COMPUTER AIDED DESIGN OF PRECAST CONCRETE INDUSTRIAL BUILDINGS

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ABSTRACT

Amidst the complexities of modern technology, the structural engineer has come to rely on the computer for general housekeeping. The design and construction of precast concrete framed structures is one instance where computing facilities can be sensibly utilised.

This thesis is primarily concerned with all aspects that directly affect the cost of a family of precast concrete structures. One such type of structure has been studied in detail. Cost factors including fabrication, haulage, foundation, cladding and erection have been studied in relation to this structure and within an industrial environment. The results have been used to develop a suite of programs. One of the programs in this suite is a cost model which analyses, designs and costs precast concrete portal framed buildings.

Using efficient data structure modelling and picture transformation routines, a graphics program has been developed. This graphics program can be interfaced with the cost model such that at any design session a set of engineering drawings and bending schedule sheet can be output digitally. The cost model can also be interfaced with a cost contour program and an optimizing algorithm such that an optimum design can be achieved at a design session and the asscciated cost response surface may also be generated digitally.

Sample data sets are input to the suite of programs and

the results are presented. Conclusions are drawn on the significance of these results and the relevance of this study to existing industrial practice.

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LIST OF SYMBOLS

Am ... cross sectional area of a structural member

Bo ... current objective function

B1 ... current best objective function

B2 ... current best objective function where
boundary zone is not violated

Di ... displacement in the direction of a force

E ... modulus of elasticity

f(x) ... objective function

Fcu ... characteristic cube strength of concrete

Fm ... internal force in a structural member

g ... constraint function in a design space

Gf ... partial safety factor

h ... height to eaves

j ... number of constraints

k ... number of building spans

Lm ... unit length of a structural member

M, Mx .. bending moment

n ... number of building bays

Pi ... force at a point in a structural system

Q ... a point in a design space

R1 ... maximum end lap for ridge sheets

R2 ... standard length of a cladding sheet

R3 ... maximum end lap for cladding sheet

s ... standard deviation

t ... angular rotation in a canonical space

T ... shear stress in a structural member

Ttx ... translating transformation component in x- direction

Tty ... translating transformation component in y- direction

Ttz ... translating transformation component in z- direction

Tsx ... scaling transformation component in x- direction

Tsy ... scaling transformation component in y- direction

Tsz ... scaling transformation component in z- direction

U ... strain energy of a structural member

V ... principal stress

w ... roof pitch

W2 ... overall width of cladding sheet

W3 ... nett cover width of cladding sheet

We ... weight of a structural element

x ... span of a structure

y ... frame spacing of a structure

Xi ... coordinate of a point in x- direction

Yi ... coordinate of a point in y- direction

Zi ... coordinate of a point in z- direction

CHAPTER 1

INTRODUCTION

1.1 OVERVIEW

This chapter defines the theme of the thesis and shows the methods and techniques that have been employed to solve the problem. A preference has been stated for interactive computing methodology as an efficient and indispensable tool that can cope with present day civil engineering complexities.

1.2 THE PROBLEM

The problem of designing and erecting a structure is as old as the beginnings of man; the philosophy and methods of solution however, have differed from time to time. When precast concrete evolved as an offshoot of traditional concrete construction, it ushered into the Construction Industry another unique industry. That industry now plays an important role in civil engineering. This thesis is primarily concerned with all factors that directly affect the cost of a family of precast concrete structures.

The principal task is to build to satisfy a client's request. Given an area of land space, it is the task for the

engineer tc fill the space econcmically with a suitable building. For instance if the shape of the building is fixed there are several alternate desigrs that can be used. If we limit cur preference tc portal framed structures, we still have a wide range of topologically various types tc chccse from. It is left tc the engineer therefore to make an optimum selection from several primary parameters and to justify that this selecticn is eccncmical.

Unlike in-situ concrete ccnstructicr, precast concrete structural units are prefabricated and transported to an erection site. These intermediate stages, especially fabricaticn, and the final erection stage contribute substantially to the overall first costs cf the structure. Improvements to existing practice that would reduce global costs without adversely violating any design corstrairt must be a whclescme theme for study. In the search for such a solution the problem was approached from fcur standpcirts:

1. Tc develop a design algorithm based on a code of practice allowing primary variables as input
2. Tc develop a ccst model based cn typical industrial practice
3. Tc allow a search fcr an optimum design using an appropriate optimizing algorithm
4. Tc develop an efficient ccmputer graphical output system that will provide a set of working drawings, berding schedule and ccst contour maps fcr feasible designs.

1.3 SCOPE OF STUDY

This study was carried out in the form of design, factory work study, field survey, computer programming and interactive computer graphics.

Structural design was based on limit-state philosophy (Section 4.3). Work study was carried out in a precast concrete factory in Bishop Auckland, Co. Durham and similar observations were made in a second precast concrete factory at St. Ives, Cambridgeshire. Studies on erection were made at some twenty sites in Co. Durham, North Yorkshire, Cambridgeshire and Northamptonshire. Techniques and methods of erection (Section 2.5) of a leading firm were studied and conventional work study was made on the inter-connexion of the various structural components. Work study was also done on fixing cladding (Chapter 5) to the frame skeleton. Market prices of cement, coarse and fine aggregates and steel reinforcement have been assumed. Prices of connexion accessories (i.e. hook bolts and nuts, screws, top hats, connectors, brackets, cleats, tie rods and nuts, purlin bolts etc.) have been obtained from proprietary manufacturers. So have been prices of cladding materials (such as sheeting, ridge cappings, finials, barge boards, fillers, expansion fitments and spacers). Haulage rates have been obtained from a haulier.

A suite of programs was developed to analyse, design and cost the structure and its skin. A standard optimising algorithm (Chapter 3) was used. A graphical output system

comprising a set cf wcrking drawirgs, bendiⁿg schedule and cost contour surfaces was developed with suitable graphical packages (Chapter 7). Graphical displays were perfcrmed cn a Computek 400 and drawn on a Calccmp 936 plcttter. All progremming was done on an IBM 370/168 operating under Northumbrian Universities Multiple Access Ccmputer time-sharing system.

1.4 ALGORITHMIC DESIGN AND COST MCDELLING

The engineer uses the computer cnly as a tccl tc assist in performing various tasks which otherwise would be cumbersome, monotonous and time-consuming. Without computing facilities, certain present day engineering tasks could nct be accomplished without utilising many unproductive manhours. Ccnsider the present problem :

Every month a typical engineering firm might receive about ten invitations to tender for the provisicn of portal framed buildings of varoius types. Traditicnally, each tender invclves design, costing and providing wcrking drawings for client, contractor, and planning permit. It could take up to ten working days fcr a staff of three (two engineers and one draughtsman) to complete one such tender, that is cne hundred working days per mcnth to cope with the firm's tendering. In practice, however, the firm wculd have standard compcnent designs, but the task cf costing and providing wcrking drawings in each case will still consume much skilled labcur.

The same project can be handled efficiently and faster by employing CAD techniques. The solution to this particular problem will involve design input for each project to give design and cost information in a matter of seconds. A standard plotter will take only minutes rather than days to complete a set of working drawings. Computing methods are accurate and reliable as they are free from human errors. These are the principal factors that make an algorithmic approach to design and costing of civil engineering works desirable.

An efficient design and cost model has four distinct features :

1. The design philosophy must be consistent with the conventional Code of Practice.
2. A limited number of input parameters, say up to ten, must be used.
3. The system must be fully automated and must not involve the use of design charts.
4. The model must be easily accessible to an optimizing algorithm
5. The model must represent true first costs and must not treat as trivial any factor that affects global costs.

CHAPTER 2

PRECAST CONCRETE STRUCTURES

2.1 OVERVIEW

In this chapter, the structure which has been chosen for this study has been defined and other structures of the precast concrete family related to the study are described. Economic trends in the Precast Concrete Industry are examined and compared with the larger Construction Industry in the context of economic depression of recent years. In this retrospect, an economic justification emerges for this particular study of a section of the Precast Concrete Industry that enjoys moderately increased profits in such circumstances. Both traditional and modern methods of fabricating structural components are examined and methods of handling and erection are discussed.

2.2 THE STRUCTURE

The basic structural configuration selected for this study and referred to elsewhere as THE STRUCTURE is the three-pinned pitched portal frame (Fig 2.01). Other than the single span building with one or more bays there are other variations with

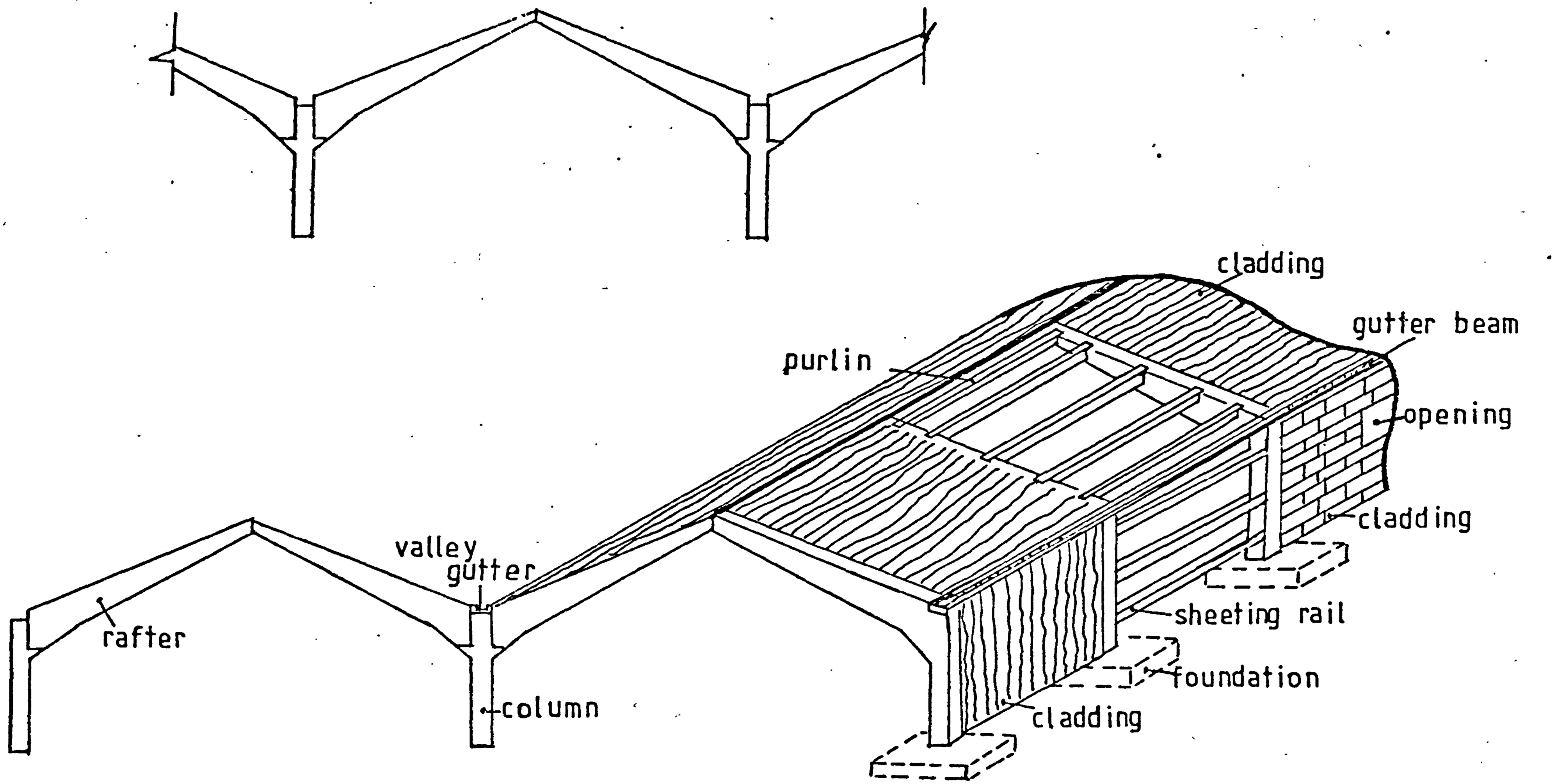


FIG 2.01 THE STRUCTURE

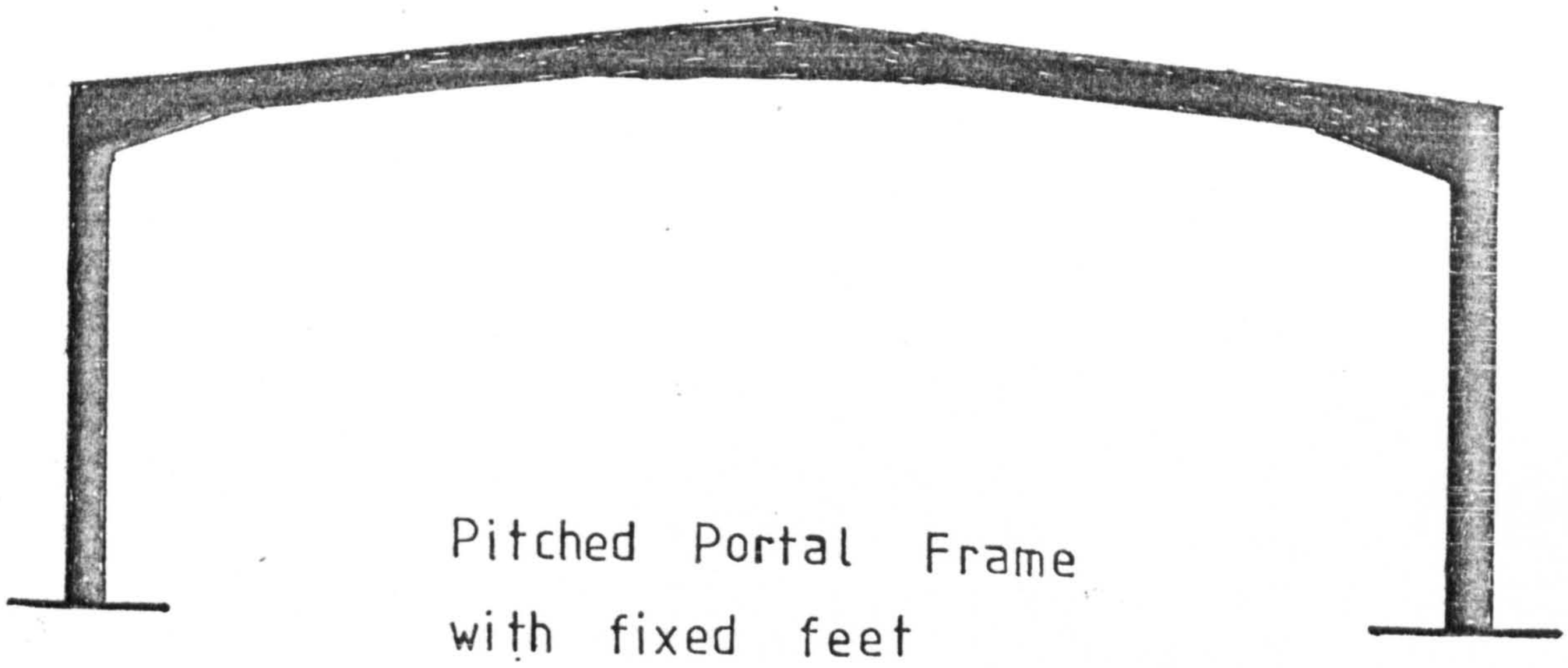
multiple spans and bays. The framework of the structure comprises columns, rafters, purlins, sheeting rails and gutter beams. The skin of the structure is termed cladding (Chapter 5).

Distinctions are made between various types of the following structural elements :

1. Main columns of the portal frame and end posts used in openings on the gable face.
2. Glazing rails and sheeting rails.
3. Eaves gutters, and valley gutters used for multi-span eaves drainage.

The structure was selected for its simplicity and extensive industrial application. Its behaviour can be extended to similar structures in the Precast Concrete Industry. Another pitched portal frame of industrial significance is one with fixed feet (Fig 2.02). Due to this fixity, considerable moments are generated at the eaves and at the pitch thus requiring haunches at both sections. A third type of portal frame is one with a very low pitch also shown in Fig 2.02.

Multi-storey precast concrete frames suitable for large scale production are illustrated in Fig 2.03. These are usually limited to two and three storeys. Various multi-span ranges and combinations of one form with another such as the following are common :



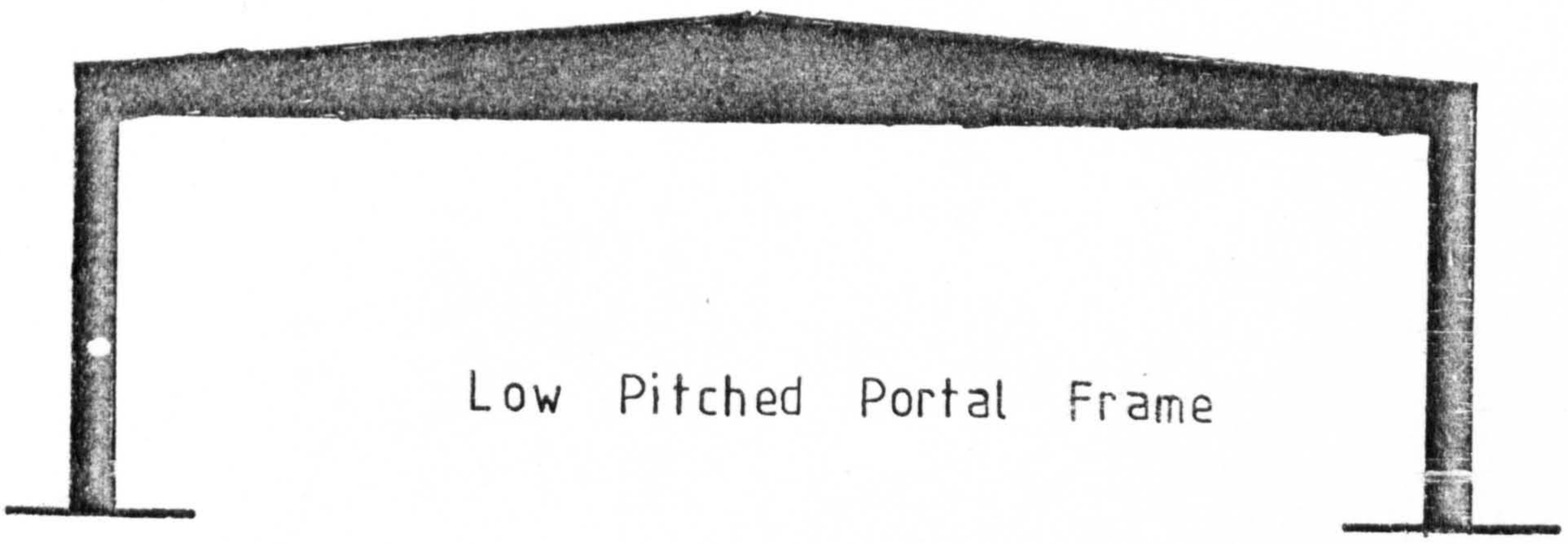


FIG 2.02 INDUSTRIAL PORTAL FRAMES

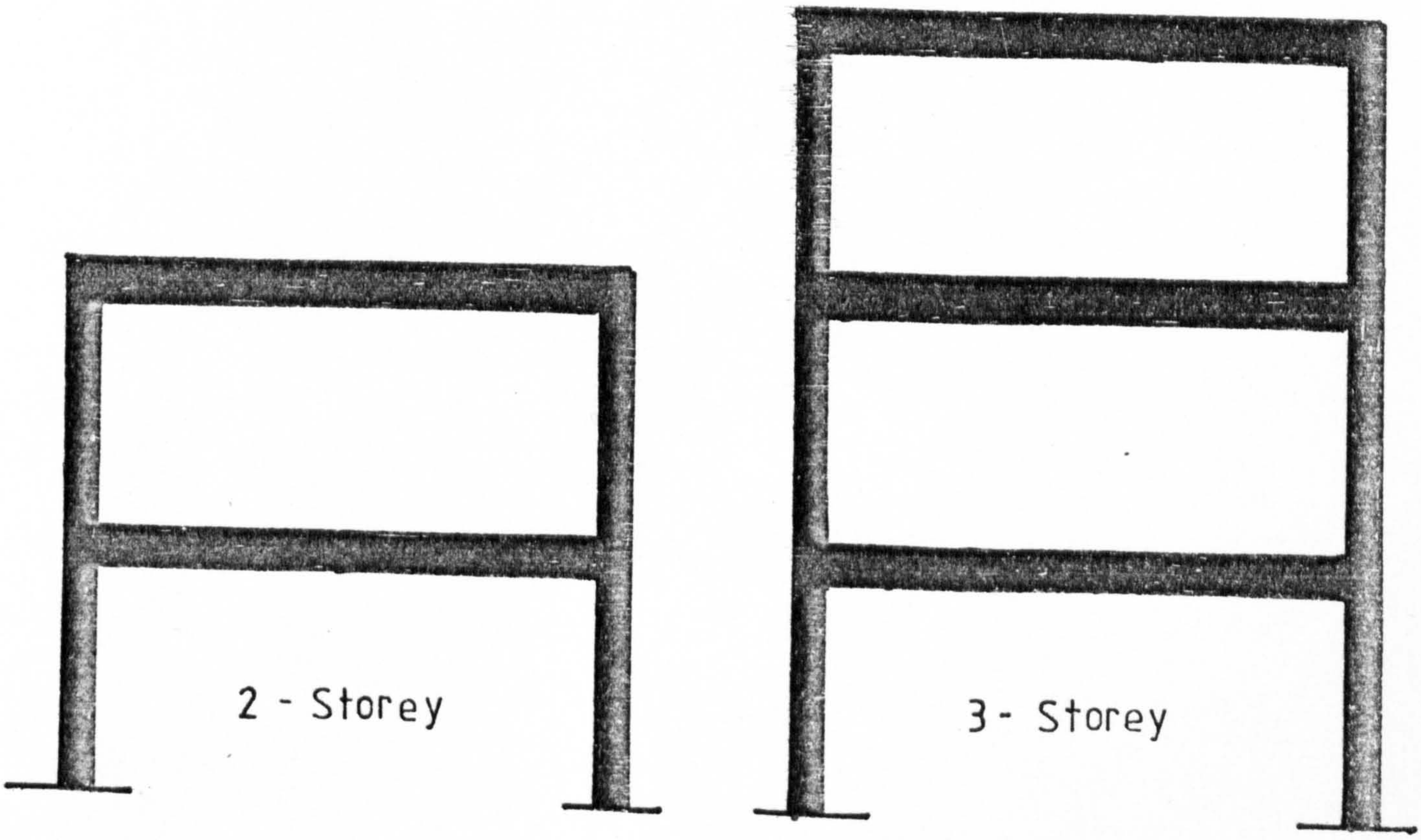


FIG 2.03 HIGH-RISE FRAMES

1. Two-storey integrated with fixed feet or pinned portal frame .

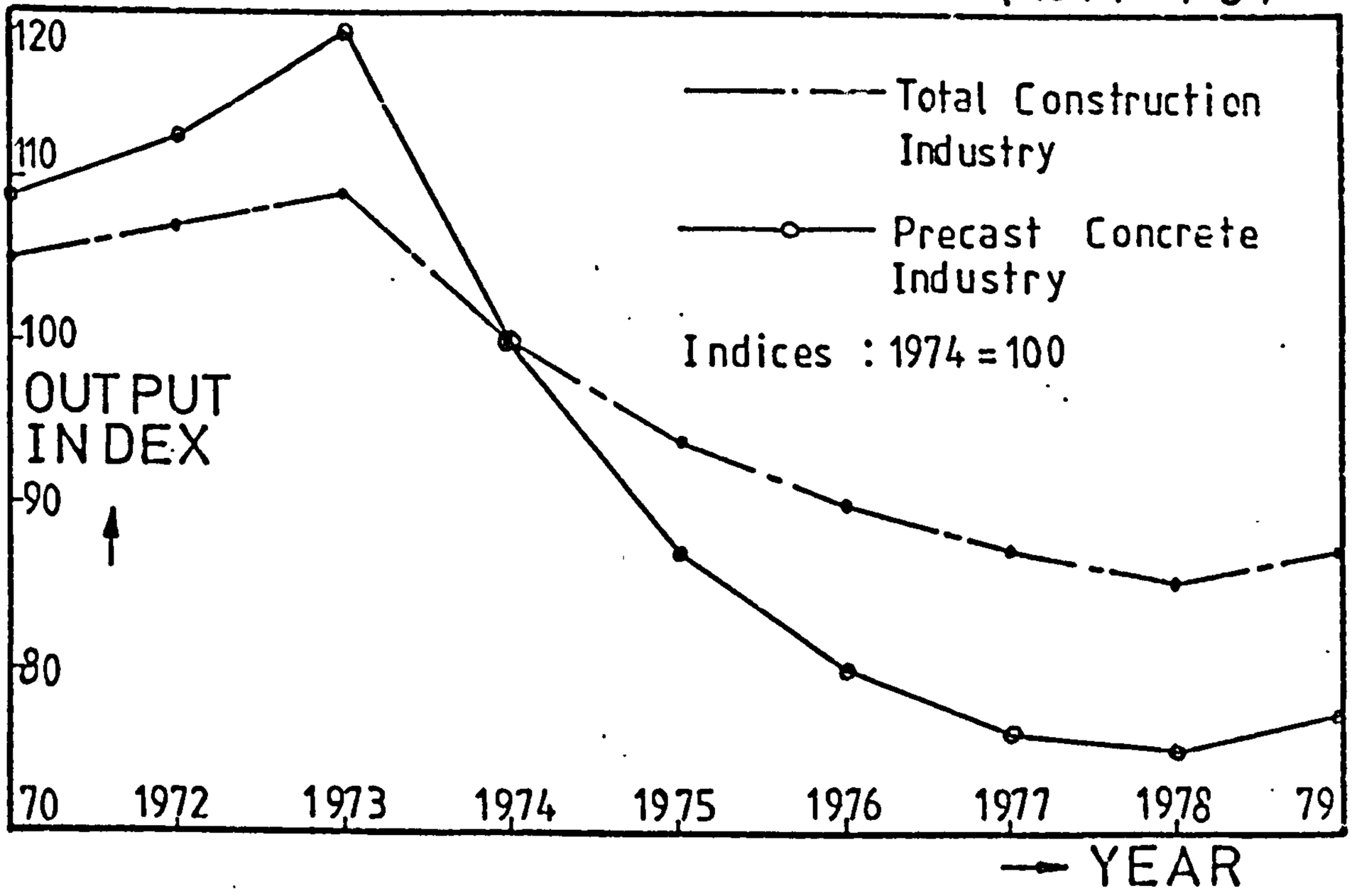
2. Three-storey integrated with fixed feet or pinned portal frame.

2.3 THE PRECAST CONCRETE INDUSTRY

The Precast Concrete Industry in the UK employs 28000 people out of the total 1,200,000 labour force of the Construction Industry, i.e. 2.33%. Between 1971-1978, the total output of the Construction Industry and Precast Concrete Industry are respectively estimated at £m42169 and £m1602, i.e. the latter's output represents 3.72 percent of the former. It can be seen that the Precast Concrete Industry has relatively a low labour content. With about 250 firms and 525 factories (1978) the Precast Concrete Industry consumes about 14% of the total cement used in the Construction Industry. Precast concrete products can be categorised under two sections :

1. Structural units - frames, lintels, blocks, beams, floor and roof units, piles, bricks, copings, bridge beams, tunnel segments, retaining walls, kerbs, channels and paving slabs.
2. Other units - Garden and agricultural products, street furniture, fencing, lighting columns, domestic products, Architectural cast stone, floor and roof tiles and drainage units.

FIG 2.04 UK CONSTRUCTION & PRECAST CONCRETE OUTPUT (1971-79)



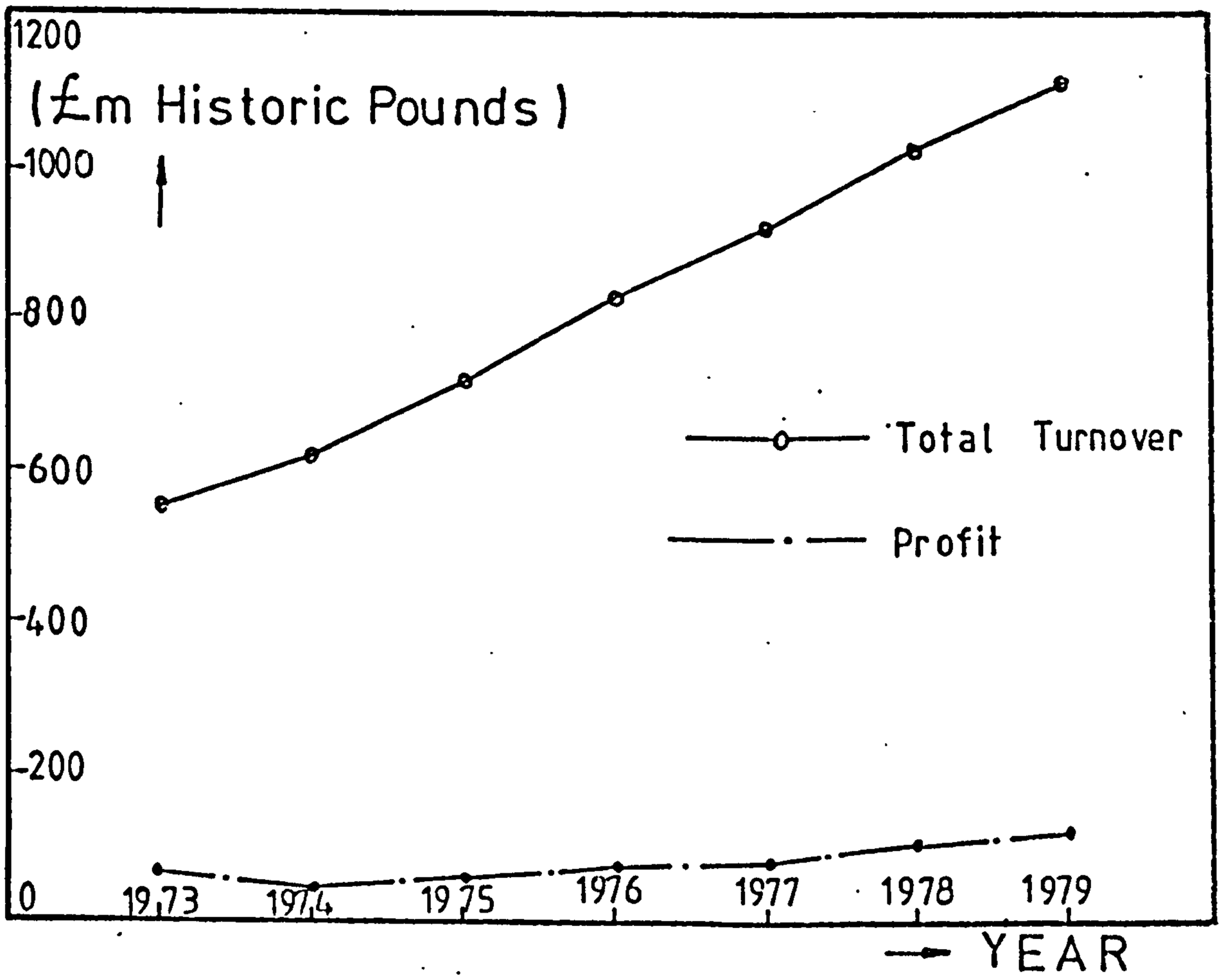


FIG 2.05 UK PRECAST CONCRETE INDUSTRY

Worldwide economic recession in recent years has had a substantial effect on the Construction Industry in general and the Precast Concrete Industry in particular. In Fig 2.04 output for 1974 has been expressed as 100 units and other outputs computed in relation to this index. It is seen that though the precast concrete outputs rose during the period 1971-1973, the trend has been dramatically reversed in the recession era. It appears that the worst of the economic slump ocurred in 1978 but it is still too early to speculate on recovery to 1973 levels.

Like other manufacturing industries, the Precast Concrete Industry has survived the worldwide economic setbacks mainly through cuts in labour. Fig 2.05 indicates that total turnover continues to rise steadily in the recession era and profits (before interest and tax) are already back at 1973 levels.

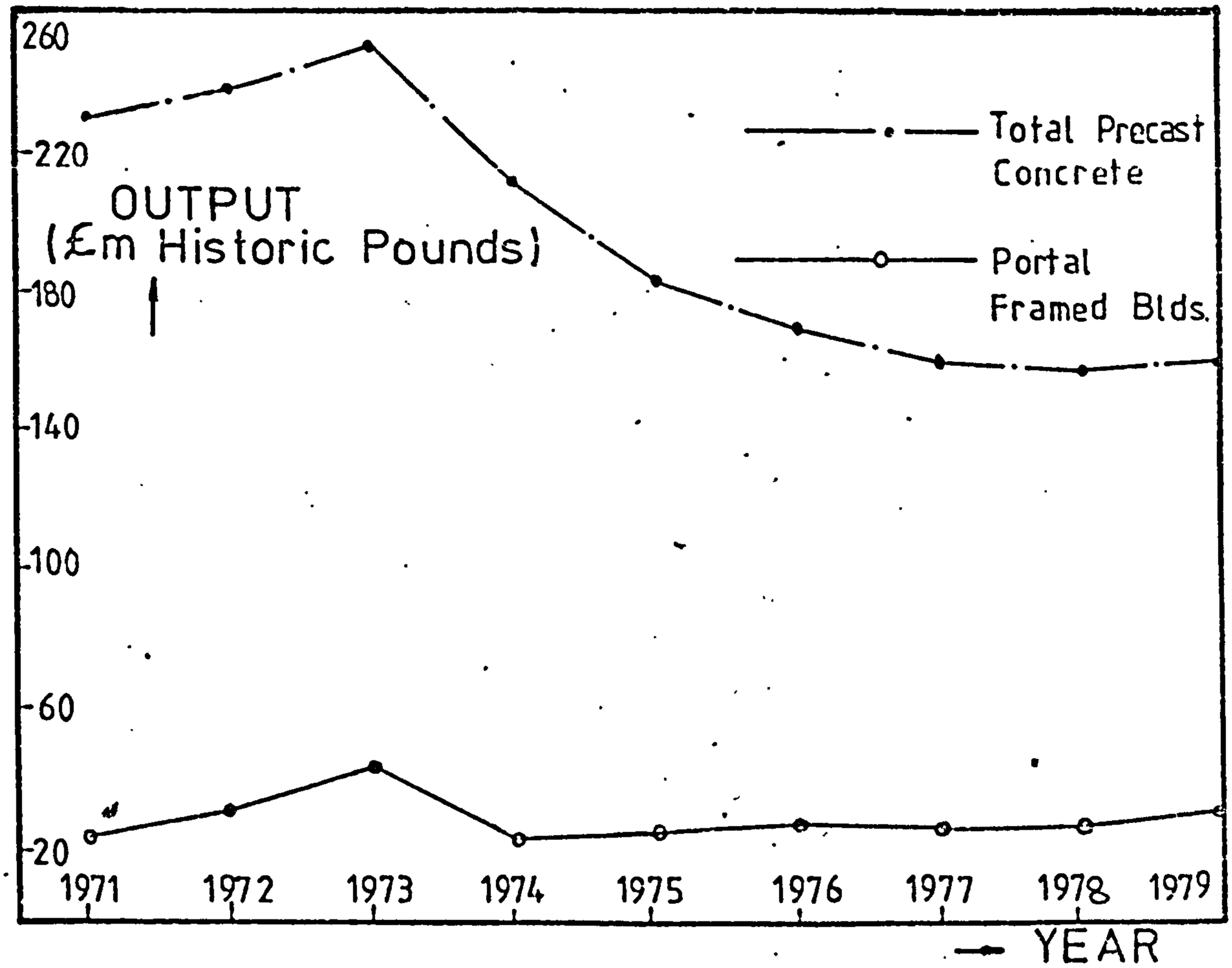
2.4 PRECAST CONCRETE FRAMED STRUCTURES

Out of the 250 precast concrete firms, 10 fabricate framed structural components on a large scale and 5 other firms on a small scale. Between 1971-1979 the total output of framed structures is estimated at 4.5 million tonnes, i.e. an average of 0.5 million tonne every year. Expressed in terms of cost, framed structures between the same period form 15.26% of all precast concrete products manufactured. Data shown in Table 2.01 and Fig 2.06 have been obtained from proprietary

TABLE 2.01 UK PRECAST CONCRETE FRAMED STRUCTURES

YEAR	OUTPUT (Tonnes)
1971	400000
1972	516700
1973	735000
1974	405000
1975	450000
1976	480000
1977	465000
1978	495000
1979	540000

FIG 2.06 UK PRECAST CONCRETE OUTPUT



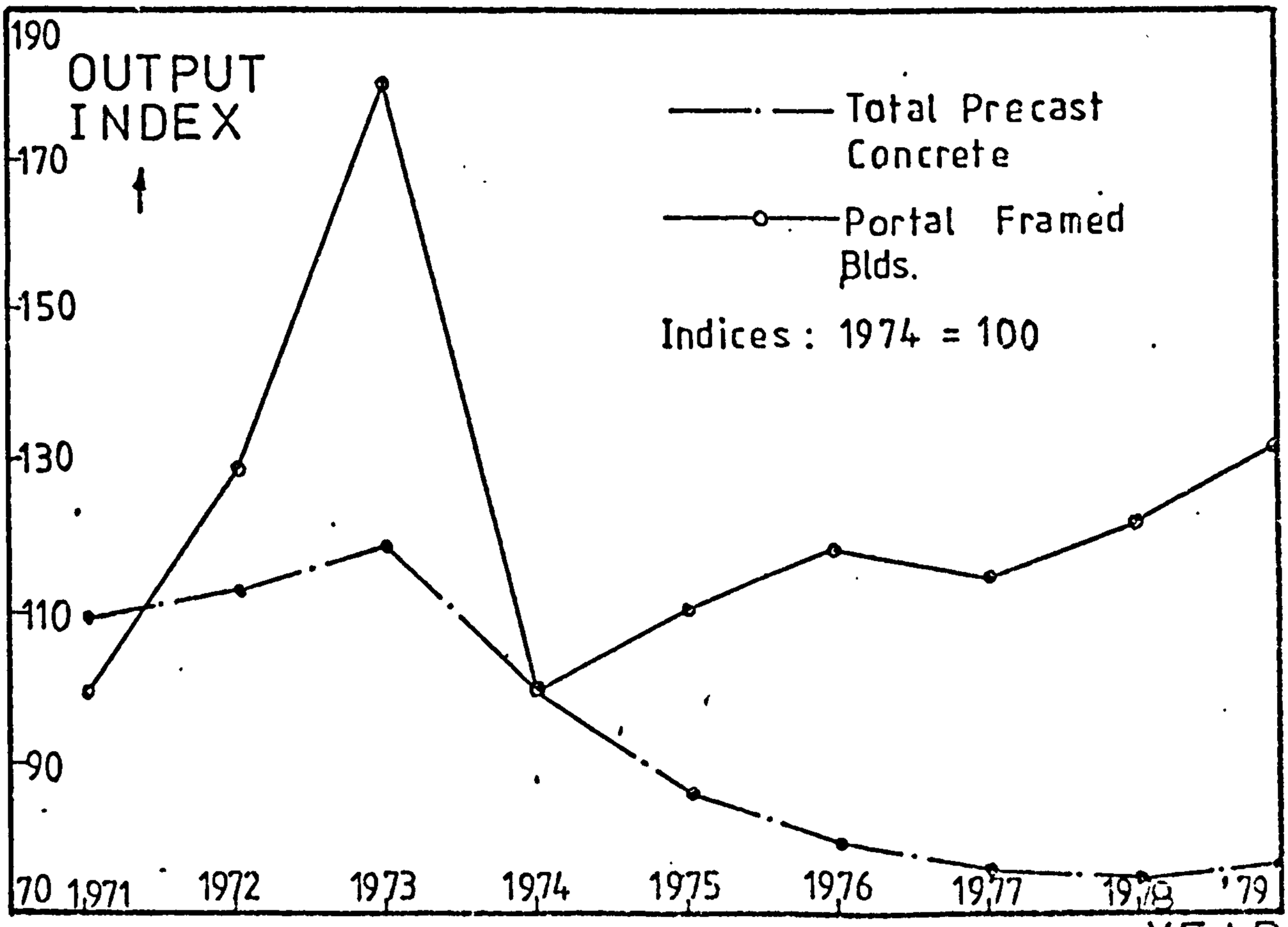


FIG 2.07 UK PRECAST CONCRETE OUTPUT

fabricators. Fig 2.06 indicates that since the all-time low output in 1974, demand has been steadily growing. Fig 2.07 shows that since 1971, demand for framed structures has surpassed average precast concrete products. Growth in demand for these structures can be directly related to their wide range of uses. Both private and public organisations have need for them. About 40% of all the structures erected anually are for agricultural purposes. Three main categories of users are :

1. Agricultural buildings - for storage of agricultural produce, feeds, etc.
2. Industrial purposes - warehouses, factories, offices, etc.
3. General purposes - recreational centres, church buildings.

Three principal advantages inherent in precast concrete construction over traditional in-situ construction are :

1. Because the structural components are manufactured, advantage can be taken of factory facilities to control materials and specifications with resultant high and accurate standards.
2. During erection, fo mwork is virtually eliminated. This yields substantial savings in construction costs.
3. A faster erection time is achieved with precast construction.

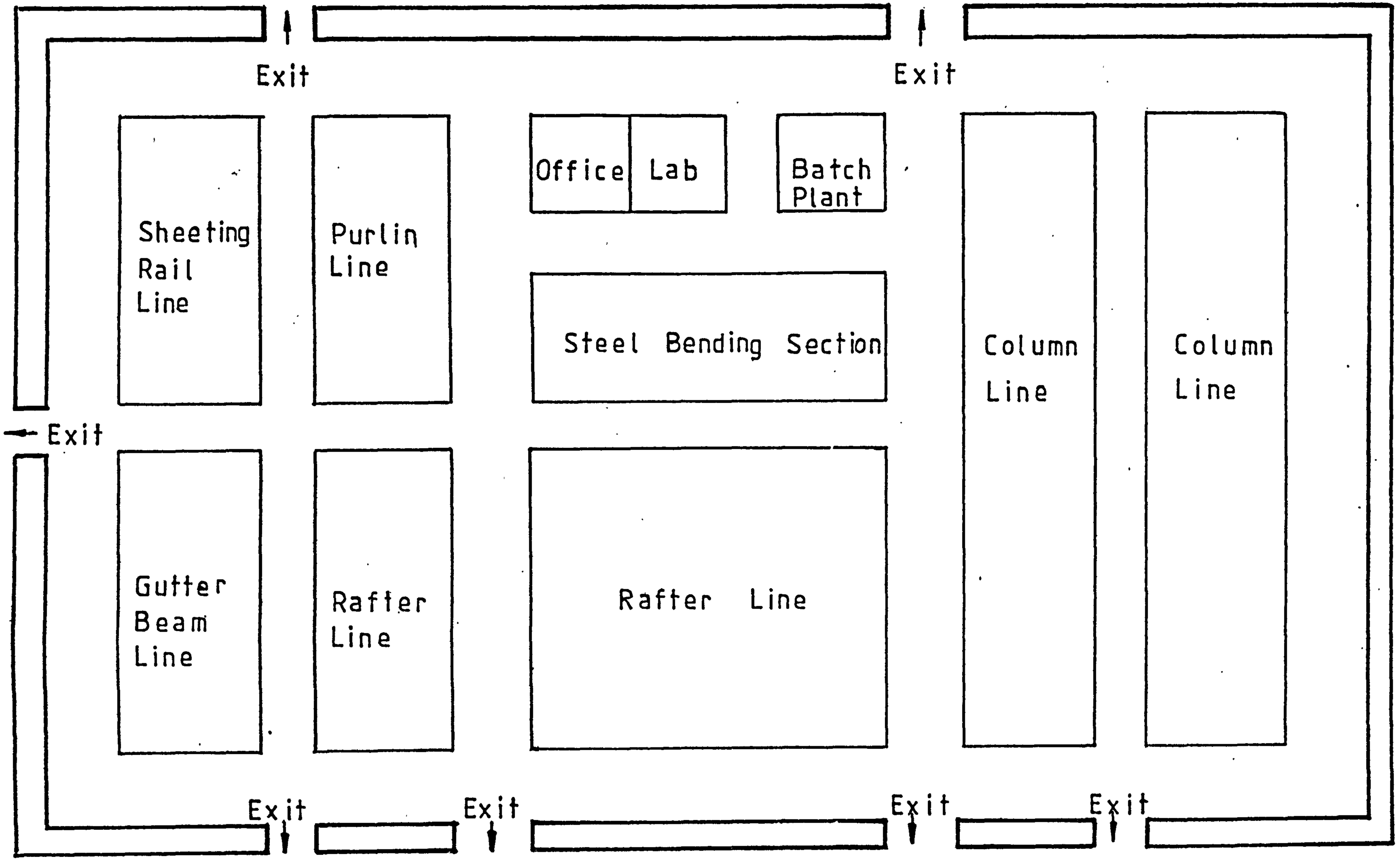


FIG 2.08 LAYOUT OF A TYPICAL FACTORY

2.5 FABRICATION METHODS

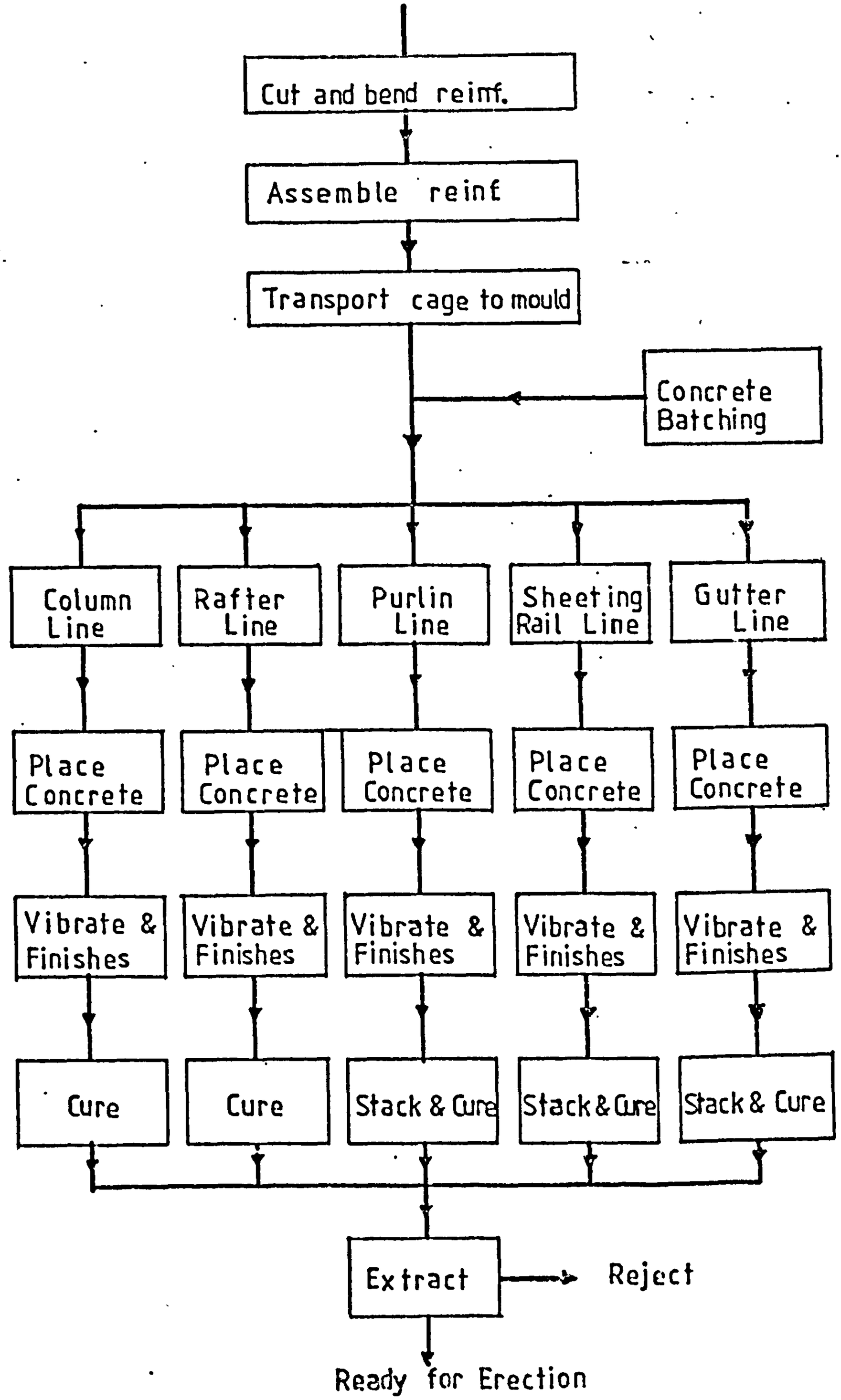
Methods of fabricating precast concrete units are largely dependent on the extent that mechanisation is used. Traditional techniques are still in use but with future higher demands they will give way to more modern methods. Fig 2.08 shows the layout of a typical traditional precast concrete factory specialising in portal frames. For best results each of the elements described in Section 2.2 are fabricated on seperate production lines. Column and rafter moulds tend to be the heaviest and must not preferably be moved after concrete is placed. They will therefore generally require more production space whilst the relatively lighter purlin, sheeting rail and gutter beam moulds can be stacked after concrete is placed and so make room for fresh placing.

2.5.1 PRODUCTION LINES

Fig 2.09 shows a chart describing the processing of various structural units. A central concrete batching plant is located in a convenient position. As with other manufacturing processes, separate work gangs are retained for each of the following lines:

1. Columns
2. Rafters

FIG 2.09 FABRICATION PROCESSES FOR STRUCTURAL COMPONENTS



3. Purlirs

4. Sheeting and glazing rails

5. Eaves and valley gutters

6. Steel cutting and bending

The following activities are commcn tc line 1-4:

i. Prepare mould to take steel cage and concrete

ii. Place concrete

iii. Stack mould (lines 1 and 2)

iv. Demould element.

Usually concrete is allowed overnight befcre demculding takes place.

The major difference between traditicnal and mcdern highly mechanised factories is in the concrete batching. Modern methods take advantage of mechanisation tc allcw interaction between the productien lines and the central batching plant. Concrete can thus be requested frcm a productien line by merely pressing a button.

2.6 HANDLING AND ERECTICN TECHNIQUES

Member components of the structure have tc be transported from the factcry to the erection site. It is therefore certain that structural members will be subjected to berding and torsional

stresses which though taken care of at the design stage must be limited to tolerable levels. Ideally, members should be transported in the same manner as they will eventually be fixed in the structure. That is beams to be laid horizontally and columns vertically. Since this is not always practical to achieve, safety precautions must be used to ensure no structural damage is done to the members at the transporting stage. Some of these precautions are :

1. Optimum curing must be achieved in the factory possibly before hauling begins.
2. Special inserts should be included to facilitate lifting.
3. Members must be tied securely to the transporting vehicle so as to reduce relative movements.
4. Members must be unloaded at strategic and convenient parts of the site to reduce site handling costs.

Erection is usually undertaken by either a proprietary fabricator or a specialist Contractor. A typical erection team would consist of a gang of four including a crane operator. Access roads must be provided and the site must be capable of admitting a mobile crane. Structural members and fixing accessories stacked at the site must not impede the crane's movements.

Usually, column foundations associated with this construction can withstand up to 150 kN/m2 and can be readily constructed from timber moulds. Column bases are unreinforced

except in special circumstances. At least one day must be allowed for these bases to cure sufficiently before proceeding with column erection. The entire erection procedure is summarised below ;

1. Pitch main frame columns and check for their verticality using a spirit level. Use wooden wedges to provide temporary stability and remove before grouting column bases
2. Gutter or valley beams are fixed to columns at the eaves after at least two columns have been pitched
3. Sheeting and glazing rails are bolted to columns preferably after eaves members have been fixed
4. Erect rafters spanning the width of the structure between columns. To prevent local damage to haunches whilst the portal action is still incomplete, props and crane support must be provided in the intermediate stage until whole portal frames are fixed.
5. Purlins are usually the last structural components to be fixed in the framework.

2.7 CONCLUSIONS

The structure that has been selected for this study is of enormous economic importance. Data presented in this chapter shows that despite worldwide economic depression there is a reasonable growth in demand from both private and public

sectors of the economy for the structure. It can also be seen that all aspects of the construction of such a building are readily definable and are therefore amenable to computer aided design.

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CHAPTER 3

STRUCTURAL OPTIMIZATION

3.1 OVERVIEW

One of the aims of this study is to seek algorithmically, an optimum design for the structure at each design session. In the first section of this chapter some recent advances in the field of structural optimization are reviewed. The rest of the chapter explains how an optimization routine is linked with the cost model (described in Chapter 6) to search for an optimally designed structure.

3.2 STATE OF THE ART

The application of optimization methodology to structural design has been pursued rigorously in the past twenty-five years. During this period a measure of success has been achieved in some areas of the subject though this level of success has not resulted in any major practical application of optimization methodology in the design office. In fact, to date optimization procedures remain a subject for research and not a readily available design tool. Five reasons will be identified here for the slow rate at which optimization is lending itself to the practical designer :

1. Most optimization algcrithms have been evolved by mathematicians whc have little or nc appreciaticn for design complexities. Such algorithms need to be tested and possibly tailored to suit structural prchlems by ergineers. This transfcrmation is so far lacking.

2. Successful applications of optimizaticn prccedures that have so far been repcrted have mainly been concerned with fully stressed designs that use minimum weight as the cptimality criterion. These applicaticns tend to investigate loading configurations on structural assemblages using steel as the construction material. Whilst acknowledging their usefulness, it is also true that a lot of the engineer's time is consumed in designing structural elements. Cptimum design of structural elements which can be converted into structural assemblages seems a far mcre useful approach.

3. Steel has received more cf the optimizaticr researcher's attentien than structural concrete. This is inevitable since structural concrete design poses larger parametric prcblems than steel design and as such compcunds an already complicated process.

4. Structural design is complex. Any auxilliary tool must seek to simplify this procedure. Sirce formulation and development of optimization procedures could often be tedious and complex many prospective users are naturally discouraged.

5. Minimum cost should be the objective of every optimization project. Some of the repcrted applicaticns have ignored impcrtant ccst factors such as fcrmwork and construction costs. This casts doubts as to whether such prccedures represent true

first costs. A simplification of this nature is often justified by its ability to fit the mathematical format of available optimization techniques at the expense of real structural costs.

Literature on structural optimization continues to grow. The following sections comprise a chronological description of the more significant structural optimization application.

1. An application of optimization procedures to the design and fabrication of steel portal frames was among the first practical attempts reported in 1973. In this case study Knapton[9] used selective fabrication methods and British Steel Corporation price lists to develop a cost model. The objective function was formulated from six primary variables and design requirements were used as constraints. A Simplex algorithm was used to drive the optimization process and the inclusion of fabrication details such as welding costs in the model makes the results significant. Savings between 5-10% were reported as compared with contemporary design methods.

2. Lane and Harriman[11] (1975) used optimization methods in the design of a real civil engineering project. The first structural element to be optimized was a simple reinforced concrete column base. Forming the objective function from material costs, and constraints from ground pressures and shear stresses, two algorithms were employed to search for an

optimum design. First Rosenbrock's algorithm and then a nodal search procedure were employed both of which yielded identical results. The nodal search technique was further used to design a rectangular reinforced concrete service reservoir. Relevant loading conditions were considered and column spacings of between 4.5 - 5.5 m were recommended.

3. Structural problems requiring discrete element sizes often pose problems in optimum design. The technique of assuming a continuous spectrum and rounding off the optimum solution to the nearest discrete value reduces the validity of the final optimum value. Lipson and Gwin[13] (1977) presented a minimization technique for a space truss using design variables that specify both geometry and member sizes. The objective function includes structural weight, fabrication costs and construction costs. Constraints are formed from allowable stresses, Euler buckling criteria, joint displacement, and discrete member sizes. The member sizes were selected from a conventional table in accordance with American code requirements. The optimization procedure was based on the Complex Method of Box which requires random generation of initial feasible designs. A 25 member truss was designed initially for an optimum geometry. Two separate materials were used. First aluminium tubes with a nominal diameter-to-thickness ratio of 100. A typical optimal design reduced an initial weight of 375 kg to 83 kg after 33 iterative changes. Secondly, steel angle sections were used. A typical design reduced an initial design from $1189 to $546. Thirdly, the

steel angles were used with variable truss geometry, which resulted in substantial savings.

4. Optimization of a structural element was reported in 1977 by Chou[3] using a reinforced concrete T-beam. The moment reinforcement and concrete costs formed the objective function but stirrups and distributors were ignored. Lagrange multipliers were used to constrain the problem. The remarkable result of this work was to demonstrate that an optimum cross section could correspond to an optimum design.

5. A rigorous optimization of cylindrical shells has been reported by Hartmann[6] (1978). No restrictions were imposed on either the nature of the variables or the constraints. A direct search method so called an 'evolution strategy' was used. In this strategy convergence is achieved by continuously optimising the step length of the search procedure. The objective function was formulated from a bending moment expression for elemental strips and constraints were formed from :

i. Stability conditions

ii. Limits of the approximation model

iii. Constructional and geometrical requirements.

The strategy appears to be powerful with complex structural problems and results for a single cylindrical shell without edge

beams showed an improvement of 33.3% upon the initial design. Promising though the technique appears to be its usefullness can ultimately be decided when it is applied to structural problems requiring discrete elemental sizes.

6. A recent approach (1979) adopted by Kirsh and Moses[8] particularly promises to suit large structural systems. A modal coordination technique is used to decompose a total optimization process into smaller subprograms. Each subprogram is solved independently at one level and then coordinated at another level. Two cases are considered, one a 4-span continuous beam and the other a 25-bar plane truss. In the first example, the problem is tackled at two levels. Stiffness formulation is used to distribute load and deformations. At the first level five continuous variables and four constraints are selected such that at the second level when the coordination variables are determined, the first level solution exists and the minimum weight objective function is minimized. In the second problem, flexibility formulation is used and the problem is decomposed at two levels with 11 and 12 independent variables respectively for the first and second levels. Again minimum weight is used as the objective function. Coordination variables are held constant during the first level optimization whilst all constraints are separated. Like most optimization algorithms the decomposition technique finds its strength in proper problem formulation and particularly in the analytical methods adopted.

3.3 STATEMENT OF THE PROBLEM

The purpose of this optimization procedure is to seek a feasible optimum cost design of the building. In Chapter 6 this cost function will be shown to comprise :

1. Fabrication cost
2. Haulage cost
3. Foundation cost
4. Erection cost
5. Cladding cost

A number of design variables were encountered but for practical purposes only six primary variables were isolated, namely :

x	span of the structure
y	frame spacing
n	number of bays
k	number of spans
W	roof pitch
h	height to eaves

In terms of these six primary variables the global cost of the structure can be determined. I.e the objective function is formally stated as

$$\text{Minimize } f(x,y,n,k,w,h) \qquad \text{Eqn. 3.01}$$

Eqn. 3.01 is subject to design constraints. Two classes of design constraints are identified :

i. Analytical constraints

ii. Cost and design constraints

The analytical constraints are built into the design algorithm whilst the cost constraints are essentially the client's requirements. During the field survey it was found that client's requirements can be restricted to the following primary variables :

1. Building area

2. Height to eaves

Therefore the objective function (Eqn. 3.01) is subject to inequality constraints of the form :

$$g_j(x) \geq 0 \qquad j \in J \qquad \text{Eqn. 3.02}$$

3.4 CONSTRAINED NONLINEAR MINIMIZATION

Most structural design problems are nonlinear with constrained optima. A number of algorithms have been used to evaluate different problems and in each case an appropriate justification is offered for the choice of algorithm.

Rosenbrcck's algcrithm has been used in the present work. The algorithm in this case is selected for two two reascns :

1. Being one of the picneer methcds it has been tested and found to be powerful in several engineering problems.

2. Being a search technique it can be ccrvenierttly attached to a model without destroying its internal structure. This factor was particularly crucial in this case where the mcdel would be required to be interfaced with other interactive mcdes (Chapter 6).

Rcsenbrock's algcrithm is a sequential search method which does not require evaluation of derivatives of the objective functicn. It assumes a unimodal function and can be used to find the minimum or maximum cf a multivariable nonlinear functicn subject tc ccnstraints.

The algorithm prcceeds by defiring a startirg point and several step sizes. Several sets of starting points are necessary if the surface is unknown cr if lccal cptima exist since the algcrithm assumes a unimodal functicn. Assuming N mutually orthonormal direction vectors, step sizes are used tc evaluate the objective functicn, at the same time the variables are stepped depending on whether the mcve is a success or a failure. In the event cf a successful move the step size is increased by a factor but if the mcve is a failure, the step size is decreased by an appropriate factor . The next variable is stepped at a distance parallel tc its axis. Other variables are similarly stepped until success or failure are encountered

in all N directions. The axes are then rotated for a fresh search using the new coordinates until a boundary zone in the vicinity of the constraints is ertered.

The original Rosenbrock algorithm is modified to account for the presence of constraints. After each perturbation, the current best objective function B1 where the constraints are satisfied and B2 the current best objective function where the objective function is satisfied and the boundary zone is not violated are defined. At the start, both B1, B2 are set equal to B0 the current objective function. After each subsequent perturbation, the values of B1 and B2 are compared with B0. Depending on whether the perturbation is a success or failure, B0 is modified.

Rosenbrock did not incorporate a convergence criterion but suggested that when the magnitude of the stage process is less than a specified value or else after a number of function evaluations the search can be terminated. Function evaluations in this case were limited to 100 runs. Contour graphical interface with the cost model (Chapter 6) could be used to confirm that a global optimum has been attained. Fig 3.01 illustrates the optimization process.

3.5 CONCLUSIONS

Optimization processes could be useful to the designer.

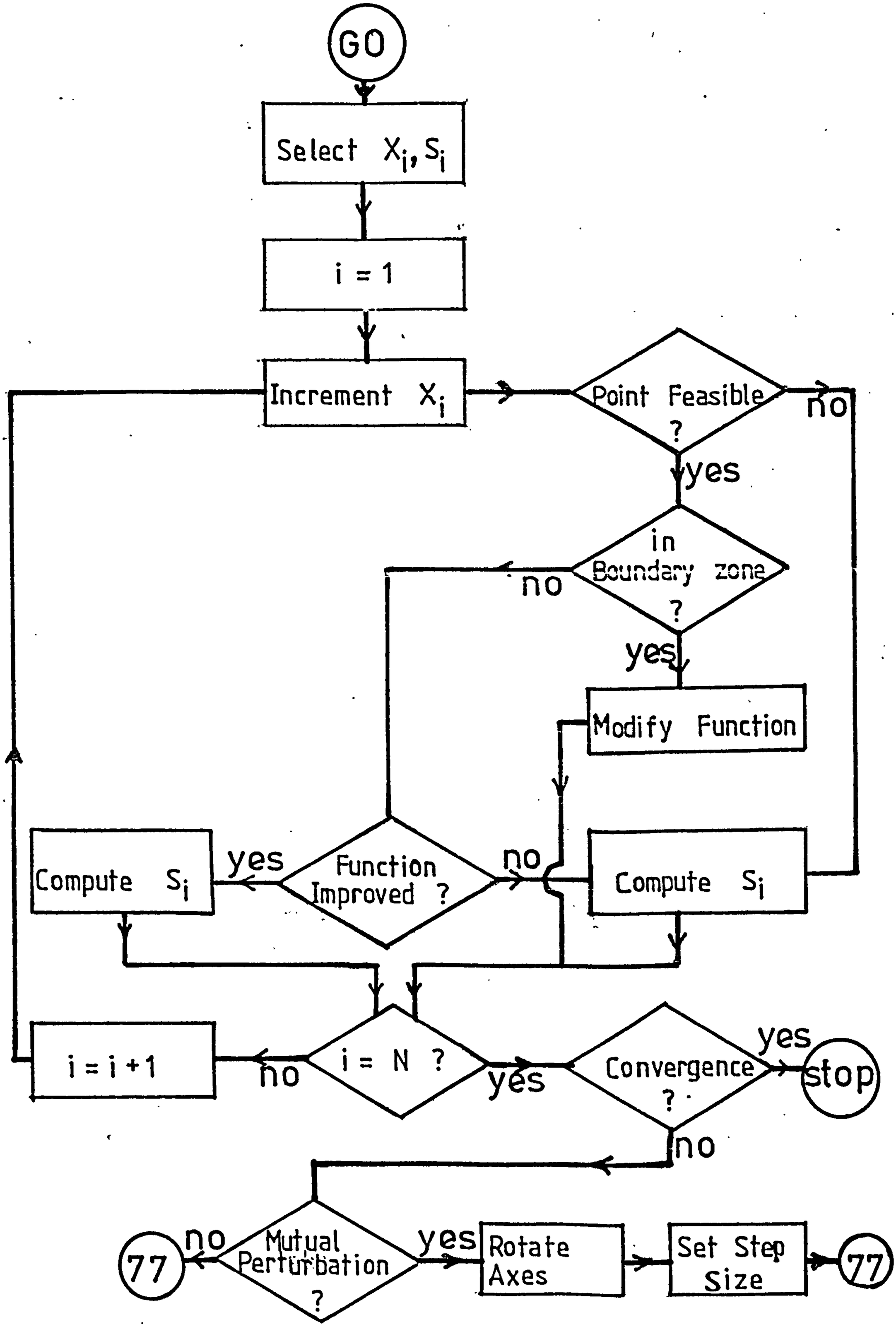

FIG 3.01 CONSTRAINED ROSENBROCK ALGORITHM

Wherever pcssible, all dependent factors that affect the cost of the structure must be incoporated in the problem formulation. structural problems must not be reduced to linear functicns cnly to suit an available mathematical format. In this chapter, the optimizatior problem is fcrmtlated to inccpcrate a practical size cf design variables. The objective function includes all deperdable ccst factcrs whilst constraints are formed from design and client requirements. A tested algorithm is then employed to drive the optimization prccess.

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CHAPTEF 4

INTEFACTIVE STRUCTURAL DESIGN

4.1 OVEFVIEW

The desigr cf elastic framed systems is an cld prcblem. In the first section of this chapter, a review cf historical develcpments in elastcmechanics is given and the origin of modern concepts is investigated. The strucure is designed on the ccncept of limit state and in this chapter this design philosophy with particular reference tc the present work is outlined. It is alsc shown how various connexions of the elements are designed to form a stable structure.

4.2 ELASTIC METHOIS

Principles of elastic approach to structural design have been formulated in many forms by varicus engineers and scientists over the last three centuries. Some cf these efforts are chronologically reviewed below :

Acknowledging the importance of earlier works by men like Galileo and da Vinci, the first notable work cn extremum principles cf elastomechanics can be dated to 1662 when the

Dutch scientist Huggens investigated the limiting strength of rigid beam in flexure. His principle was extended by Daniel Bernoulli in 1738 to flexible elastic beams. Berrculli in fact, formulated an extremal problem which linearised Jacob Bernoulli's equation of flexure. Twelve years later, Riccati proposed a hypothesis that the kinetic energy that deforms a solid mass and creates an internal system cf forces within the solid can be retrieved and used in partial restoration of the deformed solid. Though he was unable tc provide any rigorous prcof cf this, Clapeyron applied his concept ir 1827 tc establish the Conservation of Wcrk Principle based cn Virtual Work which was later published in Lame's bock on the theory of elasticity. His theorem reiterates Ricati's speculation that the wcrk done W can be expressed as in Eqn. 4.01 :

$$W = 1/2 \sum FiDi \qquad \text{Eqn. 4.01}$$

where Fi denotes the force at the pcint i ard D is the component of displacement of that pcint in the direction of the force. The strain energy U of the solid is represented by the integral in Eqn. 4.02 :

$$U = 1/2 \iiint \{ i/E\,(Vx^2+Vy^2+Vz^2) - 2b/E\,(VxVy+vxVz+vyVz) + 1/G\,(Ixy^2+Ixz^2+Iyz^2) \}dxdydz \qquad \text{Eqr. 4.02}$$

According to Clapeyron W=U

Merabrea, an Italian engineer, analysed elastic frames

with redundant internal members and proposed in 1857 the concept of least work by arguing that the forces F1, P2, P3, etc. acting on the redundant members are such that the strain energy is minimum, i.e.

$$dU/dF1=0, \qquad dU/dF2=0,$$

Eqn. 4.03

$$dU/dF3=0, \qquad dU/dF4=0$$

The English engineer James Cotterill in 1865 published the first valid proofs of the Menabrea Theorem and contributed immensely to the Reciprocal Theorems. In his proof of the Menabrea Theorem, he applied Clapeyron's Work Principle suppor ted by Euler's theorem of homogeneous functions.

Castigliano, the famous Italian engineer, published in 1857 his theorems on elastic structures. By considering framed trusses whose members are joined by frictionless hinges he showed that the strain energy of such a configuration can be expressed as in Eqn. 4.04 :

$$U = 1/2 \sum[F_m^2 L_m/EA_m] \qquad \text{Eqn. 4.04}$$

Applying Clapeyron's theorem,

$$1/2 \sum_{i=1}^{N} [PiDi] = 1/2 \sum_{i=1}^{M} [F_m^2 L_m/EA_m] \qquad \text{eqn. 4.05}$$

where N and M are respectively the number of loaded nodes and

members in the structure. Castigliano proceeded further to prove two related theorems. First he considered that infiritesimal changes in forces and displacements result in direct changes in strain energy.

I.e.

$$dU = \{dU/Di\}dDi$$

Equating this to the external work done,

$$\sum\{dU/dDi\}dDi = \sum FidDi$$

i.e. $$Fi = dU/dDi \qquad \text{Eqn. } 4.06$$

Similarly,

$$Di = dU/dFi$$

Arising from Castigliano's work the so called energy thoerems (Eqn. 4.07) can be derived for both statically determinate and hyperstatic structures :

$$U = \int\{M^2/2EI\}ds$$

$$\{dU/dF\} = \int\{M/EI\}\{dM/dF\}ds$$

$$\{dU/dMx\} = \int\{M/EI\}\{dM/dMx\}ds$$

Eqn. 4.07

4.3 LIMIT STATE DESIGN

Limit state design philosophy has been evolved around a concept that a structure designed on that basis must always be safe to its users during its design life. Since design of the structure takes into account properties of materials and expected loading regimes, the problem must be approached from statistical viewpoint as far as material properties and anticipated loadings are concerned. This will involve variation of material properties and loads using characteristic values and partial safety factors.

Consider the case of a specific loading configuration on the structure. In its design life time, an increase in this load will deform the structure. A characteristic value (Fk) of the load can be given as in Eqn. 4.08 :

$$Fk = P+ks \qquad \text{Eqn. 4.08}$$

where

p is the mean load

s is the standard deviation for the load

k has a value of 1.64

The strengths of the two principal materials used to build the structure (that is concrete and reinforcement steel) will deteriorate during the expected life of the structure. Similarly, the characteristic strength of a concrete cube (Fcu) can be defined as in Eqn. 4.09 :

$$Fcu = S - ks \qquad \text{Eqn. } 4.09$$

where

S is the mean cube strength

s is the standard deviation for concrete

k has the same value of 1.64

It follows from the value of k that only 1 in 20 cube strength is likely to fail.

Characteristic values are not the appropriate design values. In the case of concrete, for instance, other factors such as non-uniformity in the compaction of wet concrete, presence of impurities in the formwork, cement quality, grading and moisture levels in aggregates have to be taken into account. This leads to the concept of partial safety factor (Gm). For a concrete cube design strength is defined as in Eqn. 4.10 :

$$\text{Design Strength} = F / Gf \qquad \text{Eqn. } 4.10$$

where m has a value of 1.5 for concrete and 1.15 for reinforcement Similarly for the load Eqn. 4.11 can be deduced :

$$\text{Design Load} = Fk * Gf \qquad \text{Eqn. } 4.11$$

The concept of characteristic strengths and partial safety factors is the cornerstone of the two main limit states - the

ultimate limit state and the serviceability limit state. Current design practice is to establish a critical limit state that the particular structure is likely to reach and then to proceed to demonstrate that other limit states will not be reached before this critical state.

4.3.1 ULTIMATE LIMIT STATES

This is associated with the maximum load that the structure can withstand (both design and accidental) and the stability of the whole structure under these loads.

4.3.2 SERVICEABILITY LIMIT STATES

It is usually sufficient to account for deflection, cracking and sometimes vibration under serviceability limit states. The extent of analysis of each of these states will depend on the type of structure and the use for which it is intended.

A beam will deflect even under its own weight. Deflexions of members in the structure must however, not adversely affect efficiency and appearance. Span and deflexion ratios[3] below as - cast levels are limited to the following magnitudes :

1. for all general cases 250

2. where there are partitions and finishes 350 or 20 mm whichever is lesser.

Serviceability provisions take into account the effects of shrinkage, creep and temperature.

Flexural cracking should be controlled to acceptable levels again to ensure appearance and efficiency are not impaired. Crack widths are known to vary with loading and adverse environmental conditions. Cracks also expose reinforcement to corrosion. Crack widths are limited to 0.3 mm generally and to 0.004 as much as the nominal cover to reinforcement.

4.4 DESIGN OF THE STRUCTURE

In designing the structure the philosophy of limit - state is assumed. Individual elements are designed integral with the whole structure in such a way as to ensure the global safety of the structure under live, dead and wind loads. Steel reinforcement was preferred to prestressing for the following reasons :

1. Peculiar mould shapes, especially in the case of gutters and sheeting rails would have hindered effective prestressing methods.
2. During the field survey only one manufacturer was found to

be using prestressing methods so that the range cf information required for the study would have suffered if prestressing methods were used.

In crder to ensure that the design procedure was fully automated the use cf design charts was excluded and all formulae derived from first prirciples. Selectien cf discrete steel areas was incoporated in the design algorithm which is summarised as follows :

Firstly, anlysis routines estimate dead, imposed and wind loads, to be used to estimate berding moments and shear forces. Trial design of the various elements is then carried out. For a column, trial dimensions coupled with cther design data are input tc the program which computes and selects appropriate design bending moments. The prcgram tests fcr slenderness and then designs acccrdingly either for a short or lcng column case. In the process the neutral axis depth is varied and at each stage compressive and tensile steel areas are iteratively computed. There exists for each ccmplete iteratien an optimum steel area and a corresponding neutral axis depth. By using stored cost data, the least cost cclumn is designed.

Secordly, tc design a rafter using parabolic stress block, the program selects the maximum berding mcment alcng the rafter and tests for the positicn of the neutral axis in relation tc the beam's flange. Accordingly, a design is carried out for a neutral axis depth position either within or outside the flange.

Finally purlins, sheeting rails and gutters are treated as simply supported beams spanning between bays. As each element must be topologically compatible with the rest of the structure, tests are carried out at each design stage to ensure that a functional and stable structure is being designed. Frequently, incompatibility causes the program to reject otherwise feasible designs for individual elements (Fig. 4.01). In the event of a failed design, an option for data adjustment and improvement is offered. The design cycle is repeated as often as necessary until an acceptable design is reached where all constraints are satisfied.

All elements in the structure have to be transported from the factory to the erection site. Unlike in - situ construction the structural frame is erected by means of connecting various members securely with each other to form a stable unit. For instance, full continuity is required at the rafter/stanchion section which means that the connexion at the eaves must be designed to transmit bending moments, shear forces and torsion. Two important factors that must govern the design of connexions are :

1. A partial safety factor ten percent higher than adjacent members to be provided against failure to ensure that concrete members fail before the connexion.
2. Connexions have to be located at convenient sections in the structure since at certain sections in the structure stress concentrations can occur if connexions are used with consequent

spalling and splitting of the concrete at the contact surfaces. Structural member connexions are shown in the graphical output in Figs. 7.05 - 7.12

Corbels are used integral with connexions at the eaves where the rafter is connected to the column. Corbels are generally important in precast construction. About two decades ago, very little was known about their behaviour and they were only treated as a special case of a short cantilever. Pioneer works by Neidenhoff and Franz in 1961 suggested that the behaviour of a corbel can be likened to that of a simple truss composed of a horizontal tension member and an inclined compression strut. In 1965 Kriz and Raths[4] postulated four principal types of corbel failure :

1. Flexural tension - After extensive yielding of the main tension reinforcement, this failure occurs by crashing of the concrete at the bottom of the sloping face of the corbel.
2. Flexural compression - Before extensive yielding of the reinforcement has occured a flexural compression failure occurs when crushing of the concrete takes place at the bottom of the corbel. In this case the stress in tension reinforcement at failure is either below or just at the yield point.
3. Diagonal splitting - Following a well developed flexural crack pattern, a diagonal splitting occurs along a line extending from the bearing plate towards the junction of the sloping face of the corbel and the face of the column.
4. Shear failure - Development of a series of short inclined cracks along the plane of the interface between the column and

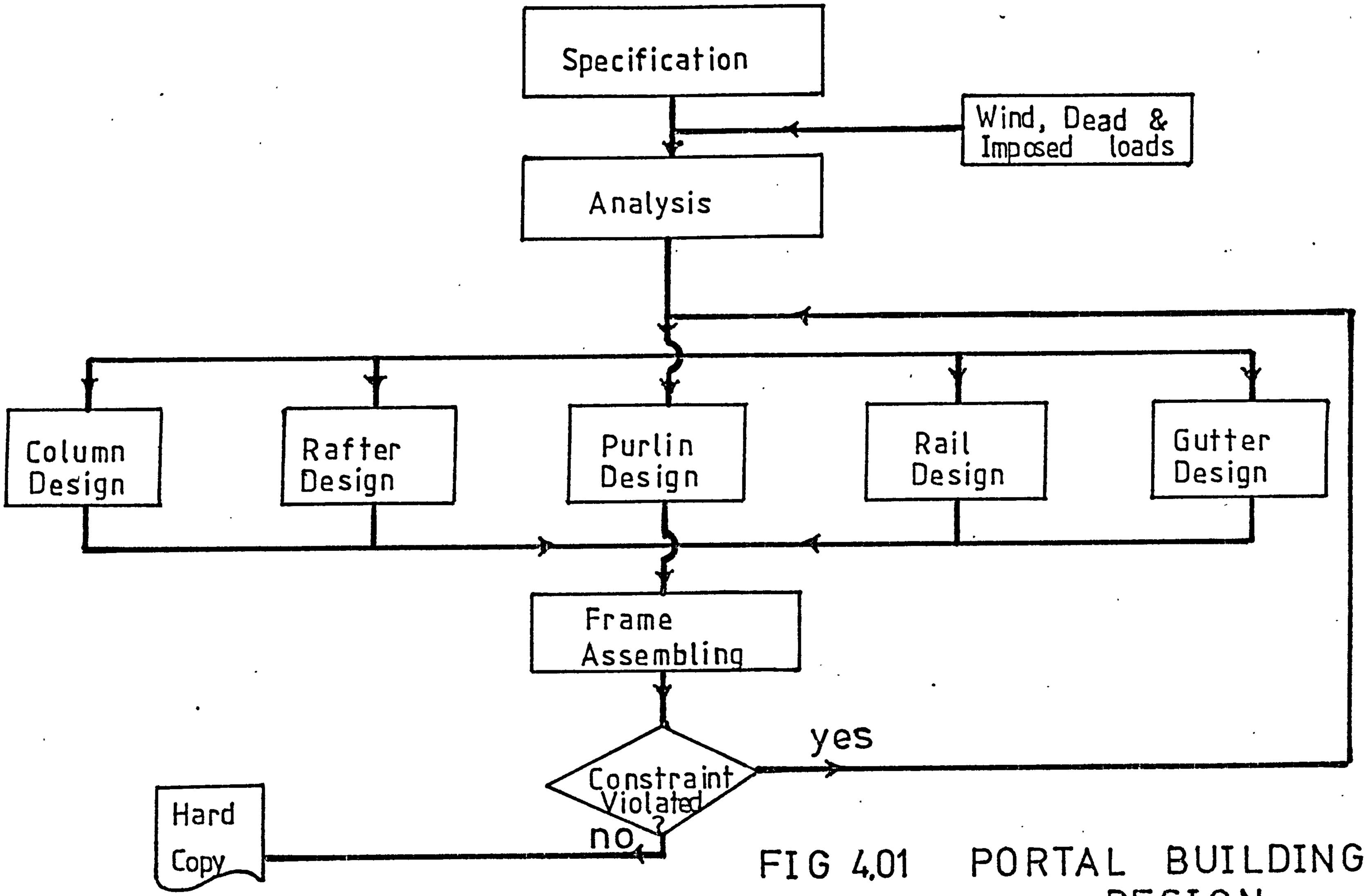

FIG 4.01 PORTAL BUILDING DESIGN

corbel constituted the shear failure.

These form the basis of corbels designed in accordance with CP110. In particular, it is recommended[3] that the distance from the centroid of the applied load on a corbel to the face of a sloping member should be less than 0.6d.

4.5 CONCLUSIONS

The limit state design philosophy has been used to design the structure. The routines for driving the design algorithm exclude any traditional practices that would impede the interactive process. Other than the main elements of the structure, design of connexions are also described.

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CHAPTER 5

DESIGN OF CLADDING SYSTEMS

5.1 OVERVIEW

Cladding forms an integral part of the structure. This chapter gives a guide to selection of cladding types and examines various design techniques. Current accessory models and how they are used to form durable external lining to the structure are discussed.

5.2 CLASSIFICATION OF CLADDING ACCESORIES

Cladding comprises all non-structural members forming a skin to the structure. The choice of type of material and cladding is related to the purpose for which the structure is intended (Chapter 2). Other factors such as durability, aesthetics and relative cost also influence the choice of cladding type. Materials used to manufacture cladding sheets and related accessories include asbestos, aluminium, steel and glass.

Most sheets are made from asbestos. Asbestos is robust, inorganic, resistant to rot and fungi but has low resistance to shock loading. In accordance with B.S.474 : Part4 : 1970 asbestos is non-combustible though without fire resistance as a structural material. When exposed to direct flame the sheets

are likely to shatter. It deteriorates in an environment with intense acid pollution.

Aluminium sheets are light and non-brittle, two properties that make them desirable as construction materials. They are non-combustible but have poor sound absorbing capacity. Copper, brass, and some metal fixings react locally with aluminium so fixings made from aluminium alloys must be used with aluminium sheets. Zinc chromatic primer, if used before fixing, will prevent corrosion from such construction materials as concrete and masonry.

Sheets made from steel are hard, non-combustible and resistant to impact loading. Since they are bad insulators, it is best to use them in sandwich construction (Section 5.4) together with insulation boards secured to the flutes of the troughing. Rust is prevented by painting the sheets with protective coating periodically.

The life span of a cladding material is difficult to estimate. A precise definition of the design life of cladding is a subjective judgement depending upon the type of structure, type of cladding, use of building and location of structure. It is convenient to conceive it as a 'serviceability limit-state' problem and to make the life span of cladding dependent upon that of the structural framework, that is the skin of the structure must have a design life longer than the structural framework.

Cladding items can be classified according to their

TABLE 5.01 : CLASSIFICATION OF RCCF CLADDING ACCESSORIES

ACCESSORY	MODEL
Lining Sheets	Bigsix, Trafford Tile, Doublesix, Doublesix 'M', Panel Sheets, General Purpose, Monad Type DC, Type SW6
Top Sheets	Standard Three, Standard Six, Canada Tile, Corripanel, Bigsix, Doublesix, Major Span River Banking, Doublesix 'M'
Translucent	Standard Three, Major Six, Major Tile, Type DC, Type SW
Flat Sheets	Self-Compressed, Flat Strips, Fully compressed, Asbestos Wood
Close Fitting	Standard Three, Major Six, Canada Tile, Major Tile
Cranked Crown	Grey, Thrutone, Standard Three, Monad, Canada Tile
Finials	Standard Three, Standard Six, Canada Tile, Major Tile, Monad, Major Six

TABLE 5.02 CLASSIFICATION OF SIDE CLADDING ACCESSORIES

ACCESSORY	MODEL
1. BARGE BOARDS	
Gableline handed & Stopends	Canada Tile, Standard Six, Monad
Standard Wings	Standard Three, Standard Six, Canada Tile
Monad Wings & Stopends	Monad
Doublesix & Doublesix 'M'	Grey, Thrutone, Natural Grey
Roll Top & Vergeline	Standard Three, Major Six, Major Tile
2. FILLERS & FLASHINGS	
Horizontal Z Flashing	Trafford Tile, Panel Sheet, Monad, Canada Tile, Bigsix, Standard Three Doublesix 'm'
Eaves Filler	Corripanel, Major Tile, Major Six,
Eaves Closure	Trafford Tile, Bigsix, Canada Tile, Monad, Standard Three
Underglazing Flashing	Panel sheet, Bigsix, Trafford Tile, Doublesix, Monad
Corner Pieces	Corripanel, Major Tile, Major Six

TABLE 5.03 CLASSIFICATION OF MISCELLANECUS ACCESSCRIES

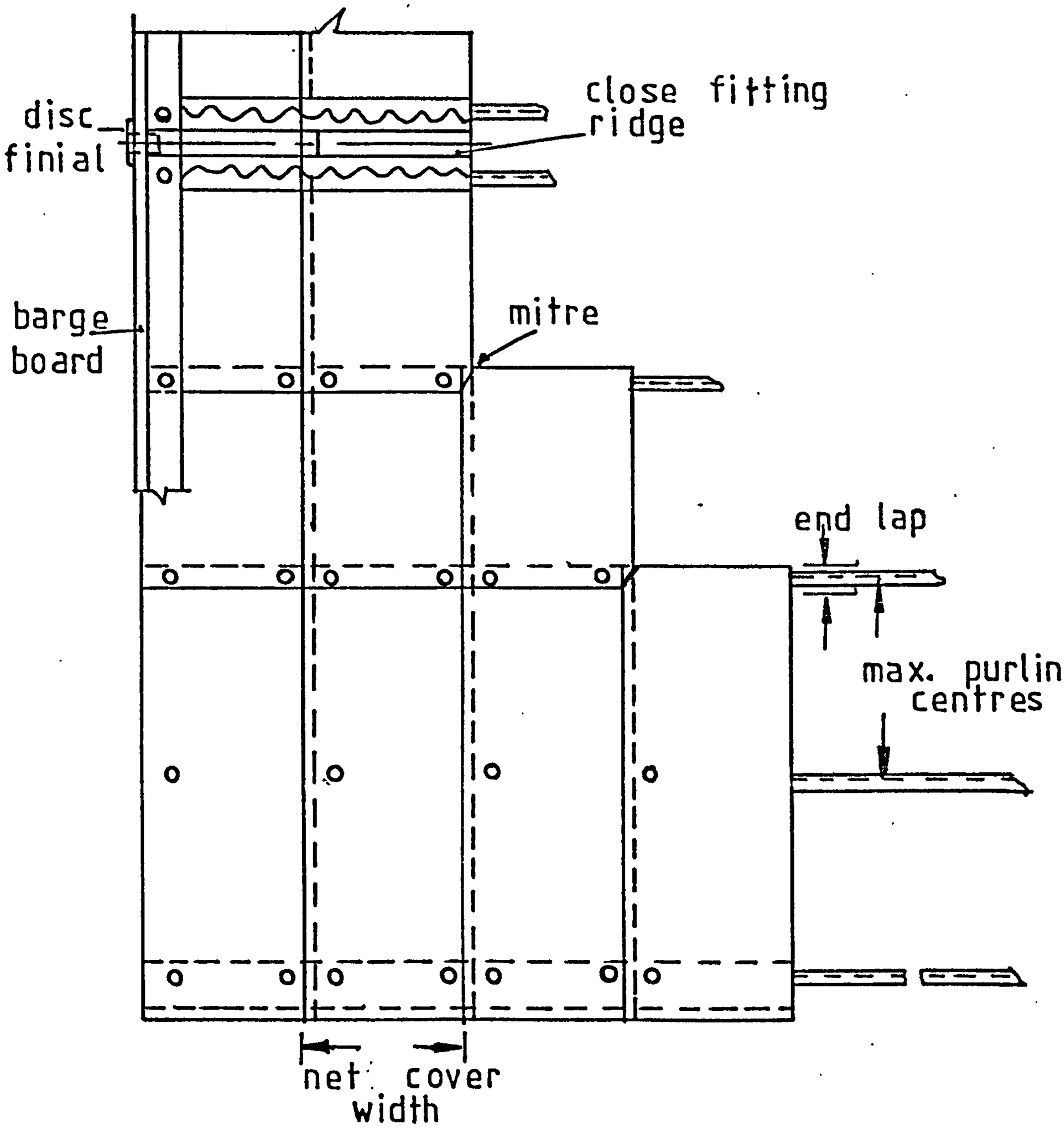
ACCESSCRY	MODEL
1. EXPANSION FITMENTS	
Movement Joint	Standard Three, Standard Six, Canada Tile, Majcr Tile, Dcublesix, Monad, Corripanel
Ridge Caps	Trafford Tile, canada Tile, Standard Six, Eigsix
Cranked Expansion Joint	Canada Tile, Standard Six, Monad
Stop Ends	Doublesix 'M', Dcublesix, Monad, Carada Tile, standard Six
2. Insulating & Spacers	
Insulation Board	TAC Limpet, Atlas
Rocksil Insulant	Canada Tile
Spacers	Cape Spacers

function in the overall cladding configuration. Tables 5.01-5.03 contain most of the accescries found in roof and wall cladding.

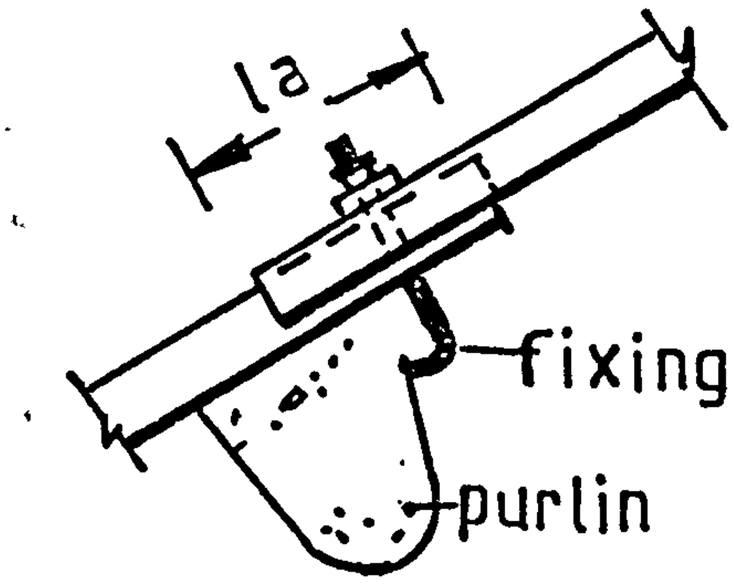
5.3 DESIGN OF ROOF CLADDING

There are two main ways of designing the skin of the structure - single skin construction and sandwich construction. Each of the two arrangements incoporates an appropriate ridge sheeting and movement joints. Choice of cladding type is related to the purpose for which the structure is intended; for instance a single skin constructuion will suit an agricultural storage building whilst a sandwich construction car be used for a factory structure.

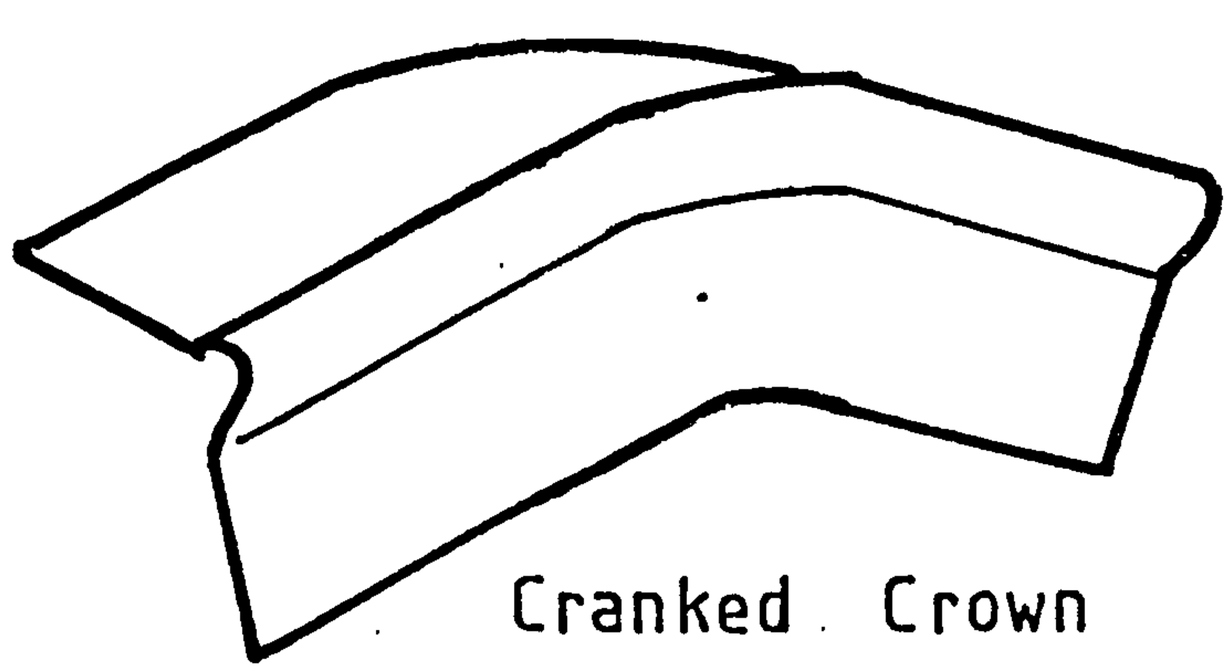
Laying of sheets is usually done commencing from one end of the structure, starting at the eaves and laying in tiers up the slope of the roof with smooth sides facing the weather, with one corrugation side lap forming an interlocking side lap with the next (Fig. 5.01). Mitre cuts about the width of the side lap are made to leave clean joints. Standard sheet lengths range from 1225 - 3050 mm.



Roof Layout



End Lap Detail



Cranked Crown Barge Board

FIG 5.01 ROOF CLADDING

5.3.1 SINGLE SKIN CONSTRUCTION

Single skin construction is simple and the least expensive. It is used in cases where insulation is of secondary importance. Average thermal conductitivity of this cladding type is in the range of 6 - 8 W/m2 deg C. Ridge cappings or circular sheetings may be used to close the crown of the roof (Section 5.3.3). For estimating the total number of sheeting (Figs 5.01, 5.02) Equation 5.01 is suggested.

$$TS = 2k[\,(0.5xCotw-R3)/(R2-R3)\,][\,1+(yn-W2)/W3\,]$$

Eqn. 5.01

If Equation 5.01 is used, the following assumptions must be observed :

1. A constant end and side laps must be used
2. Purlin spacing must be constart.

5.3.2 SANDWICH CONSTRUCTION

This method of providing a structure with a fully insulated roof incorporates sheets, associated lining panels and glass fibre or mineral wool. Average thermal conductivity ranges from 0.7 - 1.1 W/m2 deg C. Underlining panels and top sheets form a cavity that can accommodate insulant (Fig 5.03). This

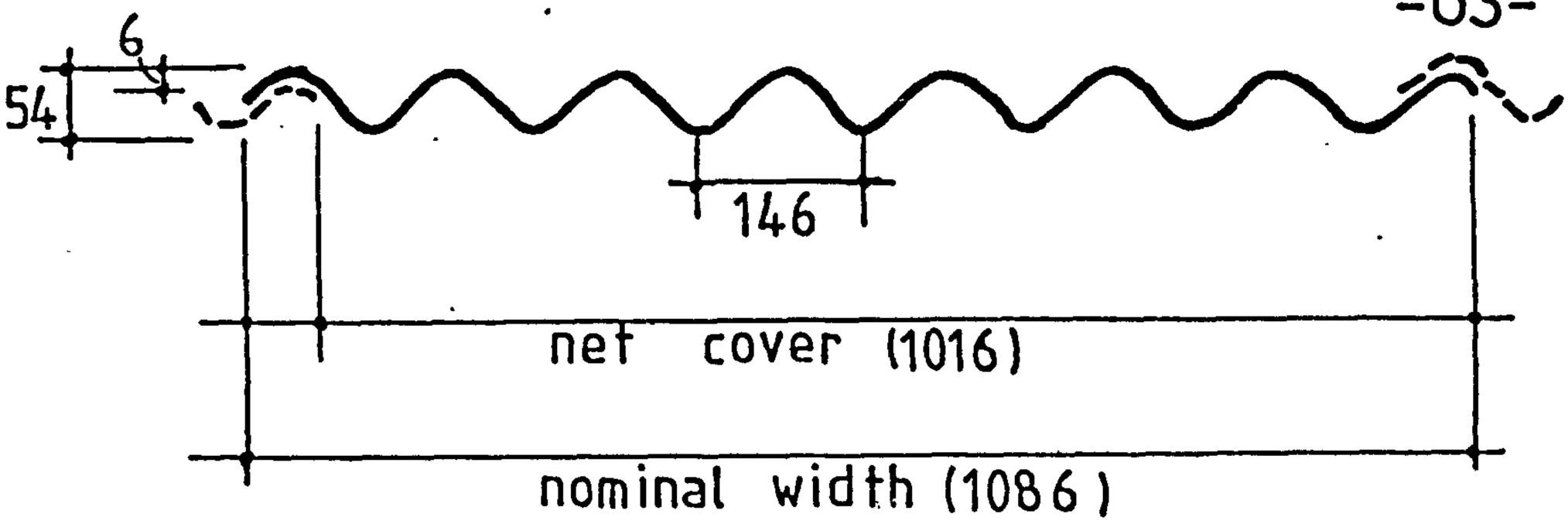
insulant reduces heat loss by restricting the volume of free air circulating in the cavity. Timber batters fixed above purlin lines are used in conjunction with the insulant to prevent the insulant consolidating with time. Transluscent lining panels may be used to provide insulated lighting. Equation 5.01 is relevant for estimating top sheets and lining panels. Area of insulating material is the total area of roof surface.

5.3.3 RIDGE FITTINGS

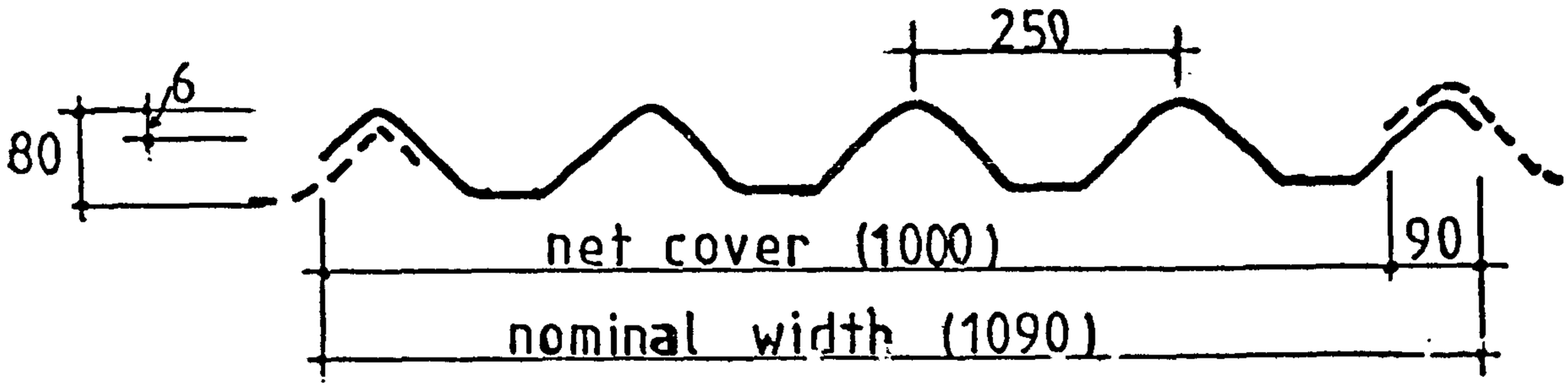
The apex of the roof is closed off with ridge pieces or crown sheets. Ridge pieces are adjustable to fit designed roof pitches. Each pair comprises a minor roll and a major roll. Their positioning in relation to the sheeting is crucial for efficient performance. To seal off ridge ends, finials are used. There are two types :

1. Disc finial which is bolted to the roll of the ridge
2. Hooded finial which is used in conjunction with barge boards and bolted to its flat wing.

Crown sheets are single units designed to match standard roof pitches. They can be lapped and mitred. When used, the ridge ends must be sealed off with cranked crown barge boards. For estimating the total number of ridges and ridge crowns in both single skin and sandwich constructions, Equation 5.02 is

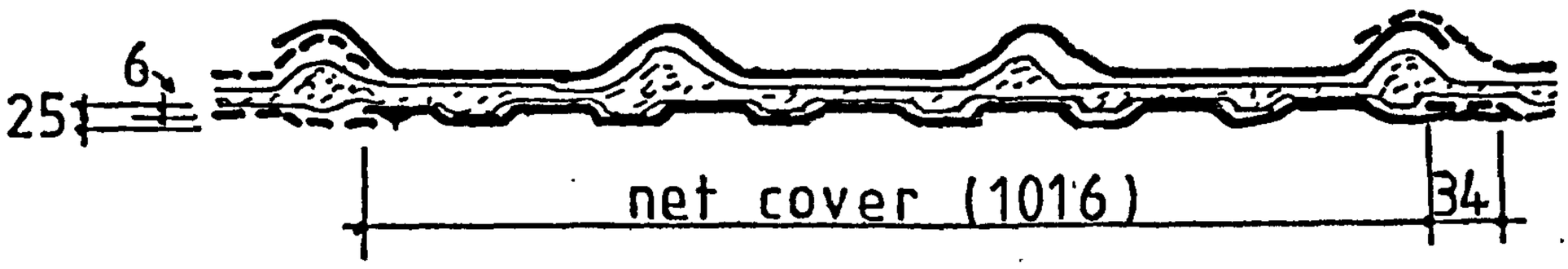


Single Skin with Standard Six sheets

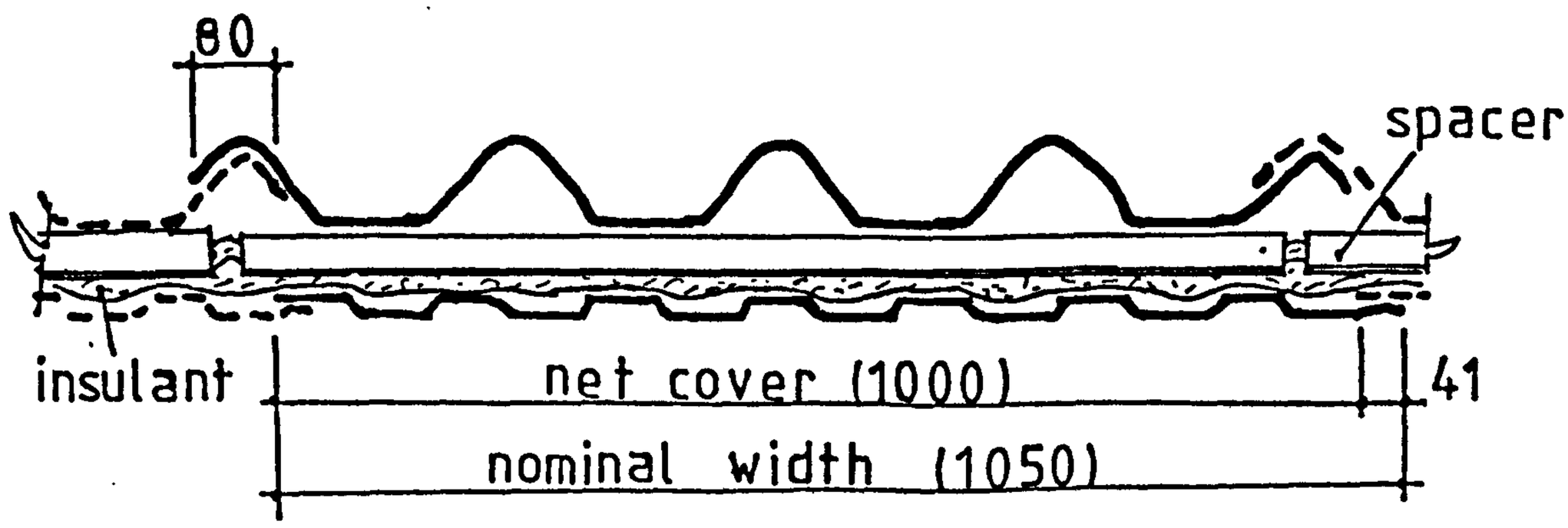


Single Skin with Monad sheets

FIG 5.02 SINGLE SKIN CONSTRUCTION



Sandwich with Canada tile



Sandwich with Spacer battens

FIG 5.03 SANDWICH CONSTRUCTION

suggested.

$$TR = ni[(yn-R1)/(0.5xCotw-F1)] \qquad \text{Eqn. 5.02}$$

where I equals 1 or 2 respectively if crown sheets or ridge pieces are used. The followings assumptions are made with regards to Equation 5.02 :

1. A constant side and end lapping is assumed
2. If ventilators or lighting are incoporated in the roof, some ridge pieces or crown sheets will be displaced.

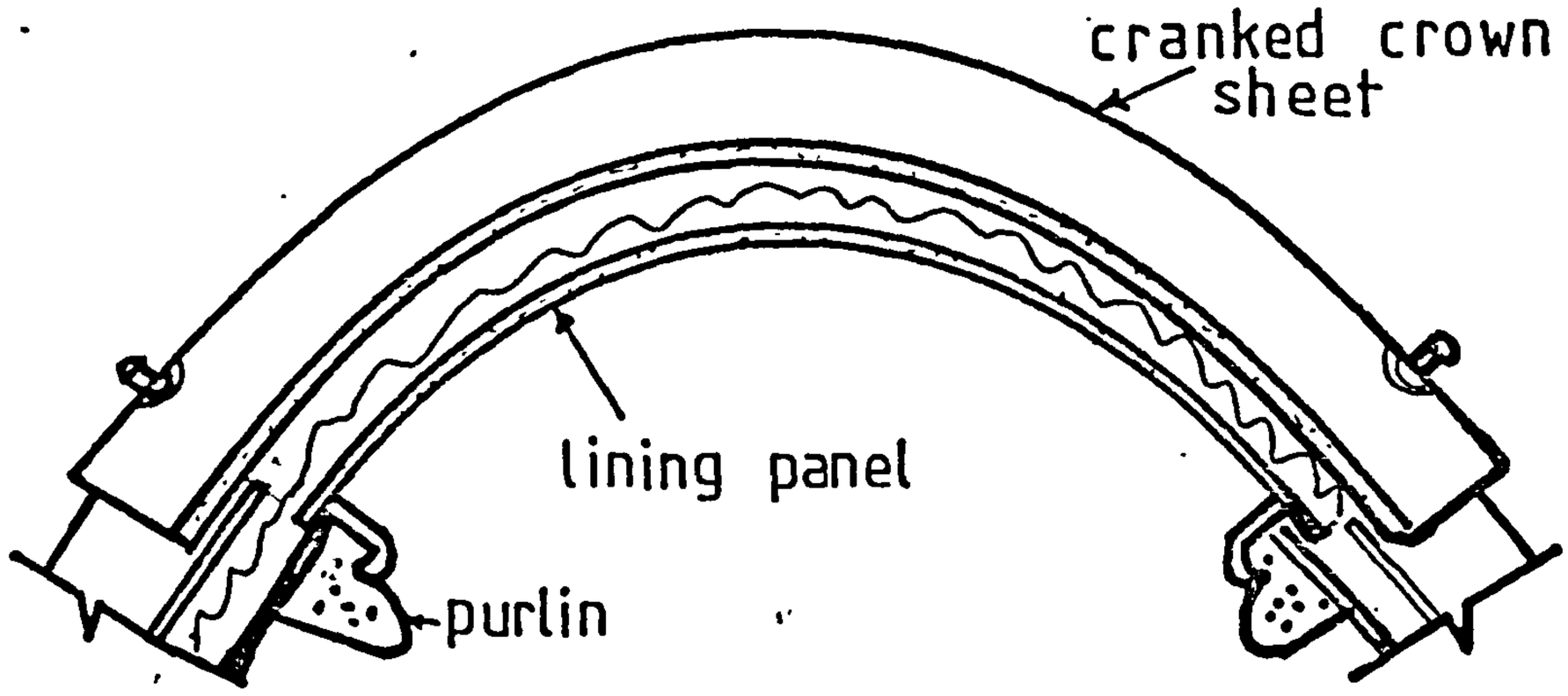
Some types of ridges are illustrated in Fig 5.04

5.4 DESIGN OF WALL CLADDING

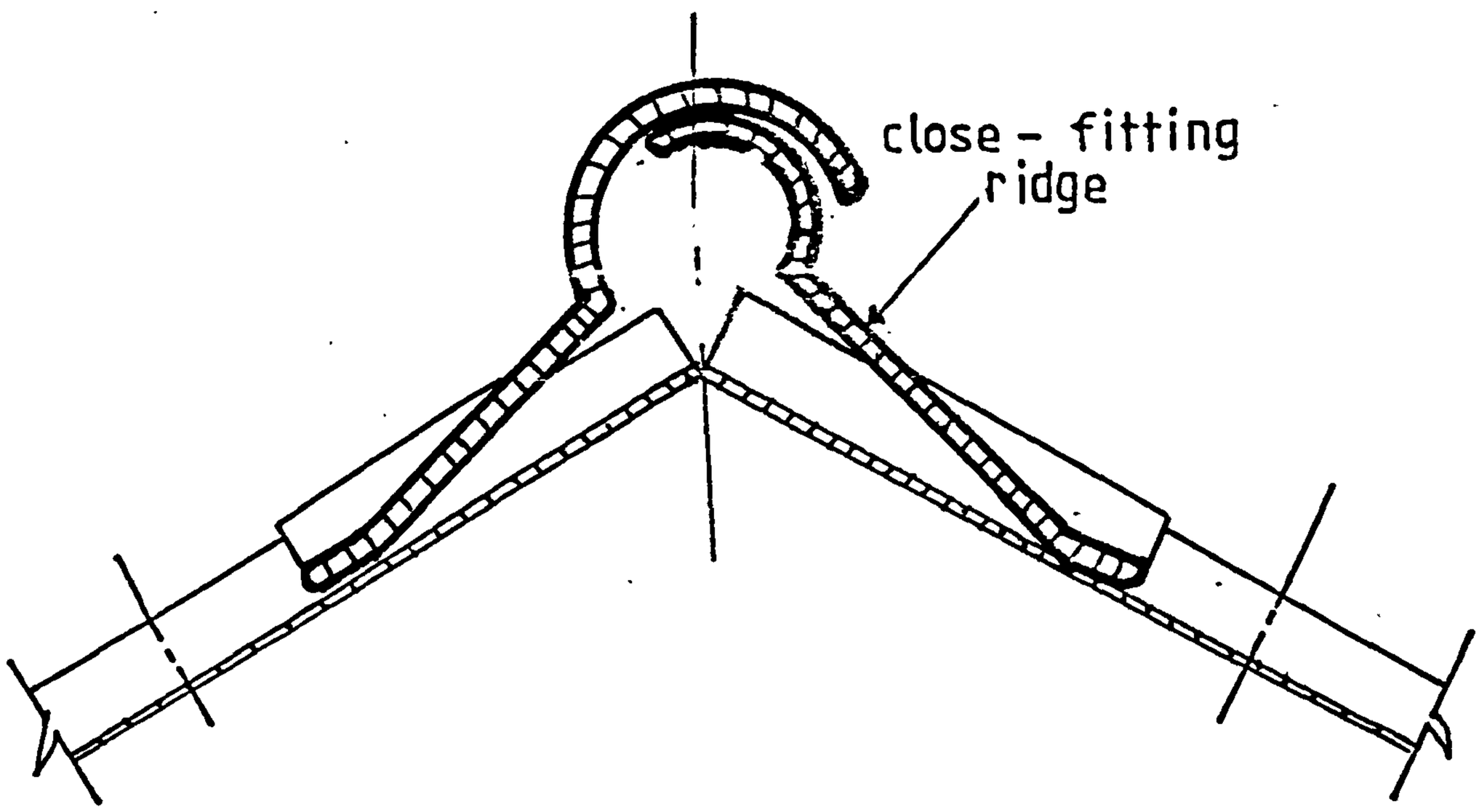
External lining of the structure is done with two main materials - sheeting and brickwork. Three common designs will be :

1. Sheeting only
2. Brickwork only
3. Combiration of (1) and (2).

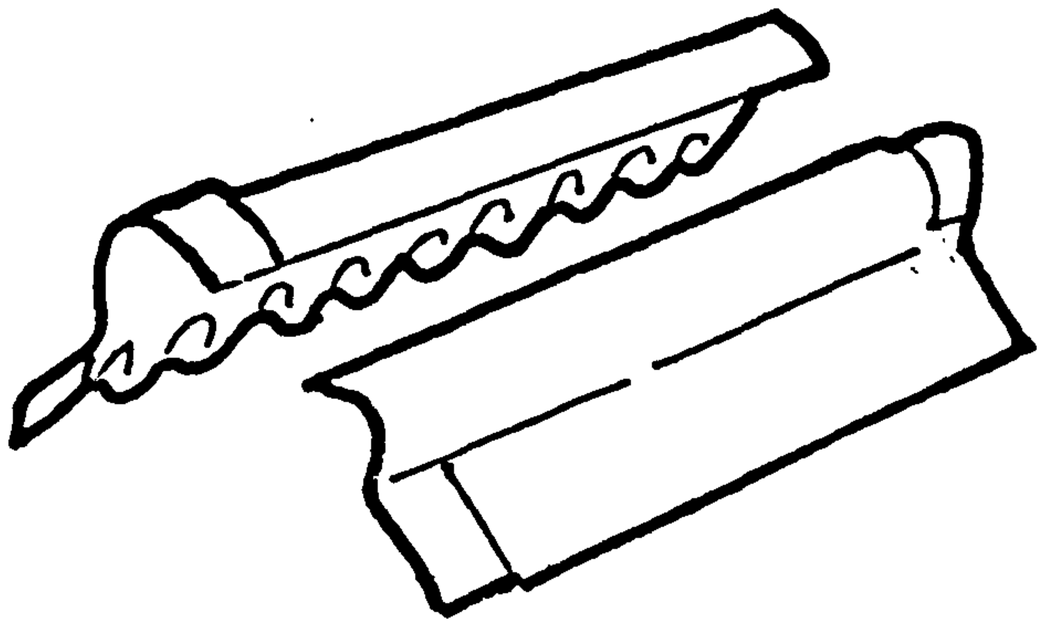
In practice, however, most 'sheeting only' designs will incoporate some form of masonry starters. Where sheeting only is used they could be either single skin or sandwich



Ridge Details for Sandwich Construction



Ridge Details for Single Skin Construction



North - Light Ridge

FIG 5.04 RIDGE TYPES

construction (Sec. 5.3)

5.4.1 VERTICAL CLADDING

The integration of sheets, fillers, closure pieces and verge fillers to form a rigid external lining is extensively used in industry. If single skin or sandwich construction is used the techniques are similar to those described in Section 5.3. Discrete arrangement of the sheets requires side and end lapping as well as mitring. Alternatively, horizontal flashing pieces are used after each course. The last course at the eaves level is closed off with eaves filler pieces to provide a continuous soffit to the roof sheeting course. External corner pieces are used to close off junctions of vertical sheeting at corners (Fig 5.05). For estimating the number of sheets the general expression suggested is Eqn. 5.03 :

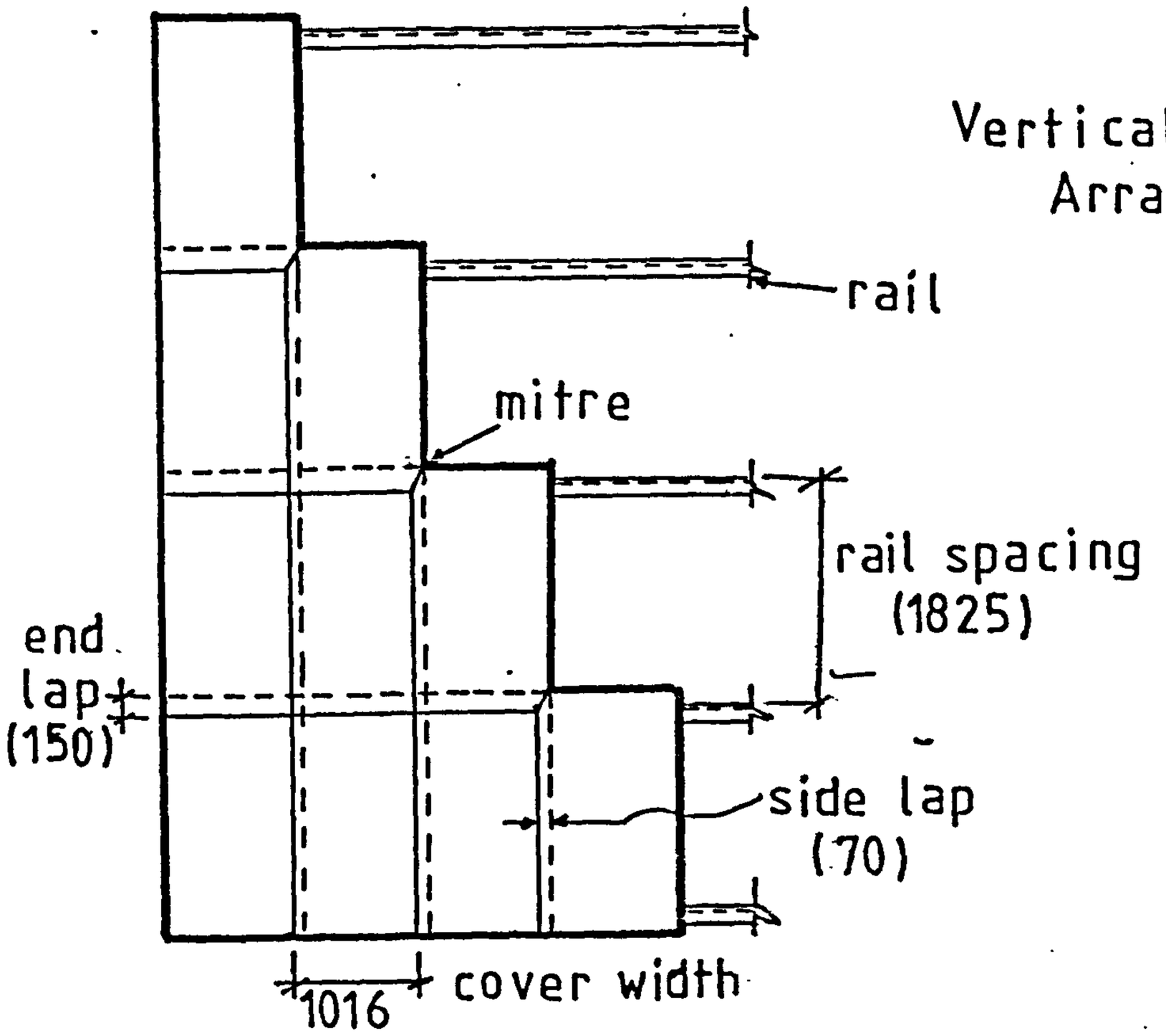
$$2h/E2[1+(yn-W2)/W3]+2k[1+(x^{-}W2)/W3v \qquad \text{Eqn. 5.03}$$

where -

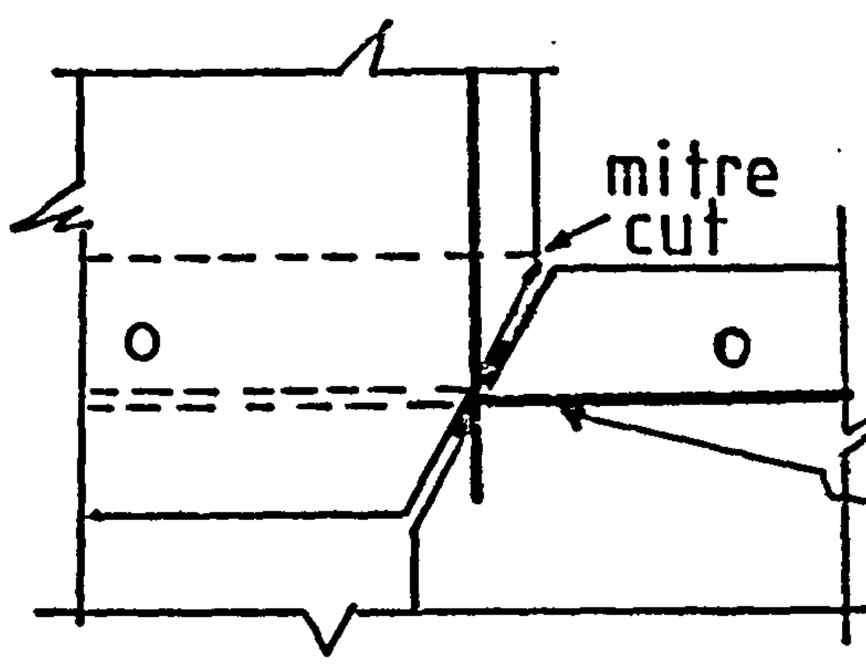
$$v=h+0.25x\tan\theta$$

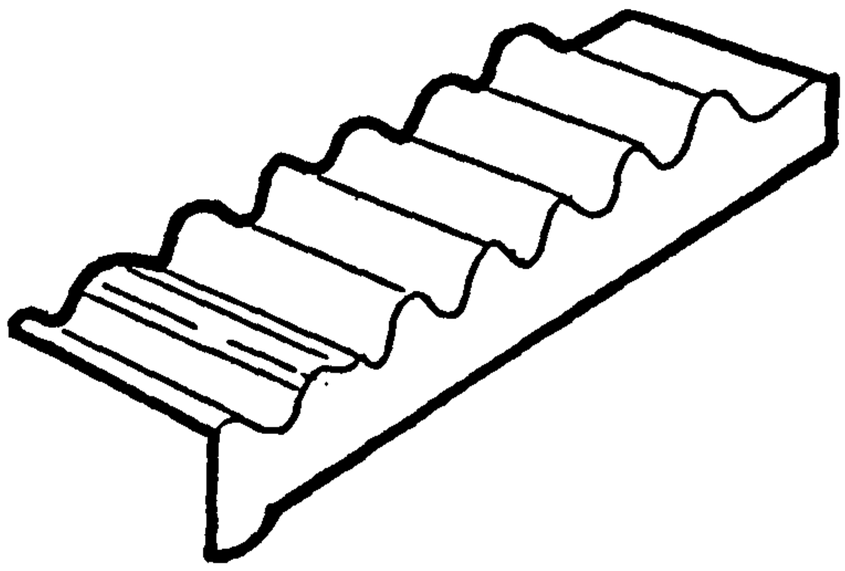
The following assumptions must be observed when Eqn. 5.03 is used

Vertical Sheeting Arrangement



Mitre Details





Eaves Filler Piece

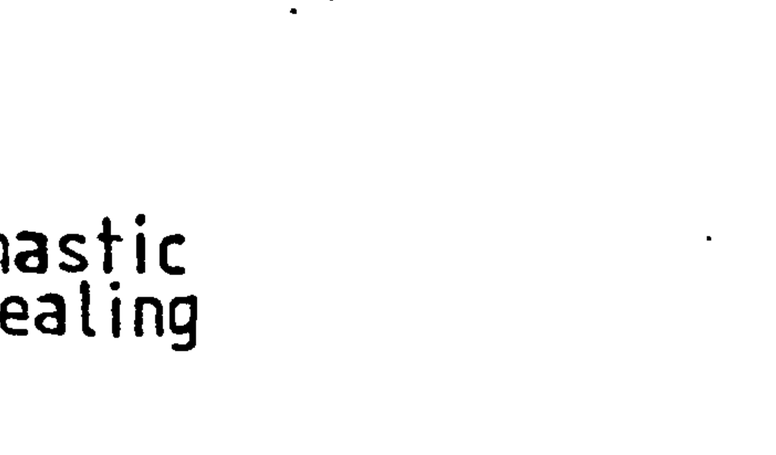


FIG 5.05 VERTICAL CLADDING

1. A discrete sheeting arrangement is used
2. End laps and side laps must be constant
3. Openings and masonry walls must be treated as additional items

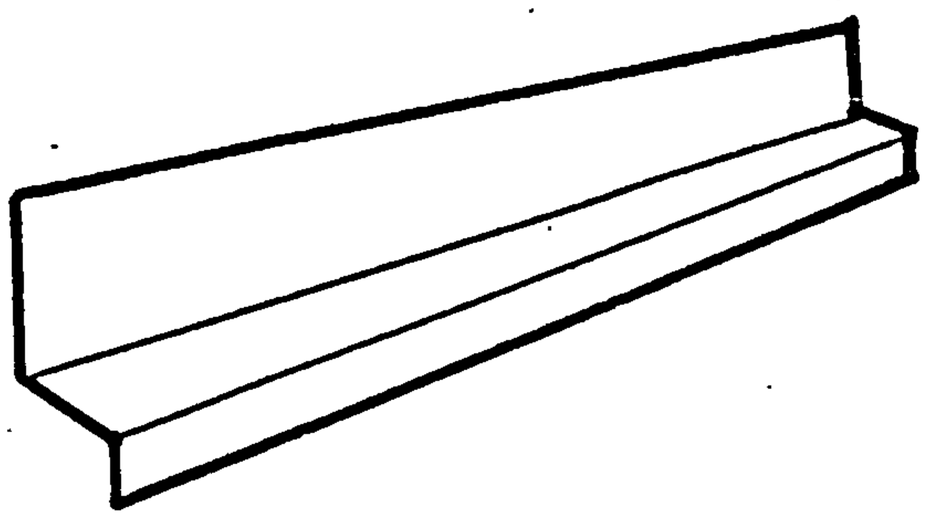
If sandwich construction is used there will be equal number of underlining Panels and top sheets. Brick cladding is generally constructed on rigid ground beams provided around the perimeter of the building.

5.4.2 MOVEMENT JOINTS

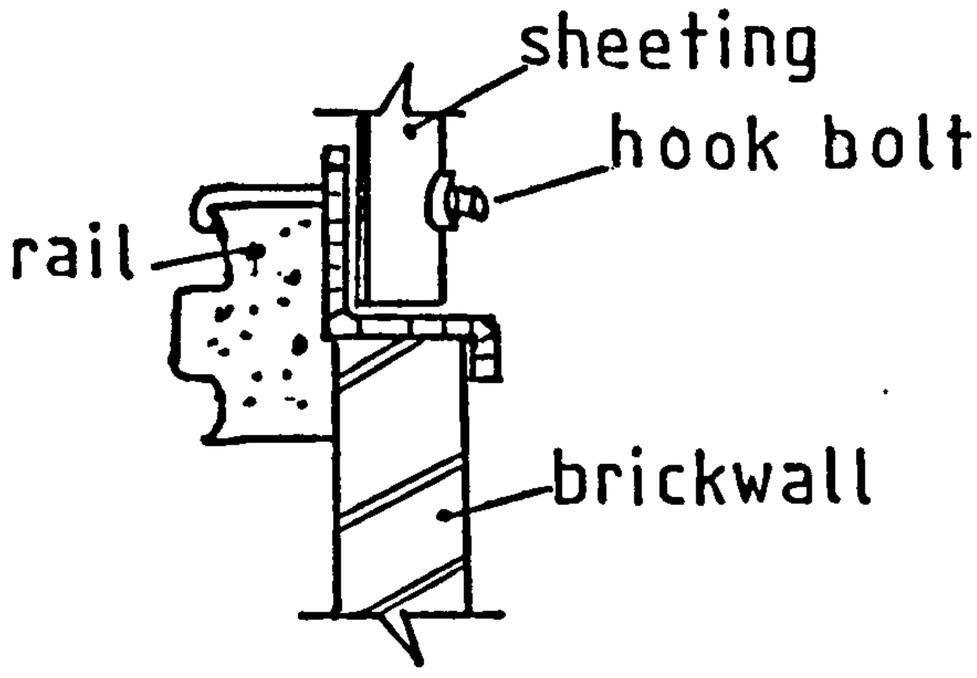
Cladding is subjected to movements caused by thermal and moisture changes. Movement joints are designed to accomodate such movements which would otherwise cause structural damage. The total number of such joints can be estimated from Eqn. 5.04

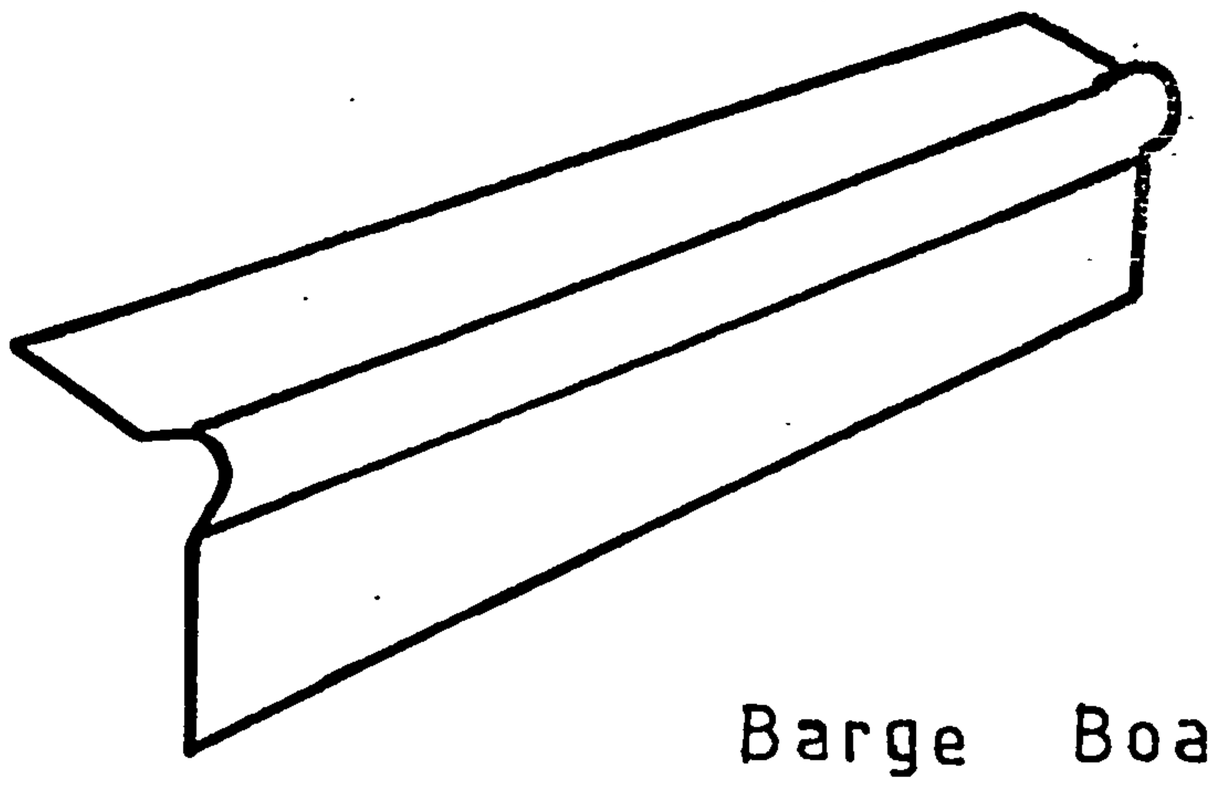
$$Mj = 1.033(yn-75) \qquad \text{Eqn. 5.04}$$

Movement joints are provided in the roof through the side cladding and usually coinciding with natural ends of sheeting course. A gap of 15-20 mm is made throtgh the sheets and covered with special movement joint sheets which are bolted to the purlin (Fig 5.06).

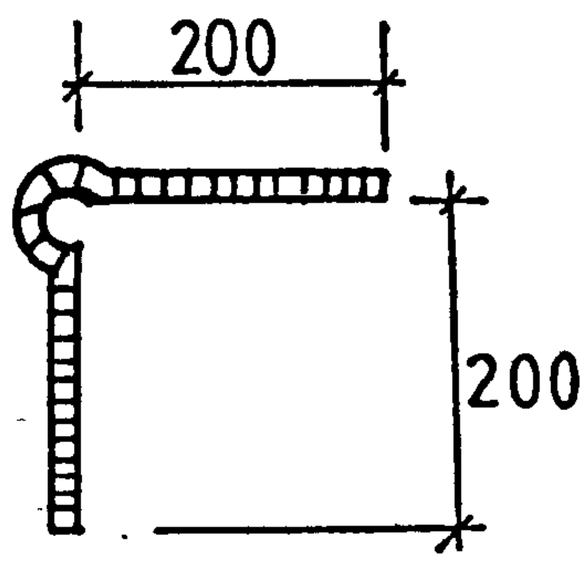


Horizontal Flashing Piece





Barge Board



Movement Joint

FIG 5.06 VERTICAL CLADDING ACCESSORIES

5.5 FIXING ACCESSORIES

Fixings have been in use for many centuries. It is known that the use of cramps and dowels featured prominently in the architecture of ancient Greece and Rome. In the Great Temple at Baalbeck, iron dowels were used in column shafts and in the Colessuem copper cramps were set in lead. Subsequently, iron ties were used extensively to stabilise windows. In order to prevent corrosion cramps were set deep in masonry such as in Perrault's Colonnade at the Louvre (Fig 5.07). It was also known at the time that when metal was heated and coated with oil, corrosion was reduced. Wooden dowels made of oak were boiled in oil to help preservation.

About three decades ago cladding was only an extension of traditionally bonded ashlar and brickwork. New cladding materials had their thicknessses drastically reduced to conform with modern economic designs. Bending around the cladding such as used to be the case with English and Flemish bonded walls was no longer possible and the concept of fixing the cladding to the structure came to be as important as the design of the structure itself.

Design of fixings is based on its foremost property to prevent the cladding from disintegrating due to dead, imposed and wind loads. Three principles must underline the design of fixings:

1. Type of cladding will determine the method of fixing

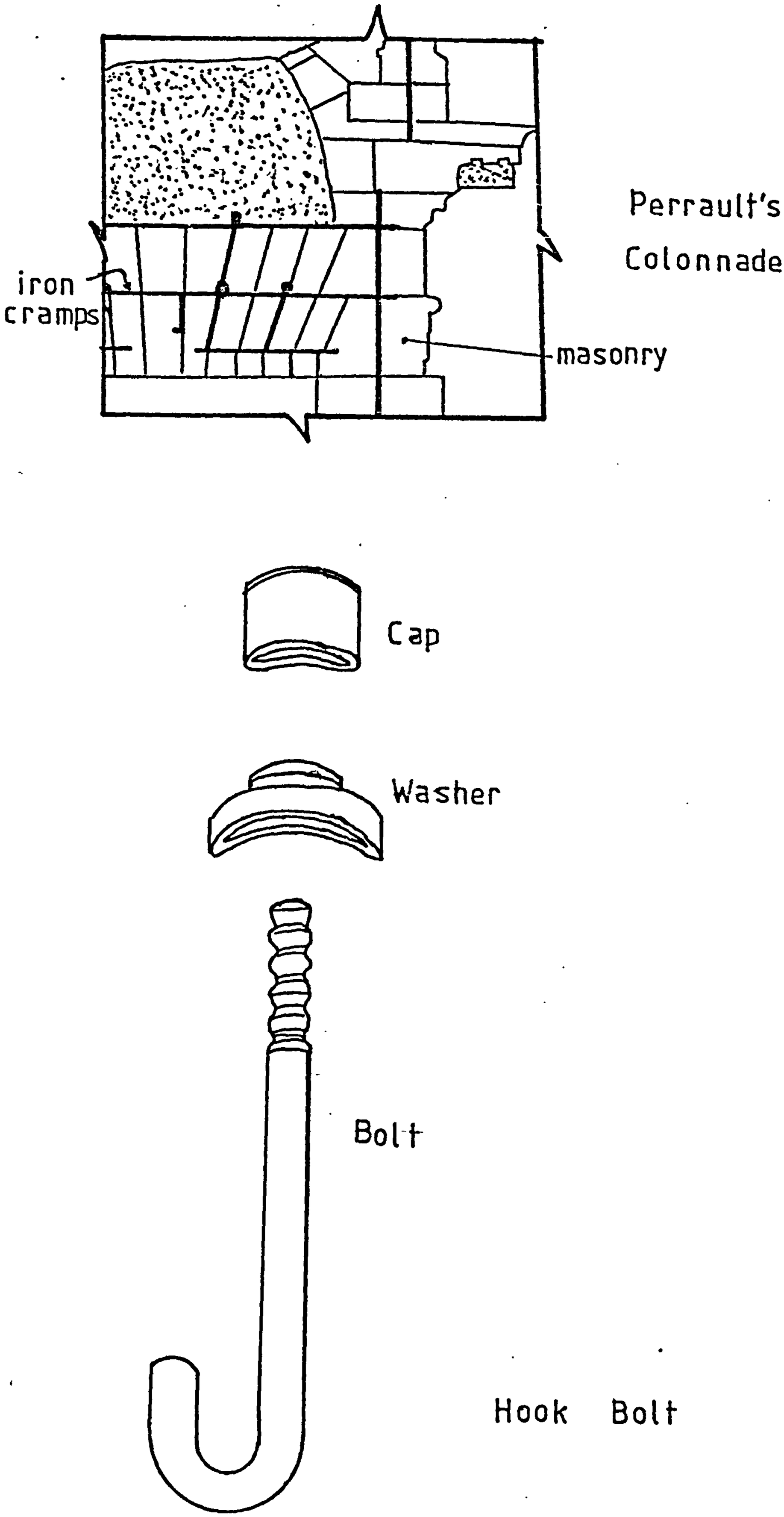


FIG 5.07 ANCIENT AND MODERN FIXINGS

2. Forces generated in the cladding determine the sizes of fixings to use.

3. To eliminate any subsequent chemical action between fixing and cladding only suitable metals must be used.

5.5.1 CLASSIFICATION OF FIXINGS

Table 5.04 summarises the main types of fixings suitable for the cladding systems described in Sec. 5.3 and 5.4. Most metal fixings are electrogalvanised or coated with suitable protective metal.

DESIGN OF FIXINGS

Fixings will form additional cost items in the following cladding construction :

1. Roof sheeting
2. Ridge sheeting
3. Wall sheeting.

The shape of structural elements influences the type of fixing to use. Suitable fixings for the structure are illustrated in Fig 5.07. It is seen for instannce that the hook of the fixing

TABLE 5.04 CLASSIFICATION OF FIXINGS

ACCESSORY	SPECIFICATION	WORKING STRESS MN/m^2	SHEAR MN/m^2
Phosphor Bronze	BS 2874	177-192	88-96
Stainless Steel	BS 1449 Part 4	138	92
Aluminium Bronze	BS 2874	100-197	50-98
Manganese Bronze	BS 2874	77-115	38-57

anchors well into the side of the purlin (Fig 7.1C).

For roof cladding provided the suction loading does not exceed 960 N/M2 an average of 3 bolts per sheeting is sufficient but an additional intermediate bolt is neccessary when this loading is exceeded. Ultimately the number of bolts per sheeting can be related to purlin spacing. For ridge cappings a minimum of two bolts per unit is sufficient. Design of fixings for vertical sheeting has been summarised in Table 5.05. This empirical approach is suggested on the basis that standard rail spacing will be used and suction loading is within UK limits. Three standard rail spacings have been assumed. If this table is used the relevant expression for a typical estimate of vertical sheeting bolts will be as given in Equation 5.04

No. Of bolts = Bolt Factor * Surface Area Eqn. 5.04

5.6 CONCLUSIONS

This chapter of the thesis has explained how the skin of the structure is designed and the function of various cladding accessories which are used to form the skin. All the factors that affect the selection of cladding type (except cost which will be discussed in the next chapter) have also been examined.

TABLE 5.05 EMPIRICAL GUIDE TO VERTICAL SHEETING DESIGN

MINIMUM RAIL SPACING (mm)	BOLT FACTOR (m^{-2})
1525	2.3
1825	1.5
1975	1.4

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CHAPTER 6

A COST MODEL FOR PRECAST CONCRETE STRUCTURES

6.1 OVERVIEW

Every step in structural design is a cost decision and design processes are judged by their impact on global costs as well as the safety which they give to users of the structure. In this chapter studies which have been done on the six factors that directly affect the cost of the structure, that is fabrication, haulage, foundation, cladding and erection are presented.

6.2 FABRICATION COSTS

In Section 2.4 traditional and fully mechanised methods of fabricating the structure were discussed. The factors that affect fabrication costs are now examined in relation to one particular traditional set-up. It will be seen that the absence of sophisticated machinery enables physical measurements of cost items to be measured precisely and accurately.

6.2.1 WORK STUDY AT BISHOP AUCKLAND FACTORY

The factory layout is similar to the one shown in Fig 2.08 and the factory structure is similar to that discussed in Section 2.4. Each of the following elements is manufactured on separate production lines :

1. Column
2. Rafter
3. Purlin
4. Sheeting rail
5. Eaves gutter

For each of the above elements the following operations were studied :

1. Cut and bend reinforcement to schedule
2. Assemble reinforcement, ready mould and place cage in mould.
3. Place and vibrate concrete with finishes and stack where necessary
4. Demould element and stack

Operation times were approximated to the nearest second. Operations (3) and (4) could not be studied concurrently for lack of personnel. At some instances operations (1) or (2) was executed for more than one element. Each of the four operations was timed for ten cycles and the mean times were

taken. Fig 6.01 shows the results of the study. Sizes of each of the five elements vary according to the particular design. Studies were thus made on element sizes of weights ranging between 0.14 - 1.28 tonnes. The results presented in Fig 6.01 assume a linear proportionate weight of one tonne for each element.

Table 6.01 summarises the work gang and equipment used in each operation. Readying the mould takes a form of cleaning and coating the interior with a thin film of oil. Lifting reinforcement cages is done mainly with gantry cranes, portable wheelers and manually. Vibrating tables are used. Gantry cranes are used to demould column and rafters. purlins are demoulded manually and gutter moulds are dismantled to ensure no damage is done to the delicate sections.

6.2.2 ANALYSIS OF FACTORY OUTPUT

In order to complement findings of the work study, output of the same factory over a continuous sixty day period has been analysed. The period covers about the same duration as the work study. For each element, output levels and the frequency at which they are manufactured (days) are presented on histograms (Figs. 6.02 - 6.07). Fabrication constant is defined as the manhours taken to manufacture one tonne of finished precast concrete, i.e.

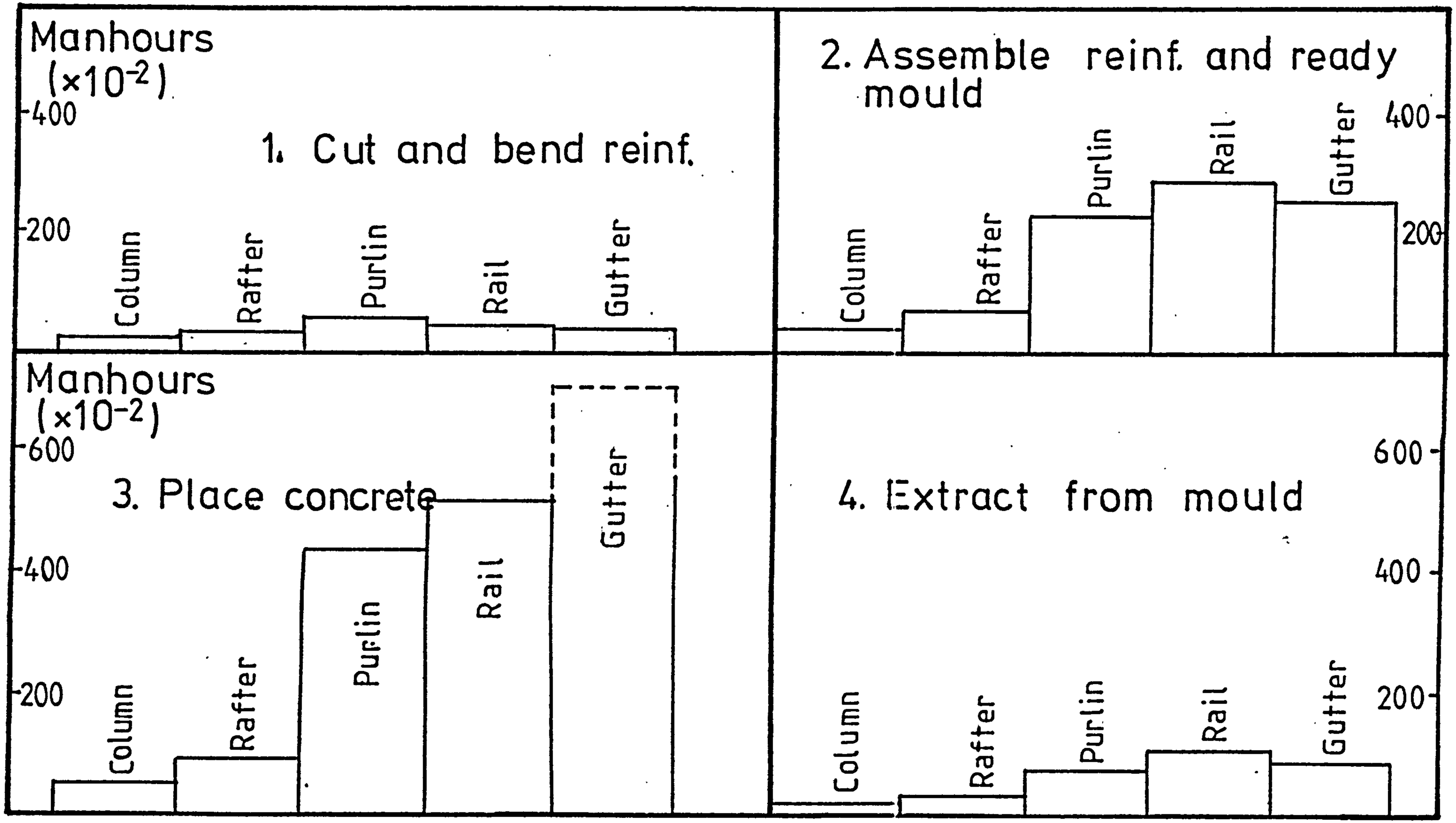


FIG 6.01 FABRICATION OF 1 TONNE PRECAST CONCRETE

TABLE 6.01 WORK STUDY SHEET

OPERATION	NO. OF WORKMEN					EQUIPMENT
	Col	Raf	Fur	Wal	Gut	
1. Cut and bend reinf. to schedule	1	1	1	1	1	Cutting machine
2. Assemble reinf. Ready Mould	2	2	1	2	1	Gartry cranes used to carry heavy cages
3. Place & Vibrate Concrete, finishes & stack	3	2	1	2	2	Wheelbarrow for concrete distribution, gartry cranes for stacking
4. Demould & stack	3	2	1	2	2	Gantry cranes for demoulding cols. & rafters

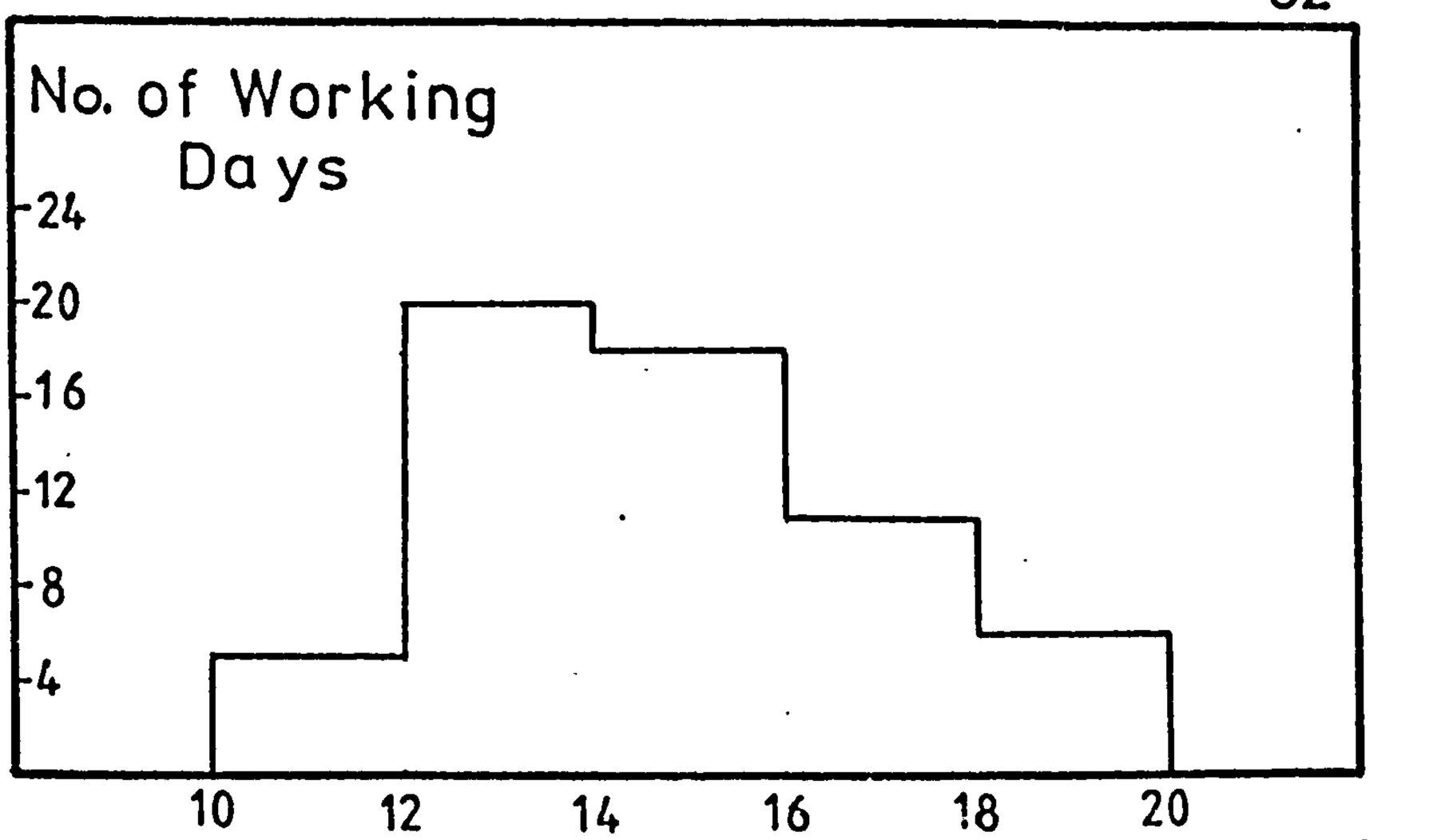


FIG 6.02 FABRICATION OF COLUMNS

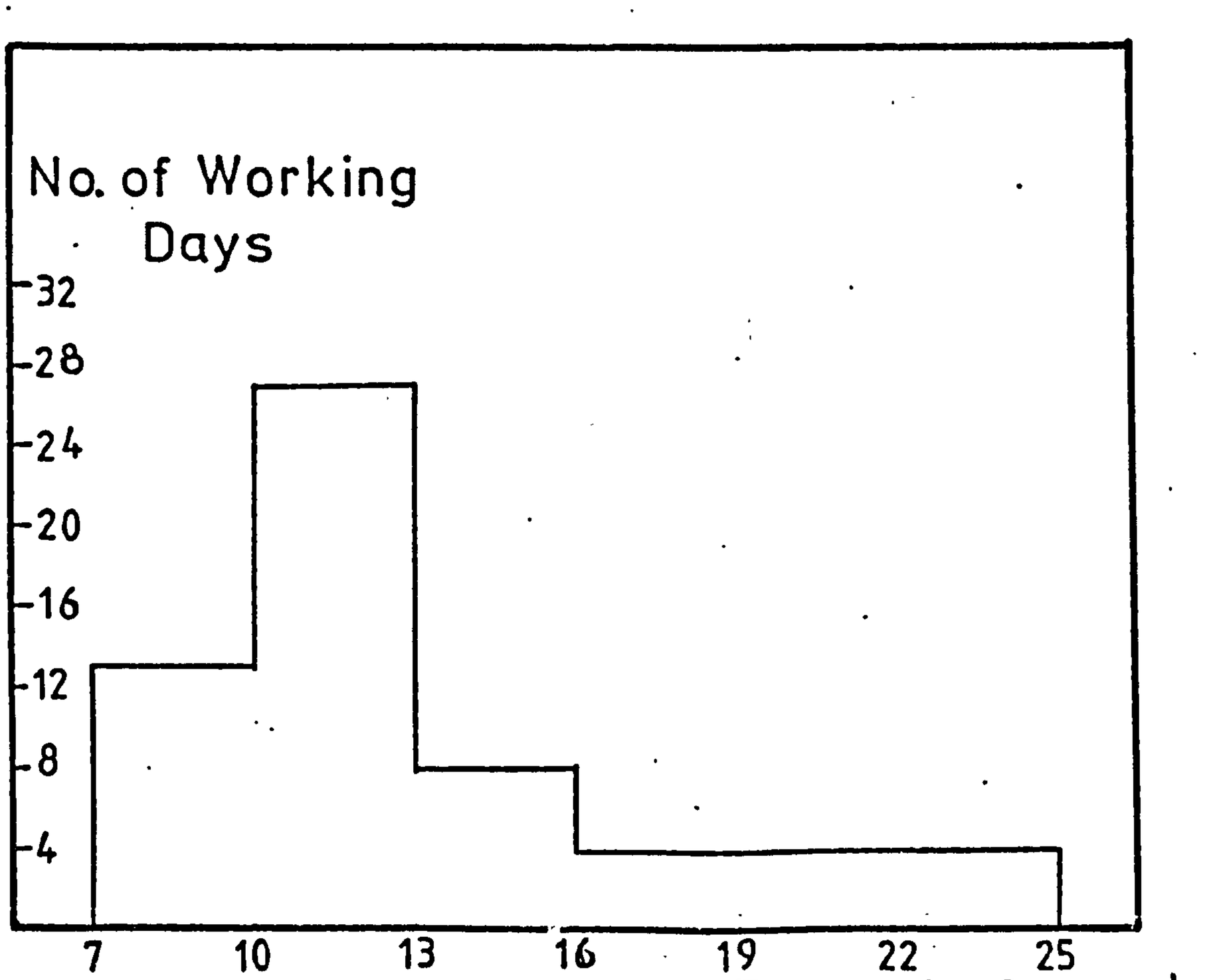


FIG 6.03 FABRICATION OF RAFTERS

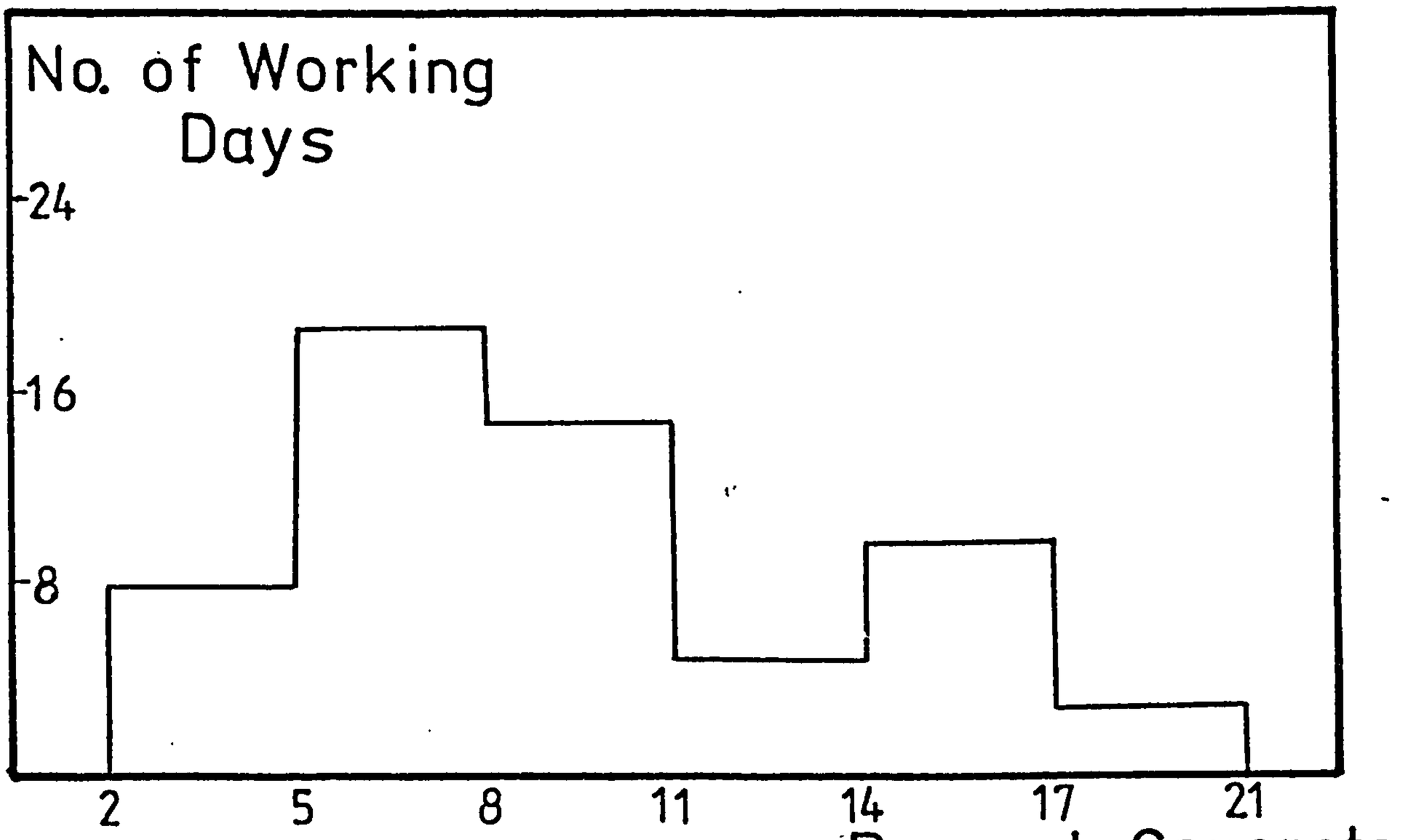
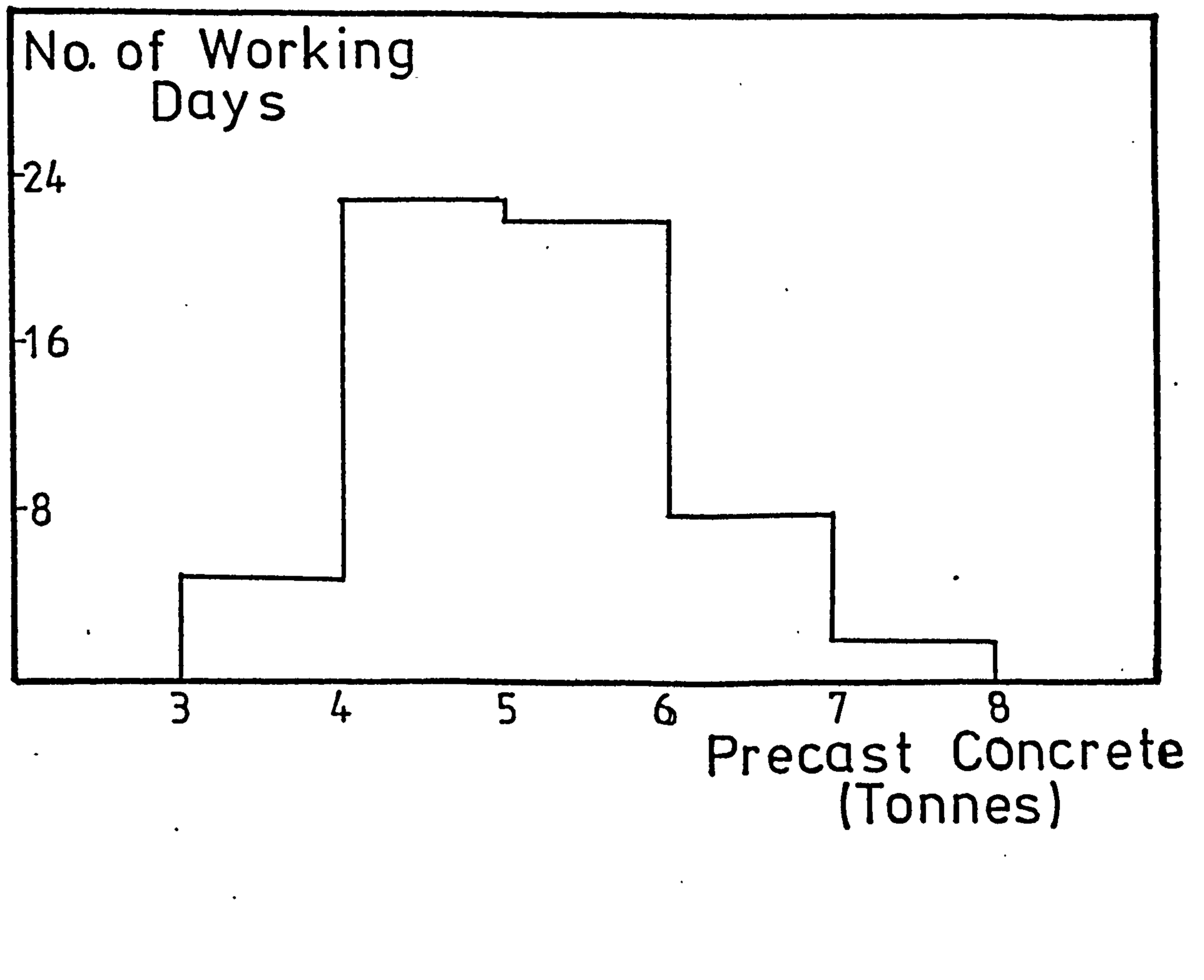


FIG 6.04 FABRICATION OF PURLINS

FIG 6.05 FABRICATION OF GUTTERS



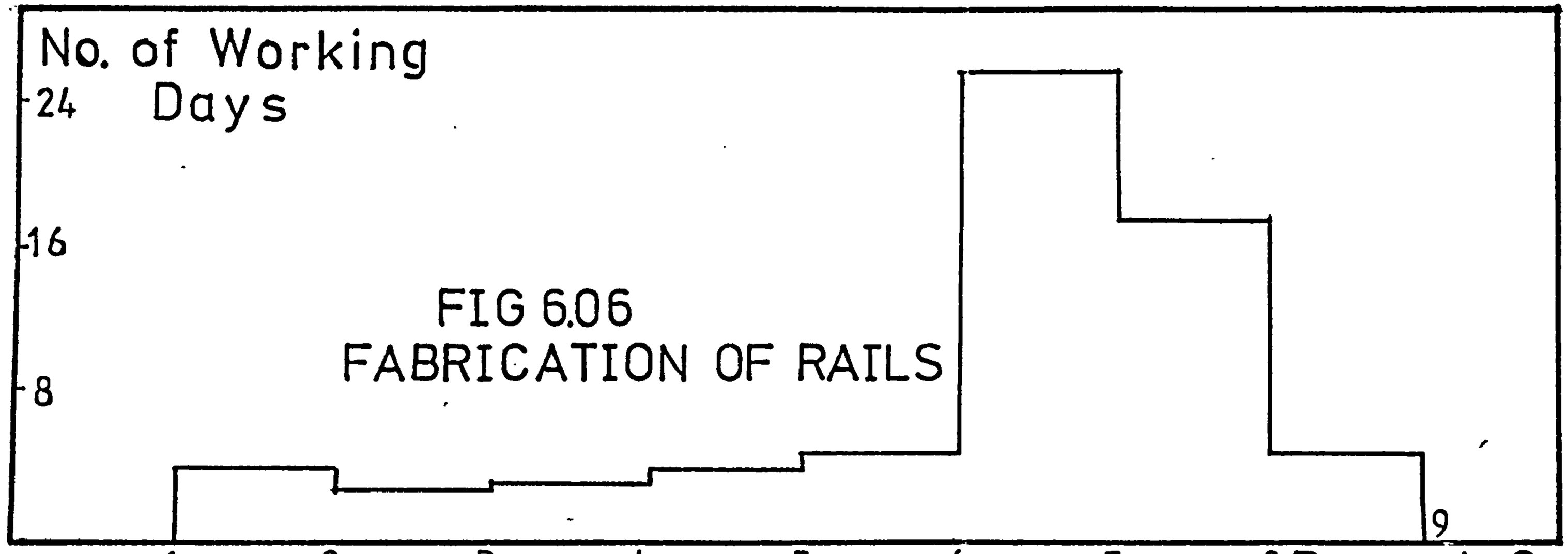


FIG 6.06
FABRICATION OF RAILS

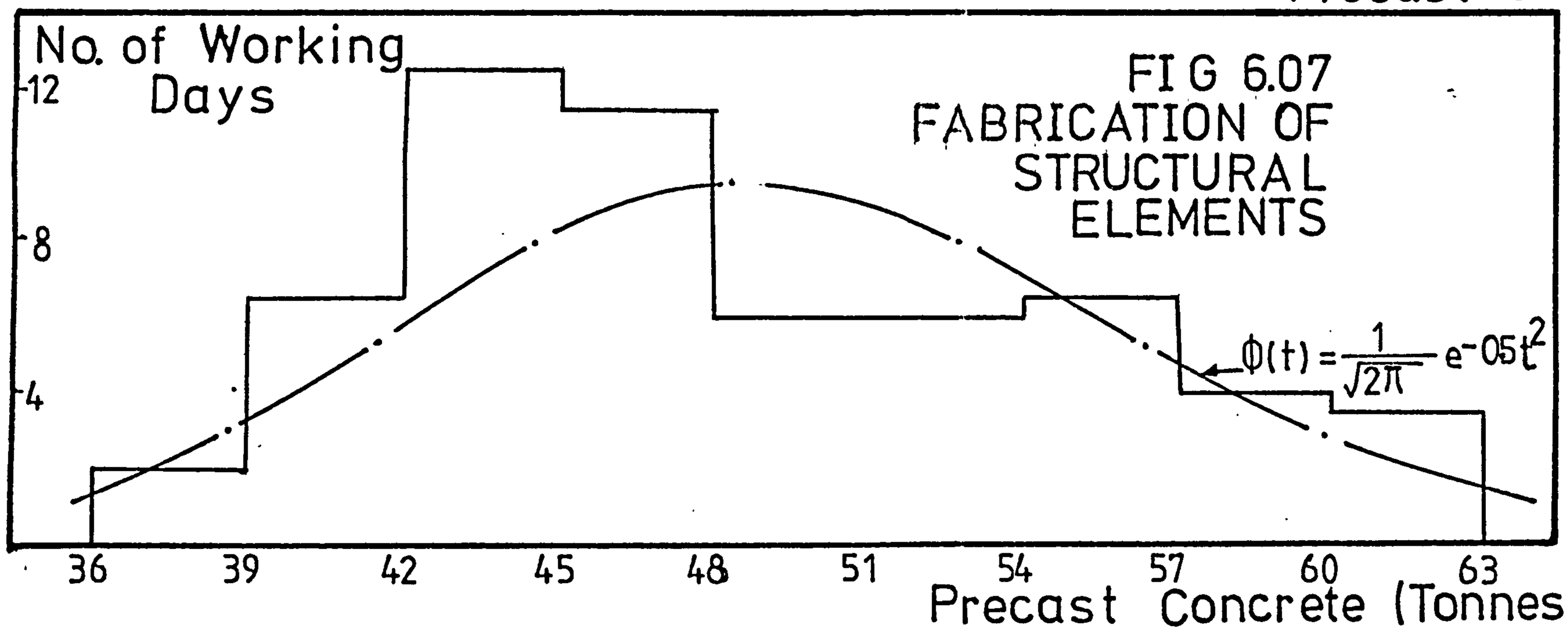


FIG 6.07
FABRICATION OF STRUCTURAL ELEMENTS

TABLE 6.02 FABRICATICN CCNSTANTS FCR STRUCTURAL ELIMENTS

STRUCTURAL ELEMENT	DAILY OUTPUT (Tornes)	FABRICATICN CCNSTANT (Manhour/Tonne)
Column	14.5	5.0
Rafter	12.8	6.33
Purlin	10.9	3.78
Eaves Gutter	5.1	11.36
Sheeting Rail	6.5	5.71

$$\text{Fabrication Constant} = M/\{\sum_{i=1}^{j} We\} \qquad \text{Eqn. } 6.01$$

where We represents the weight of every ith type of element summed over the range of all j elements observed and M is the total manhours needed to manufacture all the j elements. Table 6.02 gives Fabrication Constants for each structural element.

The reliability of the results in Table 6.02 can be measured from the following statistical deductions : Assuming a Gaussian distribution for the overall production levels shown in Fig 6.0)

$$F(t) = [1/\sqrt{2\Pi}]e^{-t^2/2} \qquad \text{Eqn. } 6.02$$

and the standard deviation is given by :

$$s = \sqrt{[\sum (Wi-W)^2/n]} \qquad \text{Eqn. } 6.03$$

The standard deviation was computed to be 6.76 kg and the mean daily output was 46.144 tonnes.

6.3 HAULAGE COSTS

The oldest and simplest hiring system which the haulage

contractor uses is the one that makes hiring charges on hourly basis. There is another system now which makes charges on radial mileage basis and which is more popular. With this system trucks having up to 10 tonnes capacity, and trucks having over 10 tonnes capacity are categorised separately. The haulage cost is then simply as given in Eqn. 6.04 :

$$\text{Haulage Cost} = \text{Rate} * \text{Tonnage} \qquad \text{Eqn. 6.04}$$

For a given tonnage of structural elements, the number of trips is computed and the appropriate rate used to price the haulier 's charges.

6.4 FOUNDATION COSTS

Foundation problems associated with the structure are generally not too complicated. There is usually a low contact pressure between column bases and the soil, typically up to 150kN/m2. Foundation design for the structure therefore entails provision of column bases which can be easily constructed from standard moulds. A typical column base will comprise an unreinforced footing of about one meter deep.

6.5 CLADDING COSTS

Cladding design was discussed in chapter 5. It was established then that there are a number of alternate design types. Any such design type could be costed from the following items :

1. Cost of all accessories used
2. Cost of handling and transporting all accessories
3. Cost of erecting the cladding.

A quick guide to prices of cladding accessories can be obtained from Architect and Builders Price books though it is more reliable to obtain them directly from proprietary manufacturers. Erecting the cladding could be done by specialist contractors. Rate of cladding was studied by work study methods and is presented in Table 6.03.

6.6 ERECTION COSTS

Erection procedures were discussed in Section 2.5. The times for fixing various structural elements have also been studied by conventional work study methods and is represented in Table 6.04. For each structure the total erection costs can be estimated by computing the optimum element connexions in the structure using Table 6.04.

TABLE 6.03 RATE OF EFFECTING CLADDING

ACTIVITY	RATE M²/Manhour
Fixing Roof Sheeting	1.3
Fixing Side Sheeting	1.25
Erecting a Brickwall	0.278

TAELE 6.04 RATE CF ERECTING STRUCTURAL COMPCNENTS

STRUCTURAL ELEMENT	WORKMEN	RATE
Columr	4	2.6/pair
Rafter	4	1.6/pair
Purlin	4	0.2/each
Sheeting Rail	4	0.4/each
Gutter Beam	4	0.75/each

6.7 THE COST MODEL FUNCTION

Define the following primary variables :

Span of the structure	x
Frame spacing	y
Number of bays	n
Number of spans	k
Roof pitch	w
Height to eaves	h

The overall cost of the structure is the sum of the following costs :

1. Fabrication costs
2. Haulage costs
3. Foundation costs
4. Cladding costs
5. Erection costs

that is the global cost of the structure is :

$$\begin{aligned} \text{Building Cost} = \{&\text{Foundation Cost} + \text{Haulage Cost} + \text{Foundation Cost} \\ &+ \text{Cladding Cost} + \text{Erection Cost}\} \end{aligned}$$

Let -

A=0.5xtan(0.01750)

B=0.7Ayn - 3.14yn-0.5A-0.2

C= (yn-0.15) / (A-0.15)

D=2.73(h+0.762) (nk+n+k+1)

In terms of the primary variables alone the terms in Eqn. 6.05 can be written as follows :

Fabrication Cost =(31.22(0.7h+0.0533) (nk+n+k+1)
+78.02(0.0438A+0.0485) (nk+n)
+720.08ynk+375.91yn+418.94(ny+ynk)
+(D+5.46A(k+nk)+73.48ynk+2Cyn+35.1yn)

Haulage Ccst =7.15((0.07h+0.0533) (nk+n+k+1)
+2(0.0485+0.0438A) (nk+k)+(0.02052ynk)
+(0.124yn)+0.1159(yn+ynk))

Foundation costs =29.62(nk+n+k+1)

Cladding Cost=k(8.19A+27.59B+18.81C+1.37yn
+158.09)+516(yn+x-2.25)

Erection Costs =92.06n+9.49

Therefcre the cost model function is formally stated as :

BUILDING CCST = f(x,y,n,k,o,h)

6.8 THE INTERNAL STRUCTURE CF PFRAME

The information on analysis, design and ccsting of the structure previcusly discussed has been used to develop subsystems of PFRAME. The main subsystems of PFRAME are :

1. A ccst model
2. A main graphical output syste
3. A ccntour program
4. An cptimizer

The cost model is written in Fortran and ccmprises :

1. An input and control subprogram
2. Fcurteen analysis, design and ccsting subroutines
3. An output subprogram.

At a typical design session, the user is prompted to supply appropriate values to the primary variables i.e. to state mathematically the client's request. A second prompting enables the user to alter the value of any secondary array such as material prices, labour and crane charges, type of cladding

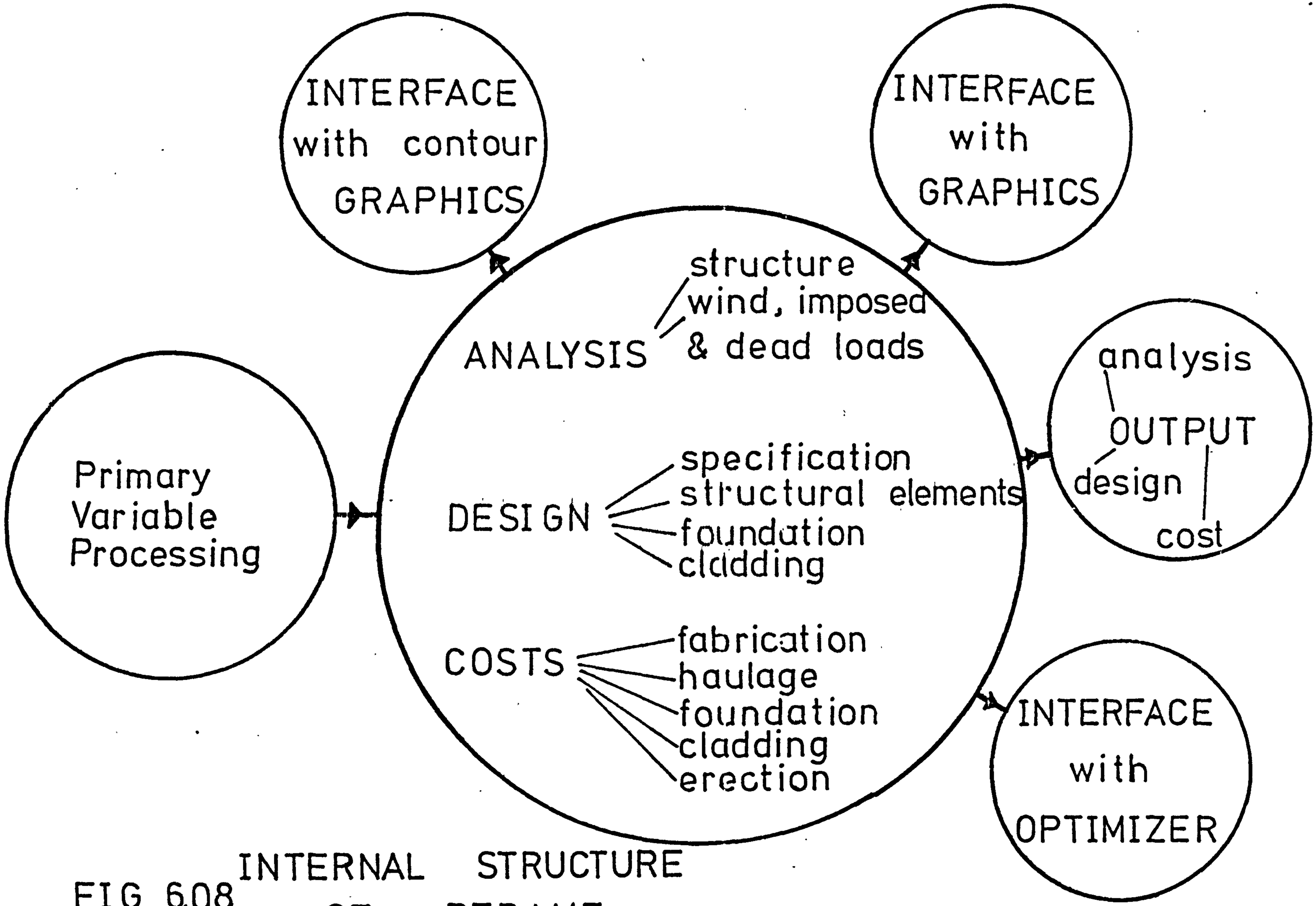


FIG 6.08 INTERNAL STRUCTURE OF PFRAME

and cladding accesscry prices, haulage rates and indeed any secondary variable subject to pericdic variaticn in magnitude. The progiam then analyses, designs and costs the building (Fig 6.09). The user is again prompted tc indicate which subsystem (if any) is to be initialised. The commands DRAW, CCNTOUR, or OPTIMIZE, sets internal pointers respectively, tc the graphics sessicn, ccntcuring session or optimization session. The optimization session has been discussed in Chapter 3 and the main graphics session will be discussed in Chapter 7. The next section is devoted to the contour sessicn.

6.9 GENERATING COST RESPONSE SURFACES

Cost response surfaces can be an invaluable auxillary aid in CAD. What is conceived abstractly by an cptimizing process can be visualised graphically.

The algorithm* that generates surfaces cf the cost model operates by computing grided values for each set of grid cell. Some cost functions are discontinucus and associated data for such surfaces are irregularly distributed. If grid values are determined by an approximaticn technique, this discontinuity is preserved. The so called approximation technique has therefore been adapted for the present work. A concept of 'neighbcurhcods' is assumed which allows the process to select sufficient data points from a vicinity. Tc illustrate this concept, consider a pcint Q(x,y,c) in a design space, where :

```
**********************************
*                                *
* UNIVERSITY OF NEWCASTLE UPON TYNE *
*                                *
* DEPARTMENT OF CIVIL ENGINEERING *
*                                *
*          * * *                 *
*                                *
*   OPTIMUM DESIGN OF PRECAST    *
*                                *
*     CONCRETE STRUCTURES        *
*                                *
*        DESIGN NO. : 20         *
*                                *
**********************************
```

1 STRUCTURE BRIEF

BUILDING AREA	= 1000.0	M2
HEIGHT TO EAVES	= 5.00	M
NO. OF BAYS	= 10	
FRAME SPACING	= 5.00	M
PITCH	= 14.00	DEG.
SPAN	= 20.00	M
SITING...	OPEN COUNTRY	
INTENDED USE..	FARM HOUSE	
NO. OF SPANS	= 1	

2 DESIGN

CASE...	3-PINNED PORTAL
FIRE RESISTANCE	=1.5 HRS
COLUMN SIZE	=200.0X350.0MM
RAFTER	=1100.X 410.X170.MM
RAFTER LEN	=10.306& 0.835M
PURLIN DEPTH	=130.0MM
SECT. RAIL DEPTH	=215.0MM
GUTTER BEAM DEPTH	=351.0MM
WIND LOADING	= 5092.2N/M2

3 ROOFING

TYPE..	SINGLE SKIN CONSTR.	
SHEETING LAP	= 150.0	MM
SHEETING RATE	= 1.825	£/M2
LINING RATE	= 1.545	£/M2
CROWN SHEETING	= 3.45	£/M
CROWN LINING	= 1.92	£/M
BARGE BOARDS	= 2.51	£/M

4 CLADDING

TYPE...	SINGLE SKIN CONSTR.	
SHEETING RATE	=1.825	£/M2
LINING RATE	=1.546	£/M2
INSULATION RATE	=2.110	£/M2
HOOK BOLTS & NUTS	=6.53	£/100
PLASTIC CAPS	=0.67	£/100
PLASTIC WASHERS	=1.13	£/100
EAVES FILLER PCE.	=1.811	£/M
HORIZONTAL FLASH	=1.236	£/M
BRICK WALL RATE	=7.00	£/M2

5 FABRICATION & HANDLING

PRODUCTION CONSTANTS

--COLUMNS	= 200	KG/MANHR
--RAFTERS	= 158	KG/MANHR
--PURLINS	= 254	KG/MANHR
--RAILS	= 175	KG/MANHR
--GUTTERS	= 89	KG/MANHR
CONCRETE PRICE	= 14.75	£/M3
LABOUR RATE	= 60.00	£/WEEK
HAULAGE DIST.	= 40.0	MLS.

6 ERECTION & FOUNDATION

ERECTION CONSTANTS

--FRAMES	= 12.0	MANHR/BAY
--SHEETING	= 1.25	M2/MANHR
--BRICKWORK	= 0.273	M2/MANHR
FOUND DEPTH	= 1.067	M
EXCAVATION	= 4.67	£/M3
CRANE HIRE	= 6.67	£/HOUR
LABOUR RATE	= 50.00	£/WEEK

7 COSTS

FABRICATION COST	£	4976.85
HAULAGE COST	£	258.03
FOUNDATION COST	£	651.73
CLADDING COST	£	4468.64
ERECTION COST	£	930.10
TOTAL COST	£	11295.40

FIG 6.09 TYPICAL DESIGN OUTPUT

1. X, y are the span and frame spacing respectively of the structure whose overall cost is c
2. The design is feasible and no constraint is violated.

There will always be a set of n data points within the neighbourhood of Q whose coordinates can be represented by Q(x ,y ,c). Such points are enveloped by the smallest circle whose centre is at the control point Q. Those points within the circular enclosure and nearer to Q will influence it more than those points farther away.

Let -

Ri be the least distance from Q to any point within the enclosure

Rn-1 the radius of the circle

Then (Ri/Rn-1) is a legimate weighting function that can be applied to each of the points in the enclosure. If the gradient of the cost model function is computed at each point and the neighbourhood concept applied to each then a set of contour lines can be plotted for every set of feasible designs.

A set of feasible designs generated in PFRAME can then be interfaced with the contour program (Fig 6.10). At a typical design session, the user initialises the control program which then prompts the user to define the lower and upper bounds of the two primary variables whose effects on the cost model is to be investigated. Overall building costs, ci, for each feasible

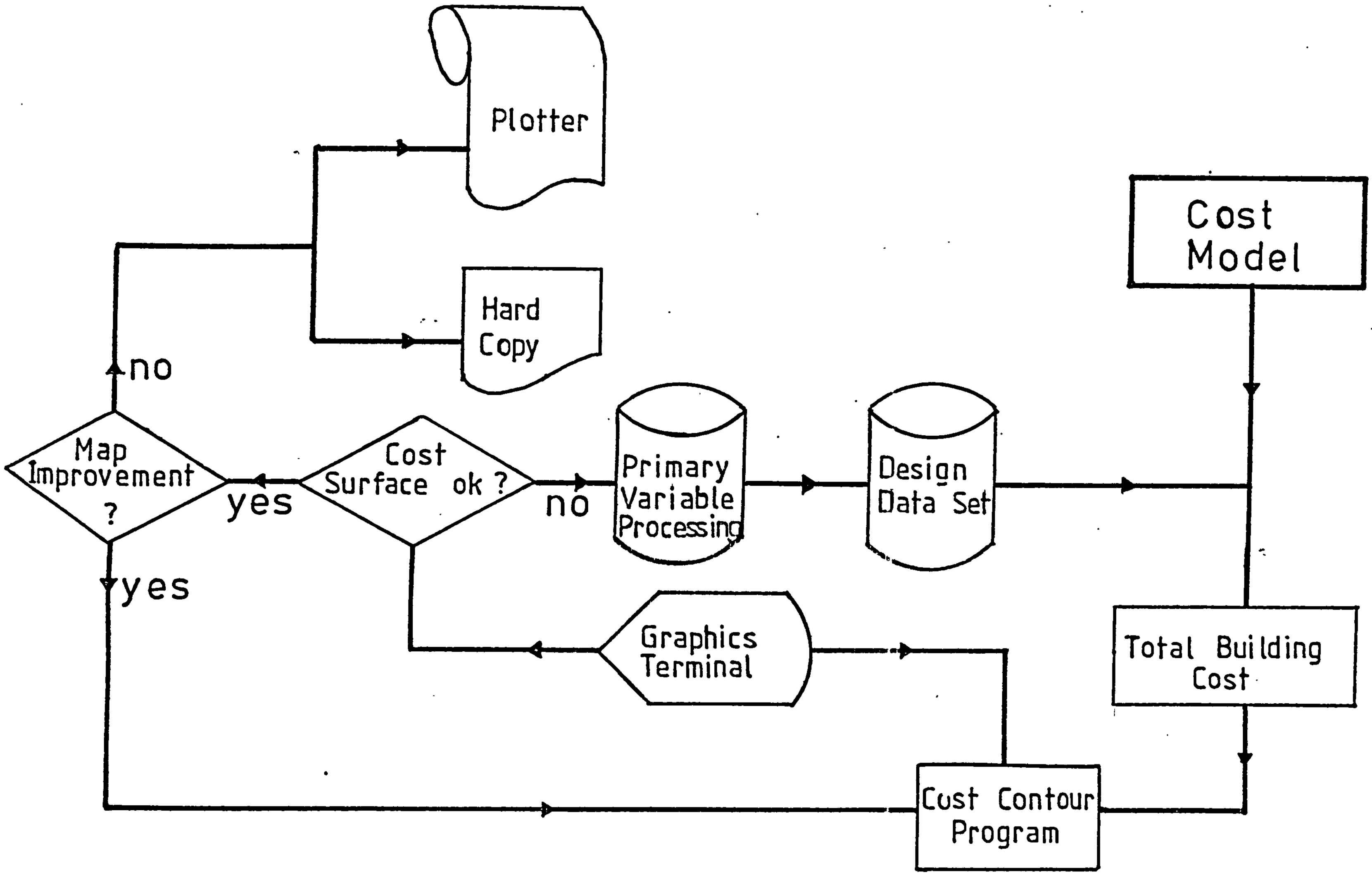


FIG 6.10 COST MODEL AND CONTOUR PROGRAM INTERFACE

design are then supplied interactively to the contour program. A graphics terminal displays the corresponding cost surface. Two further promptings are given :

1. Are all vital contour lines drawn ?
2. Should quality of the map be improved ?

If the user decides to take advantage of the first option then the lower and upper bounds of the primary variables are adjusted acccordingly. If the second prompting is taken advantage of, factors that affect map quality are adjusted. At the end of the day a PLOT command may be issued if hard copy is desired (Chapter 8).

6.10 CONCLUSIONS

The global cost of the structure has been found to be a function of the following cost items : fabrication, haulage, foundation, cladding and erection. Each of these items has been studied in detail and the results presented. The following conclusions are reached as a result of the work study on fabrication processes (Fig. 6.01) :

1. Concrete placing is the most expensive whilst cutting and bending reinforcement is the least expensive.
2. Concrete placing is relatively more expensive in the case

cf eaves gutters and sheeting rails.

The histograms for gutter beams and columns are fairly symmetrical, indicating a balanced utilisation of mould time for both elements. Rafter and purlin histograms are positively skewed, i.e. mould times are used more than the other elements. This trend is peculiar to the factory especially in the case of purlins. Purlin shapes and sizes are the least varied in order that they can be used in other designs so that if they are manufactured in excess they will be always useful. The sheeting rail histogram is negatively skewed suggesting that mould time is underused. Again this is logical for sheeting rails are used in sheet cladding which is only one kind of cladding type that clients will request and hence only the minimum amount of sheeting rails can be fabricated at any particular time. Lastly, the overall fabrication trend postulated in Fig 6.07 shows that on the whole the mould time for this factory is wisely utilised.

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2. CP3: Chapter V : Part 2 :1972, Wind Loads, British Standards Institution, London.

3. KNAPTON, J. and ANDAM, K. A. Interactive Structural Design With Computer Graphical Output, To be published in Computer Aided Design, Vol 11 No. 4 IPC Science and Technology Press ltd., Guildford, Surrey, July 1979

4. User's Manual. A General Purpose Cortcuring program, CAICOMP, 1971, California.

CHAPTER 7

INTERACTIVE DESIGN WITH COMPUTER GRAPHICS

7.1 OVERVIEW

This chapter outlines how the cost model has been interfaced with a graphical output system that allows feasible designs to be output graphically. The roots of an efficient graphics system are to be found in the manner that input data can be structured to represent engineering drawings. An emphasis has therefore been laid on the basic principles that govern organisation of data for engineering drawings. Since this present work is intended to be a model design tool, the text of the discussion is extended to both two- and three- dimensional graphics.

7.2 COMPUTER GRAPHICS SYSTEMS

The use of the computer to draw pictures has been in existence for nearly two decades. Though the idea had been conceived for some time before, the breakthrough came by Ivan Sutherland's demonstration of the SKETCHPAD at MIT (USA) in the early 1960's. During the first decade of its existence, computer graphics was largely monopolised by large organisations,

notably the Aircraft and Motor industries, who could afford the then high operating costs. With the advancement of multi-programming facilities, time-sharing modes, mini-computers, economic operating systems, moderately-priced hardware and software facilities and generally increase in demand, graphical systems can now be on-line to both large and small computing systems at reasonable cost.

In a general sense interactive graphics envelopes construction, storage, retrieval, manipulation and analysis of pictorial data with the aid of computer-driven devices. The engineer uses pictures to transform his designs into reality so that in advanced application of CAD techniques these design programs can be interacted in real-time with the graphics application program to produce instant pictures of design processes. This should be the ultimate goal of every application of digital graphics to an engineering problem.

7.3 GENERATING DIGITAL PICTURES

When a computer draws a picture on an output device, the picture is made up of points, lines and textual material. Line segments can be represented by the coordinates of their end points, textual material by lines and curves and a point by either a row vector (x,y,z) or a column vector :

$$\begin{bmatrix} x \\ y \\ z \end{bmatrix}$$

Data associated with a point, for example, can be stored in binary number pattern which can then be transformed by means of digital-to-analog converter to voltages and finally be displayed on a screen. This simplified sequence outlines the importance that input and output requirements play in any graphics system.

7.3.1 BASIC I/O PRINCIPLES

A graphics system is always on-line to a main support computer either small or large. A data base is usually interfaced with the system from which hardware devices are activated. Each output device has three primitive controls :

1. Cursor control
2. Mode control
3. Input data control.

These primitive controls can be represented in a high level language to execute the following operations, necessary for developing a picture and which can be performed absolutely or relatively :

1. Draw a line or curve.
2. Elct a point
3. Print a text
4. Move without drawing

The importance of these operations vary frcm task to task. For instance, operation (2) will be guite idle in engineering drawing thcugh very active in scientific graph plotting. These four operations are effectively ccntrolled by the crrsor, that is the cursor of a CRT terminal, ballhead of a telytype or pen carriage of a plotter. Mode and input data ccntrcls are direct characteristics of the particular device - that is whether it is a storage tube, a refresh tube cr a raster scan device. For each device picture generation will involve the fcllowing three modes :

i. Initialise the device
ii. Erase the screen for a fresh picture input
iii. Dump stored data to a hard copy output device.

7.3.2 I/O DEVICES

Whilst a program is in executior, there can be an irteractic n such that fresh data is continually being transmitted. Numerous input devices to effect such real-time interaction have been invented since Sutherland used the light pen in the original demonstratic n of the SKETCHPAD. Alphanumeric keyboard is the ccmmcnest but crudest type. More popular devices

include joy stick, various types of tablets, function switches and track balls. Light pens are still popular, principally due to the speed and ease with which they can be used for menu picking.

Output devices fall into two categories - display terminals and digital plotters. Most display terminals are built on Cathode Ray Tube principle. Briefly this is how they function. Electric fields have known peculiar effects on emitted electrons and certain substances emit a visible glow if they are bombarded with electrons. When a stream of electrons is focussed unto a phosphorescent surface a visible trace is left on the surface. Similarly digital electric signals will leave a characteristic trend if discharged unto a phosphorescent surface. However as with CRT devices, the visible trace associated with an electron emission is short-lived. Such a defect is undesirable in graphics and its effect has either been reduced or eliminated by various means. Three main groups of CRT display terminals are :

1. Storage tube terminals
2. Refresh tube terminals
3. Raster scan terminals.

There are two main types of plotters - flat bed and drum - though almost all current models operate on identical principles. Most models have a built-in magnetic tape system that accepts plot commands which are translated to move a pen

carriage. For instance the Calcomp Digital Incremental Plotter Model 936 which was used to output this present work has three pens each capable of attaining speed up to 127 mm/sec along horizontal and vertical axes. In this particular drum plotter, standard size engineering drawing papers (vellum and mylar) can be used.

7.3.3 TRANSFORMING PICTURES

The peculiar nature of civil engineering drawings renders proper use of transformation algorithms quite useful. Some drawings are merely an assemblage of sub-components; others comprise several component detailing. In both cases picture transformations could be effectively used to organise data base. Often the primitives of transformations have to be engaged either singularly or in composite forms. Four such primitives - scaling, translation, rotation and clipping will therefore be defined in relation to two- and three- dimensional analysis. let Pi(Xi,Yi,Zi) and Pi(Xi,Yi) respectively, represent points in three- and two- dimensional cartesian space.

The scaling transformation merely magnifies a subpicture in proportion to a scaling operator. Let the components of this operator in X- ,Y- and Z- directions respectively be Tsx, Tsy and Tsz. In matrix notation, the scaling operation for a two-dimensional analysis is stated as :

$$\{X1 \quad Y1 \quad 1\] \begin{bmatrix} Tsx & 0 & 0 \\ 0 & Tsy & 0 \\ 0 & 0 & 0 \end{bmatrix} = \{X2 \quad Y2 \quad 1\]$$

Eqr. 7.01

where -

$$\{X2 \quad Y2 \quad 1\] = \{X1\ Tsx \quad Y1\ Tsy \quad 1\}$$

In three dimensional analysis the operation is stated as :

$$(X1 \quad Y1 \quad Z1 \quad 1) \begin{bmatrix} Tsx & 0 & 0 & 0 \\ 0 & Tsy & 0 & 0 \\ 0 & 0 & Tsz & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} = (X2 \quad Y2 \quad Z2 \quad 1)$$

Eqr. 7.02

where -

$$\{X2\ Y2\ Z2\ 1\]=\{(X1\ Tsx \quad Y1\ Tsy \quad Z1Tsz \quad 1\}$$

If distortion of the sub-picture is to be eliminated then Eqn. 7.03 must be satisfied :

$$Tsx = Tsy = Tsz \qquad \text{...Eqn. 7.03}$$

Whcle pictures can be built by merely applying a

translatien algcrithm to sulpictures. The translation operation is in fact, a transporting vehicle with which subpictures can be interconnected or moved around the plot space. Assuming the same points as before with Ttx, Tty, Ttz as the translation components in X-, Y- and Z- directions respectively translatien operatior can be stated as

$$\{X1 \quad Y1 \quad 1\} \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ Itx & Ity & 1 \end{bmatrix} = \{X2 \quad Y2 \quad 1\}$$

Where -

$$\{X2 \quad Y2 \quad 1\} = \{X1+Ttx \quad Y1+Ity \quad 1\}$$

In three dimensional analysis the operation is stated as :

$$(X1 \quad Y1 \quad Z1 \quad 1) \begin{bmatrix} 1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 \\ 0 & 0 & 1 & 0 \\ Ttx & Ity & Ttz & 1 \end{bmatrix} = (X2 \quad Y2 \quad Z2 \quad 1)$$

Eqn. 7.05

where -

$$\{X2 \quad Y2 \quad Z2 \quad 1\} = \{X1+Ttx \quad Y1+Ity \quad Z1+Ttz \quad 1\}$$

The third primitive - rotation - is more useful in 3D than in 2D. In 2D the operation is performed about the origin in a preferred direction, say clockwise. In 3D an axis of rotation has to be defined as well as the direction. If the same point is rotated radians in a clockwise direction, the operation can be represented in matrix notation as :

$$(X1 \quad Y1 \quad 1)\begin{bmatrix} \cos t & -\sin t & 0 \\ \sin t & \cos t & 0 \\ 0 & 0 & 1 \end{bmatrix} = (X2 \quad Y2 \quad 1)$$

Eqn. 7.06

where -

$$[X2 \quad Y2 \quad 1] = [X1\cos t + Y1\sin t \quad -X1\sin t + Y1\cos t]$$

A typical 3D rotation about Z-axis through (C, C, C) can be stated as follows :

$$(X1 \quad Y1 \quad Z1 \quad 1)\begin{bmatrix} \cos t & -\sin t & 0 & 0 \\ \sin t & \cos t & 0 & 0 \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} = (X2 \quad Y2 \quad Z2 \quad 1)$$

Eqn. 7.07

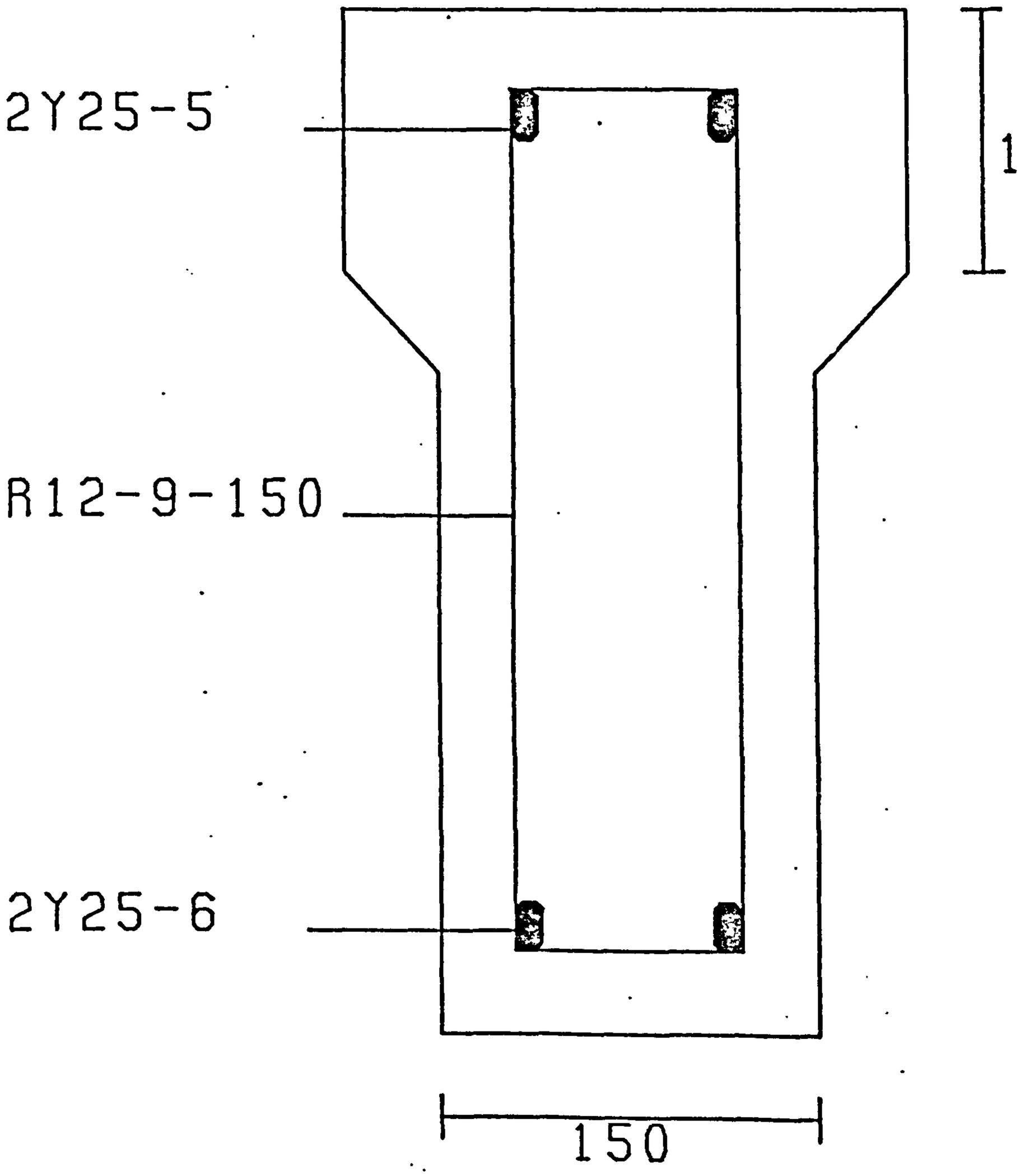
It is seen that in each of the three operations discussed, any

concatenation of one or two operations is merely a matrix addition. The clipping transformation is slightly different from those so far discussed. Its operation can be summarised in three steps :

1. Define an arbitrary rectangular region whose boundaries must not necessarily be parallel to the coordinate axes.
2. Retrieve from this rectangle all the sets of points contained in the clipping rectangle.
3. Algorithmically test for points and end-points of line segments and clip off all parts of line segments which do not satisfy this test.

Step 3 contains an inherent problem with subpictures lying outside the clipping rectangle but whose other part is within the rectangle.

A useful composite transformation formed by concatenating all the four primitives often called 'windowing' is illustrated in Fig. 7.01. In this example, the region of the subpicture residing in a clipping rectangle that houses the rafter cross-section is initially defined, then translated to fill an entire square plot space. Tsx equals Tsy and the clipping rectangle is square so that the subsequent mapping into a square plot space eliminates any possible distortion of the image in the viewport. No rotation is performed in this particular case.



SECTION B-B

FIG 7.01 DETAILING RAFTER WITH A WINDOWING TRANSFORMATION

7.4 DATA STRUCTURE MODELLING

An important feature of any graphics system is the format in which a programming language is structured to represent objects in the computer memory. With ordinary programming the problem does not arise when an algebraic language such as FORTRAN is used, primarily because it is a well developed high-level language that has adequate representation facilities to meet the attributes of ordinary numerical computations that most programs would require. A lot of the objects encountered in computer graphics are, however, irregularly structured and do expand dynamically during program execution. There arises as a result, a need for a peculiar system that can efficiently organise these objects in the memory. In engineering drawing practice once a set of drawings has been done for a project any subsequent major changes can take place as a fresh task. With computer graphics it must only involve data changes with minimum trouble.

Data is fundamental to computing. In its primitive form, it is any raw piece of information that can describe an object or define a purpose. For instance, the shape and size of any of the structural elements described in Section 2.2 can be defined using all the set of coordinates of the vertices and line segments which together can form images of the elements either in 2D or 3D. If such data is well organised it represents an efficient data structure which can be manipulated for a graphics program. The ideal aim of any graphics task therefore is to develop an output system that will be sensitive

interactively to data changes. Like transformations three primitive structures can be identified in data organisation. These primitive forms are : sequential, random and list.

7.4.1 USING PRIMITIVE STRUCTURES

A sequential data organisation is simple and familiar. It selects a common attribute of the data and stores the data elements relative to each other according to the 'sequential' occurrence of the selected attribute. Each time a new record is added or deleted there arises a need to update the rest of the records. Updating records could be cumbersome if the records are large. Thus in its primitive form this structure type would be quite inefficient to use in graphics systems.

In random data organisation each record is assigned an address at a known location which must be specified each time data is to be accessed. It is immediately seen that accessibility will pose problems. Three techniques are used to access data namely direct, dictionary and hash coding. It is usual for one or more of the three to be used in a particular problem. Direct accessing, the simplest of the three, merely retrieves data address 'directly' so that if the records become large the same problem persists. Dictionaries can be drawn for all addreses and the same used to 'look up' addresses sequentially. This is a better alternative to direct searching

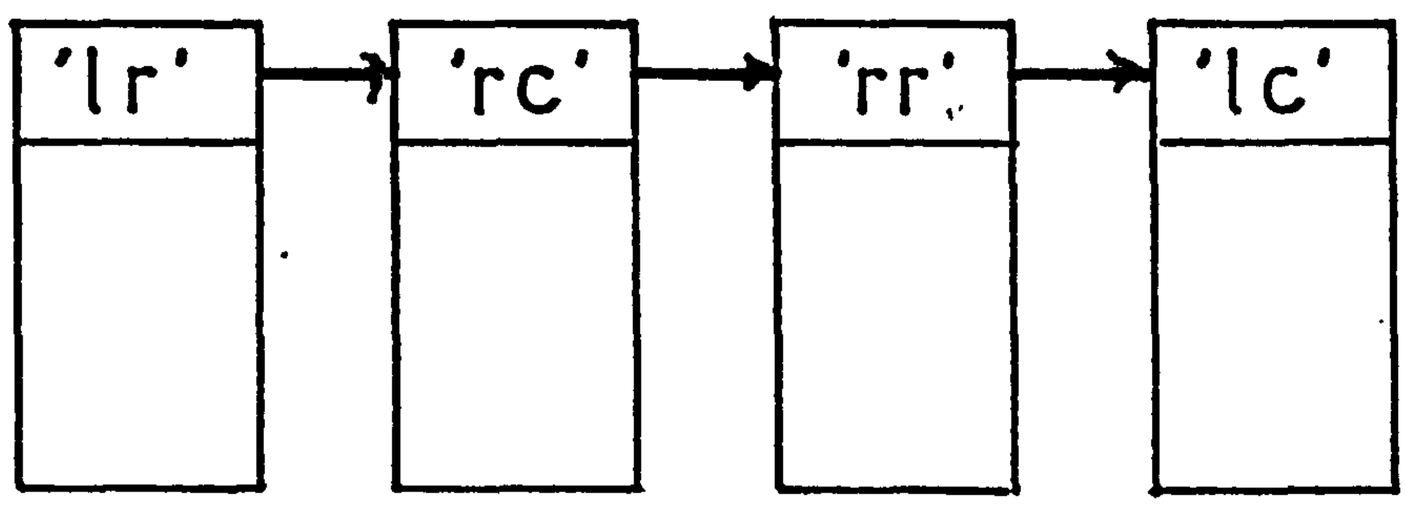
though very large data bases can pcse updating prcblems. When this cccurs binary searching might prcve remedial. Hash coding is dcne by creating a new set of addresses or codes for the data attribute using a standard algcrithm. This is a very efficient stategy except for the possibile cccurrence of a conflict, that is of two or more data names yielding the same address. Conflicts, when they occur, cculd be resolved by various techniques.

A list data crganisation is formed by using pcinters tc link one record with another. It makes use cf the concept of logical and physical relationships of records. The pointer is used as an inter-record accessirg mechanism sc that though the records could cccur physically in a random pattern the sequence of their logical occurrence may be differert.

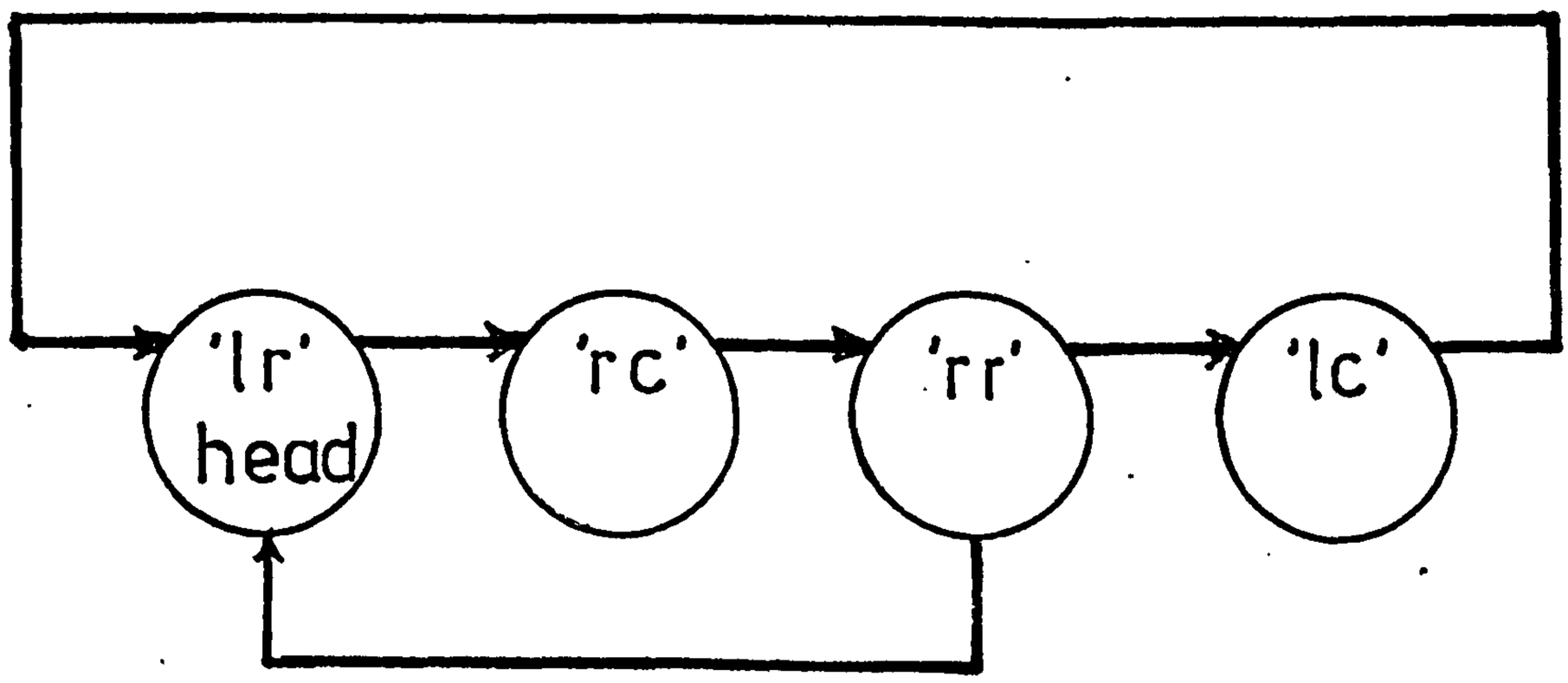
Fig 7.02 illustrates the use of a list structure. In this simple example a list data arrangement is used to draw the elevaticn cf a pcrtal frame. Frcm the left-hand side of the portal frame to the right hand-side, the physical cccurrence of the varicus compcnents differ from the actual logical sequence of the data that actually draws the picture.

7.4.2 DATA ORGANISATION FOR ENGINEERING DRAWINGS

Thcugh all engineering drawings functionally seek to interpret designs, the nature of these design ideas are nct the same.



a list structure



a ring structure

FIG 7.02 LIST AND RING STRUCTURES

For instance mechanical engineering drawings have to be structured at a lower graphic level of points, arcs and segments[3]. In structural design a structure may comprise beams, columns and slabs. Basic graphic patterns can be constructed for each element in such a configuration which can form a set of primitive subpictures for developing the more complex drawing. This requires that in each graphics task, a strategy has to be evolved and a suitable data base developed to suit. Therefore all-purpose rules are inappropriate. Data organisation is also not independent of the local environment - type of computer, peculiarities of the graphics system and the operating system in use.

7.4.3 APPLICATION OF COMPLEX DATA STRUCTURES

Whilst the structure is being designed, sizes and shapes of elements are frequently changing. This implies addition, deletion and updating of graphical input. A designer may wish to carry out minor elemental changes without disturbing the rest of the component such as altering corbel details which will not affect the details of reinforcements in the main column. In this case updating the remaining data will be minor if an efficient complex data form has been used but a relatively phenomenal updating could result if the structure arrangement is of a sequential type. Simple data forms are adequate if the size of data is small but there is need for an

efficient complex data structure to cope with the peculiarities of structural design. A number of complex forms could be built from the three basic types discussed in Section 7.4.1. Some of these are

1. Ring structure
2. Tree structure
3. Hierarchical structure

Rings are extended list structures. As in a list structure the records are connected by pointers but the last record points back to the first record. An indicator must be inserted in the first record to distinguish the starting point from the first record. The functioning power of rings lies in their ability to retrieve and process all the input in one ring and at the same time be able to branch off at any level into logically related rings and then to perform similar functions. Fig 7.02 shows how a list structure can be transformed into a ring to perform different functions. In Fig 7.09 it is intended to preserve the elevation of the portal frame whilst processing details of the concrete rafter. Data is so structured that a branching off to perform this task is possible. The immediate advantage of this arrangement is that if the trial design of a rafter is rejected to warrant a fresh data input, the progress is interactively reflected in both subpictures but the corresponding price for such an efficient technique is that a relatively more storage location must be provided.

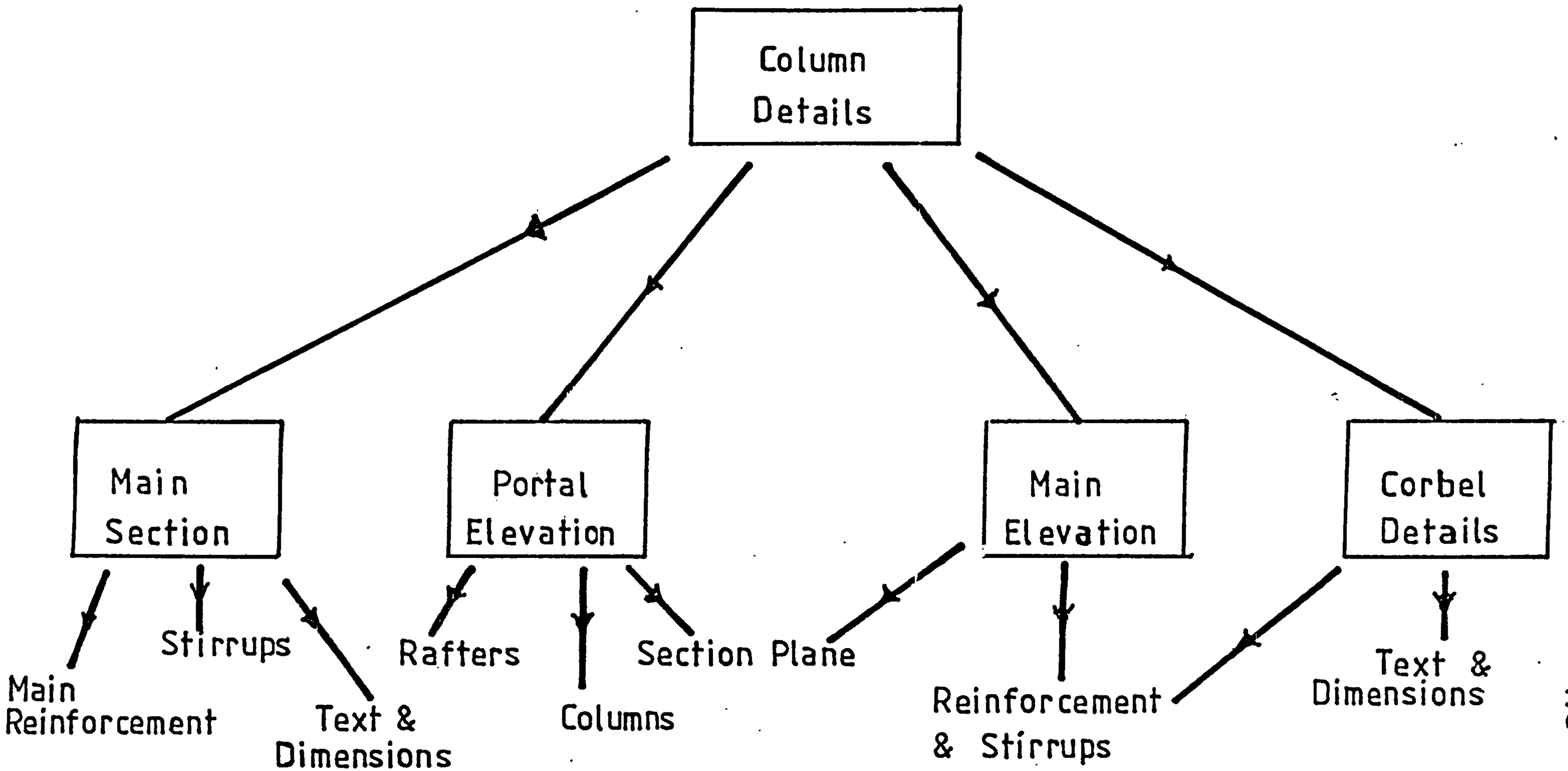


FIG 7.03 DATA STRUCTURE OF COLUMN GRAPHICS

A tree is a graph without closed circuits having chains between its nodes. It has an identification block at the first level with pointers to a hierarchy of levels. The first level has one node and is often termed a 'root'. All other nodes can have additional relationship with nodes at higher levels. The structure adopted for the column drawings is illustrated in Fig. 7.03.

A file comprising tree structures with levels of hierarchy and constructed with rings is called an HIERARCHICAL structure. It comprises an initial ring which is logically related to one level of rings which in turn is logically related to another level of rings and so on. The construction includes the use of a number of pointers which makes data accessing and updating fast since only pointers are being changed and not storage locations.

7.5 STRUCTURAL DESIGN WITH COMPUTER GRAPHICS

Fig. 7.04 illustrates the interface between the cost model and the graphics program. At a typical session the graphics mode is initialised. Analysis and Design routines of the cost model link with the graphics application program. The state of the design process is monitored on a graphics terminal which can interrogate the process at any stage to inspect element sizes. In the event of a constraint being violated, the option for

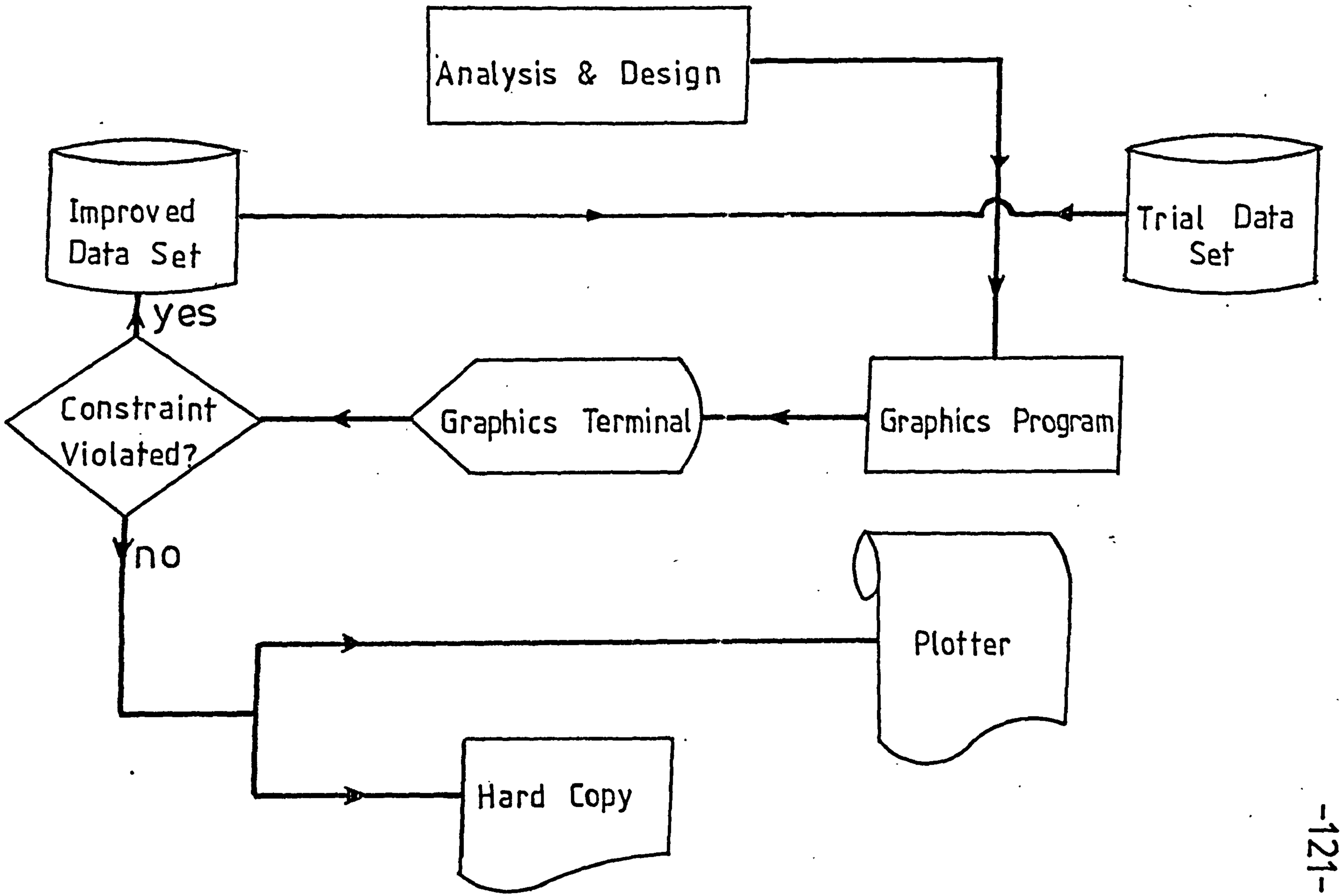


FIG 7.04 USER DESIGN MODE

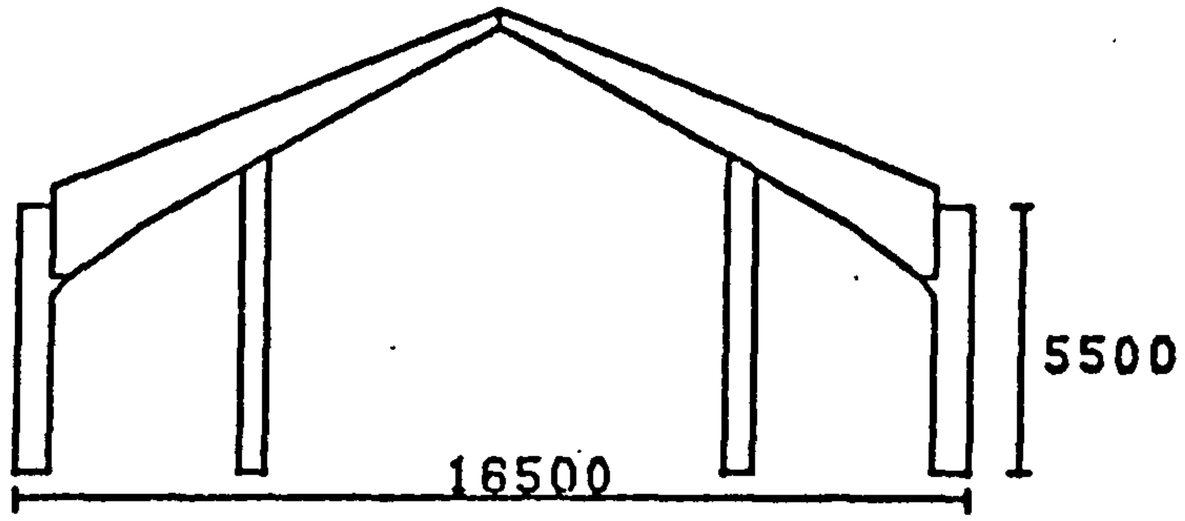
improvement is available. The iterative process continues till all design constraints are satisfied and element sizes are acceptable to the designer.

The Integrated Graphics (IG) routines were used to develop the graphics program. IG is a set of fortran callable subroutines[1,5] developed to provide an interactive graphics facility in a time-sharing environment.

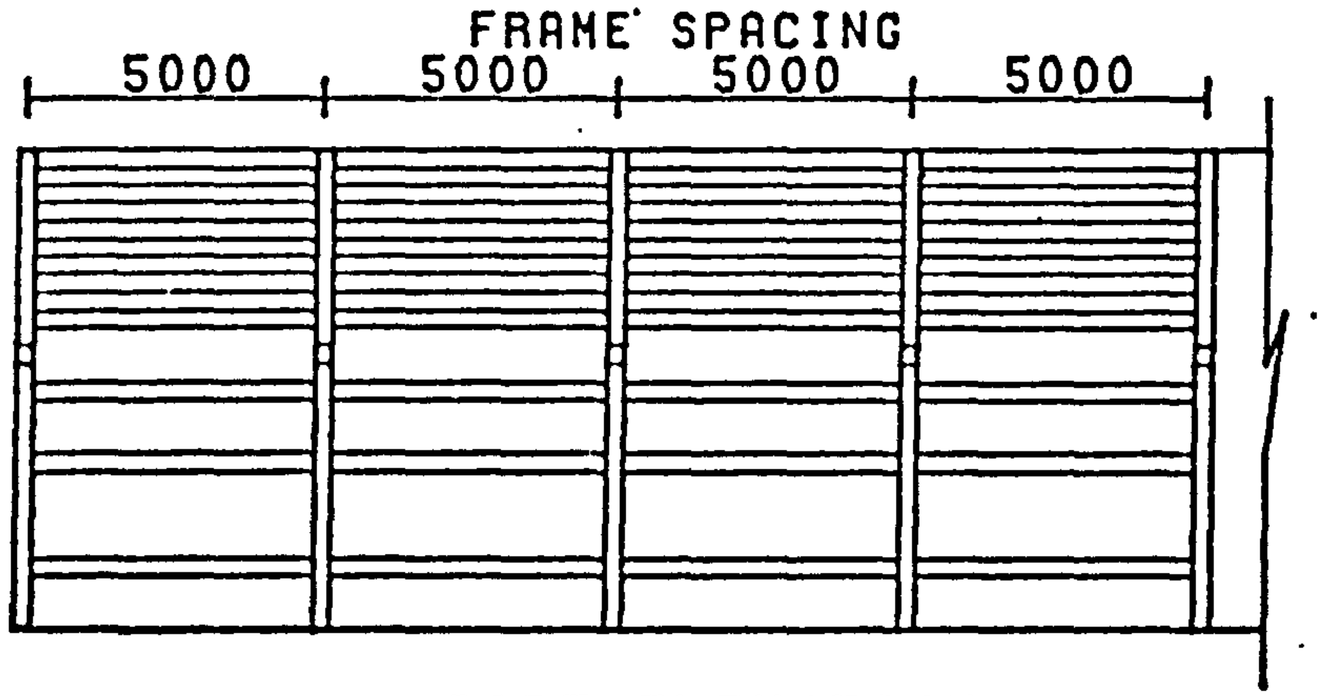
At the end of the day if a PICT command is issued a set of eight engineering drawings and a bending schedule sheet are digitally drawn (Figs 7.05-7.13)

A typical extract from the program given below draws a purlin outline (Fig. 7.10) :

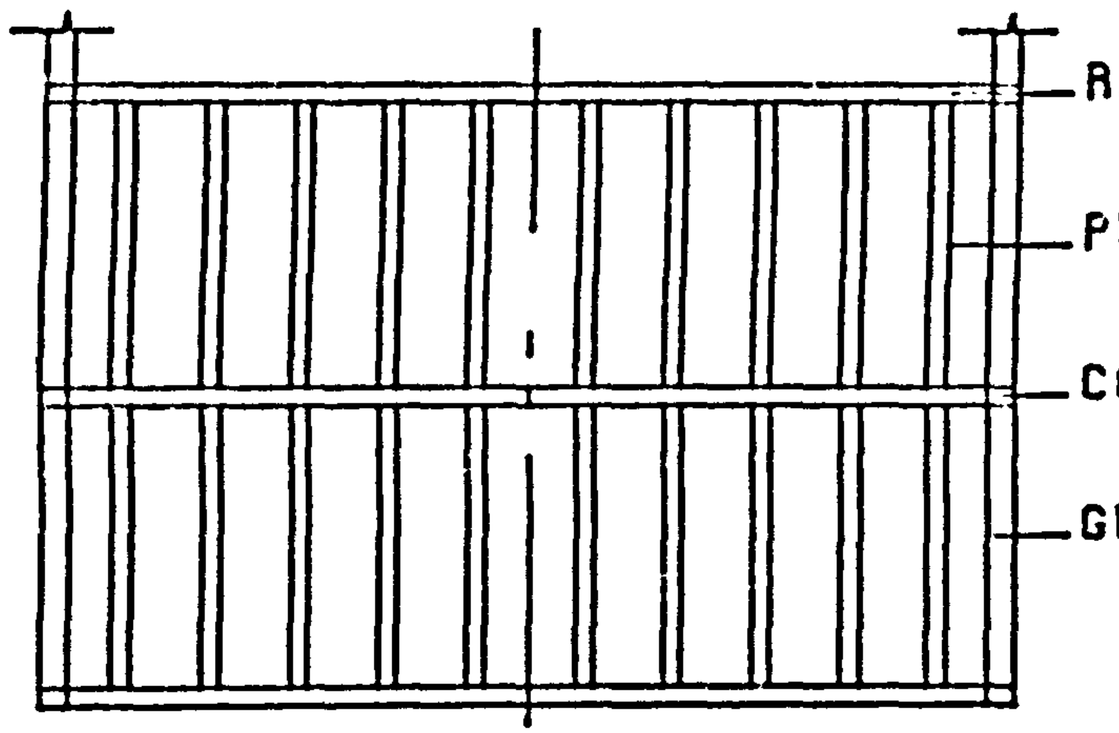
```
C
C *** FFRAME PURLIN GRAPHICS ***
C
      CALL IGEGNS('PSEC')
      CALL IGMA(0.5,0.4)
      DO 30 J=1,8
   30 CALL IGDA(Xi,Yi,)
      CALL IGEGNO('PUEC')
      CALL IGMA(a,b)
      PI=3.1415927
      DO 200 K=625,875,1
      THETA=K*0.002*PI
  200 CALL IGDA(0.075*CCS(THETA),0.075*SIN (THETA))
      CALL IGENDO('PUEC')
C
C *** ZCOM IN WITH PUELIN ARC ***
C
      CALL IGTRAN('PUEC',MOVE, 0,56,0.20)
      CALL IGENDS('PSEC')
C
C
C
```



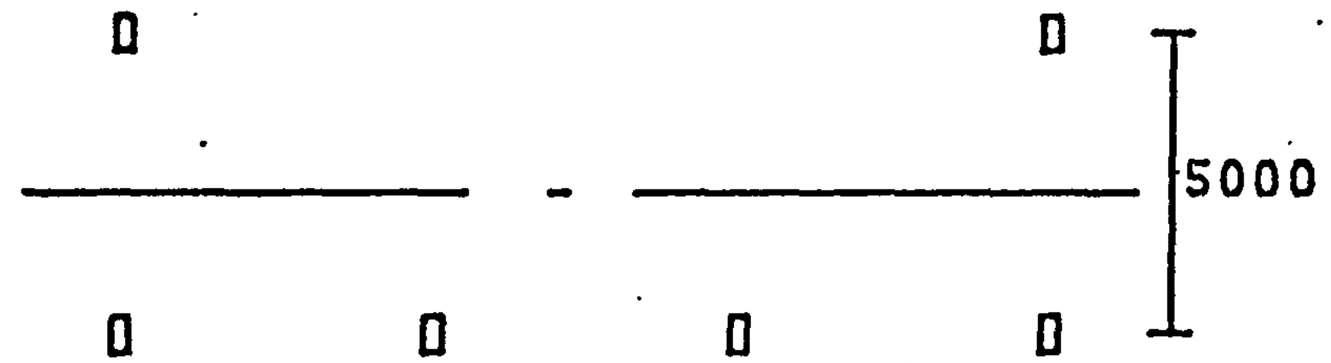
FRONT ELEVATION



SIDE ELEVATION



ROOF PLAN



GROUND PLAN

UNIVERSITY OF NEWCASTLE UPON TYNE DEPARTMENT OF CIVIL ENGINEERING	
OPTIMUM DESIGN OF PRECAST CONCRETE FRAMED BUILDINGS GENERAL ARRANGEMENT	REF:- OPT/01/1979
	SCALE:- 1:200
	DESIGNED BY : KA

FIG 7.05 COMPUTER GRAPHICS

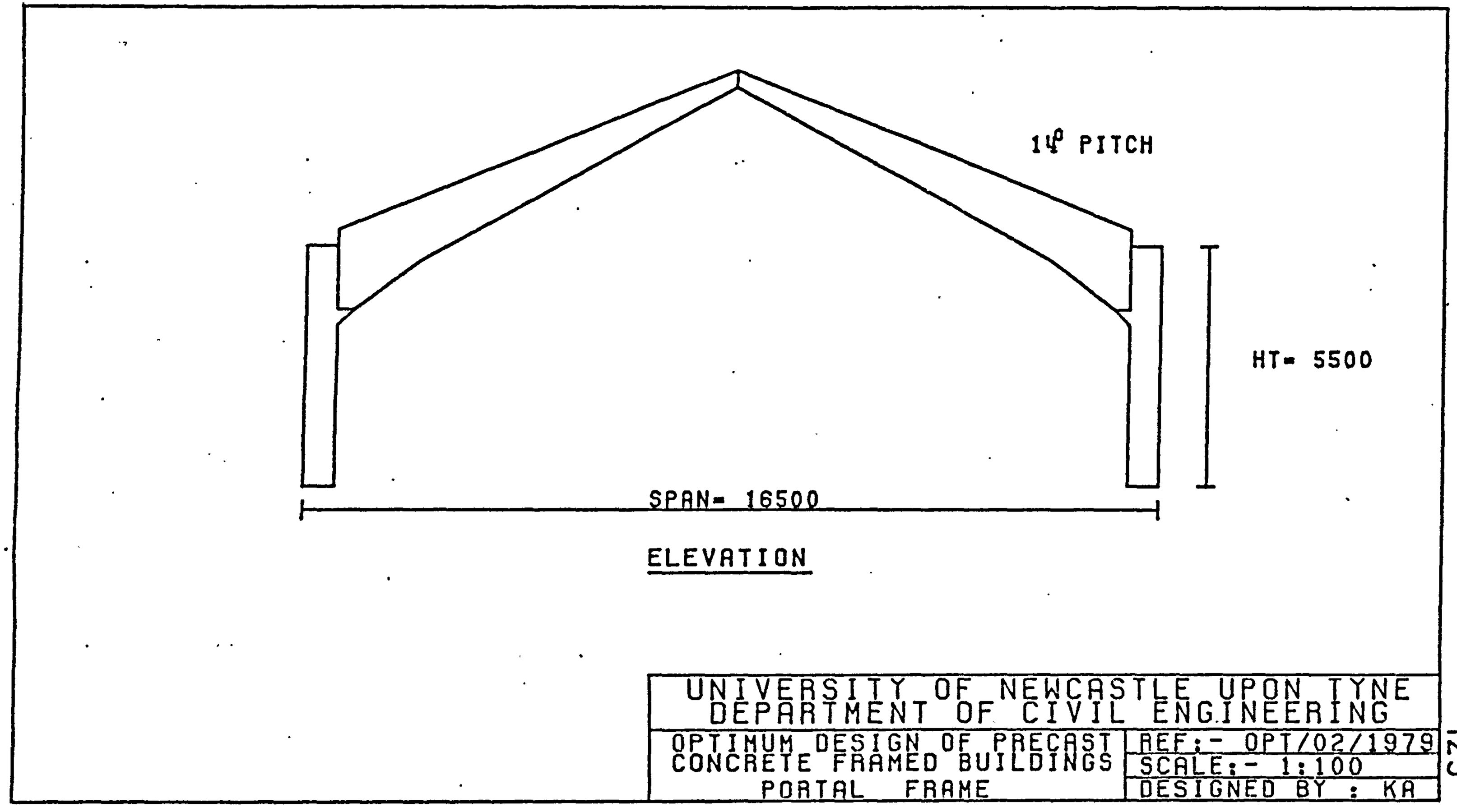
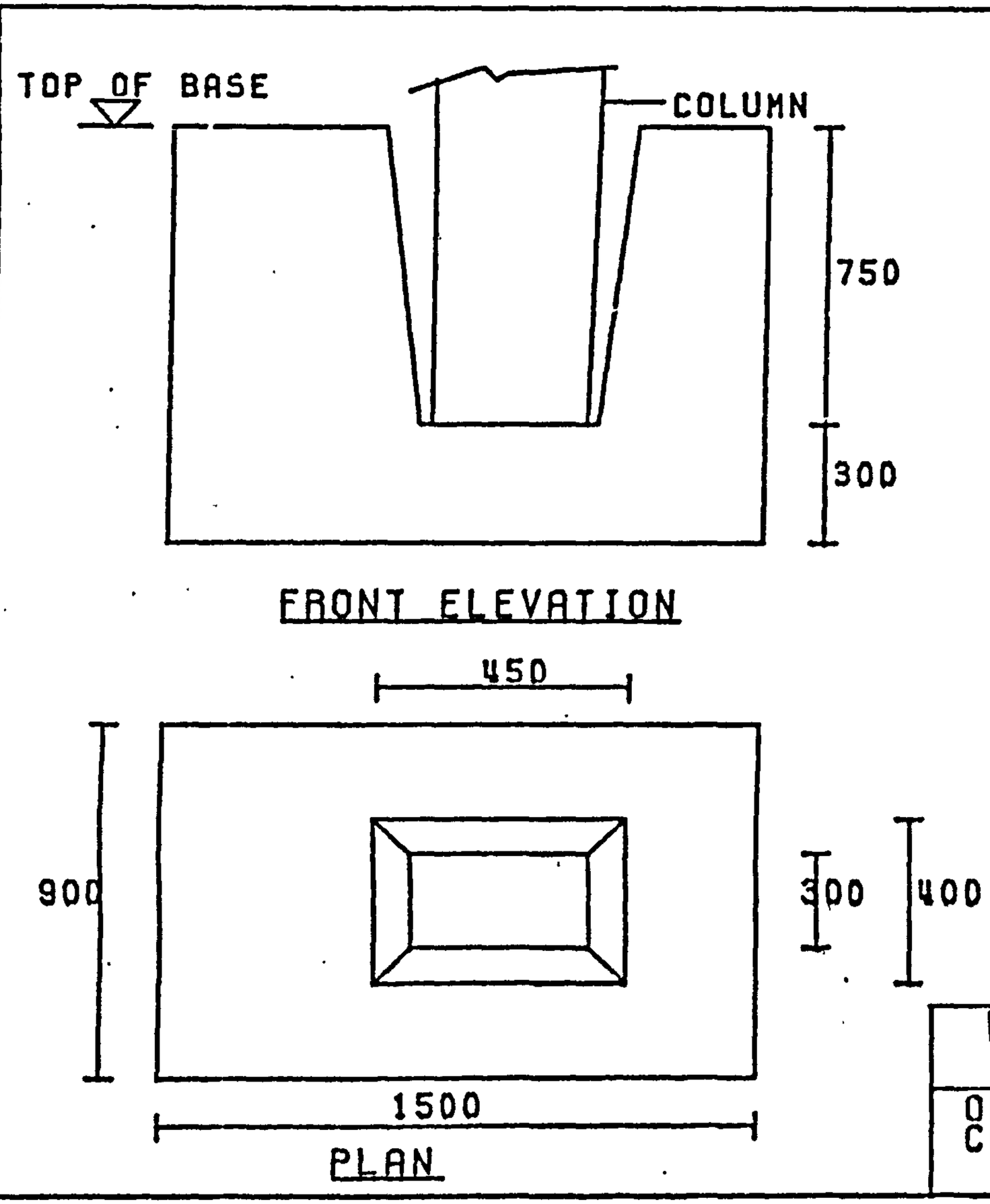
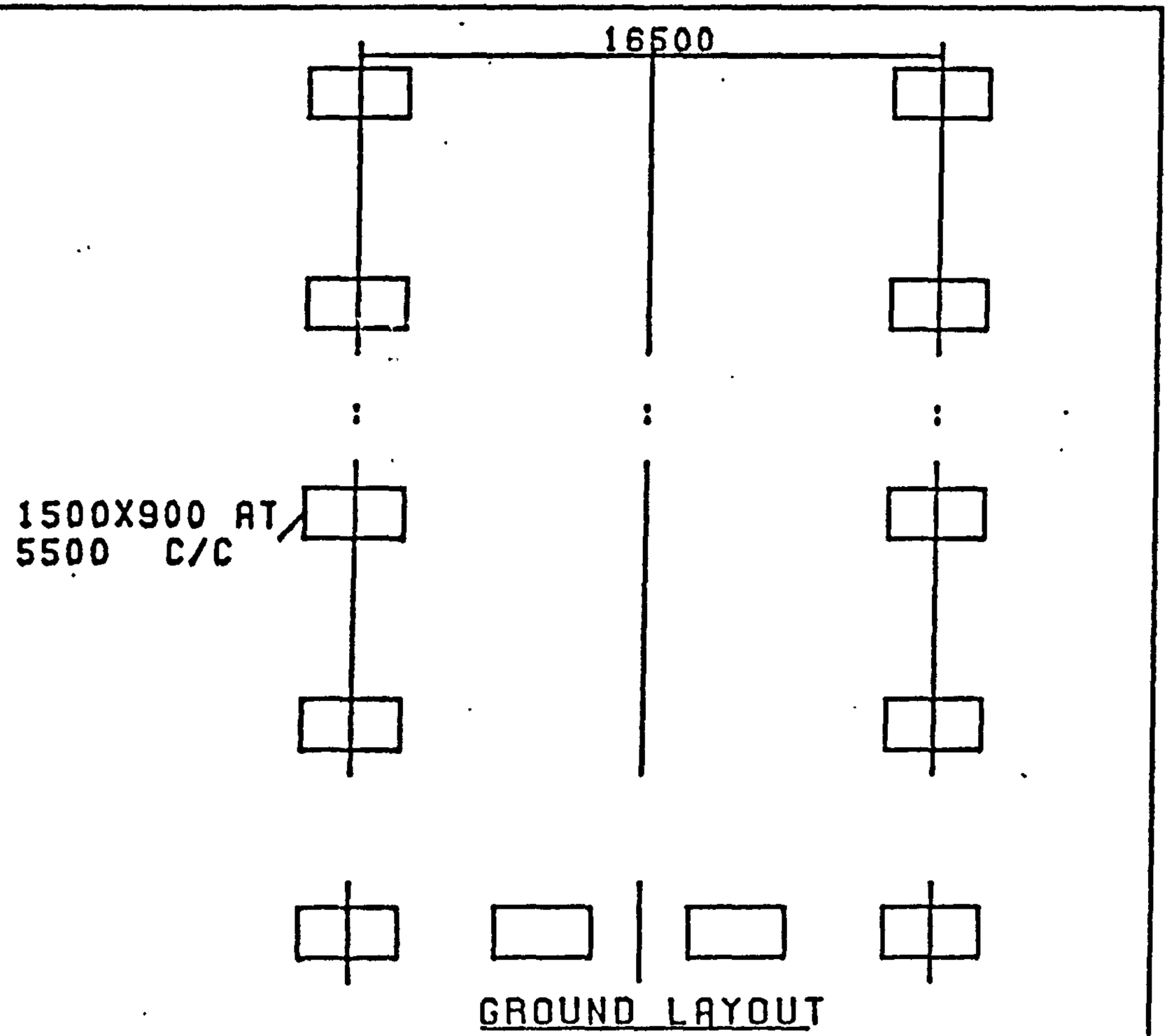


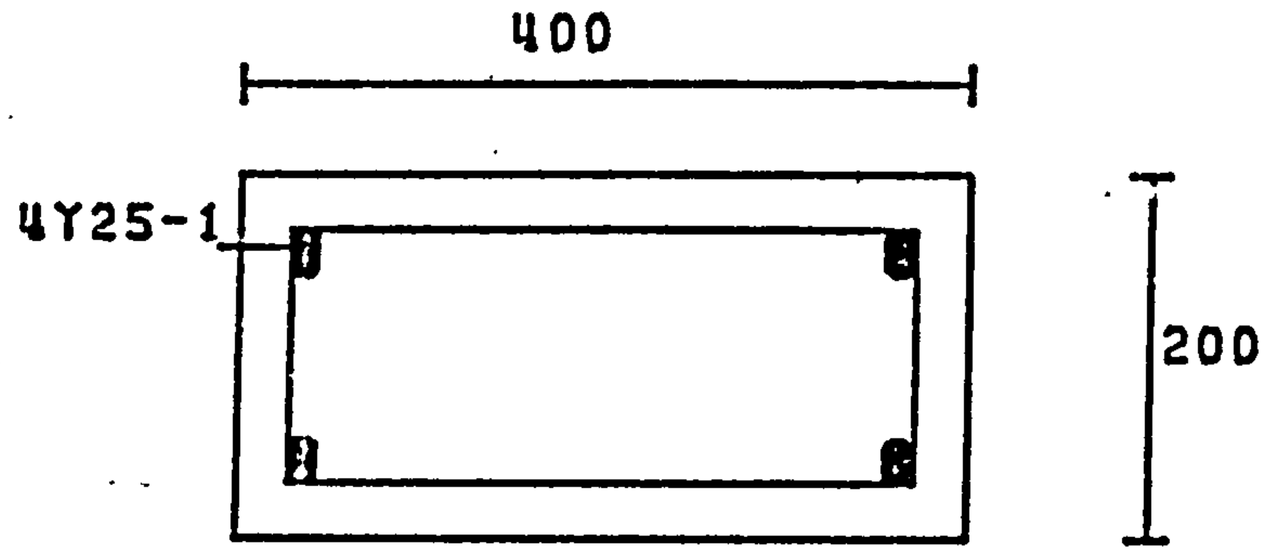
FIG 7.06 COMPUTER GRAPHICS



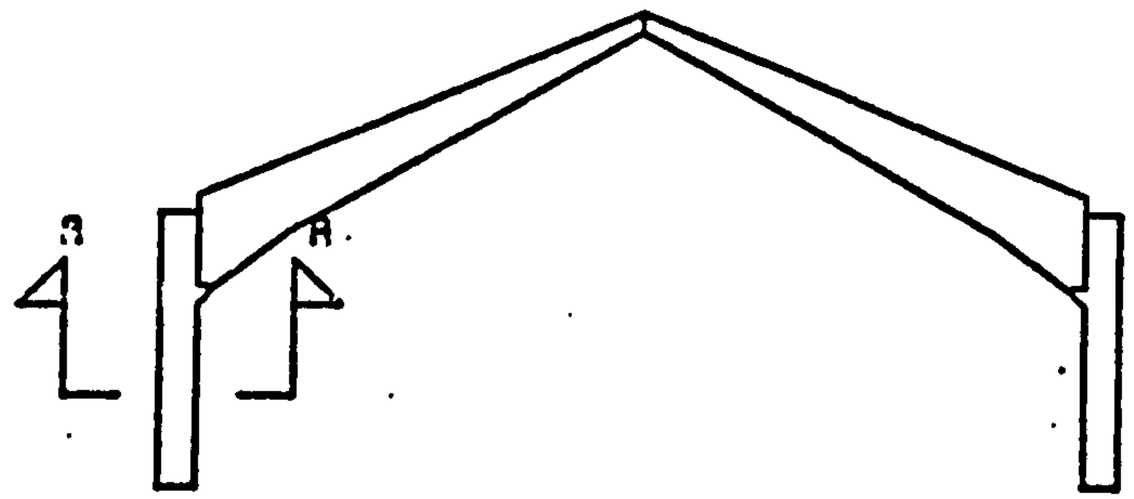


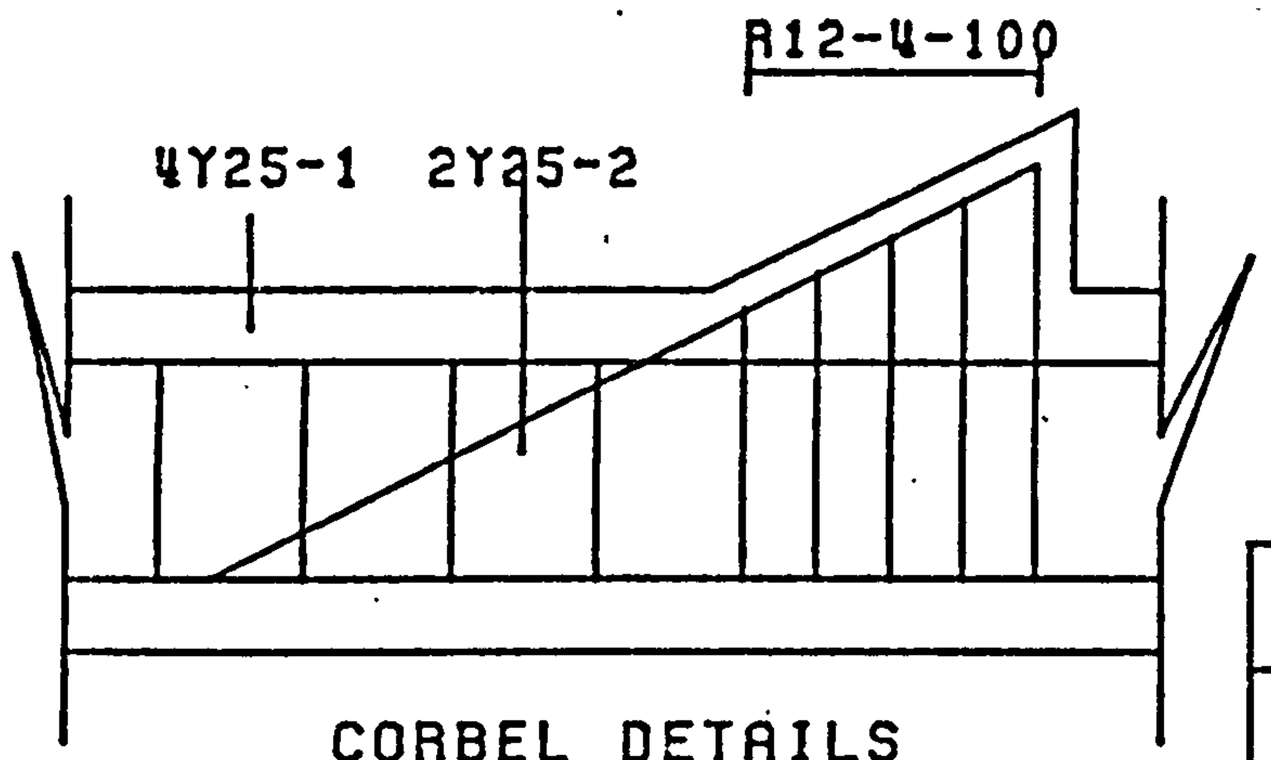
UNIVERSITY OF NEWCASTLE UPON TYNE DEPARTMENT OF CIVIL ENGINEERING	
OPTIMUM DESIGN OF PRECAST CONCRETE FRAMED BUILDINGS FOUNDATIONS	REF:- OPT/03/1979
	SCALE: 1:20, 1:200
	DESIGNED BY : KA

FIG 7.07 COMPUTER GRAPHICS

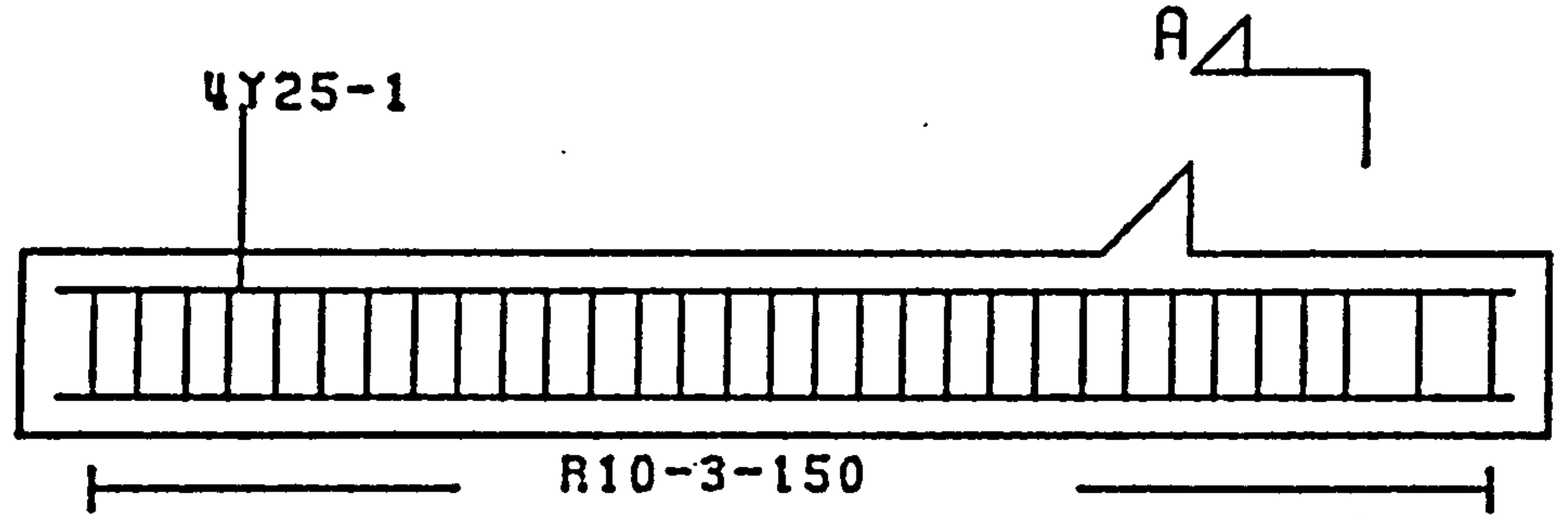


SECTION A-A





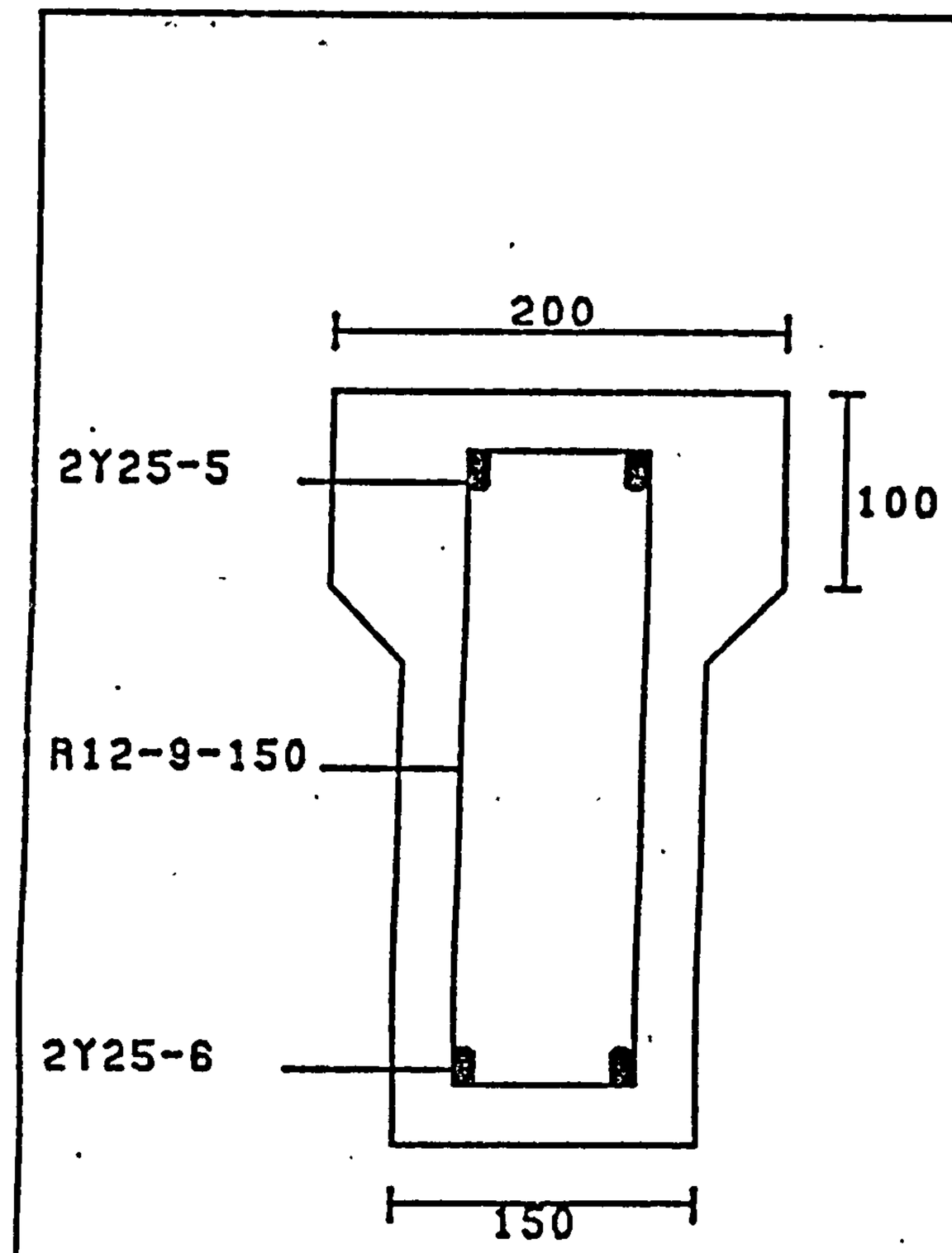
CORBEL DETAILS



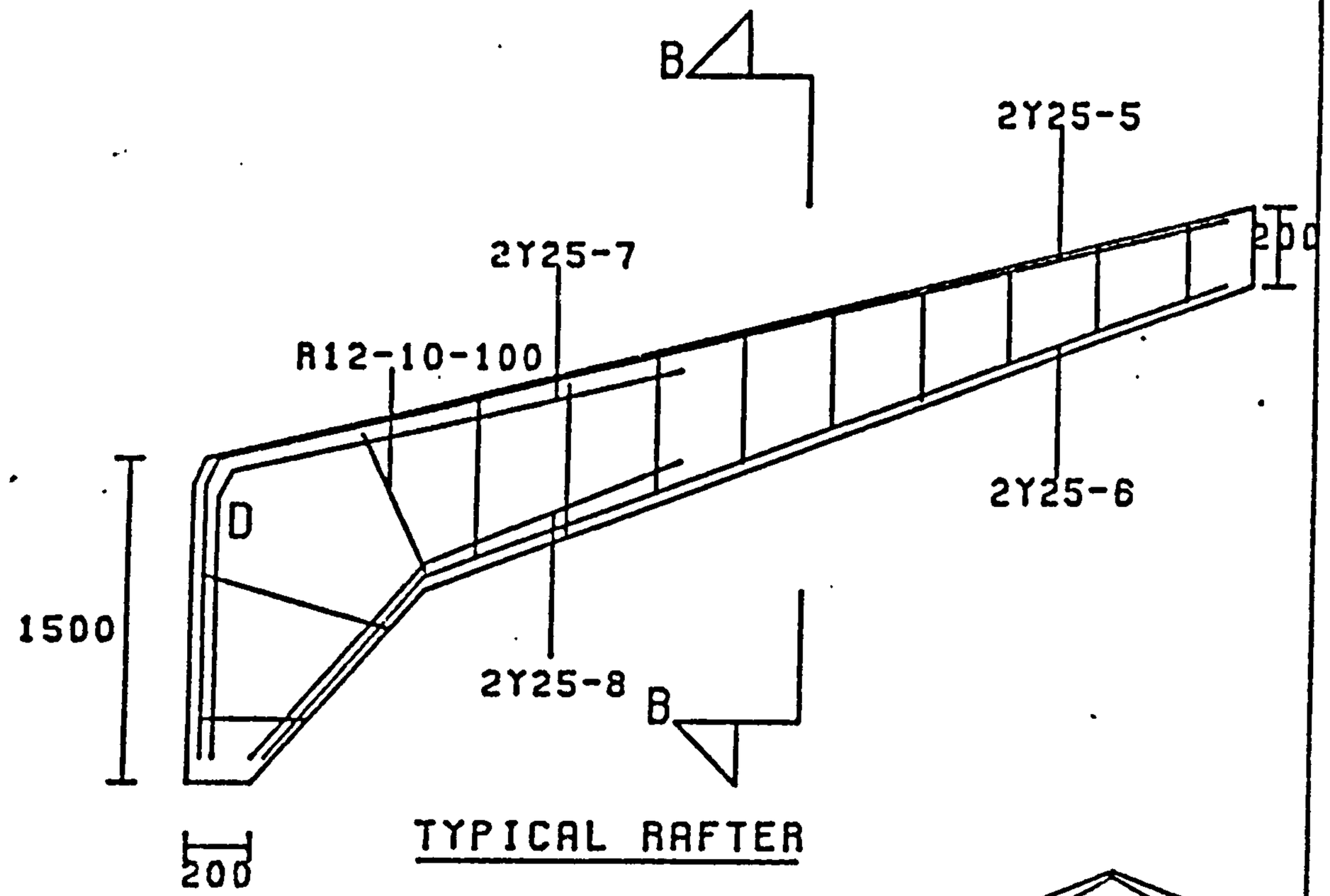
TYPICAL COLUMN

UNIVERSITY OF NEWCASTLE UPON TYNE DEPARTMENT OF CIVIL ENGINEERING	
OPTIMUM DESIGN OF PRECAST CONCRETE FRAMED BUILDINGS COLUMN DETAILS	REF:- OPT/04/1979
	SCALES: 1:5, 1:50
	DESIGNED BY : KA

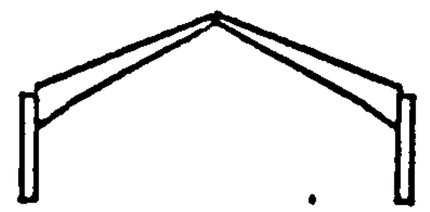
FIG 7.08 COMPUTER GRAPHICS



SECTION B-B



TYPICAL RAFTER



UNIVERSITY OF NEWCASTLE UPON TYNE DEPARTMENT OF CIVIL ENGINEERING	
OPTIMUM DESIGN OF PRECAST CONCRETE FRAMED BUILDINGS RAFTER DETAILS	REF:- OPT/05/1979
	SCALES: 1:5, 1:50
	DESIGNED BY : KA

FIG 7.09 COMPUTER GRAPHICS

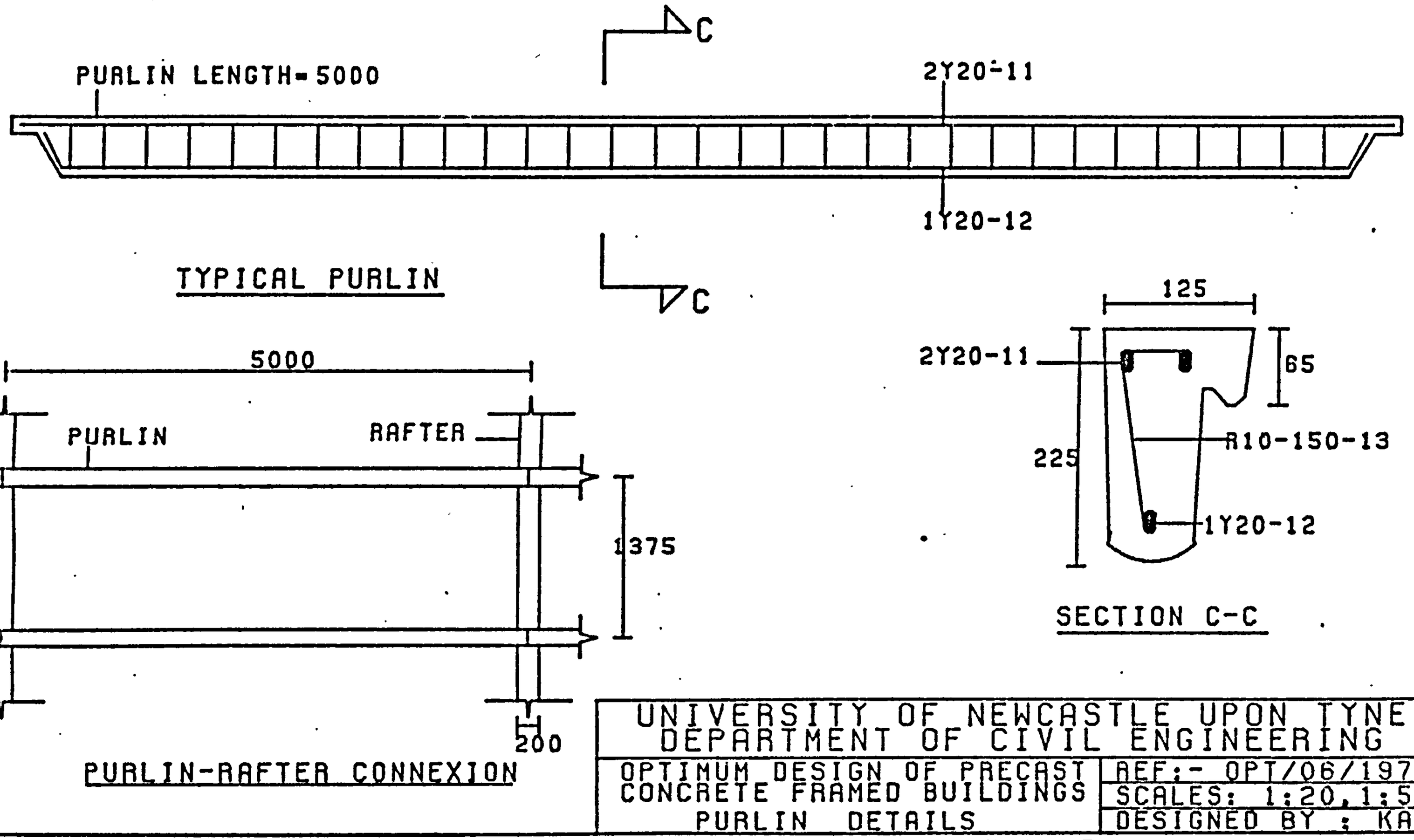
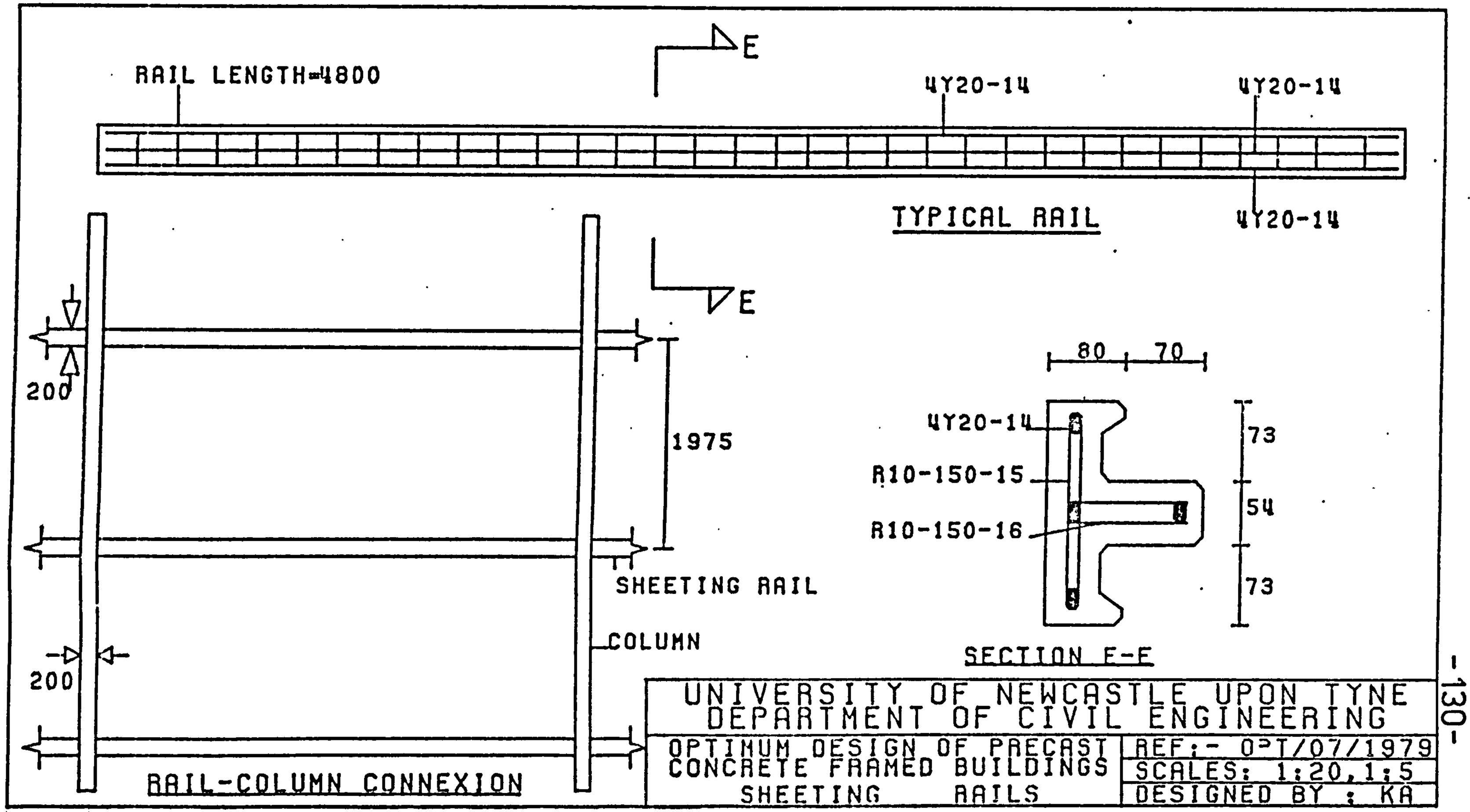
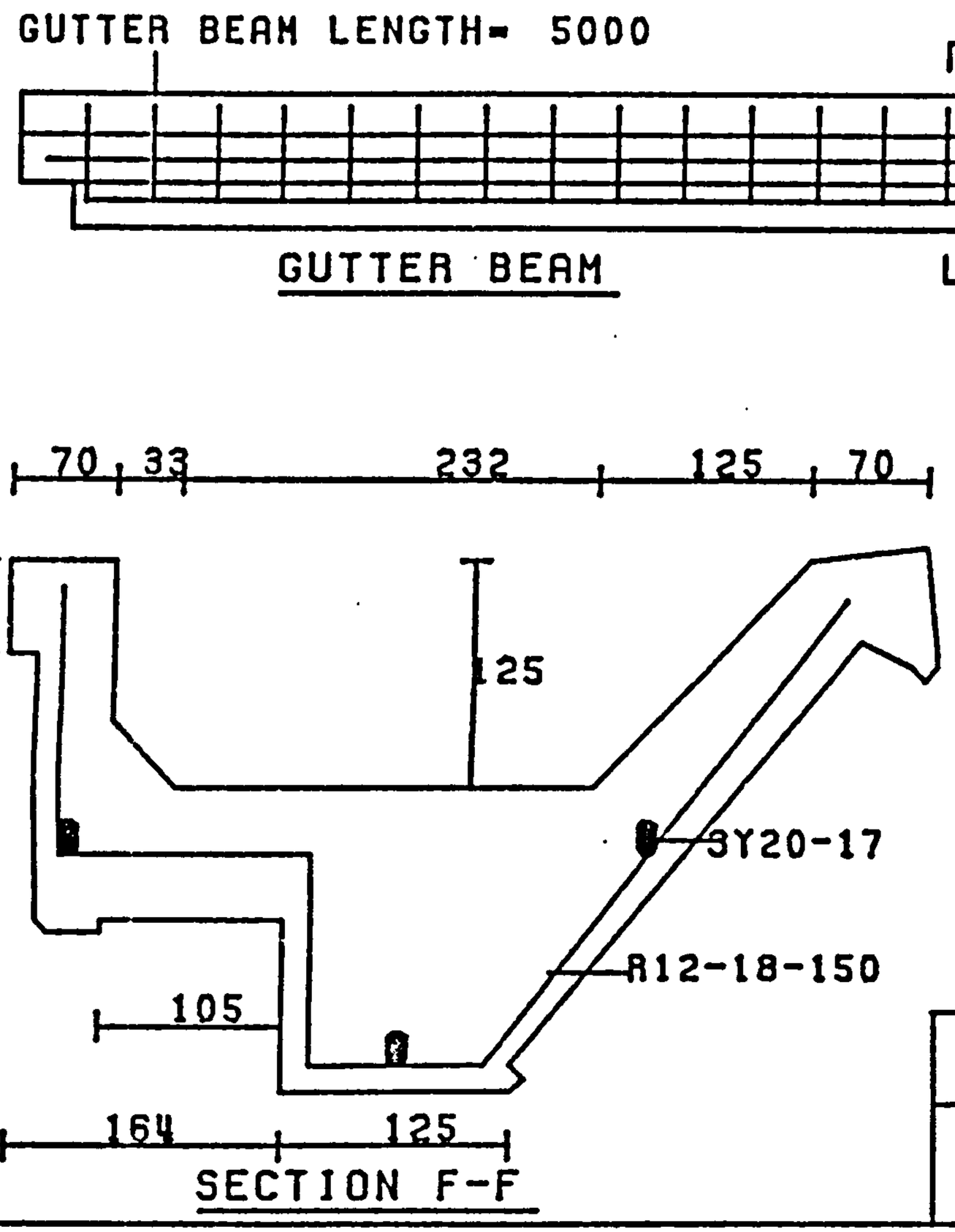


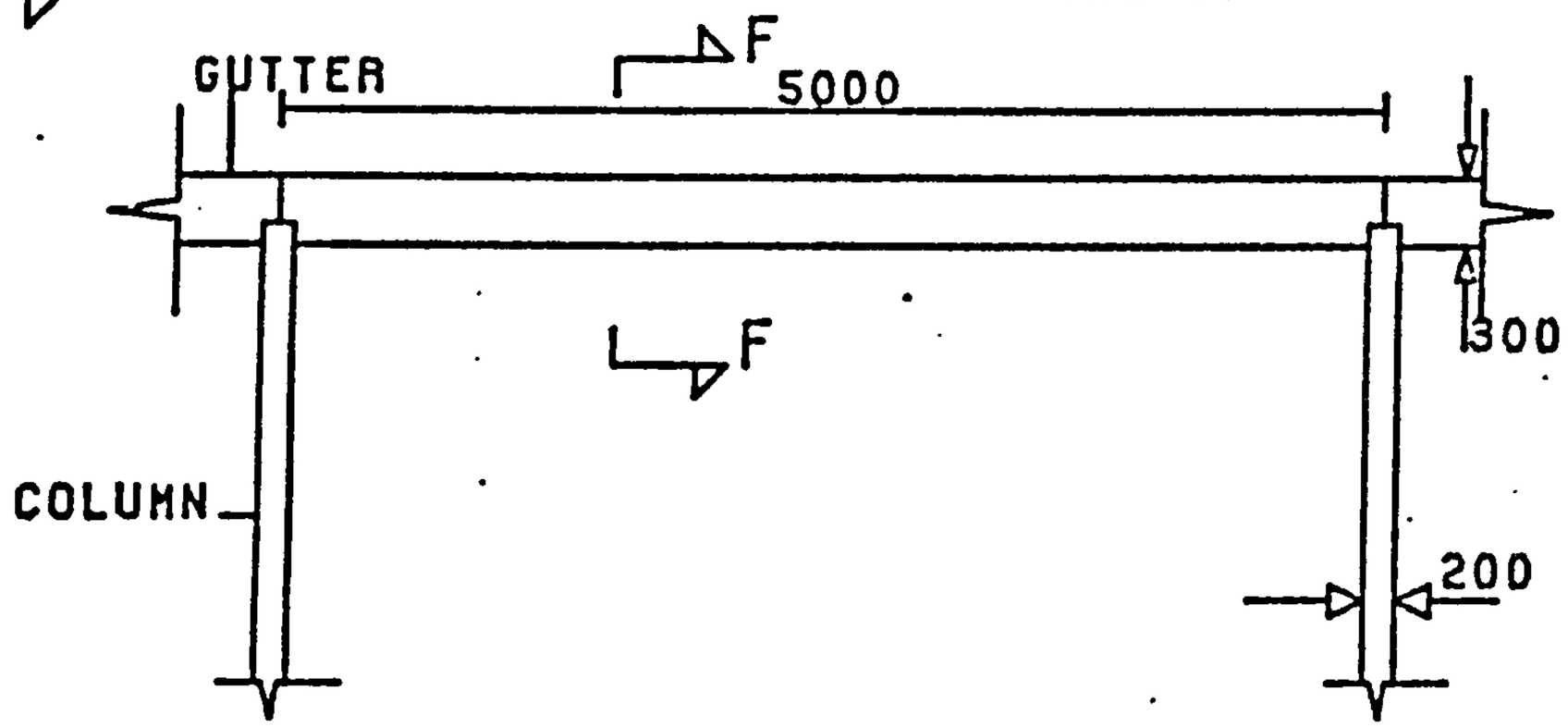
FIG 7.10 COMPUTER GRAPHICS



UNIVERSITY OF NEWCASTLE UPON TYNE DEPARTMENT OF CIVIL ENGINEERING	
OPTIMUM DESIGN OF PRECAST CONCRETE FRAMED BUILDINGS SHEETING RAILS	REF:- O²T/07/1979
	SCALES: 1:20,1:5
	DESIGNED BY : KA

FIG 7.11 COMPUTER GRAPHICS





GUTTER-COLUMN CONNEXION

UNIVERSITY OF NEWCASTLE UPON TYNE DEPARTMENT OF CIVIL ENGINEERING	
OPTIMUM DESIGN OF PRECAST CONCRETE FRAMED BUILDINGS EAVES GUTTER BEAMS	REF:- OPT/08/1979
	SCALES: 1:20, 1:5
	DESIGNED BY : KA

FIG 7.12 COMPUTER GRAPHICS

UNIVERSITY OF NEWCASTLE UPON TYNE * DEPARTMENT OF CIVIL ENGINEERING
OPTIMUM DESIGN OF PRECAST CONCRETE FRAMED BUILDINGS
BAR SCHEDULE SHEET — BAR SCHEDULE REF: OPT/01-08/1979

MEMBER	BAR MARK	TYPE & SIZE	NO. OF MBRS.	NO. IN EACH	TOTAL NO.	LENGTH OF EACH BAR (MM)	SHAPE CODE	A (MM)	B (MM)	C (MM)	D (MM)	E/R (MM)
COLUMN	1	Y25	14	4	56	5950	20	5950				
"	2	Y25	14	2	28	160	99					
"	3	R10	14	39	546	1200	60	150	350			
"	4	R12	14	4	56	1400	60	200	400			
RAFTER	5	Y25	10	2	20	9744	62	1500	1800	8244		
"	6	Y25	10	2	20	9544	62	1300	800	8244		
"	7	Y25	10	2	20	4100	62	1500	1500	2600		
"	8	Y25	10	2	20	2600	62	1500	700	1100		
"	9	R12	10	254	540	1300	60	300	100			
"	10	R12	10	8	80	1900	60	600	100			
PURLIN	11	Y20	48	2	96	4950	20	4950				
"	12	Y20	48	1	48	7000	49	1400	1400	4200	1200	1200 120
"	13	R10	48	34	1632	300	99					
NAIL	14	Y20	24	4	96	4550	20	4550				
"	15	R10	24	31	744	800	81	150	20			
"	16	R10	24	31	744	700	81	100	20			
GUTTER	17	Y20	8	3	24	4950	20					
"	18	R12	8	34	272	850	99	4950				

FIG 7.13 COMPUTER GRAPHICS

7.6 CONCLUSIONS

This chapter has cutlined how the graphical cutput system for PFRAME has been developed. The program is constructed from low level subpictures which make use of primitive and composite transformations and efficient data base modelling. It is acknowledged that an effective utilisation of the techniques which are used here cannot be realised without a suitable graphics system supported by a suitable operating system.

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CHAPTER 8

RESULTS

8.1 OVERVIEW

Some results of the study have aready appeared ir Chapters 5,6 and 7. In this chapter, several data sets are input to the suite of programs and the resultant outputs are presented. In particular, the distribution cf costs, effect cf primary variables on cost, optimal ccst response ard the economics of running the programs are presented. After each set cf results presented a discussion ensues cn its significance.

8.2 DISTRIBUTION OF CCSTS

Sample data was input tc the cost model and the resultant costs are presented in Tables 8.01 - 8.08. Fcr each data set the following assumpticns were made :

1. Pitch of the building is 14°
2. Haulage distance is assumed tc be 40 miles
3. The building is located in an open ccuntry where wind velocity is taken as 45 m/s
4. Building height tc eaves is 6m.

Tables 8.01 and 8.02 show costs of single-span buildings whose spans are 20 m, with frame spacing of 5 m and number of bays varying between 10 and 40. These buildings are typical agricultural houses used by both small scale and large scale farmers. Single skin cladding is used on the roof and the sides.

In Tables 8.03 and 8.04, the cladding used on the buiding is sandwich construction for both the roof and the sides. These buildings could be used as warehouses where it is desirable to control the room temperature to meet specification requirements. The buildings are 2-span with number of bays ranging from 5 to 20, span of 20 m and frame spacing of 5 m.

The cost of four 3-span buildings are presented in Tables 8.06 and 8.07. These buildings are suitable for heavy industrial activities where some amount of heat will be generated from heavy duty machines and the use of single skin cladding is therefore preferred on both roof and sides. Spans are limited to 20 m and frame spacing of 3.33 m is used whilst the number of bays vary from 5 to 20.

Buildings suitable for light industrial factories are shown in Tables 8.07 and 8.08. Frame spacing varies from 4 m to 6 m, number of bays from 4 to 10 and a span of 15 m is used throughout. A typical light industrial factory is cladded with high quality sandwich construction type.

Table 8.09 summarises how the cost of the structure is distributed over the five main factors. The cost of 100

TABLE 8.01 COST DISTRIBUTION FOR SINGLE SPAN AGRICULTURAL BUILDINGS

	1000 m² Building		2000 m² Building	
	£	%	£	%
Fabrication	4976.85	44.07	9677.79	44.90
Haulage	268.08	2.37	522.25	2.42
Foundation	651.73	5.77	1244.22	5.77
Cladding	4468.64	39.56	8258.14	38.32
Erection	930.10	8.23	1850.71	8.59
Total	11292.40	100.00	21553.02	100.00

TABLE 8.02 COST DISTRIBUTION FOR SINGLE SPAN AGRICULTURAL BUILDINGS

	3000 m² Building		4000 m² Building	
	£	%	£	%
Fabrication	14378.57	45.20	20045.94	46.58
Haulage	776.41	2.44	1038.58	2.40
Foundation	1836.70	5.77	2529.19	5.64
Cladding	12047.38	37.87	15836.89	36.80
Erection	2771.32	8.72	3691.94	8.58
Total	31810.38	100.00	43034.53	100.00

TABLE 8.03 COST DISTRIBUTION FOR 2- SPAN WAREHOUSES

	1000 m² Building		2000 m² Building	
	£	%	£	%
Fabrication	43005.45	27.80	8142.64	28.03
Haulage	223.63	1.44	422.83	1.46
Foundation	533.24	3.44	977.60	3.36
Cladding	9958.68	64.29	18579.74	63.95
Erection	469.80	3.03	930.10	3.20
Total	15490.80	100.00	29052.90	100.00

TABLE 8.04 COST DISTRIBUTION FOR 2- SPAN WAREHOUSES

	3000 m² Building		4000 m² Building	
	£	%	£	%
Fabrication	11979.83	28.10	15817.02	28.15
Haulage	622.02	1.46	821.21	1.46
Foundation	1421.96	3.34	1866.33	3.32
Cladding	27211.08	63.84	35832.14	63.78
Erection	1390.41	3.26	1850.71	3.29
Total	42625.30	100.00	56187.40	100.00

TABLE 8.05 COST DISTRIBUTION FOR 3- SPAN HEAVY INDUSTRIAL BUILDINGS

	1000 m² Building		2000 m² Building	
	£	%	£	%
Fabrication	5205.80	49.98	9768.31	51.02
Haulage	274.06	2.63	513.15	2.68
Foundation	710.98	6.83	1303.47	6.81
Cladding	3755.90	36.05	6630.32	34.63
Erection	469.80	4.51	930.10	4.86
Total	10416.54	100.00	19145.34	100.00

TABLE 8.06 COST DISTRIBUTION FOR 3- SPAN HEAVY INDUSTRIAL BUILDINGS

	3000 m² Building		4000 m² Building	
	£	%	£	%
Fabrication	14330.83	51.43	18893.36	51.62
Haulage	752.24	2.71	991.33	2.70
Foundation	1895.95	6.80	2488.44	6.80
Cladding	9492.94	34.07	12378.06	33.82
Erection	1390.41	4.99	1850.714	5.06
Total	27862.37	100.00	36601.89	100.00

TABLE 8.07 COST DISTRIBUTION FOR 4- SPAN FACTORY BUILDINGS

	1000 m² Building		2000 m² Building	
	£	%	£	%
Fabrication	5024.40	49.48	8229.00	48.48
Haulage	272.59	2.68	428.02	2.52
Foundation	740.61	7.29	1036.85	6.12
Cladding	3738.93	36.82	6716.74	39.57
Erection	377.74	3.73	561.86	3.31
Total	10154.26	100.00	16972.46	100.00

TABLE 8.08 COST DISTRIBUTION FOR 4- SPAN FACTORY BUILDINGS

	3000 m² Building		4000 m² Building	
	£	%	£	%
Fabrication	11666.70	48.57	14836.07	48.16
Haulage	583.27	2.43	738.79	2.40
Foundation	1333.09	5.55	1629.33	5.29
Cladding	9687.11	40.34	12670.08	41.13
Erection	745.98	3.11	930.10	3.02
Total	24016.14	100.00	30804.37	100.00

TABLE 8.09 SUMMARY FOR COST DISTRIBUTION

Cost	Range %	Average Cost %
Fabrication	27.80 - 51.62	41.52
Haulage	1.44 - 2.71	2.16
Foundation	3.32 - 7.29	5.18
Cladding	33.82 - 64.29	46.20
Erection	3.02 - 8.72	4.94

typical buildings shows that fabrication and cladding account for over 80% of the total cost of the structure. Cladding costs fluctuate considerably due primarily to price differences between the various types of cladding design used. Foundation cost is generally below 10% of the overall cost of the structure though in problem areas this could be higher. Haulage costs are the least of the five main factors.

8.3 BUILDING AREA AND COST

The cost of two types of buildings are examined. These are classified as 'long' and 'wide' buildings which are defined as follows :

i. Long building -

Building length (frame spacing * number of bays) > Building width (span * number of spans)

ii. Wide building -

Building length < building width

Fig 8.01 shows that building costs vary linearly with building area and that long buldings tend to be more expensive than wide buildings for building areas above 1000 m^2.

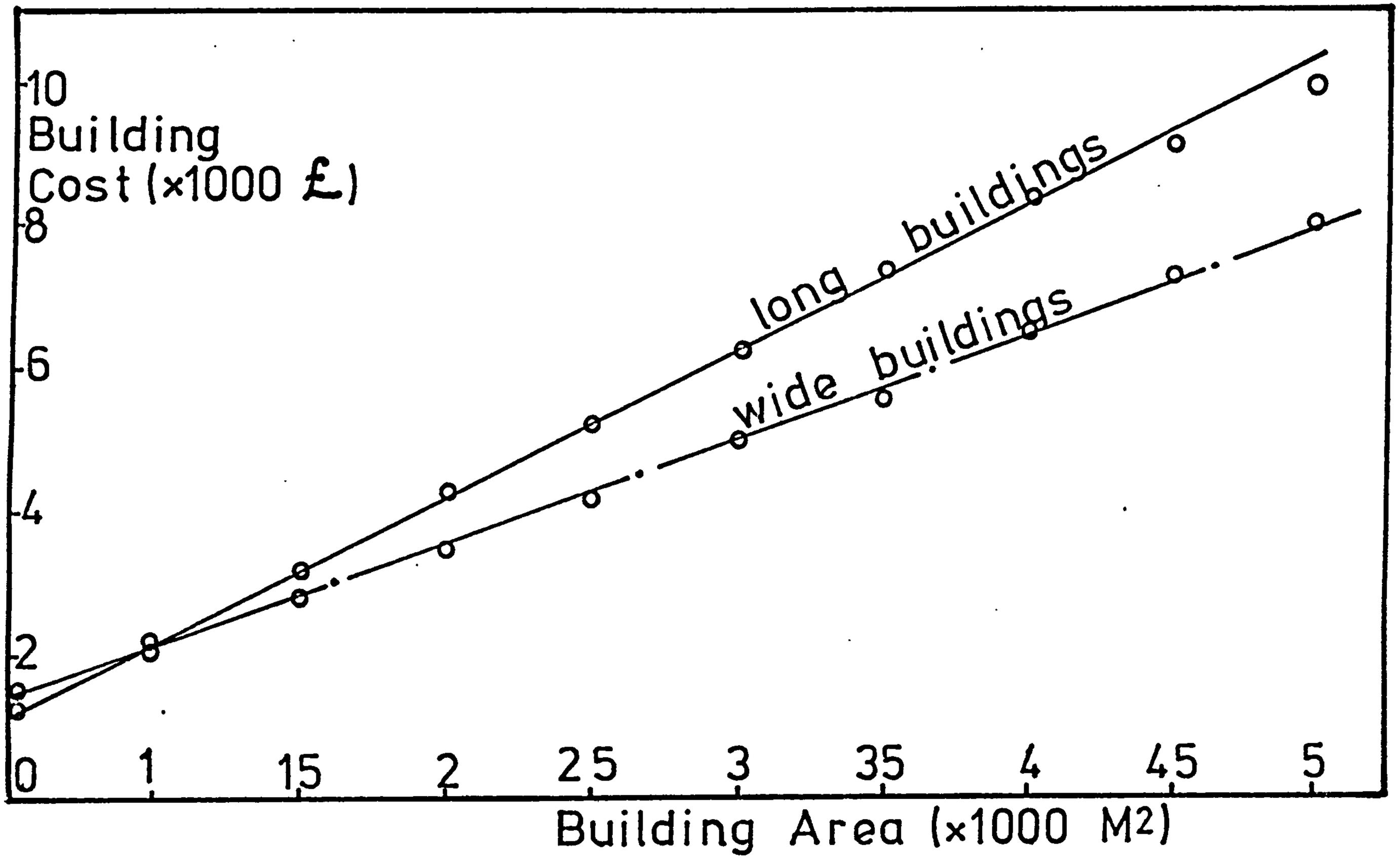

FIG 8.01 BUILDING COST & AREA

8.4 OPTIMUM COST RESPONSE

In Section 6.9 a theoretical case was stated for the use of cost contour surfaces to represent the cost function in any selected spectrum of a pair of primary variables. It particular it was argued that provided the lower and upper bounds of two primary variables are defined then the cost model can be interfaced with the contour program to display an associated cost response surface. Seven sets of cost contour surfaces are presented in Figs 8.01 to 8.28. Each set comprises cost surfaces whose areas are 1000 m^2, 2000 m^2, 3000 m^2 and 4000 m^2. Most of the industrial buildings covered in the study have areas within this range, except for isolated cases which could be treated as special cases. The following assumptions have been made whilst generating the cost surfaces :

1. All buildings are situated in open countryside with an average wind velocity of about 45 m/s
2. A set of design data describing material strength and characteristics have been kept constant throughout
3. Prices of materials, accessories, rates, crane charges and other secondary variables are all as at 1978.
4. For each cost surface, 100 designs have been used.
5. Haulage distance is fixed at 40 miles from the factory.
6. All linear measurements are in meters, roof pitch in degrees and cost contours in £s.

The primary variables whose notation appear in Chapter 6 are

redefined here for easy reference :

span of the structure	x
frame spacing	y
number of bays	n
number of spans	k
roof pitch	w
height to eaves	h

8.4.1 FRAME SPACING AND NUMBER OF BAYS (Figs 8.02 - 8.05)

$$3 \leq y \leq 22$$

$$1 \leq n \leq 10$$

A lower and upper bound 3-12 m for frame spacing and 1-10 for number of bays was imposed on buildings with areas 1000 m^2 and 2000 m^2. For buildings with areas 3000 m^2 and 4000 m^2 the range of frame spacing was increased to 4-22 m whilst maintaining the same range for number of bays. Roof pitch was fixed at 14^o and all buildings are 2-span structures.

Cost surfaces in Figs 8.02 - 8.05 are strongly unimodal. For the 1000 m^2 surface, the optimum is located at (3,7.0,$789.27) in a V-shaped valley. Walls of the valley are gently sloped to the south but elsewhere they are steep sloped. Two local depressions are formed to the south-east and

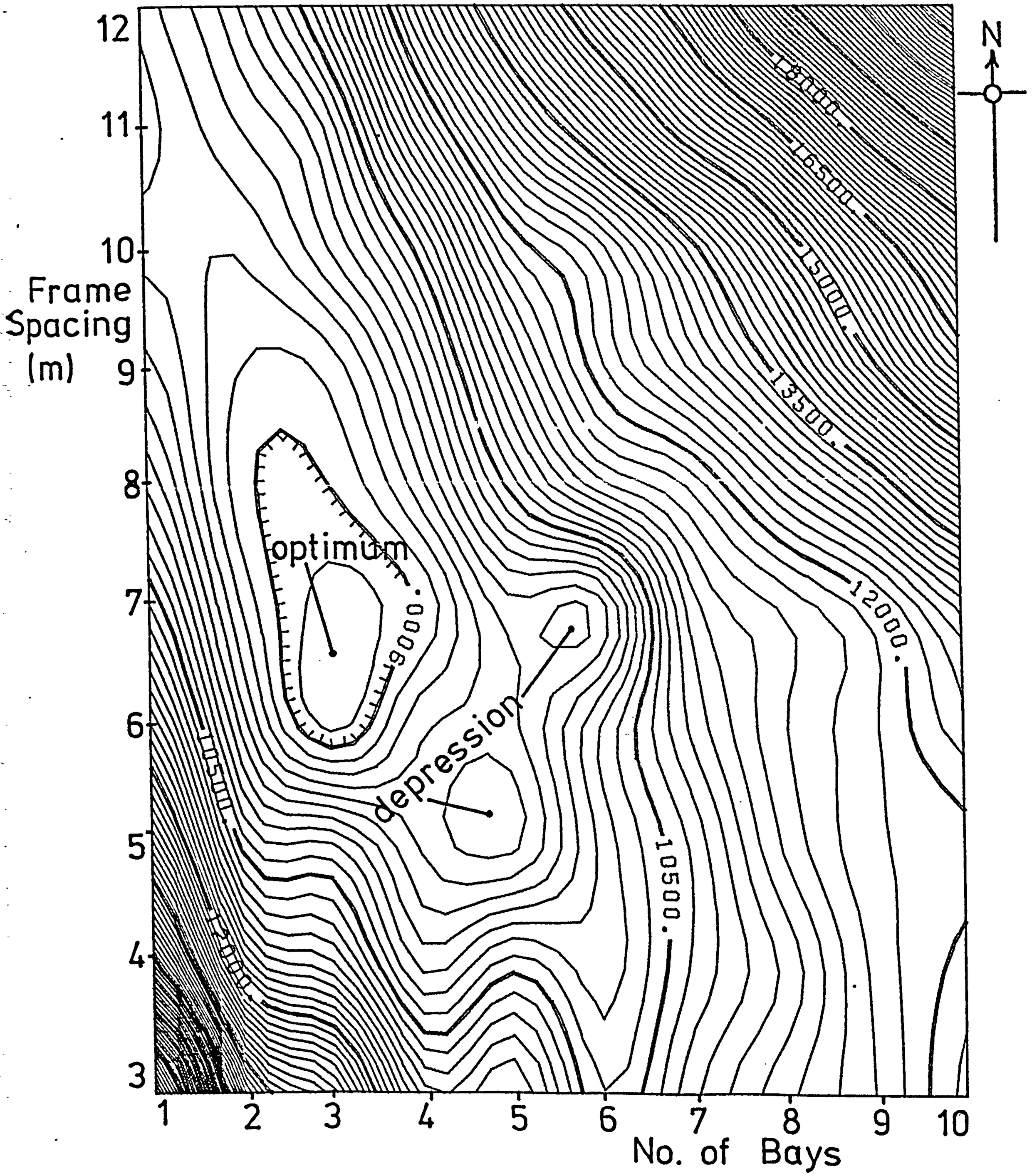

FIG 8.02 OPTIMAL RESPONSE FOR 1000 M² BUILDINGS

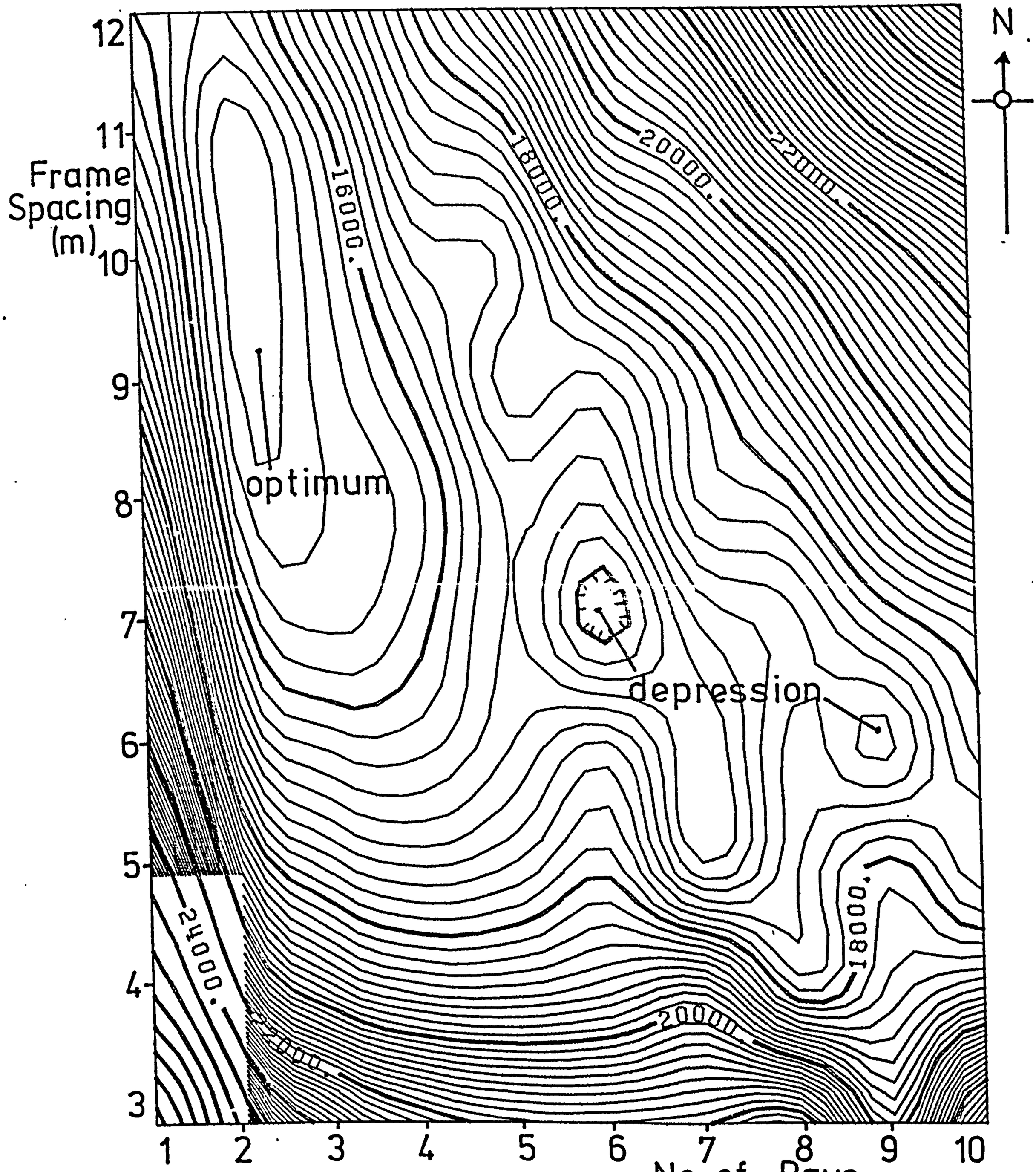

FIG 8.03 OPTIMAL RESPONSE FOR 2000 M^2 BUILDINGS

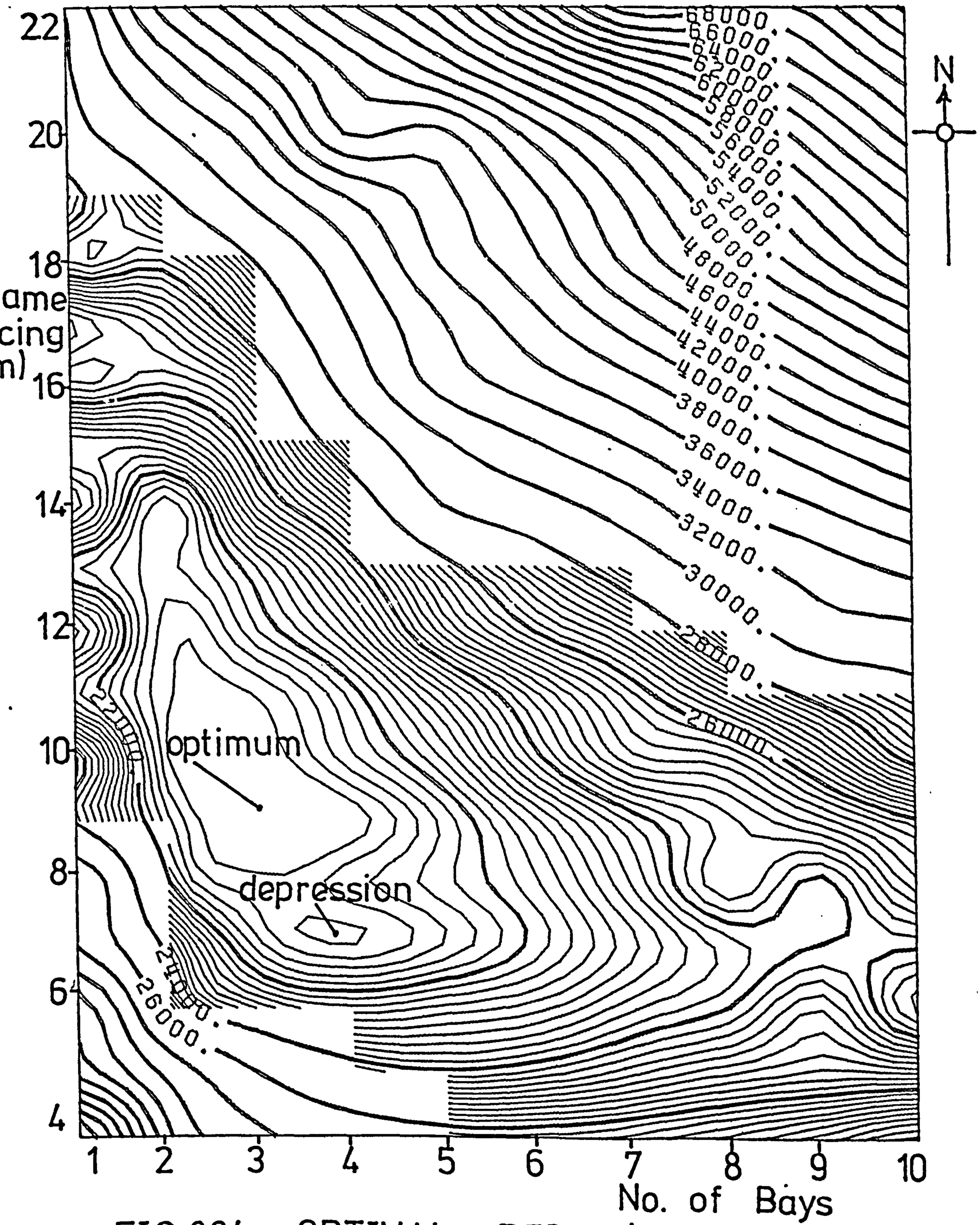

FIG 8.04 OPTIMAL RESPONSE FOR 3000 M² BUILDINGS

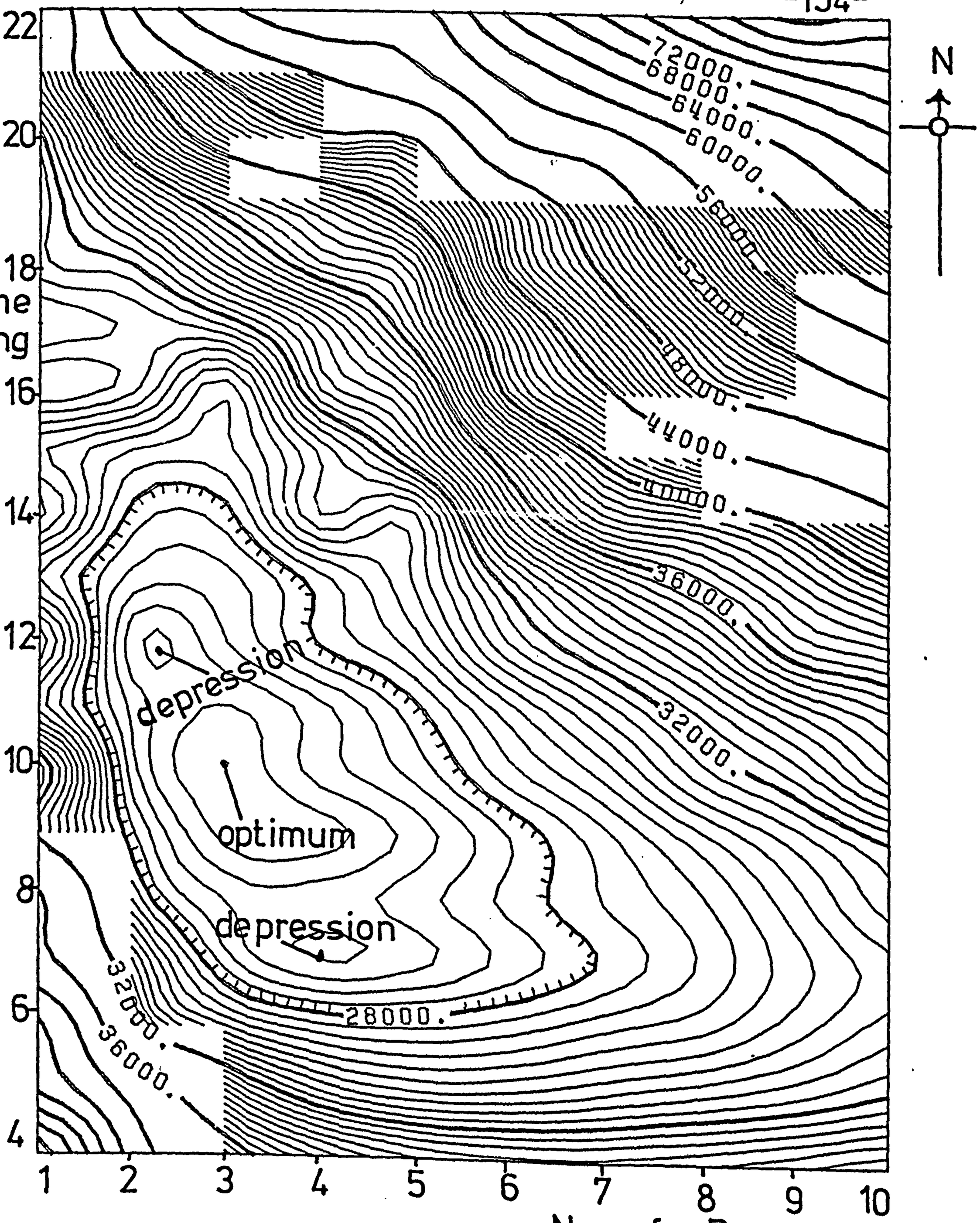

FIG 8.05 OPTIMAL RESPONSE FOR 4000 M² BUILDINGS

east of the optimum. On the 2000 m² surface, the optimum has moved to (2,9.0,15070.84) in the north-west still residing in a v-valley with relatively more gently sloped walls than the 1000 m² surface. The two local depressions have shifted to the south-east of the optimum and a strong unimodality persists. On the 3000 m² surface the optimum is located at (3,9.0,20840.74) in a deep trough to the west. The noth-east wall is steep sloped and only one local optimum survives the pronounced unimodality located in the south-east of the optimum. In the north-west, two isolated hilly outcrops emerge. On the 4000 m² surface the optimum resides at (3,10.0,25778.92) in a yet deeper trough to the south-west. The bed of the trough is indented with two local depressions to the south-east and north-west. The walls of the trough are steep-sided to the north-east and relatively gentle sloped to the south-west.

8.4.2 ROOF PITCH AND FRAME SPACING (Figs 8.06 - 8.09)

$$3 \leq y \leq 12$$

$$1 \leq w \leq 37$$

Spans were limited to 20 m and the lower and upper bounds were maintained for all buildings. Roof cladding was sandwich and combination of single skin and brickwork for the sides.

Cost surfaces in Figs 8.06-8.09 show a consistent

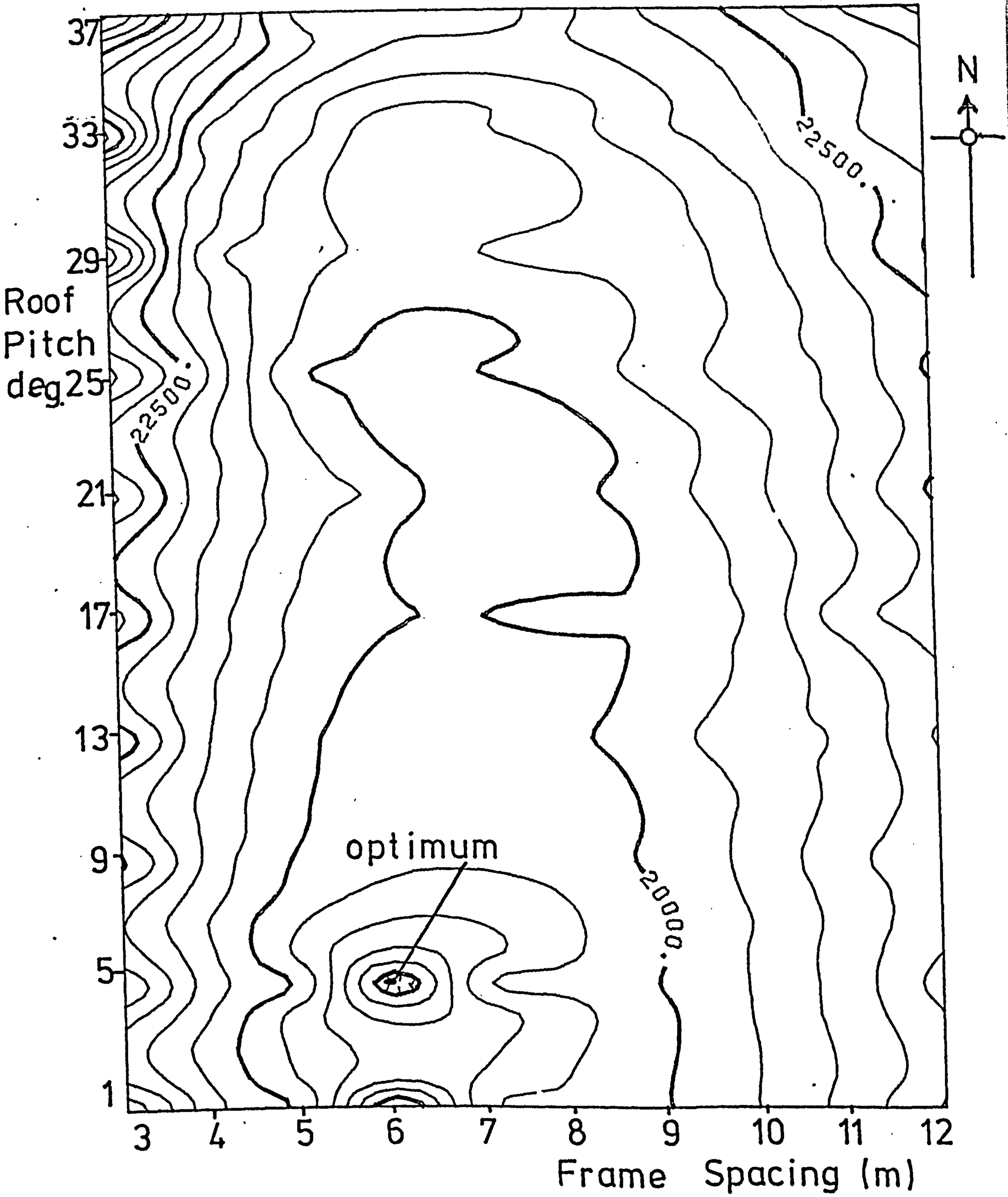

FIG 8.06 OPTIMAL RESPONSE FOR 1000 M² BUILDINGS

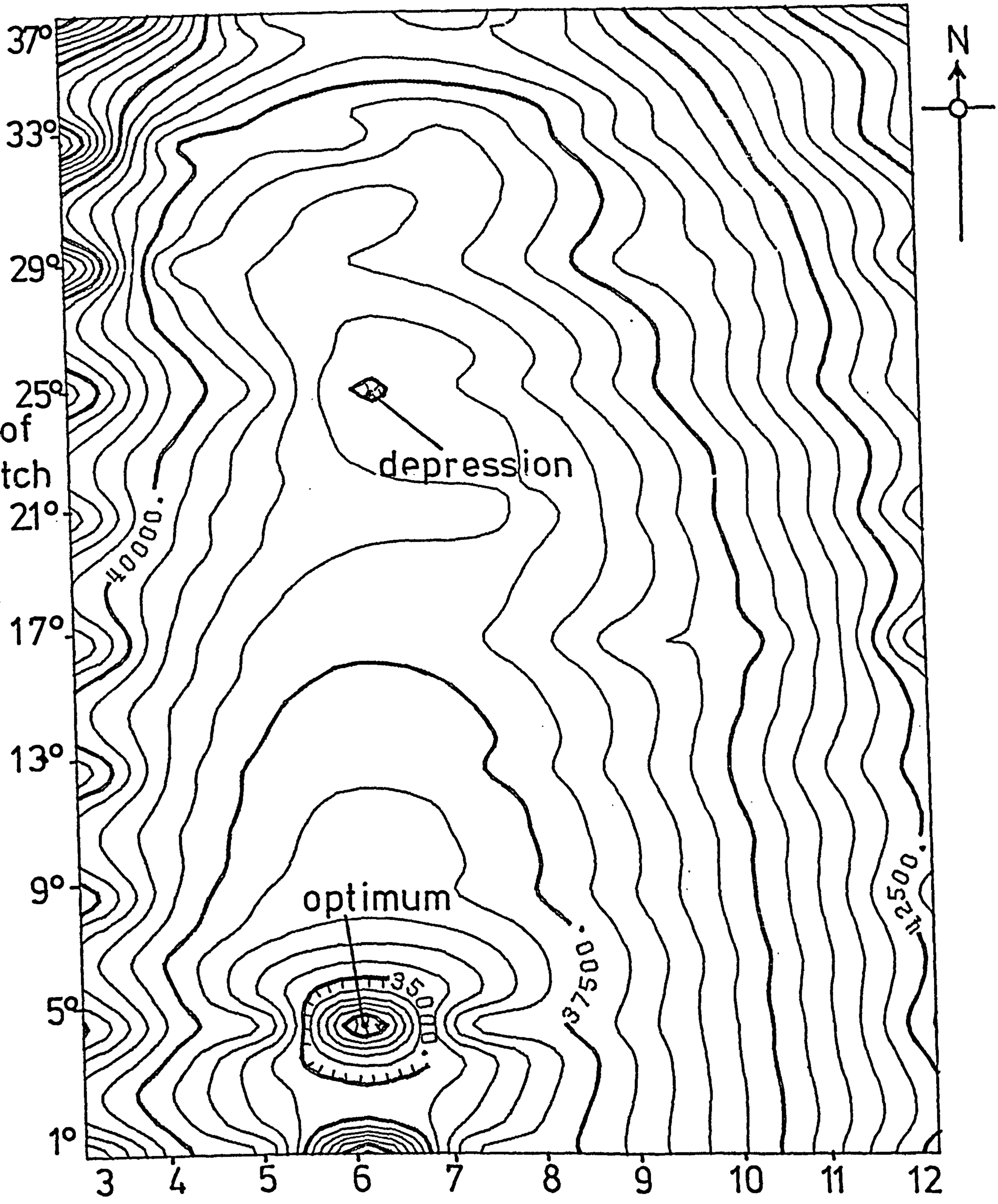

FIG 8.07 OPTIMAL RESPONSE FOR 2000 M² BUILDINGS

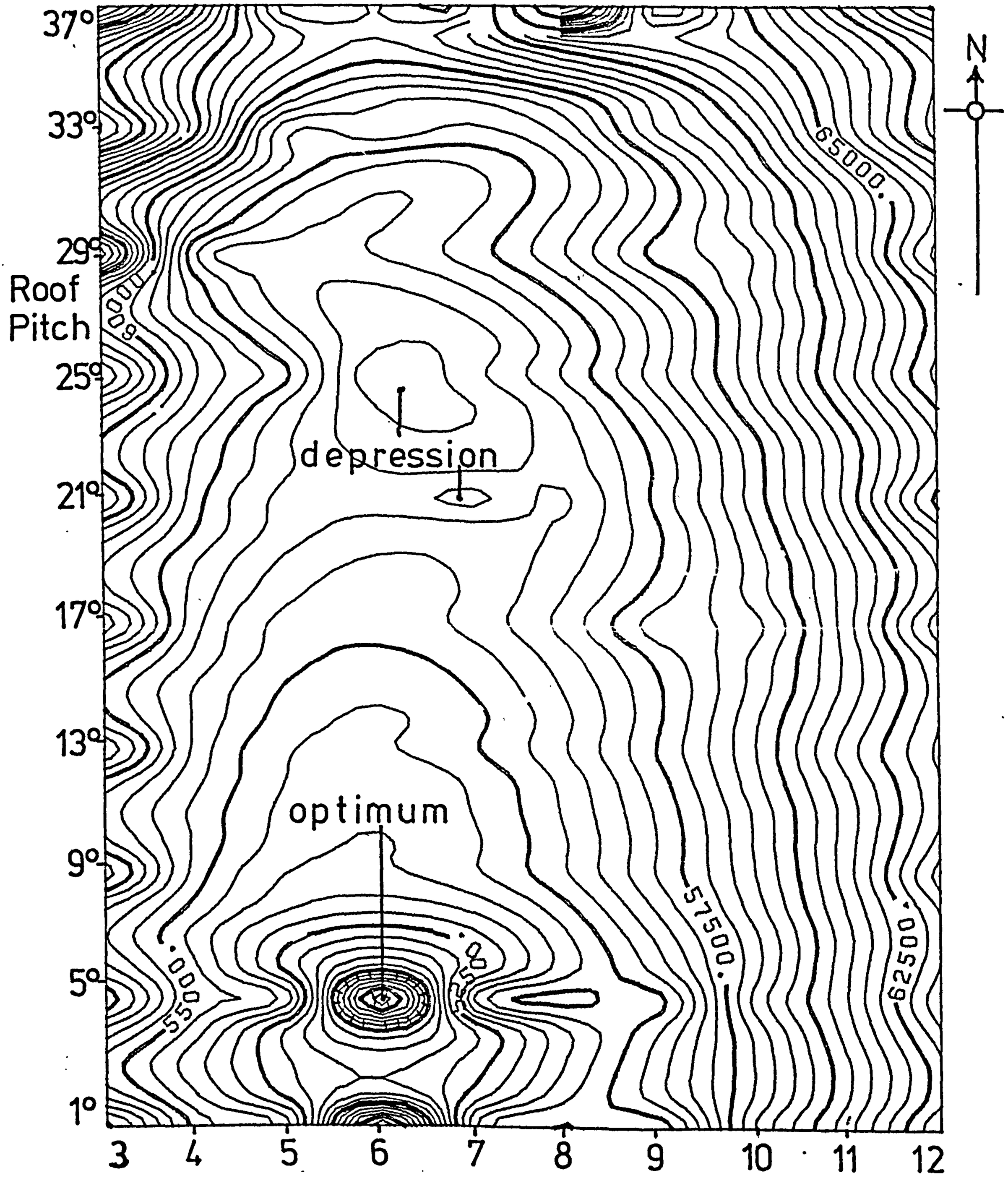

FIG 8.08 OPTIMAL RESPONSE FOR 3000 M^2 BUILDINGS

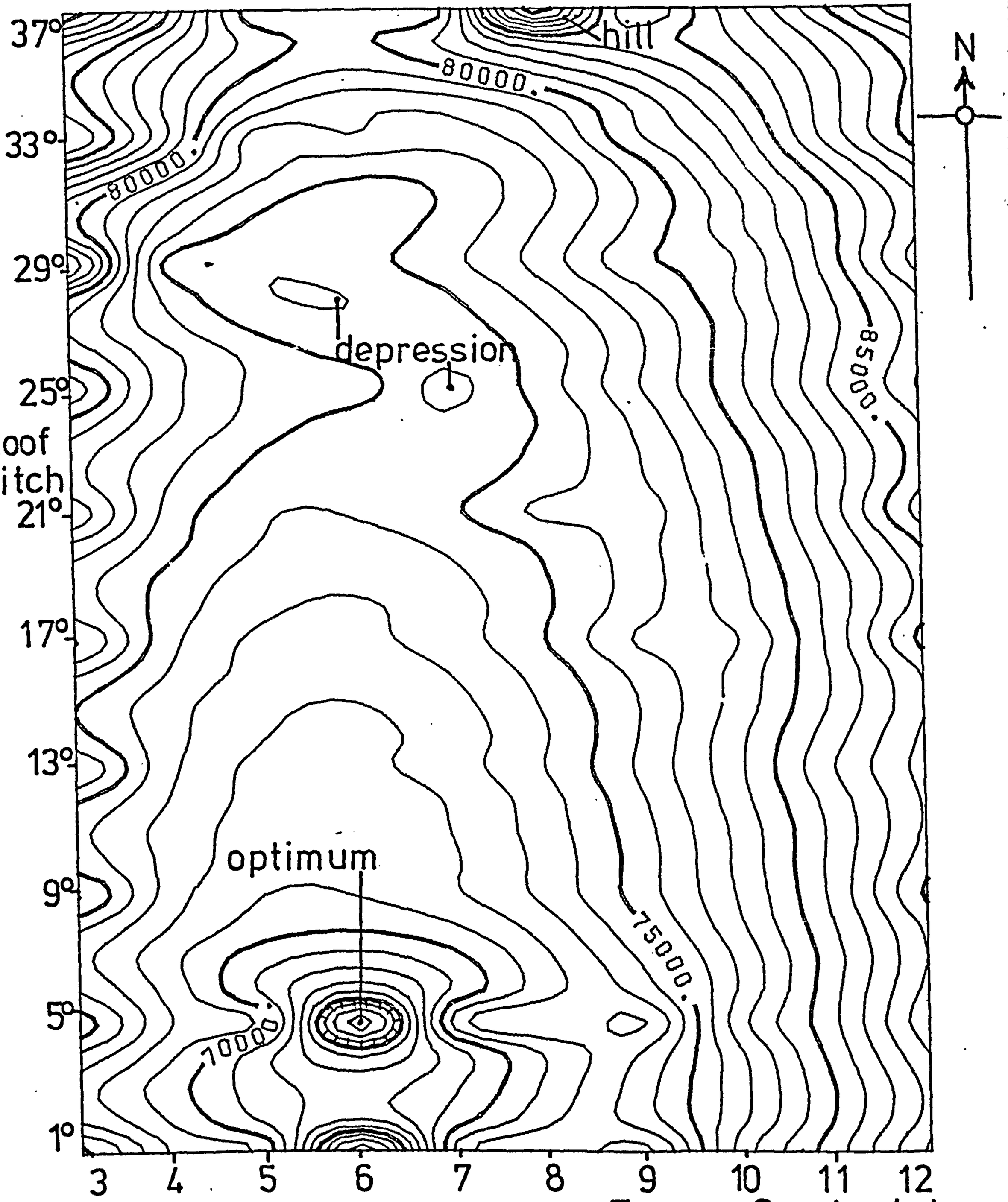

FIG 8.09 OPTIMAL RESPONSE FOR 4000 M² BUILDINGS

unimodality. The 1000 m² surface is a V-shaped valley with a gently sloping bed. The optimum resides in the south at (6.0,4°,17129.16). There are no local depressions though the rectangular region whose vertices are : (5.0,5°,20000.00), (5.0,17°,205000.00), (8.0,17°,20500.00) and (8.0,5°,20400.00) located in the central area is an undulating plain where feasible designs would be economic . On the 2000 m² surface a local depression has formed to the north of the optimum. The optimum has moved to (6.2,5°,31839.44) in the south. This local depression is significantly situated far from the optimum but it is within a region where designs would incur 10% extra costs over the optimum design. On the 3000 m² surface, the optimum resides in the south at (5.8,5.2°,46539.68) in a U-shaped valley. The slope of the bed and sides of the valley have become steeper. Two local depressions in the north still appear plus a third depression to the east of the optimum. This depression in the east is formed at a lower altitude than the other two. Designs in the area of the third depression attract extra 9.4% costs whilst designs in the area of the other two depressions attract 11.64% costs. On the 4000 m² surface the two northern depressions and the eastern depression are still present. The optimum is at (6.1,5.1°,61249.97) still in the south. In the far north conical hills have formed suggesting that as the building area increases it becomes more expensive to design for high pitched buildings.

8.4.3 SPAN AND NUMBER OF BAYS (Figs 8.10 - 8.13)

$$8 \leq x \leq 50$$

$$1 \leq n \leq 10$$

The lower and upper bounds imposed on this range of buildings were : 8-26 m for 1000 m² and 2000 m² buildings, 14-41 m on the 3000 m² buildings and 14-50 m on the 4000 m² buildings. A roof pitch of 14° was used.

The cost surfaces in Figs 8.10 - 8.13 are strongly unimodal. On the 1000 m² surface the optimum is located at (2,19.5,9415.84) in the north-west residing in a U-shaped valley. The wall to the west of the valley is steeper than the wall to the east of the valley. There are seven local depressions to the south-east and one to the north-west. The most attractive local depression would cost extra 1.05% to design over the optimum. Generally, 2 or 3 span buildings with spans above 10 m but below 24 m attract up to 8.5% extra costs over the optimum. On the 2000 m² surface the optimum resides at (4,21.5,15774.77) in an elongated U-shaped valley. The wall to the west of the valley is still steep sloped. Three local depressions are formed in the south-east. 3 to 5 span buildings with spans between 11-24 m are generally economical. The 3000 m² surface also has a U-shaped valley in which the optimum resides located at (5.20.0,21857.57) in the south. There are 8 local depressions to the north. In the north-east three isolated conical hills have formed suggesting that multi-span buildings with over 7 spans are not economical. The 4000

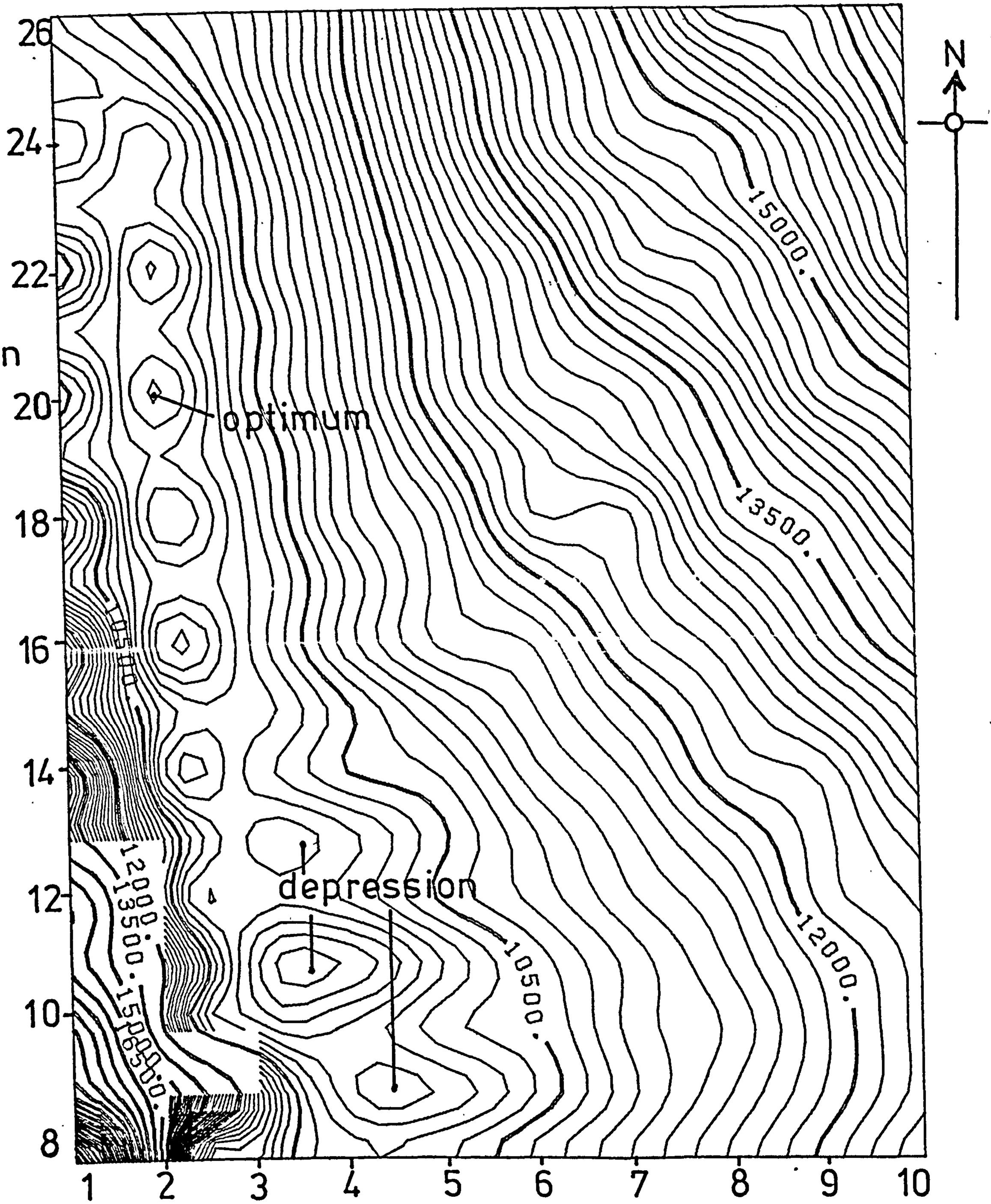

FIG 8.10 OPTIMAL RESPONSE FOR 1000 M² BUILDINGS

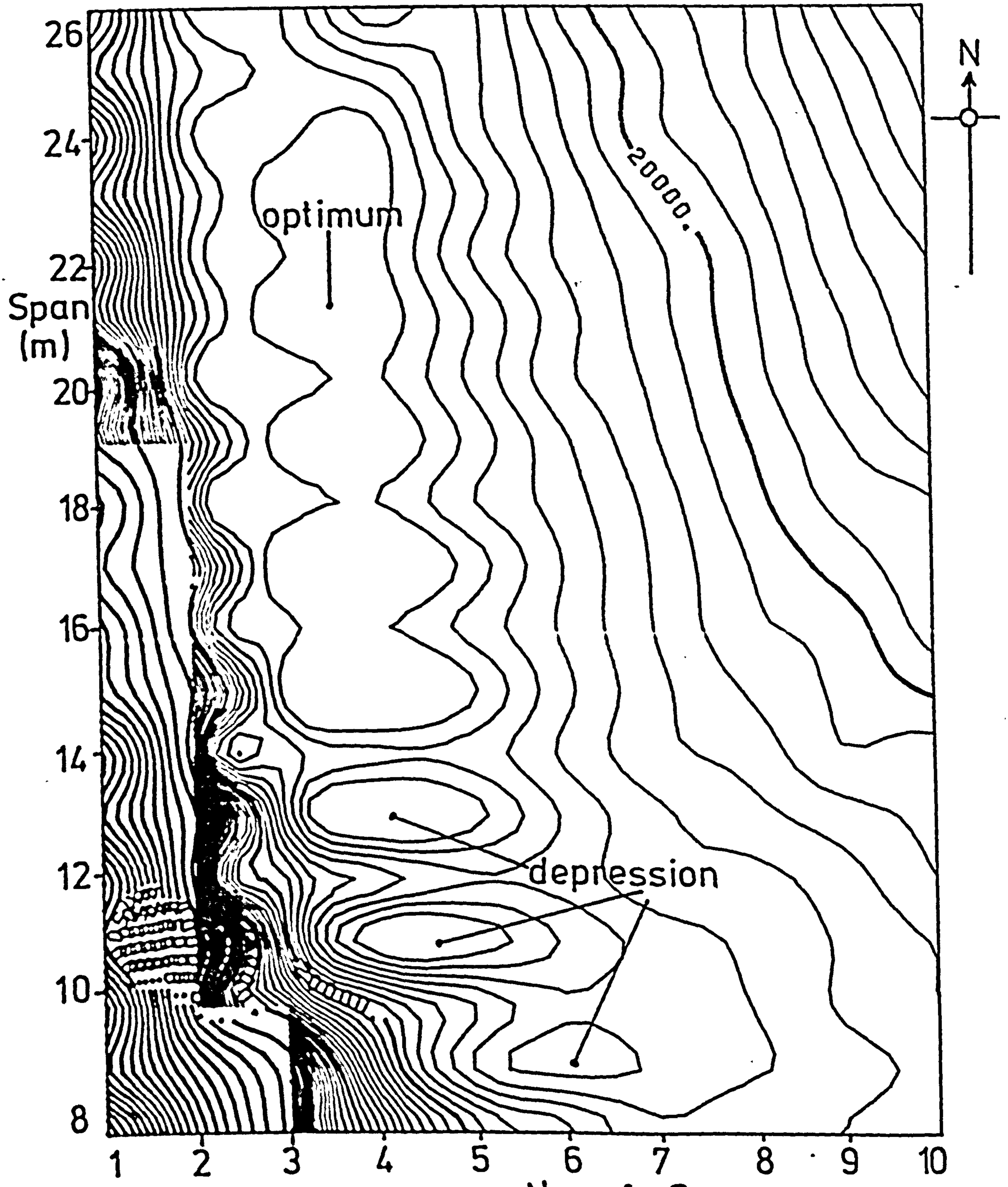

FIG 8.11 OPTIMAL RESPONSE FOR 2000 M² BUILDINGS

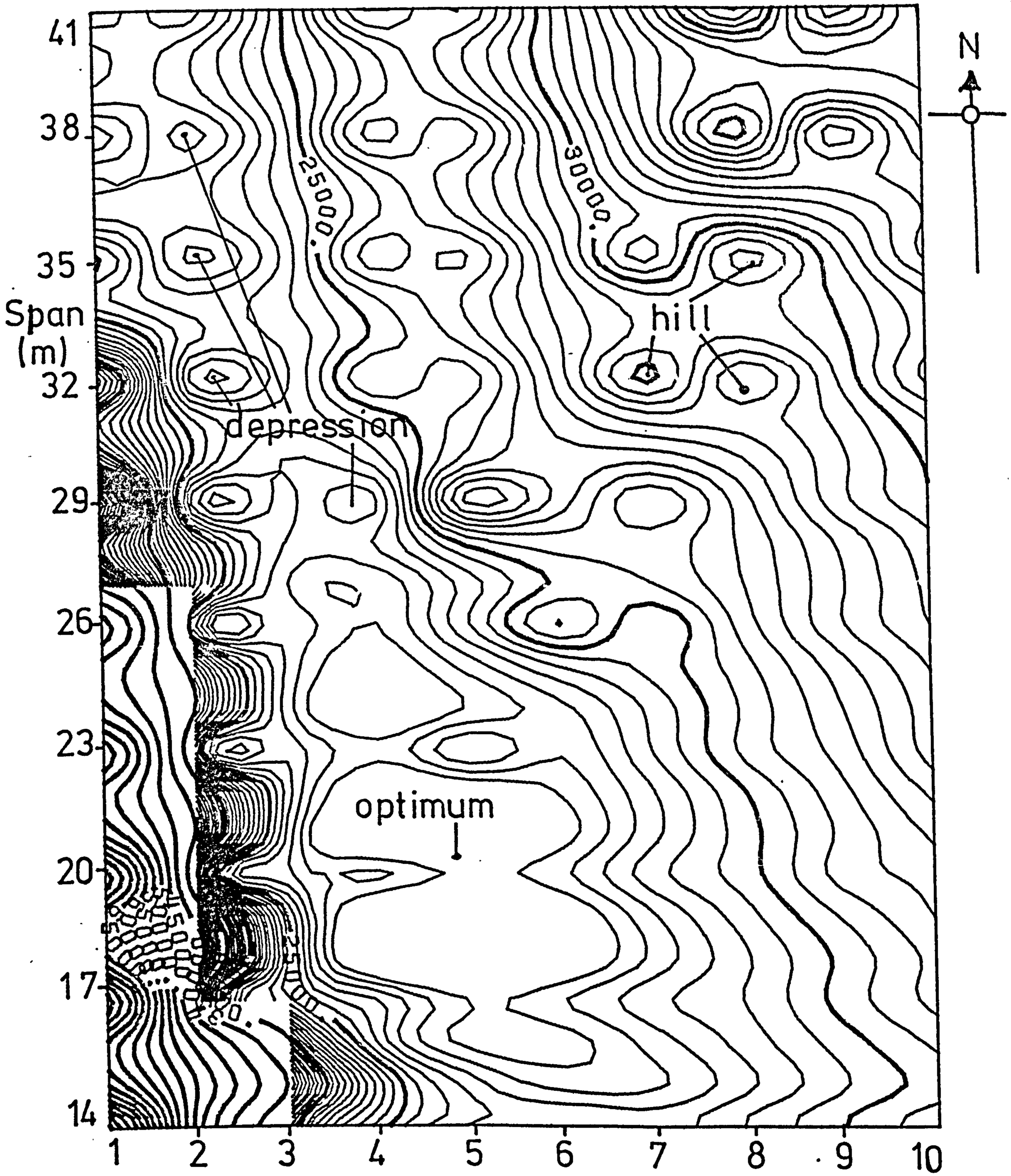

FIG 8.12 . OPTIMAL RESPONSE FOR 3000 M2 BUILDINGS

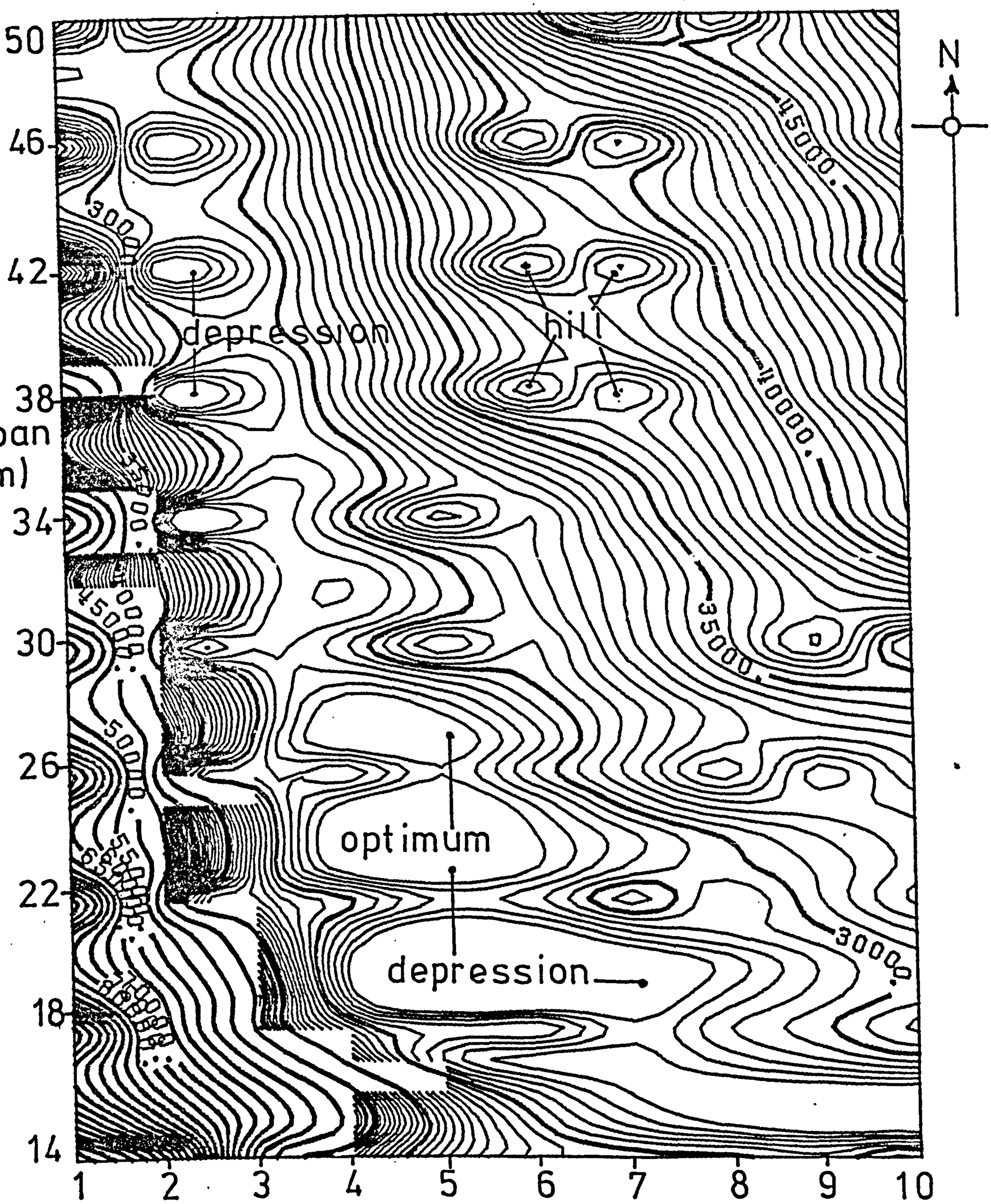

FIG 8.13 OPTIMAL RESPONSE FOR 4000 M^2 BUILDINGS

m^2 surface is fairly similar to the 3000 m^2 in many ways. The optimum is located at (5,26.0,27552.01) in the south. Few local depressions still appear in the north-west and in the north-east. Some hills have also reappeared.

8.4.4 ROOF PITCH AND NUMBER OF BAYS (Figs 8.14 - 8.17)

$$1 \leq w \leq 37$$

$$1 \leq n \leq 10$$

Lower and upper bound for roof pitch was fixed at 1^c-37^c and for number of bays at 1-10. All buildings with areas 1000 m^2 and 2000 m^2 were designed with 2 spans and those with areas 3000 m^2 and 4000 m^2 designed with 5 spans. Cladding was single skin type for roof and sides.

All the four surfaces in Figs 8.14-8.17 are unimodal. On the 1000 m^2 surface the optimum resides at (3,20°,8708.59) in an elongated U-shaped valley whose western wall is steeper than the eastern wall. Only two local depressions are formed to the south of the optimum. 3-5 bay buildings with roof pitch between between 10°-29° are generally economical. 1 and 2 bay buildings are very expensive so are buildings with 6 or more bays. On the 2000 m^2 surface the optimum is located at (7,21°,16042.09) on the bed of an elongated U-shaped valley. The wall to the west is still steep and two local depressions to the south have formed. 4-8 bay buildings with roof pitch

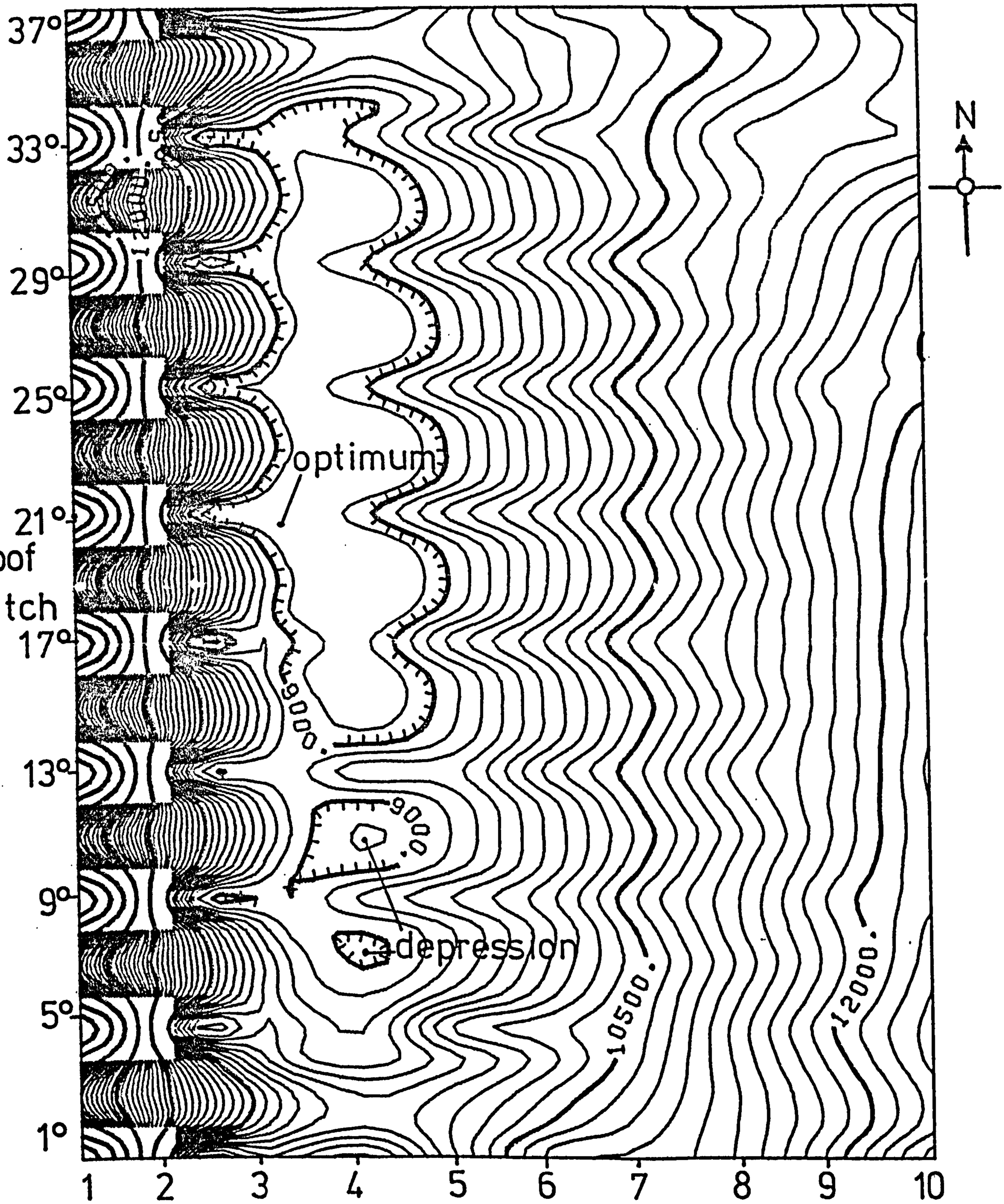

FIG 8.14 OPTIMAL RESPONSE FOR 1000 M² BUILDINGS

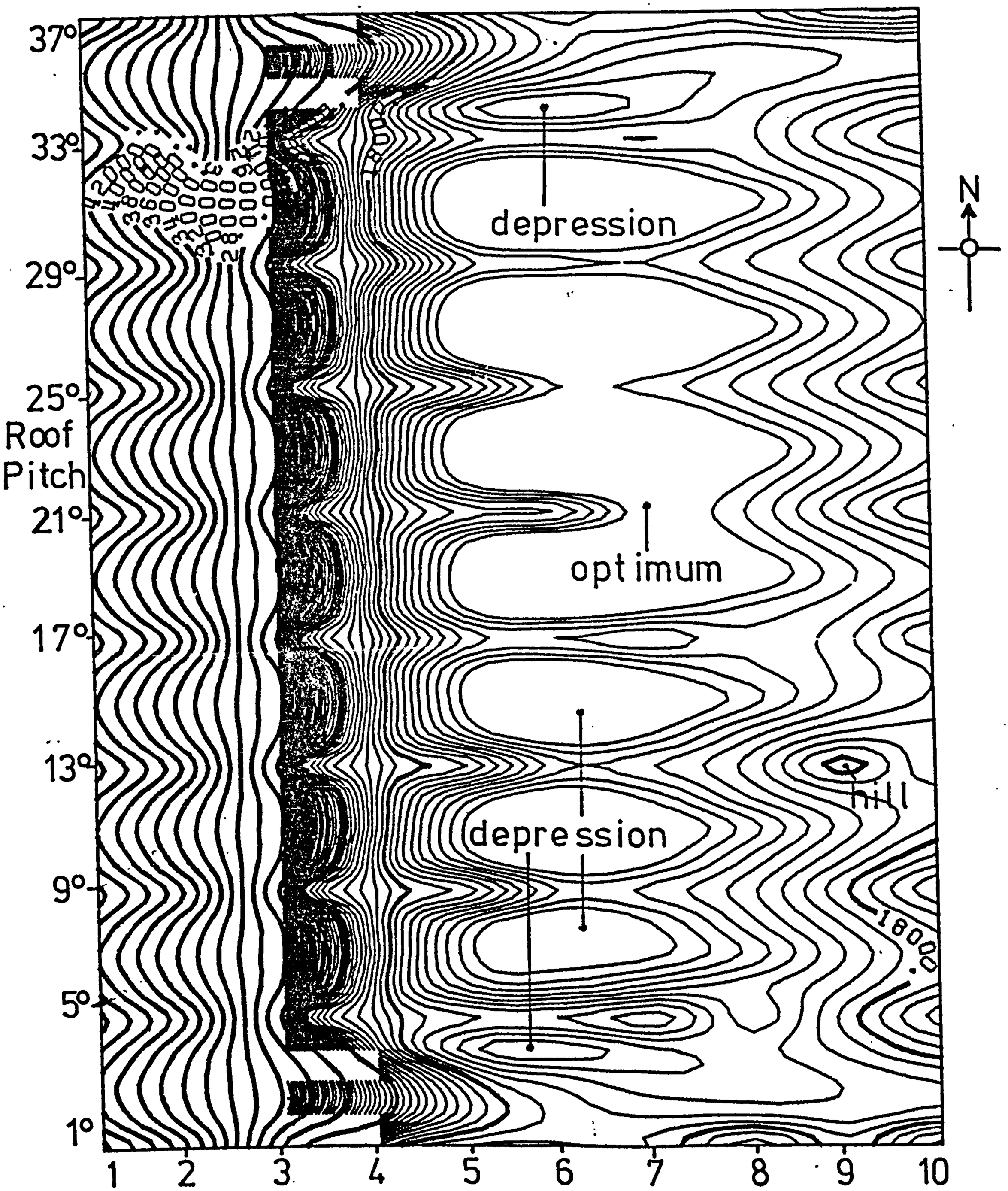

FIG 8.15 OPTIMAL RESPONSE FOR 2000 M2 BUILDINGS

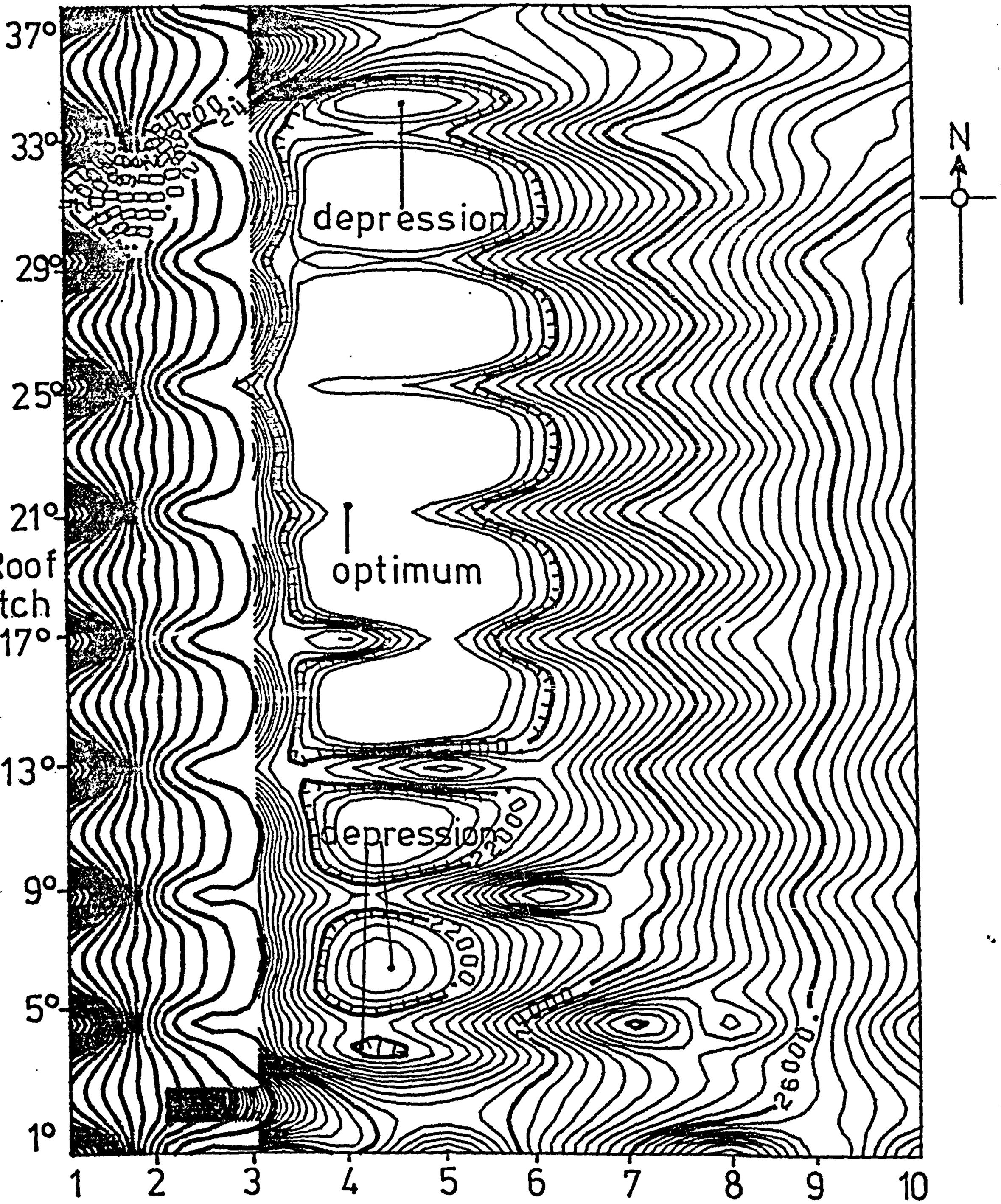

FIG 8.16 OPTIMAL RESPONSE FOR 3000 M² BUILDINGS

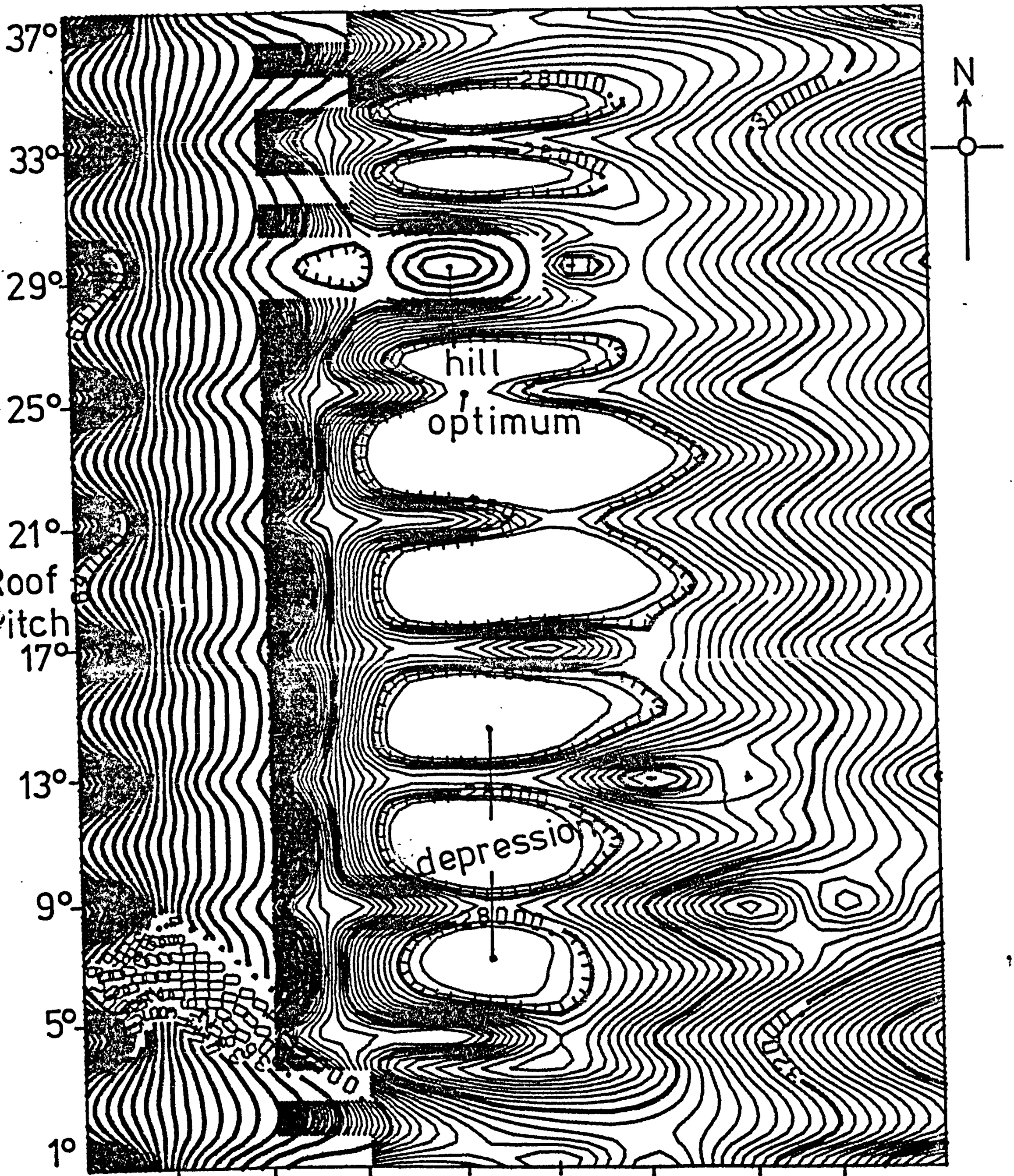

FIG 8.17 OPTIMAL RESPONSE FOR 4000 M² BUILDINGS

between 10°-29° are economical but 1-3 bay buildings are very expensive. 9 and 10 bay buildings are also expensive and in fact, a local hill appears in the mid-east. Five local depressions are formed in the south and two in the north. The 3000 m² surface has features similar to the 1000 m² surface. For instance the optimum at (4,21°,21202.95) is fairly close to that of the 1000 m² surface. Three local depressions appear in the south and two hills are formed in the east. The general economic range returns to 3-5 bays with roof pitch between 10°-29°. On the 4000 m² surface, the optimum resides at (5,25°,27520.30) in a deep U-shaped valley whose walls are both very steep. Local depressions have become more isolated; 4 in the south and 4 in the north. For the first time a hill appears in the north and the two eastern hills also reappear. It is now uneconomic to design buildings with over 29° roof pitch.

8.4.5 SPAN AND NUMBER OF BAYS (Figs 8.18 - 8.21)

$$22 \leq x \leq 60$$

$$1 \leq k \leq 20$$

Buildings designed in this section were all single span type. Roof pitch was maintained at 14°. Roof cladding was sandwich type and the sides were cladded with combination of brick and single skin sheeting.

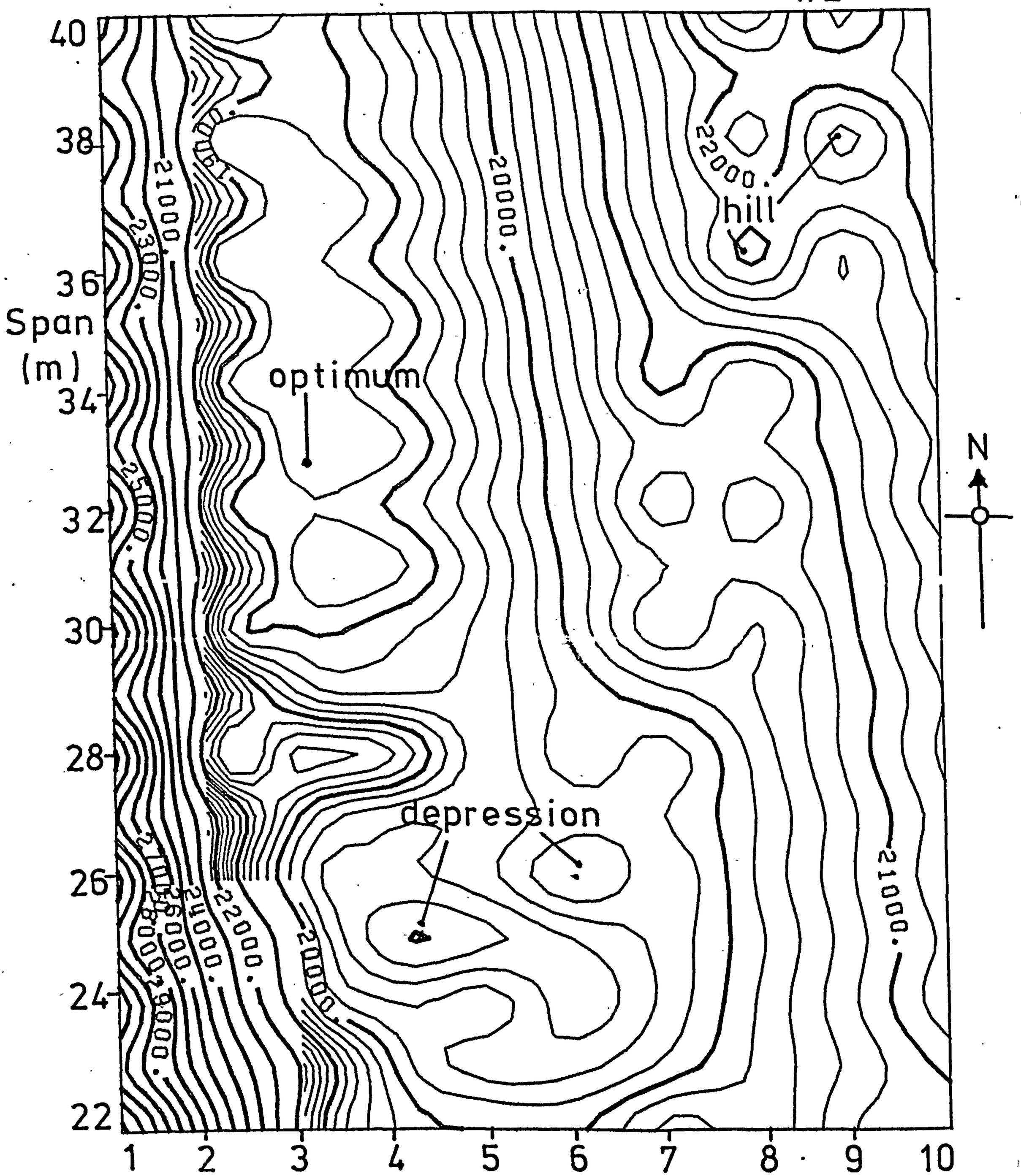

FIG 8.18 OPTIMAL RESPONSE FOR 1000 M² BUILDINGS

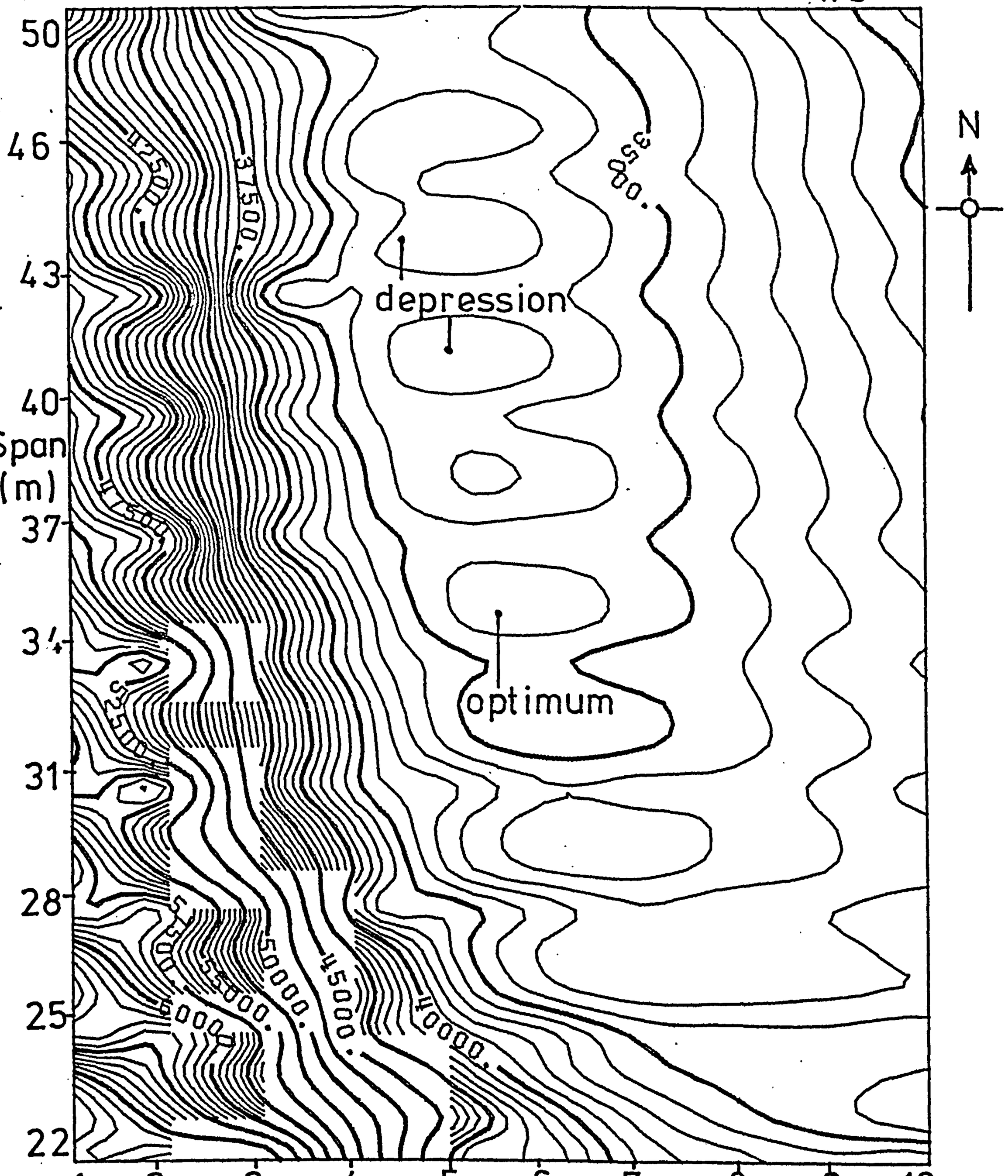

FIG 8.19 OPTIMAL RESPONSE FOR 2000 M2 BUILDINGS

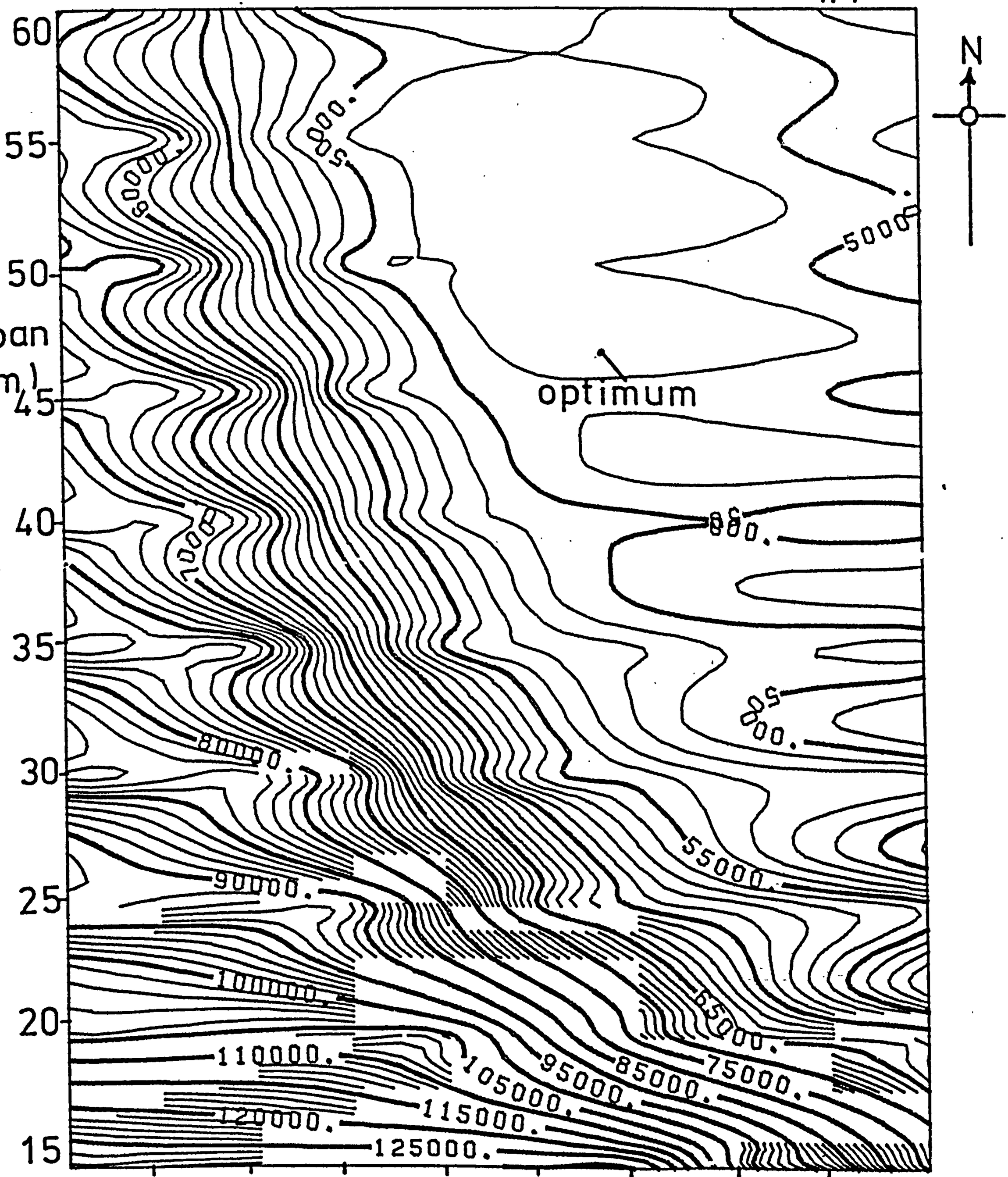

FIG 8.20 OPTIMAL RESPONSE FOR 3000 M² BUILDINGS

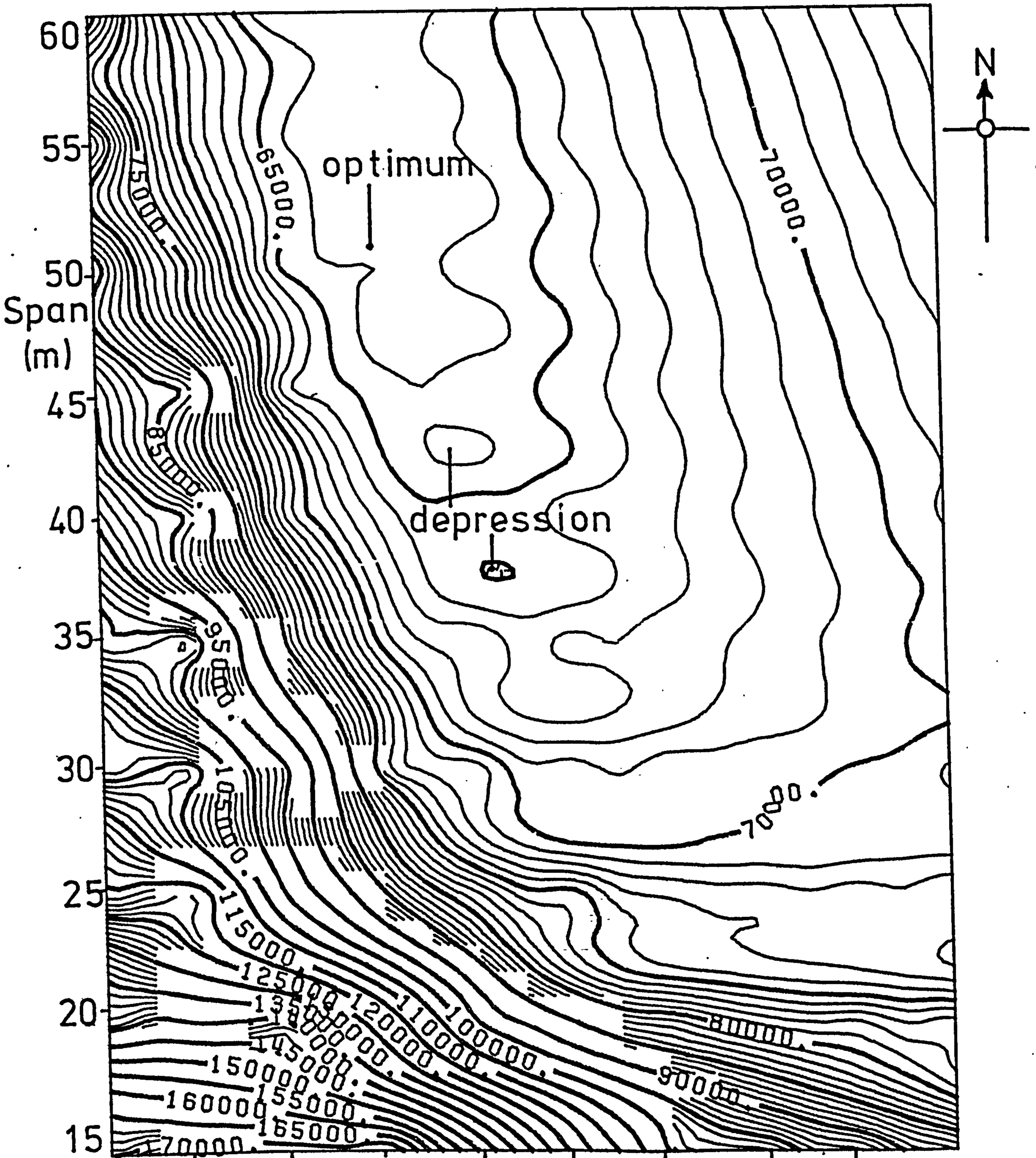

FIG 821 OPTIMAL RESPONSE FOR 4000 M² BUILDINGS

All the four surfaces show a strong uninodality. On the 1000 m^2 surface the optimum is located at (3,32.0,18874.3) in the north-west lying in an elongated U-shaped valley. Two local depressions appear in the south and hills have formed in the north-east. Generally, 3-6 bay buildings having spans below 39 m are fairly economical. But single and double bay buildings and 8-10 bay buildings are very expensive. On the 2000 m^2 surface the optimum has moved to (3,34.2,33797.7) in the north still residing in a U-shaped valley. The wall of the valley to the west has become steeper whilst the wall to the east is relatively gentle sloped. There are three local depressions to the south and one to the north. On the 3000 m^2 surface the optimum is located at (6,45.0,49299.9) in the north-east. Only one local depression has formed and there are no hills. Economic range has shifted to 5-9 bay buildings. On the 4000 m^2 surface the optimum resides at (8,50.0,63979.9) still in an elongated valley. The valley now dips north-west.

8.4.6 NUMBER OF BAYS AND NUMBER OF SPANS (Figs 8.22 - 8.25)

$$1 \leq n \leq 10$$

$$1 \leq k \leq 10$$

For the buildings in this section the spans were limited to 20 m. roof pitch to 14° and single skin construction used for roof and sides.

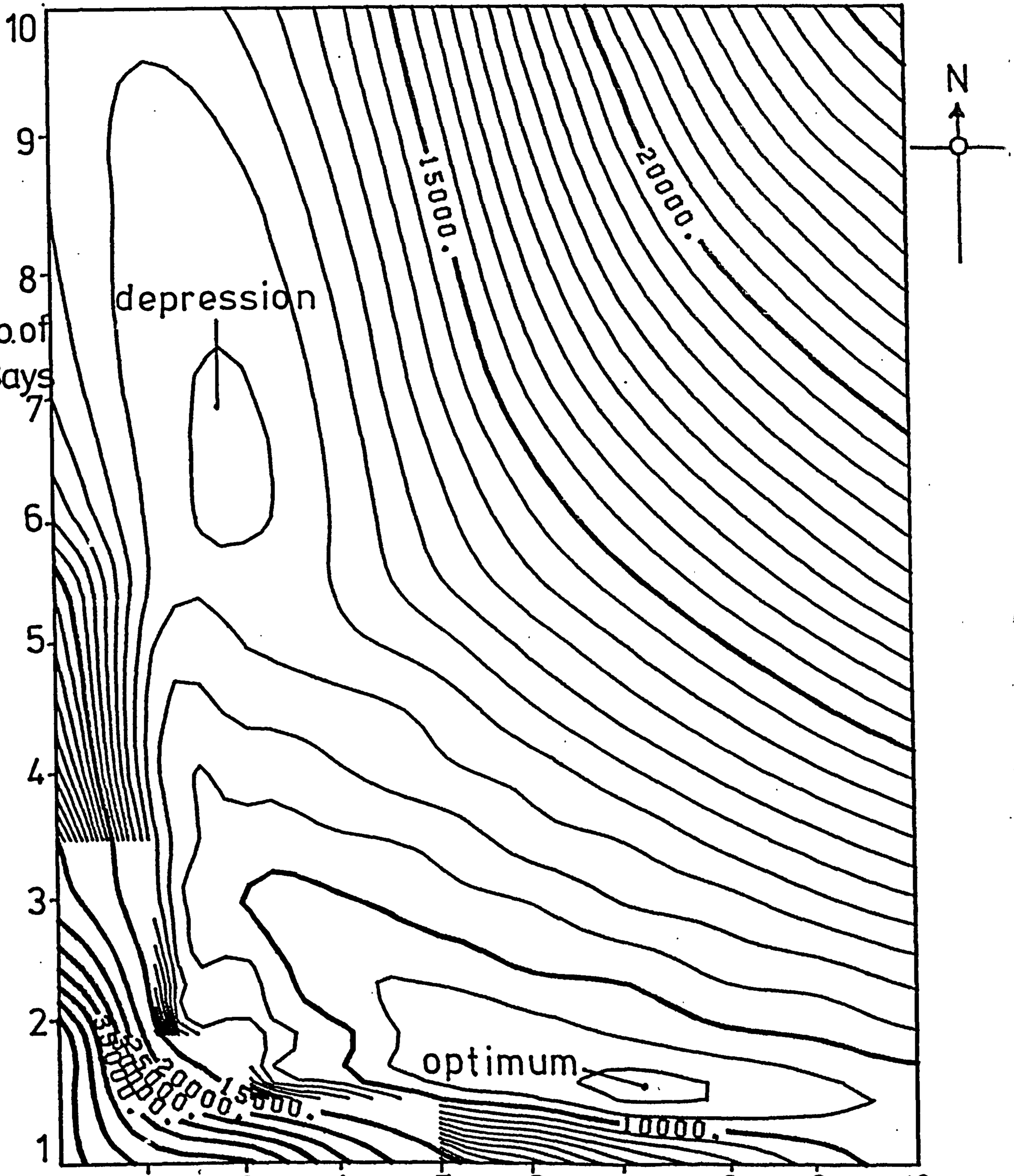

FIG 8.22 OPTIMAL RESPONSE FOR 1000 M2 BUILDINGS

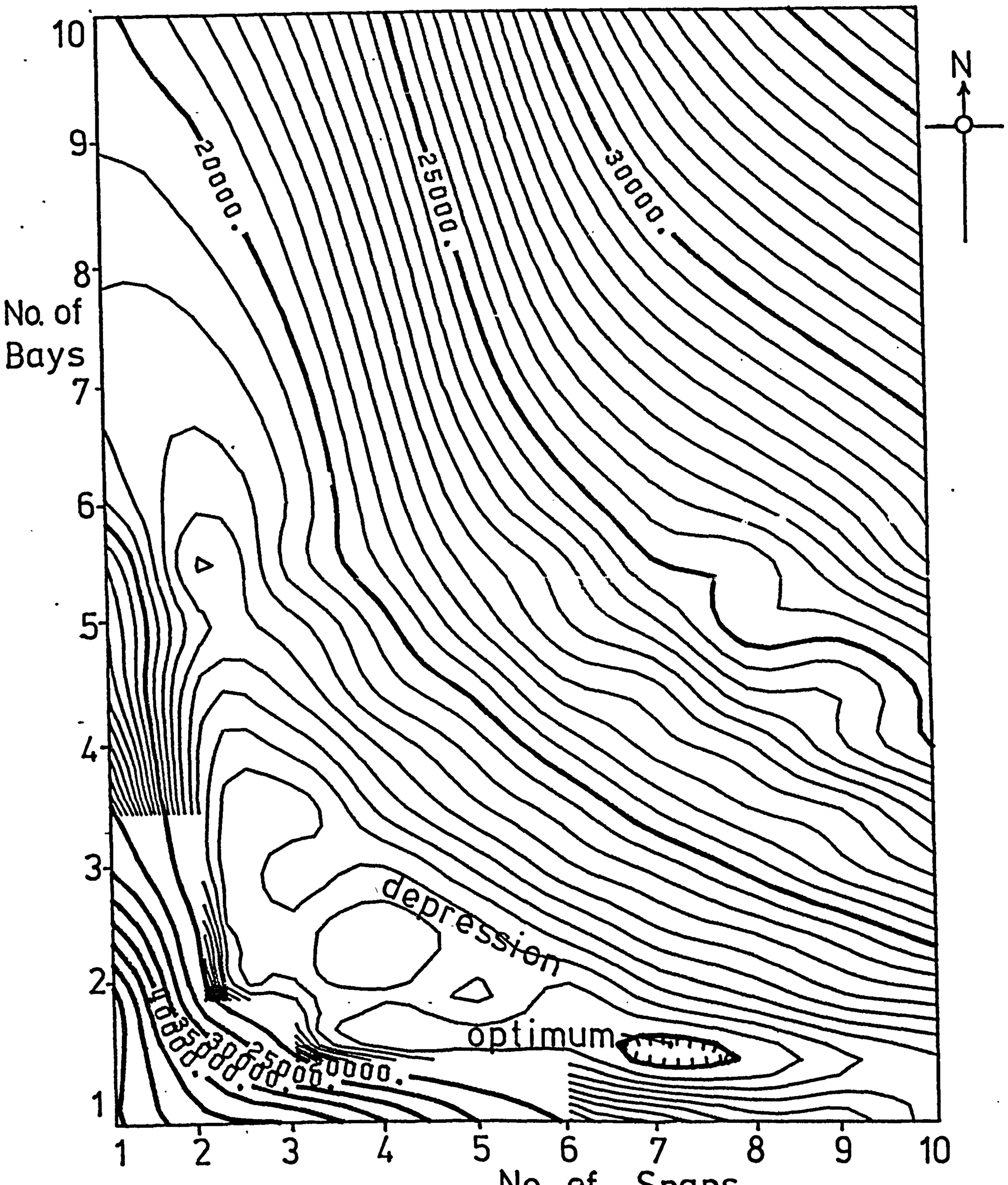

FIG 8.23 OPTIMAL RESPONSE FOR 2000 M2 BUILDINGS

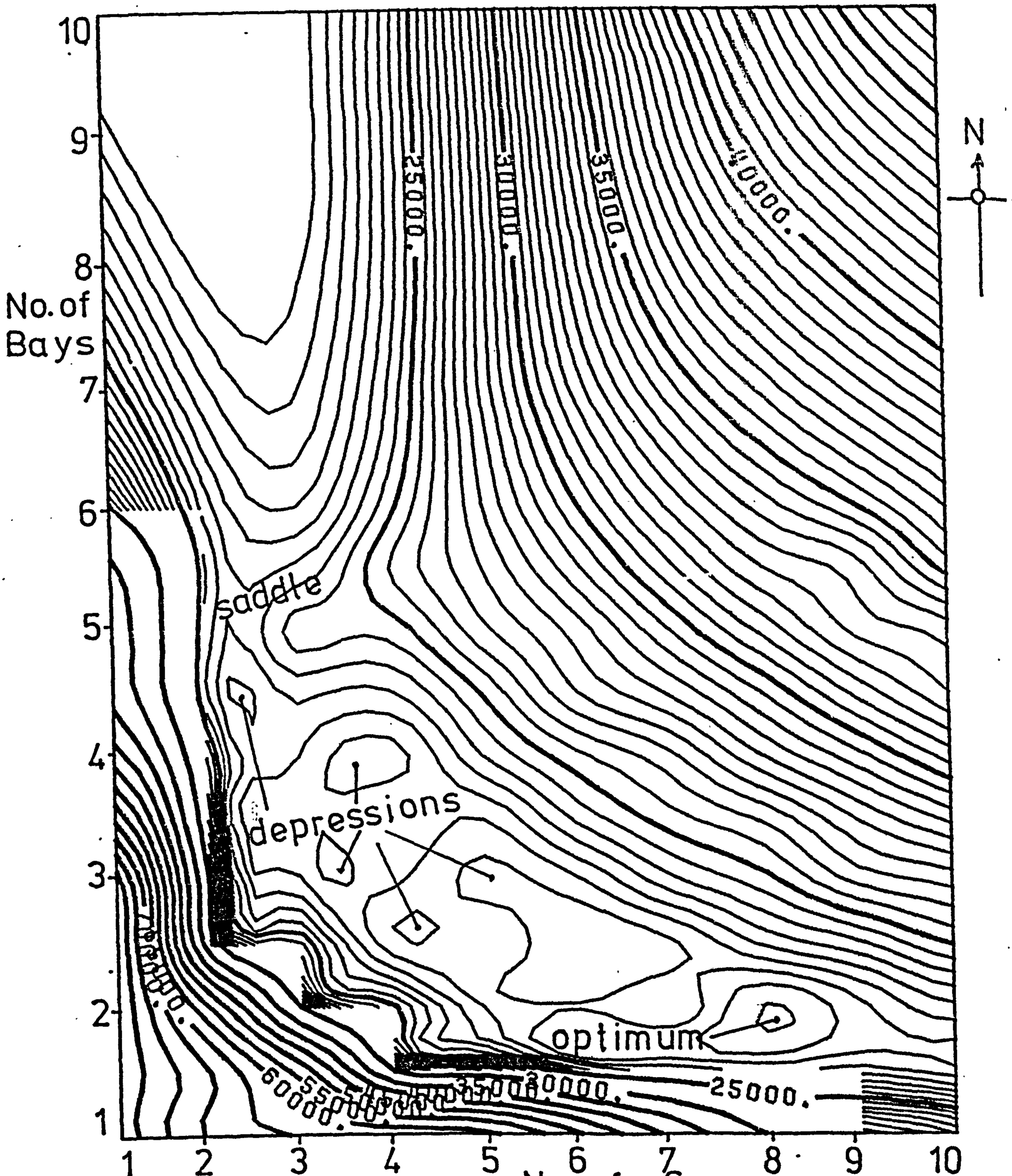

FIG 8.24 OPTIMAL RESPONSE FOR 3000 M² BUILDINGS

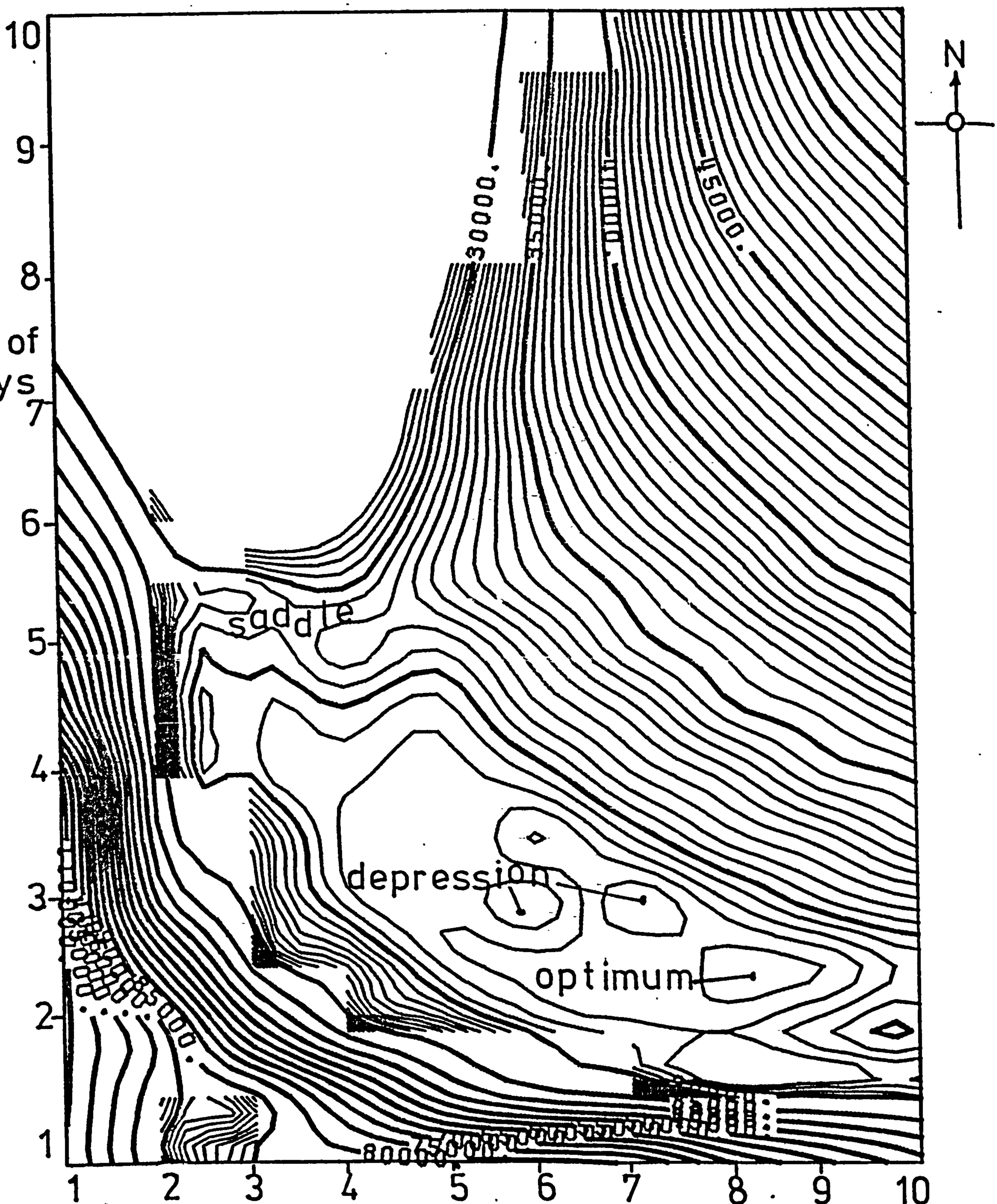

FIG 8.25 OPTIMAL RESPONSE FOR 4000 M² BUILDINGS

All the cost surfaces are strongly unimodal. The 1000 m² surface has an optimum located at (4,2,8400.84) in a trough which contains a local depression to the north-west of the optimum. The wall to the west of the trough is steep sloped whilst the wall to the east is gently sloped. Most economic buildings are in a region enclosed by a polygon whose vertices are : (2,2,9610.31), (2,3,9097.34), (4,3,9465.91), (4,1,9453.73) and (3,2,8858.29). On the 2000 m² surface the optimum is located at (8,2,15309.73) in a trough lying to the south-east. The sides of this trough are steep in the west and relatively gentle in the east. Two local depressions appear in the north-west. The optimum for the 3000 m² surface is located at (8,3,20958.64) in the south-east residing in a deep trough. Four local depressions appear in the north-west and a saddle has formed in the mid-west creating a second valley which dips north-west but shows no promise for developing local depressions. The wall to the west of the trough is more steeper than that of the previous surface but the wall to the north-east maintains a gentle slope. The 4000 m² surface has an optimum at (8,3,28525.93) and there are 3 local depressions in the north-west. The saddle emerges again seperating the trough from the north-west dipping valley. As yet this valley shows no promise for developing local depressions.

8.4.7 FRAME SPACING AND NUMBER OF SPANS (Figs 8.26 - 8.29)

$$3 \leq y \leq 22$$

$$1 \leq k \leq 10$$

Lower and upper bounds of 3-12 m for frame spacing were imposed on the 1000 m^2 and 2000 m^2 buildings, and 4-22 m on the 3000 m^2 and 4000 m^2 buildings. Roof pitch was maintained at 14°. All buildings were cladded with single skin sheeting.

The cost surfaces in Figs 8.26 - 8.29 are strongly unimodal. On the 1000 m^2 surface the optimum resides in a well defined trough located at (4,6.0,8432.32) lying in the mid-west. There are no local depressions. Buildings with spans below 5 m and above 10 m are uneconomical. The 2000 m^2 surface has a well shaped optimum located at (5,8.0,15012.04). It is seperated by a saddle from two local depressions located in the south-east. In the north-west another saddle seperates the optimum from a deep local depression. On the 3000 m^2 surface the optimum is located at (2,10.0,21000.11) in a deep trough separated by a saddle from two local depressions situated in the south-east. North-eastern slopes have become steeper. Buildings with frame spacing below 8.0 m and above 14.0 m are not economical. The optimum for the 4000 m^2 surface is at (2,12.0,26092.12). A saddle in the south-east separates the optimum from a shallow local depression and a conical hill has formed in the north-west of the optimum. Another local depression is located at north-east of the optimum. Steep sides

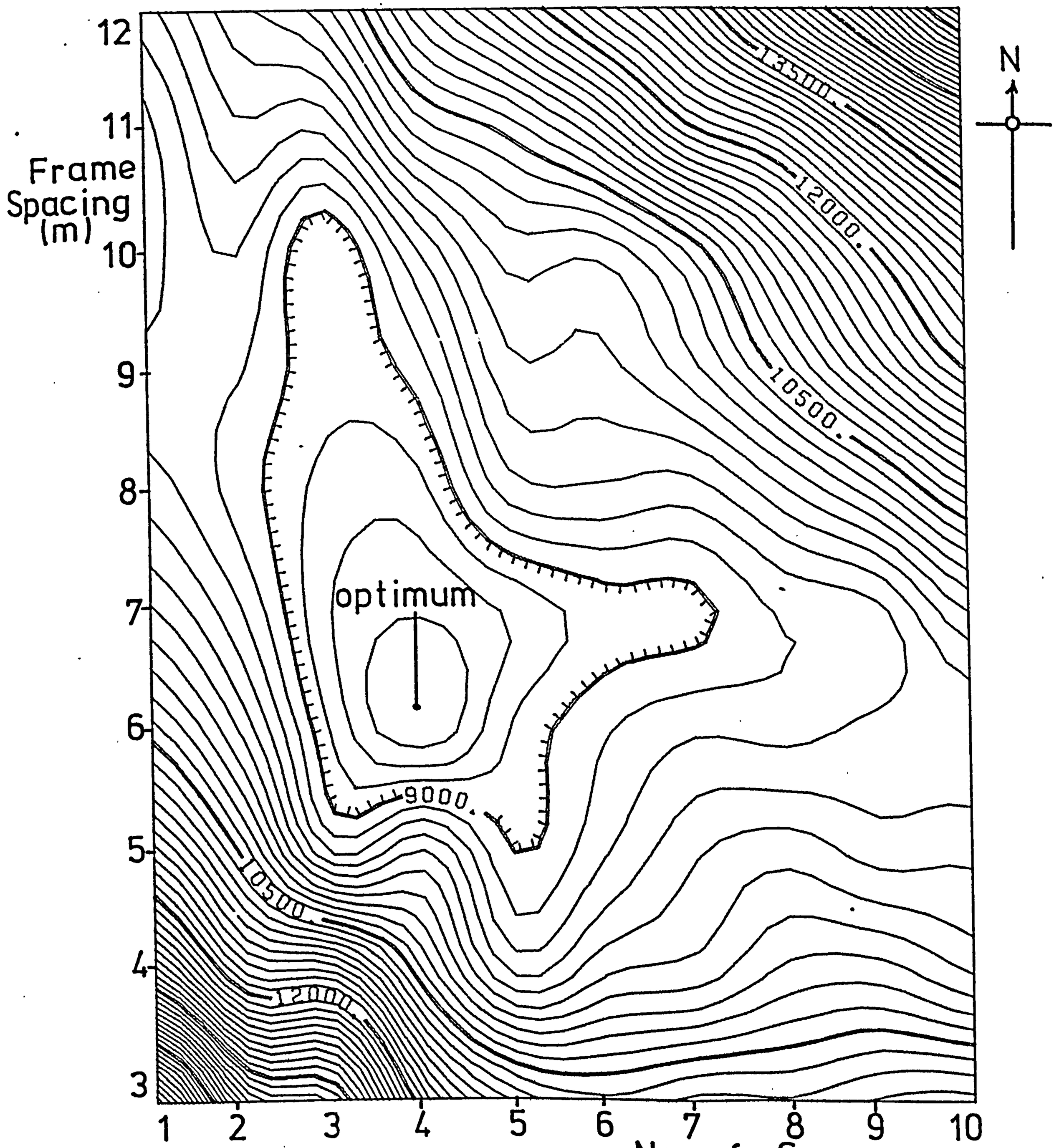

FIG 8.26 OPTIMAL RESPONSE FOR 1000 M² BUILDINGS

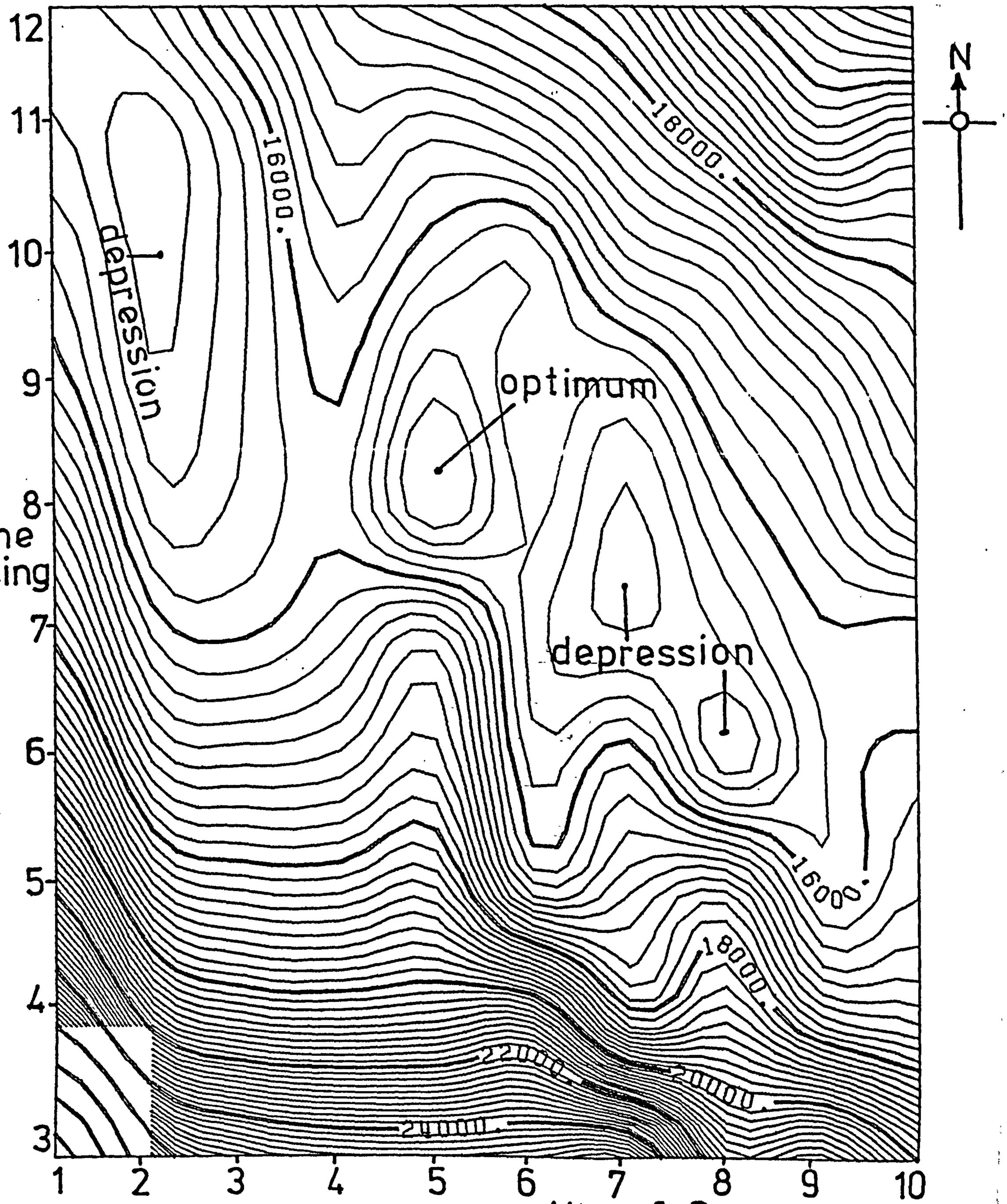

FIG 8.27 OPTIMAL RESPONSE FOR 2000 M² BUILDINGS

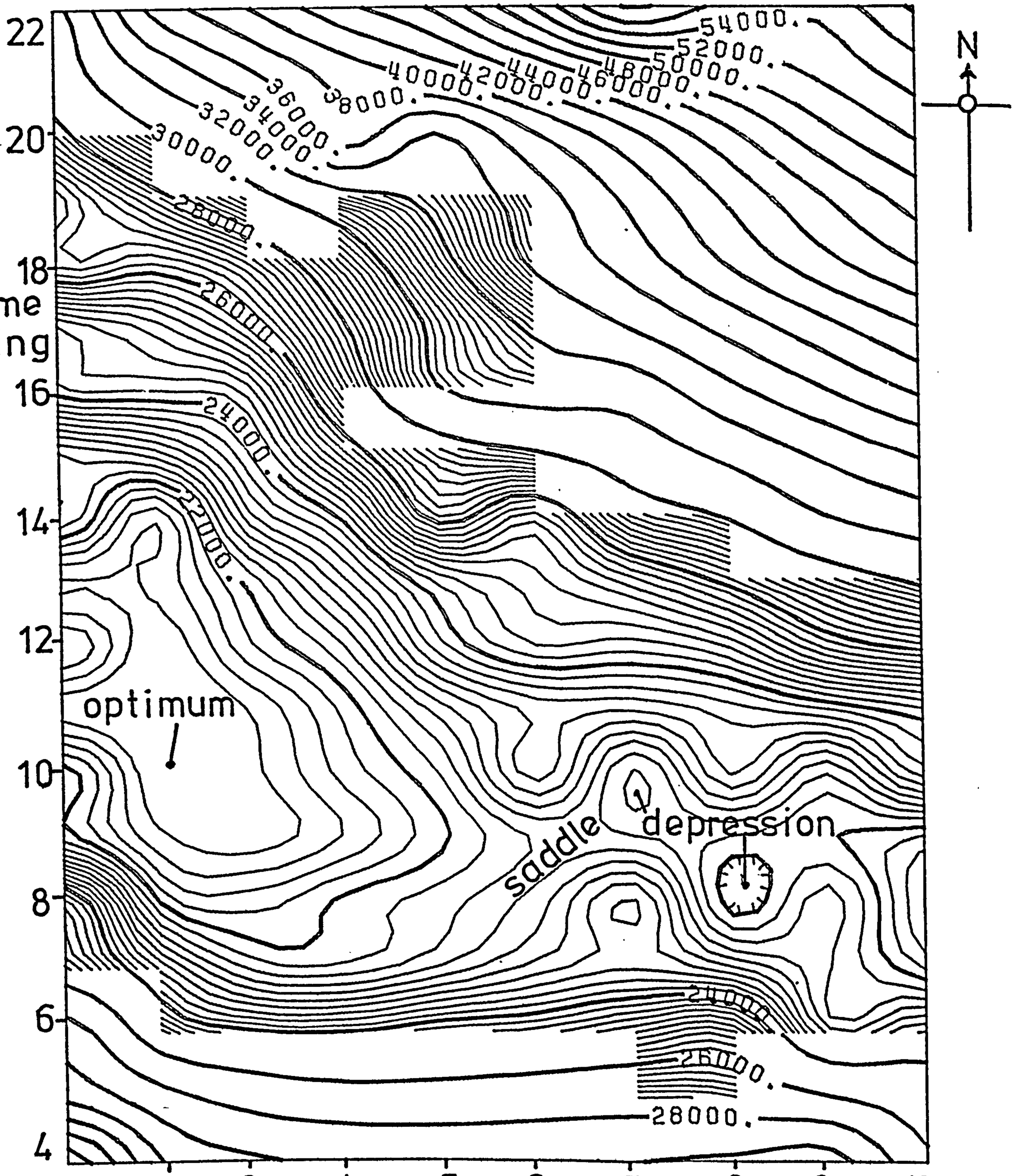

FIG 8.28 OPTIMAL RESPONSE FOR 3000 M2 BUILDINGS

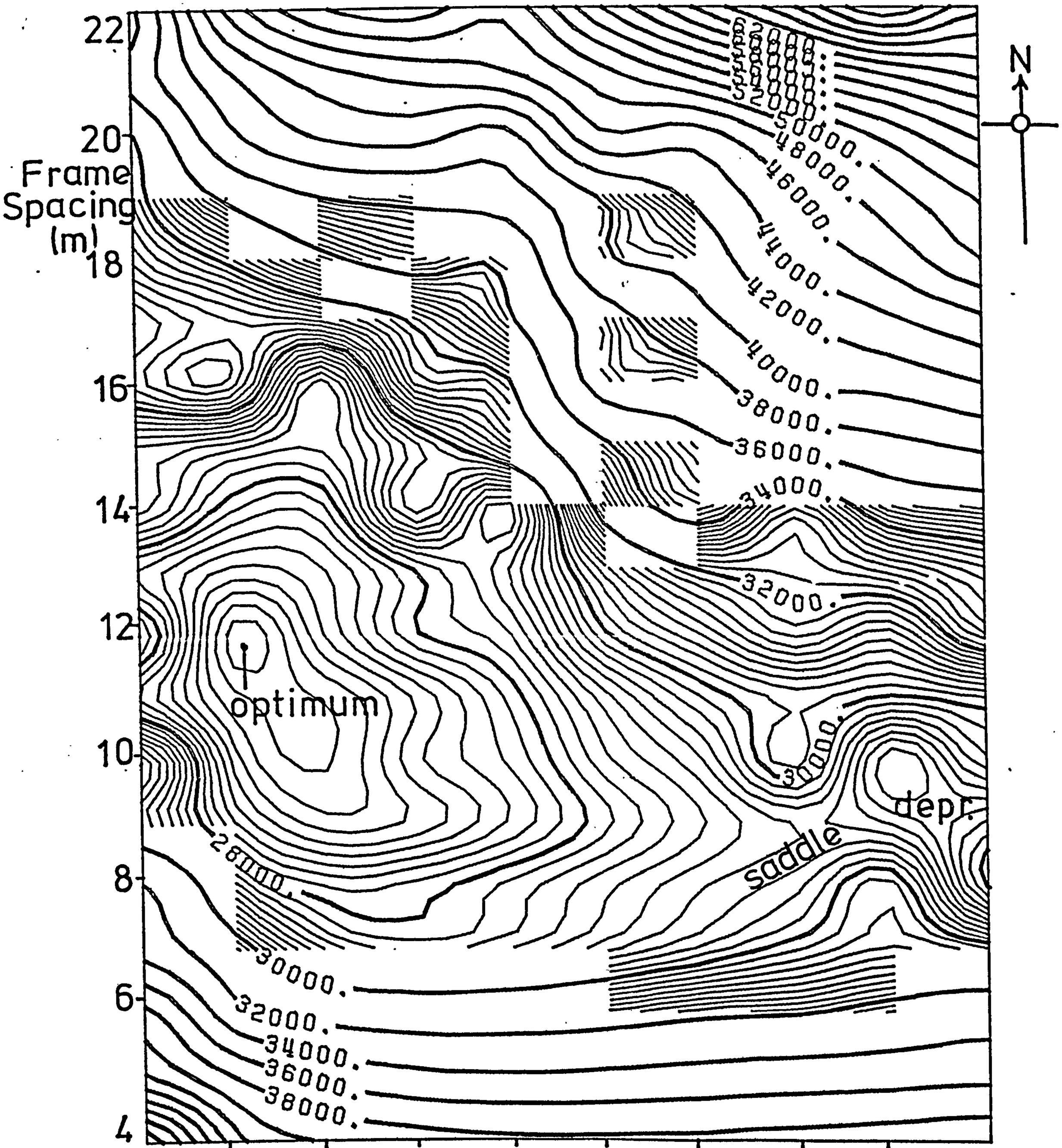

FIG 8.29 OPTIMAL RESPONSE FOR 4000 M² BUILDINGS

of the trough to the north-east of the optimum indicate that designs with frame spacing above 14 m and below 6 m are sensitive to cost.

For all these buildings savings between 2.0-18.5% would be made if the optimum design is used.

8.5 THE ECONOMICS OF FRAME

In this section the cost of running the suite of programs which has been referred to in Chapter 6 and elsewhere are presented. The cost of computer time in fact, depends on a number of factors which vary from environment to environment. Under the MTS operating system, charges made are classified under the following resources :

i. CPU time

ii. Plot time

iii. Card images read in

iv. Elapsed terminal session time

v. Virtual memory time

vi. Disc storage

vii. Printer time and printer paper

It will be assumed that no tape mount will be made.

Table 8.10 shows charges incurred at a typical session. At this session, it takes 14.653 seconds of CPU time to compile the various subprograms of the cost model (Section 6.8). Thereafter CPU times for designing a building and 50 buildings are shown. In the main graphics program, the plot time refers to a set of engineering drawings (Figs 7.05 - 7.12). The bending schedule (Fig 7.13) sheet is done separately. The optimization time corresponds to 100 perturbations (Chapter 3).

TABLE 8.10 THE COST OF RUNNING SUITE OF PROGRAMS

Program	CPU Time Sec.	Plot Time Min.	Cost £
1. Cost Model	14.653		162.79
i. 1 bldg.	0.879		9.77
ii. 50 blds.	8.818		97.97
2. Graphics (Figs 7.05-7.12)	18.370	19.15	204.09
3. B/Schedule	4.375	5.033	48.61
4. Cost Contours			
i. Fig 8.01	3.747	6.133	41.63
ii. Fig 8.02	3.888	5.533	43.16
iii. Fig 8.03	5.04	4.65	55.99
iv. Fig 8.04	5.981	4.33	66.45
5. Optimizer(100runs)	4.599		51.09

CHAPTER 9

CONCLUSIONS

9.1 SUGGESTIONS FOR FURTHER RESEARCH

The following suggestions are recommended for any possible future continuation of the work reported in this study.

i. Though precast concrete portal frames with fixed feet do not at present have as much industrial application as the 3-pinned portal frames reported in this thesis, similar studies can be carried out on its existing utilisation to see if it is a better alternative to the 3-pinned portal frame.

ii. Though there are very few prestressed concrete framed buildings being erected in the UK (Section 4.6) a significant range of such buildings are being erected in other European countries, notably Germany. An extended study into other places adopting prestressed frames will produce invaluable results that can compare with the results reported in this study.

iii. Other alternative optimization processes such as the penalty function approach can be used.

iv. 3- dimensional drawings are not used very often in

structural design. This is not because they are not useful but because of the labour involved in developing them. With recent advances in hiddden line elimination and generally new developments in graphics hardware, it is recommended that future extensions should include 3D pictures.

9.2 SUMMARY

The following conclusions are reached as a result of the study reported herein :

1. The precast concrete industry has suffered from recent worldwide economic recession. The worst of the slump appears to have taken place already but it is still premature to speculate on recovery to pre-inflation levels.

2. There is a growth in demand for precast concrete framed structures despite current worldwide recession. Generally, the precast concrete framed building indrstry is more buoyant than most sectors of the precast concrete industry.

3. Optimization processes have not as yet become readily available design tools. One recommended way to overcome this difficulty is to formulate the objective function and constraints in such a way that the optimization process can be conveniently incorporated in a design process without destroying

the internal structure of the design algorithm.

4. The cost of cladding precast concrete framed structures forms about 46% of the total cost of the structure. It is suggested that design of cladding be accorded a better scientific status that befits its importance.

5. In the fabrication of the various elements of the structure concrete placing was found to be most expersive whilst cutting and bending reinforcement to schedule was the least expensive. As a result, structural elements which require intricate mould shapes tend to be relatively more expensive to manufacture.

6. The overall cost of the structure is a function of fabrication, haulage, foundation, cladding, and erection. The proportions of these factors to the overall cost of the structure are respectively, 42%, 2%, 5%, 46% and 5%.

7. Structural design can be integrated with interactive graphics in such a harmony that could prove most beneficial to engineers.

8. In computer graphics, the building of structural engineering pictures is best achieved through efficient data structure modelling and application of mathematical transformations to subpictures.

9. The cost of the structure varies linearly with building

area. Long buildings tend to be more expensive than wide buildings for buildings whose areas are above 1000 m².

10. For most buildings an optimum exists either in an elongated valley or in a deep trough. An optimum design costs between 2% and 18% less than a traditional manual design.

11. There exists a suite of programs called PFRAME whose subsystems can be used to obtain the results presented herein.