figure 3.10   Influence surface for horizontal component of stress at (B), Cob unit**

SHED SECTION

$I_{xx} = 1.5 \times 10^8 \text{ mm}^4$

$I_{yy} = 4.7 \times 10^8 \text{ mm}^4$

D1 = 355 mm

D2 = 212 mm

COB SECTION

$2.3 \times 10^8 \text{ mm}^4$

$I_{yy} = 11.1 \times 10^8 \text{ mm}^4$

D1 = 484 mm

D2 = 212 mm

SQUARE SECTION

idealised unit
limb thickness = 260 mm

$I_{xx} = I_{yy} = 3.8 \times 10^8 \text{ mm}^4$

D1 = D2 = 368 mm

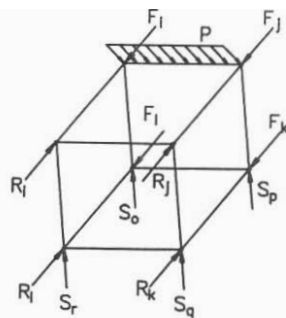**Figure 3.11   Shed, Cob and Frame unit mid-limb sections**

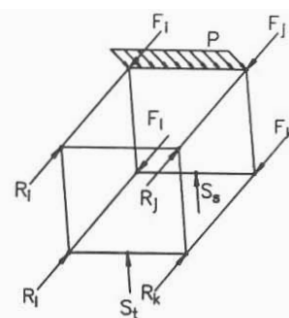Units Row Numbering

Distribution of Damage as Observed on Site

Figure 3.12   Cob breakwater observed cracked units
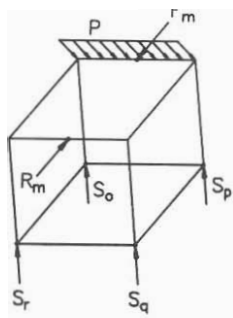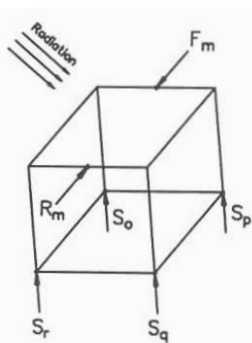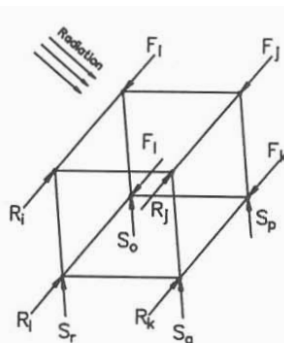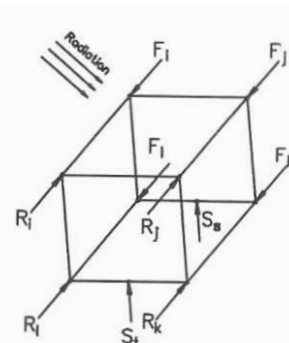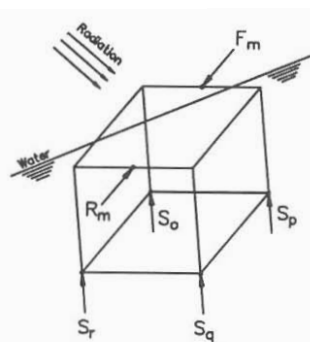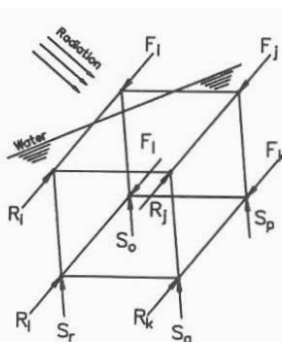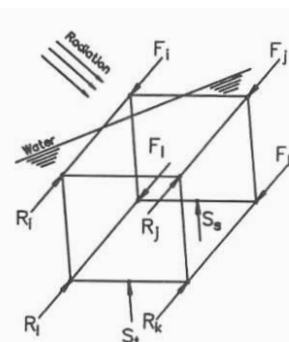
Figure 3.13 Load cases assessed for breakwater in Fig 3.12
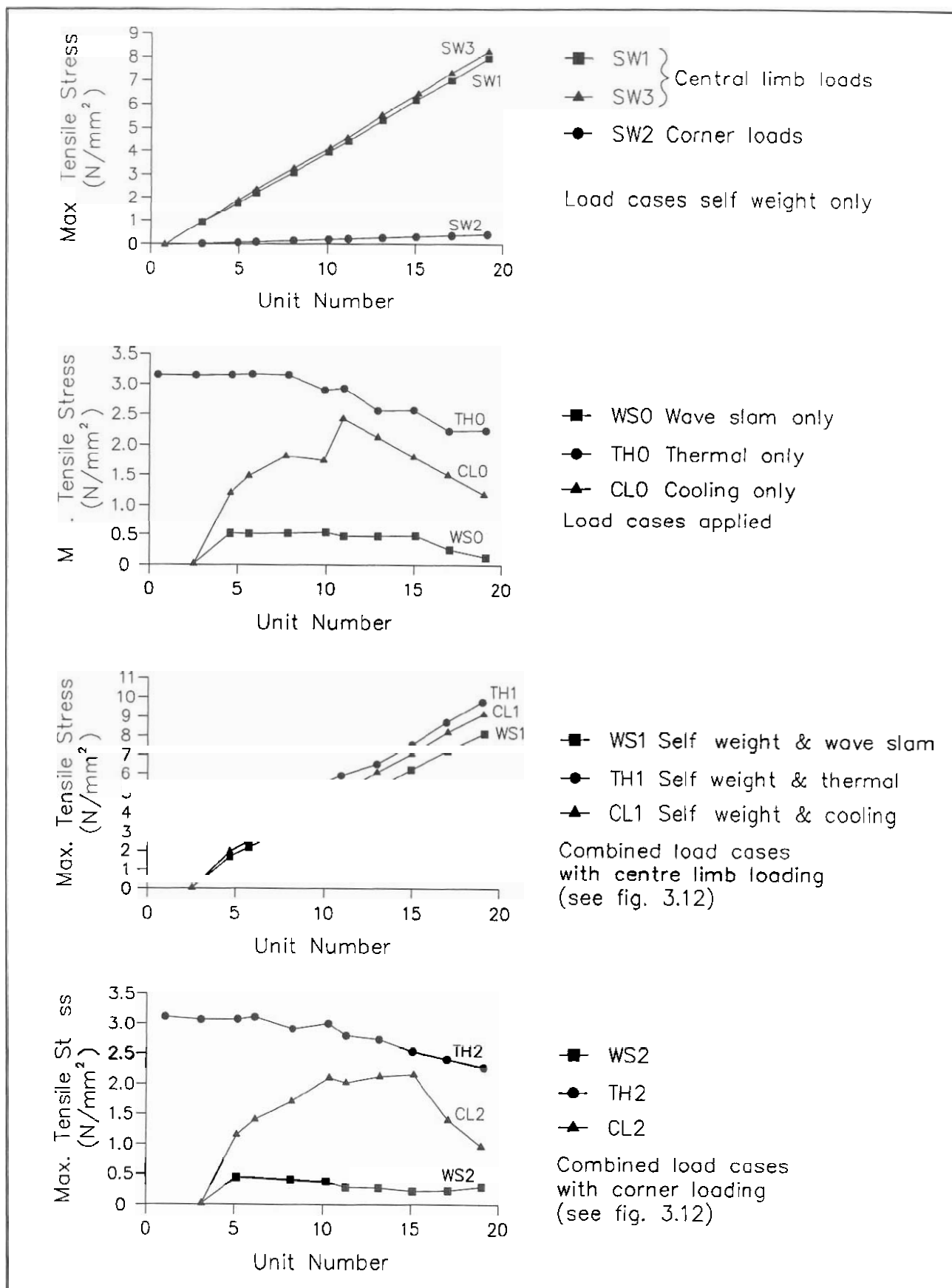
Figure 3.14   Load case results computed for breakwater in Fig 3.12

Appendices

Appendix A

Fault tree analysis : cracking and breakage of hollow cube armour units

Appendix A Fault Tree Analysis: Cracking and Breakage of Hollow Cube Armour Units

A1 Introduction

The objective of using a simplified form of "Fault Tree Analysis (FTA)" is to examine, all the potential causes which could result in either cracking or breakage of single layer concrete armour units. In so doing a better understanding is obtained regarding the "modes of failure" of a structure which is an essential requirement of good engineering design.

A2 Method

The basic reasoning behind "Fault Tree Analysis" is the consider each step in the life cycle of an armour unit and seek to identify every possible cause which could result in cracking or breakage. In short it is a cause and effect diagram. It is essential in such work to include every possible "cause" however remote the chance of it occurring, although at this stage no attempt has been made to assign numerical values to the probability of occurrence of each particular event.

The system has been used in various engineering situations, but particularly on failures of breakwaters and seawalls and has, in many instances, identified unexpected causes of failure which has led to fundamental rethinking of design and construction practice.

It is not necessary to select any particular armour unit at this stage of the analysis other than to specify a single layer pattern placement.

A3 Life cycle of armour unit

The life cycle is deemed to consist of four basic elements
- i   Design
- ii   Manufacture
- iii Construction
- iv   Service Life

Each of these elements can in turn be sub-divided as follows and as shown in Figure A.1
- a   Design - Structure design and armour unit design.
- b   Manufacture - Casting, curing, handling and storage.
- c   Construction - Storage, transportation, lifting and placement.
- d   Service Life -   In-situ forces and stresses

A4 Fault Tree Analysis

The starting point of the analysis must be the end result, ie, the cracked or broken unit, on the face of the structure so that in effect the life cycle must be considered in reverse. This does not affect or influence the possibility that breaking or cracking can occur earlier in the life cycle of a unit.

Note that the analysis which has been undertaken is of a generalised nature and not related to a particular type of breakwater or seawall but could easily be modified and extended to cater for specific conditions.

Figure A.2 shows the initial main divisions of the fault tree. Starting with the cracked or broken unit on the breakwater the principal "causes" of such a situation are identified as either "excessive load on unit" and/or "insufficient strength of unit". The row below this then looks at the potential "causes" of either excessive loads on the unit or insufficient strength of the unit. Form this point onwards the diagram tends to become somewhat large and unmanageable and separate drawings have been used to cater for each extended fault tree in Figures A.3 to A.8 inclusive.

A5   Extended Fault Tree Analysis

The example above deals with one particular aspect of the structure's stability. Extended analysis can however be undertaken to cover the structure as a whole as briefly referred to in Figure A.4 "Excessive Static Loads".
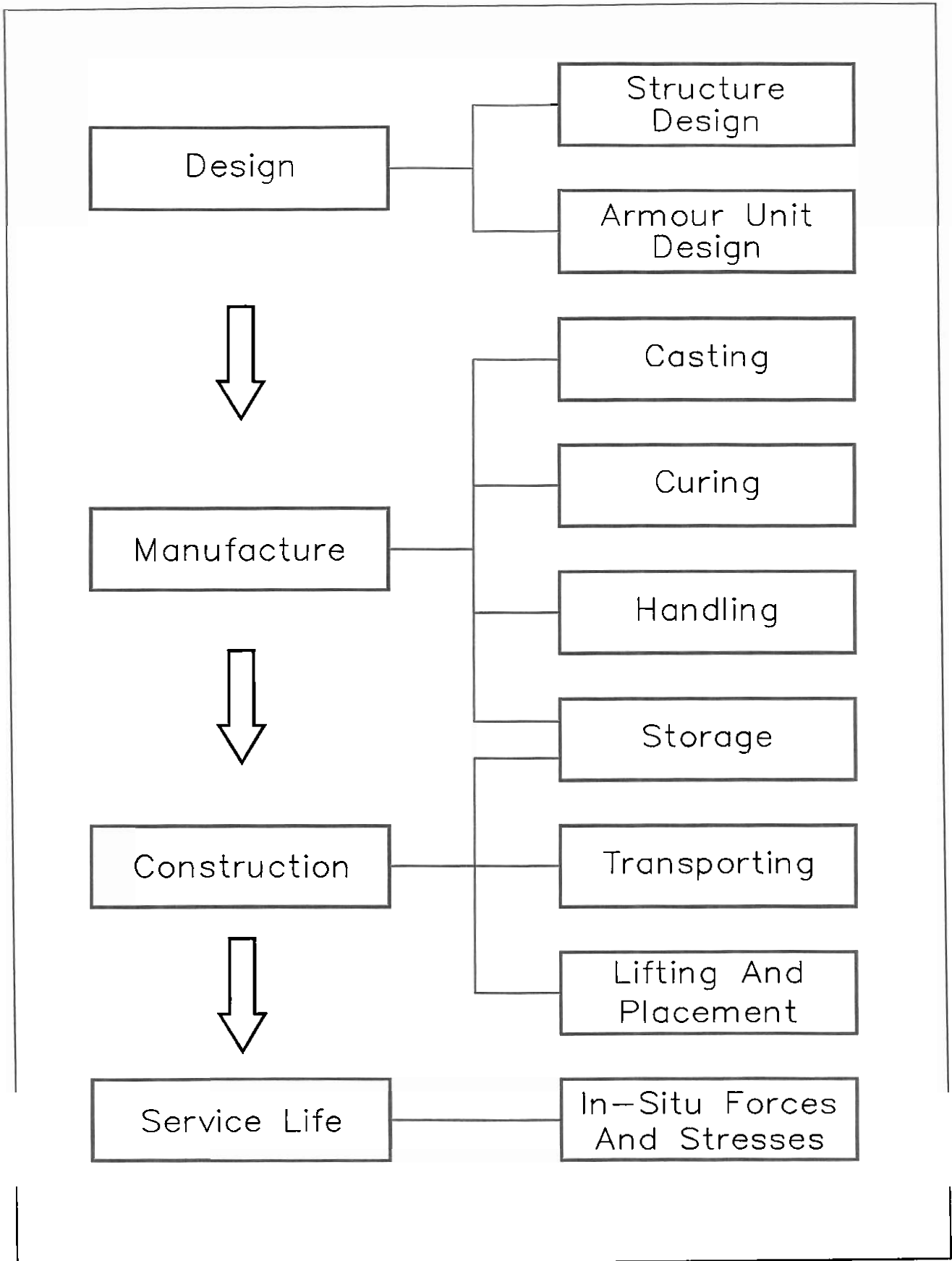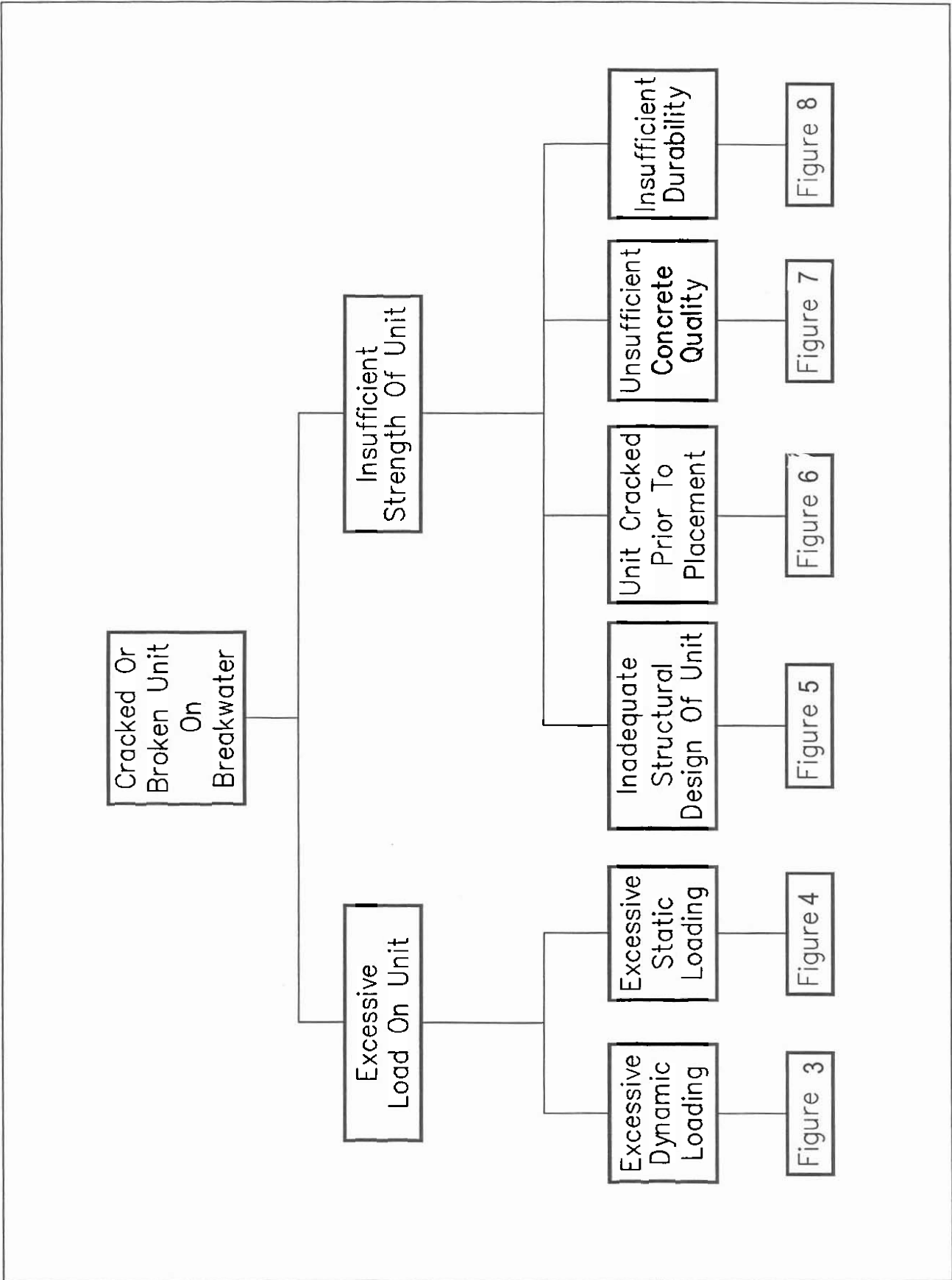
Figure A1 Life cycle of Armour unit

Figure A2 Fault tree main divisions
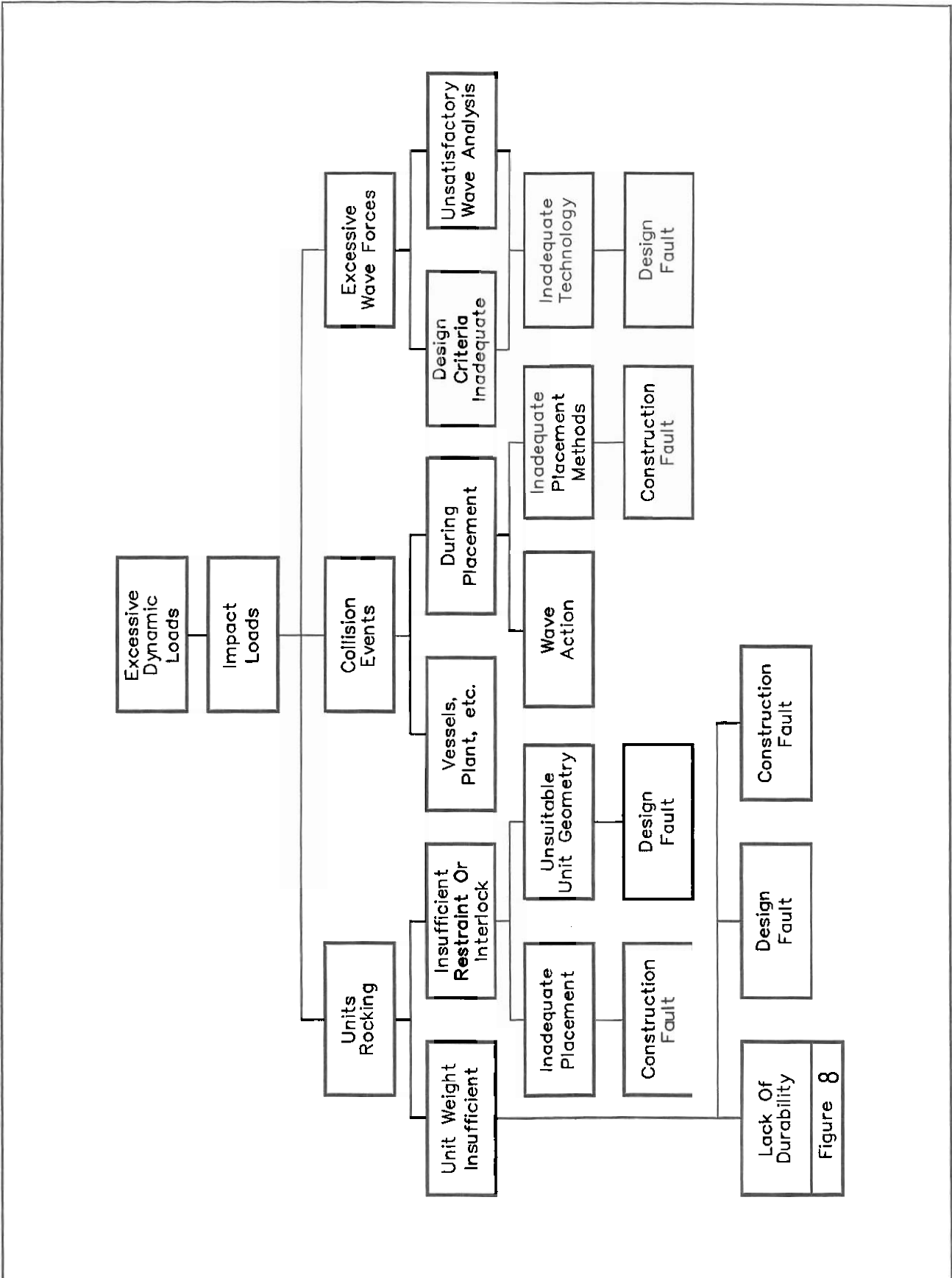
Figure A3 Excessive dynamic loads

EXCESSIVE STATIC LOADS

- Unfavourable Supporting Layers
  - Inadequate Placement
    - Construction Fault
  - Underlayer Rock Size Unsatisfactory
    - Design Fault
- Cumulative Self Weight Down Slope
  - Insufficient Strength Of Unit
  - Figures 5, 6, 7 & 8
- Excessive Loading By Surrounding Units
  - Unsuitable Unit Geometry
    - Design Fault
  - Settlement Of Armour Layer
    - Settlement Of Structure
      - Slip Failure Of Sub-Soil
      - Excessive Ground Load
    - Failure Of Toe Structure
      - Instability Of Armour Stone
      - Excessive Foreshore Erosion
      - Local Instability Of Toe
      - Erosion Of Filter Or Core Material

Further Fault Trees Required Beyond This Stage

- Hydraulic Load > Expected
  - Design Fault
- Toe Geometry Incorrect
  - Construction Fault
- Rock Weight Insufficient
  - Design Fault
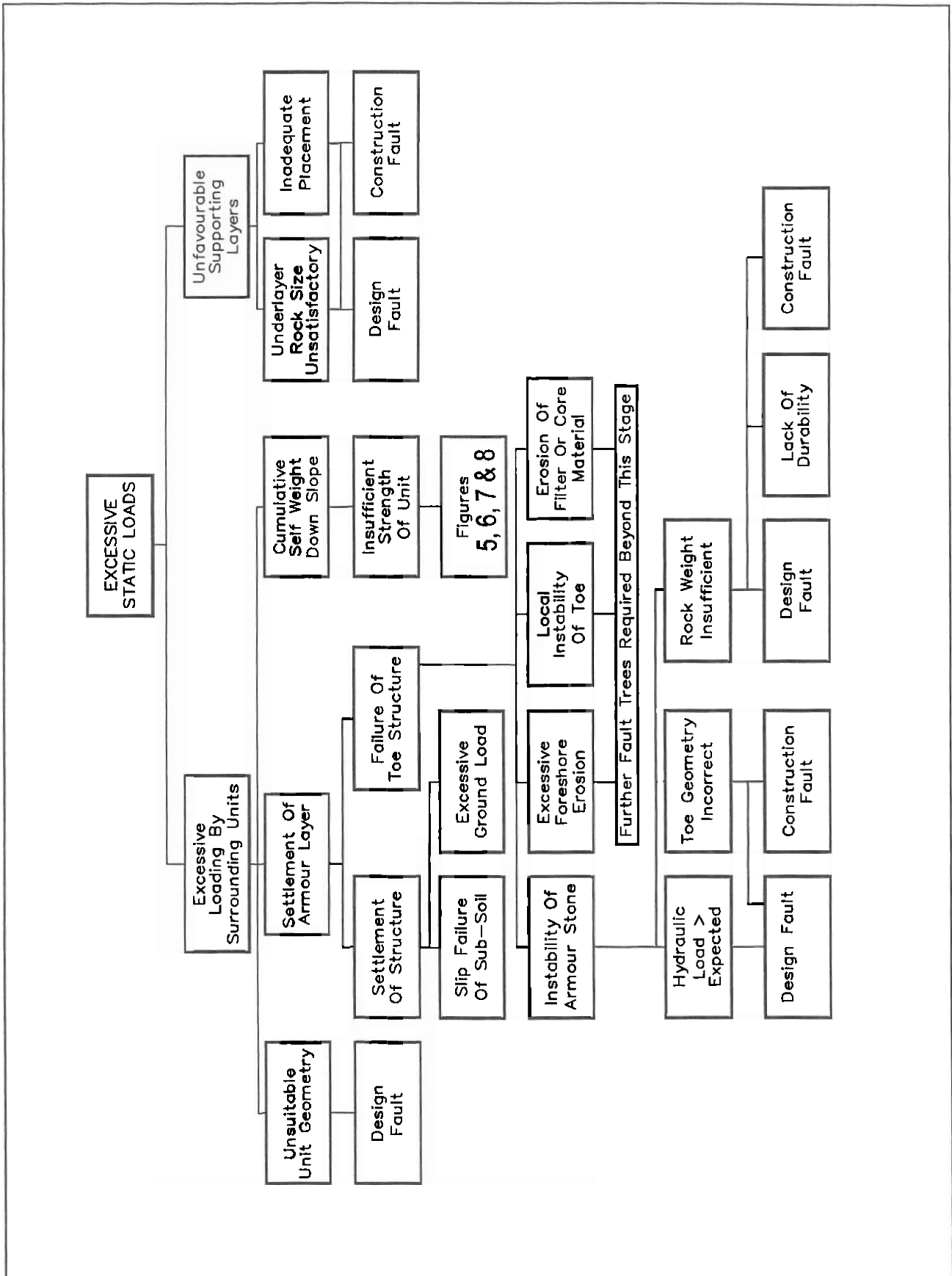 - Lack Of Durability
  - Construction Fault

Figure A4 Excessive static loads

Figure A5 Inadequate structural design of unit
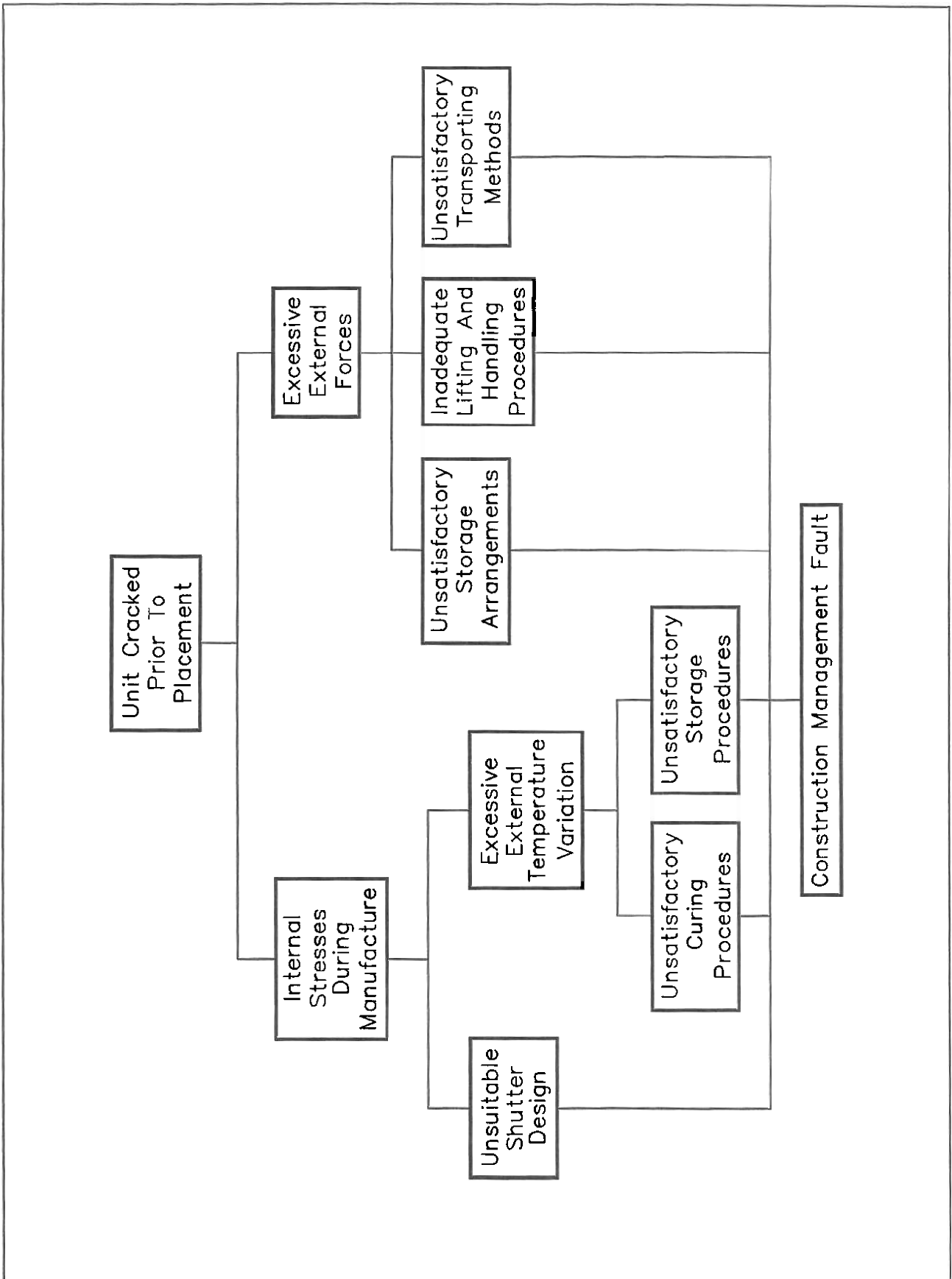
Figure A6 Unit cracked prior to placement
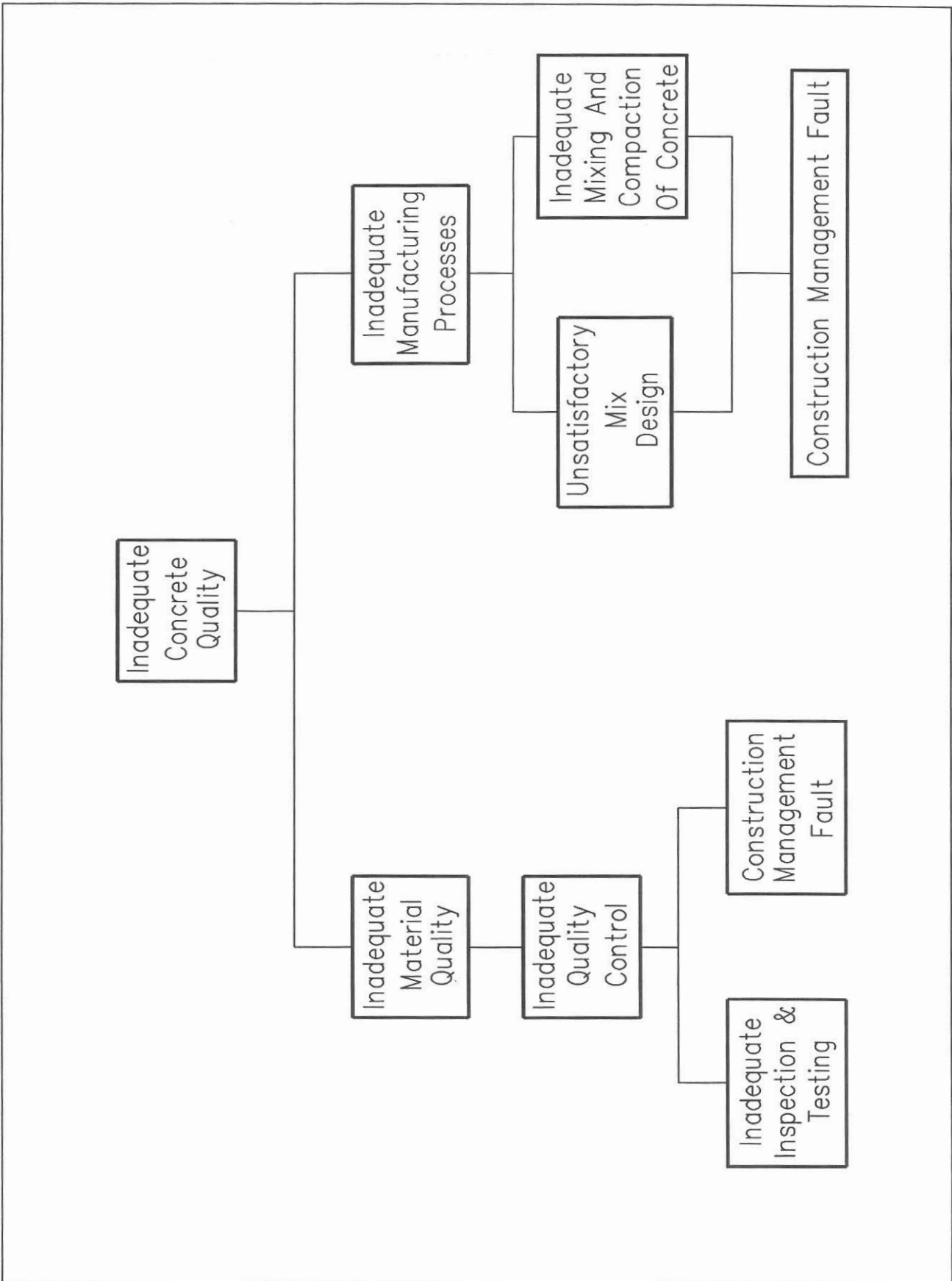
Figure A7 Inadequate concrete quality

Boxes in figure (reading the tree structure):

- Inadequate Concrete Quality
  - Inadequate Manufacturing Processes
    - Inadequate Mixing And Compaction Of Concrete
    - Unsatisfactory Mix Design
    - Construction Management Fault
  - Inadequate Material Quality
    - Inadequate Quality Control
      - Construction Management Fault
      - Inadequate Inspection & Testing
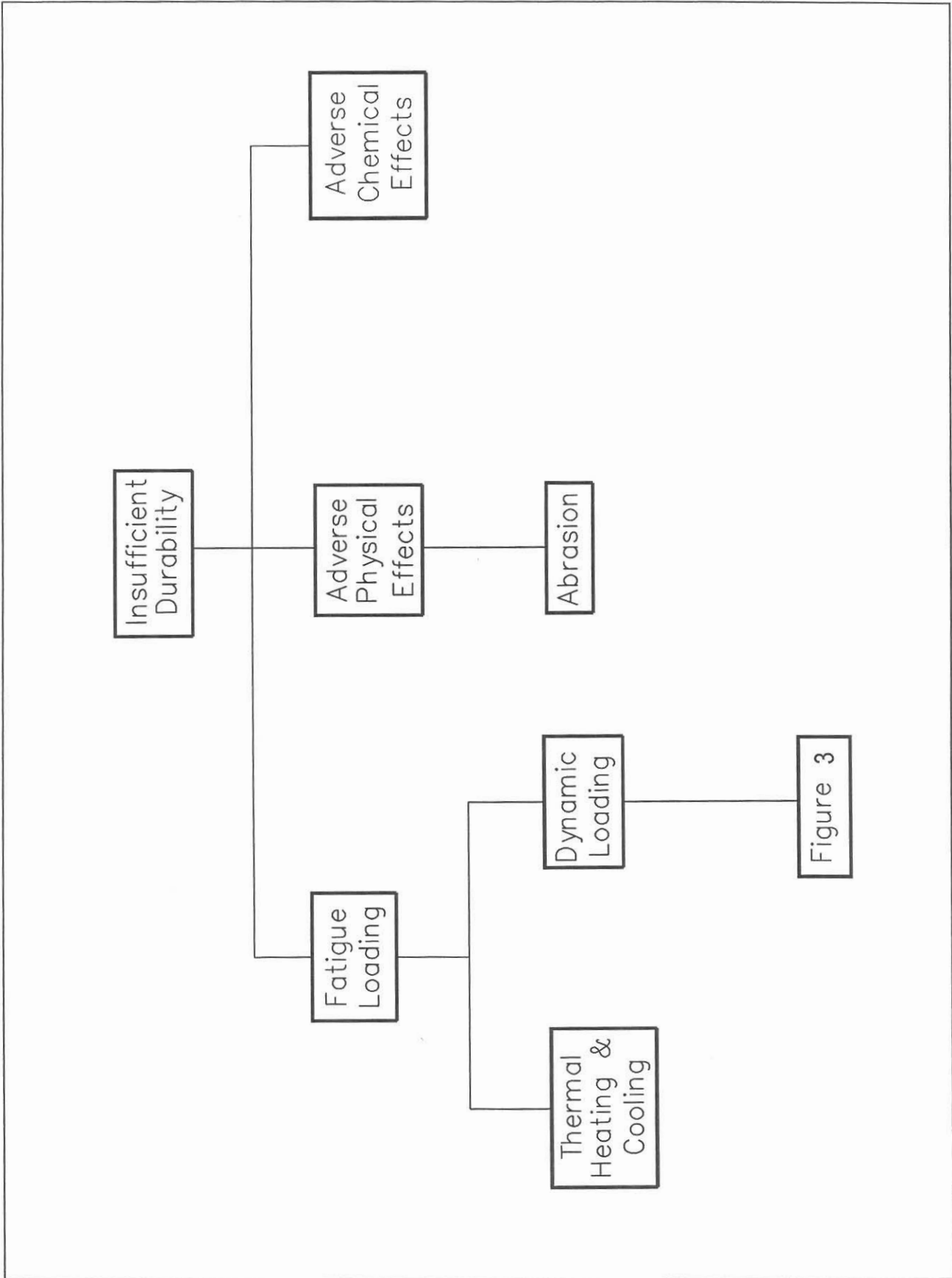
Figure A8 Insufficient durability

SR 482 27/01/97

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