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Comparative cost study of Two Way Slab Systems

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ABSTRACT

Comparative Cost Study of Two Way Slab Systems

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Shortage of materials is a very common construction problem in developing countries, especially in Africa. The problem arises from heavy dependance on imported technology and materials. This, coupled with unstable currencies and demand-supply variations, can lead to unpredicable costs of construction.

In the developed world, the costs of construction are much more stable. Therefore, economics of alternative building components can be compared for a predictable time into the future.

This study examines the economics of two way reinforced concrete slabs in rapidly changing economies by considering the current situation in a typical developing country. Interior panels are optimized and compared at different construction cost levels to establish trends in cost parameters. However, cost changes of upto 300 percent did not alter the competetive edge of the slab-beam system for normal residential or light office construction. Greater increases in labour costs and formwork costs would have to occur before the flat plate system can even compete with the slab-beam system. In general there was little or no variation in cost parameters.

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LIST OF SYMBOLS

a ---- Overall depth of beam in mm

A_{fv} -- Cross section area of single leg of a stirrup

b ---- beam web width

c ---- Column side dimension in mm.

d ---- Effective depth of section

FB --- Two way flat plate with edge beams

FP --- Two way flat plate without edge beams

FS --- Two way flat slab without edge beams

Fy --- Yield strength of reinforcement

Fc'--- 28 day compressive strength of concrete = 25 MPA

h ---- Slab or plate overall thickness in mm

hd --- Total drop panel thickness in flat slabs

hw --- Rib bottom width in a waffle system

IB,IS --- Moment of inertia of beam section ,slab section.

L ---- Center to center distance between Columns in mm

LL --- Unfactored live load = 1.9 KN/M^2

Ln --- Clear distance between supports or columns

N_s --- No. of beam stirrups

PB --- Two way plate/slab with beams between all supports

S ---- clear spacing between ribs in joist floors.

SB --- Two way flat slab with edge beams

SDL -- Unfactored superimposed dead load = 1.7KN/M^2

t,hf - Top flange or slab thickness in a waffle system

t_s --- Waffle slab solid head thickness

t_w --- Equivalent thickness of waffle system by weight

WS --- Two way waffle flat slab

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Selecting a suitable floor framing system is of utmost importance in the design of multistorey buildings. Many variations of floor systems have been developed in response to economic, aesthetic and structural constraints.

Today, however, the traditional insitu concrete slab system is still economically competetive. Due to fast construction benefits composite structural steel-beam and concrete-slab systems have found much favour in developed countries. In Europe, Russel et al(1) reported that the composite system held 50% of the market for medium (5-10 storeys) commercial developments. The authors further stated that this was so even though the basic cost of typical composite steel framed construction and concrete framed construction of a medium rise office block were comparable. Where concrete construction time matched composite steel construction time, the cost equation favoured concrete construction. That standard office panel grids of 9x6m favour composite steel framed construction. While squarer grids of 9x7.5m or 9x9m favour a concrete framed construction.

Wher the cost of steel in proportion to other material and building costs is very high, the concrete framed construction appears more economical. This is true for countries like Zambia, located in South-Central Africa where steel is wholly imported. Cement aggregates, labour and some timber are supplied locally with little effect from exchange rate fluctuations. Often construction time is not as important to a client who may seek the cheapest floor.

The historical development of reinforced concrete floor systems depicts the results of searches for more economical systems. Early systems were principally one way spanning and in a simple support manner. The beam and slab construction for reinforced concrete framed buildings slowly began to evolve at the turn of this century. The need for wider spans and improved load carrying performance led, in American, to the concept of flat slabs in the 1930's. With further increase in structural spans and loads the dead weight of slab became a significant part of total design. This resulted in development of voided or waffle slabs.(2)

The reinforced concrete floor systems in use today remains essentially the same. Cast in situ reinforced concrete framing systems still hold a monopoly in all multi-storey construction. This competetiveness is attributed to the better structural rigidity that in-situ construction offers. It does not require welding or prestressing which are unavoidable costs of precast construction. Use of the limit states designs method and development of higher strengths of concrete and steel have contributed to more economical long-span concrete slabs in multi-storey structures. Over the years more efficient construction techniques have been developed that make construction with in-situ concrete quicker and cheaper. Flying forms that cover entire bays and are transfered as large units from storey to storey are a good example of this. Generally, construction costs of cast in situ framed buildings are minimized through repetetive concepts.

1.2 TYPICAL APPLICATIONS OF REINFORCED CONCRETE SLABS

Undoubtedly, the economy of a floor system for a particular building will be determined by a variety of conditions. These conditions can be devided into groups. The first one relates to use or type of occupancy which sets the limit on the design loads and expected spans. Architectural preferences and expected performance of the floor system would also depend on the type of occupancy. The second group relates to locality and availability of materials. These two factors localise the actual cost of constructing the floor system. Costs of formwork, labour, reinforcements, and transportation, all fall into this group. In the third group are other factors, that are in no way minor, and may be inclusive of fire resistance, volume of construction, height of building, wind resistance, and foundation system.

Some categories of occupancy have been distinguished as particularly affecting the choice of a floor framing system.

Apartment buildings, hotels, dormitories all fall into the category of residential buildings. These are characterized by the presence of permanent partitions. Services, such as for heating and cooling, are handled through walls. Therefore, under-ceiling ducts are not required. Structural slab sotfit can function as ceiling. The column sizes are not critical. They can be blended into the architectural layout. The flate plate system has obvious advantages in residential construction. It also has the advantage that columns can be offset to suit the architectural layout because of the absence of beams.

Office buildings and commercial spaces are characterized by movable partitions. A flexible service network is thus a prominent feature of this category of occupancy. For flexibility in distribution of services like heating, cooling, power and telephones, the services are carried through central cores and distributed under slab systems. Hung ceilings are necessary to cover the ducts and lines. The appearance of the soffit is not critical so the floor system may be selected for economy. Flat slabs with drop ceilings are prevalent for spans 7.5 - 9m. For two way spans in the range 9.5 - 11m the waffle slab is more efficient. The waffle slab also provides an attractive ceiling where a hung ceiling is not desired.

Other occupancies include industrial buildings and garages which are characterized by heavy floor loads. The flat slab with capitals and/or drop panels is well suited. Two way slabs on beams are also common in this category.

Table 1.1 below summarizes the effects of category of occupancy on selection of floor framing systems. It is prepared in a way that distinquishes the four two way slab systems under investigation here. Additional notes relating to the competetiveness of each system have also been enclosed. The table is similar to that compiled by fling(3).

1.3 CONSTRUCTION COST ESTIMATION

Precise prediction of construction costs of designs is not possible. Therefore, the choice of the most economical floor system for a particular project may still depend on the designer's discretion and judgement.

On an overall project cost basis the cost of constructing a building may be estimated from average percentage costs. This is possible because it has been traditional to discriminate between purely structural, achitectural and service type costs of construction. A high structural cost would imply a comparatively uneconomical structural design. On average it has been found that purely structural costs account for 25%, architectural costs 45 to 60% and service costs 15 to 30% of construction costs.

SLAB SYMB.	DESCR.	RECOM SPAN	LIVE LOAD	BLDG. TYPE	ADVANTAGES	DISADVANTAGES
fp	two way flt plate with or without edge beam	5-8m	1.9 kn/m^2	hotels apt blks hospital offices	labour econ. exposed ceilings fast erection flexible column location least cost fmwk	low shear capacity larger deflections excess concrete at long spans
fs	two way flat slab with or without edgebeam	5-14m	>2.4 kn/m^2	offices, commercial bldgs, W/house industry plants.	economical for - heavy loading.	drop ceiling - may be reqd. expensive fmwk..
pb	two way plt with beams between all suppts	5-20m	>2.4 kn/m^2	offices public-bldgs	small deflection can carry concentrated load steel economy	expensive fmwk difficult to insert-services
ws	two way waffle slab with solid heads	10-20m	>4.8 kn/m^2	storage areas indust.-blds entrance halls	heavy load cap. desirable architecture steel & concrete economy	complicated fmwk- unless moulds are used.

Table 1.1: comparative record of slab systems

The percentage estimate is however, not an adquate method for estimating costs, but rather it shows the expected total construction costs of structural components.

In America and other developed countries its possible to empirically determine a cost factor per-square meter based on average of costs of similar construction at a given place and time. Such factors can be used to closely estimate costs of construction depending on designer's efficiency in use of materials and labour on the project in question. Such average rates are obviuosly limited to normal span, loading conditions and construction times.

When more accuracy is desired, estimates can be based on the volumes of materials required to do the work. Estimates are given by assigning in-place values to tonnes of steel, cubic metres of concrete and square metres of formwork required to build a structural system. Additional costs of labour and overheads must also be included. The costs of these variables vary from site to site and region to region such that precise forecast of construction costs of design may not be possible.

Simulation techniques are probably the ultimate answer to problems of cost estimating. Simulation is the process of conducting experiments on computers with models of the systems being designed. Simulation can, however, be time consuming particularly where the model is to be optimized.

1.4 SCOPE OF STUDY

Most common types of multi-storey constructions are residential and office buildings. Where two way floor slabs are used they are usually of square panels that ensure economy in construction. Typical two way floor systems currently used for this purpose are flat plates, flat slabs, slabs with beams between all supports and waffle slabs. Preliminary designs of floor systems always involve the tedious task of selecting the most suitable system. While economy is not the only factor influencing the suitability of a floor system, there are times when it is critical to the success of a project. In rapidly changing economies the comparative economics of these systems are continiously changing.

This study investigates the comparative economics of the two way slab systems with square panels for spans ranging from 5-11m. Loading conditions are those applicable to residential or light office construction. Construction costs assume a typical construction method and cost of material and labour rates for Zambia in 1987.

A step by step design procedure is used to optimize the cost of designs based on typical interior panels. Procedures were programmed for execution by computer. Designs were in accordance with the Canadian standards building codes.

CHAPTER 2

CONSTRUCTION COSTS

2.1 INTRODUCTION

Using simplified methods, like the Direct design method for two way slab systems design, some inferences into the comparative structural costs may be realised. Fortunately, a majority of normal floor systems satisfy conditions for the use of the Direct Design Method. Chapter 3 deals with the search for optimum solutions to the design of typical interior panels for given spans and other parameter using this method. It requires that one writes general expressions incorporating variables which can be optimized. In the next few sections these expressions are derived.

2.2 MATERIAL QUANTITIES

Basic construction raw materials are concrete, steel reinforcement and formwork. All the expressions symbolizing the quantities of these materials were obtained with close reference to Canadian standards design codes(4). Quantities were expressed in terms of volumes of material. Edge and corner panel conditions vary greatly such that writing general expressions for quantities inclusive of these would be impractical. Quantities for typical square interior panels were expressed as follow.

2.2.1 TYPICAL INTERIOR PANEL CONCRETE QUANTITIES

It is easier to envision the concrete quantities for an interior panel as being that of a strip bounded laterally by the centrelines of the panels on each side of an interior column as shown in fig 2.1. Table 2.1 presents a summary of concrete quantities. The expressions for flat slabs is deliberately split to show the quantity arising from the drop panel. Waffle slab quantities are a function of the solid head of thickness t_s and the equivalent thickness t_w of waffle part based on weight.

2.2.2 TYPICAL INTERIOR PANEL FORMWORK QUANTITIES

Materials used for forms in concrete structures include lumber, plywood, hardwood, fiberglass, plastic and rubber liners, steel and aluminium fiber forms just to mention a few. Other materials that ensure safety and quality are nails, bolts, screws, form ties, form clamps, anchors, form oils and other accessories. Forms frequently involve the use of two or more materials depending on what the selected system calls for.

Selection of the formwork system largely depends on the overall needs of the project. Proctor jr(5), in his article shows how consideration of formwork planning with overall project planning can best serve the interests of the project. However, whatever forming type is selected it has to satisfy the requirements of good formwork which may be categorised as quality, safety and economy.

slab system	concrete quantity(mm^3)
fp/fb	$L^2 h$
fs/sb	$L^2 h(1 + \frac{1}{36})$
pb	$L^2 h + b(a-h)(2L-b)$
ws	$\frac{L^2}{9}(t_s + 8t_w)$

Table 2.1 concrete quantity terms (for typical interior panels)

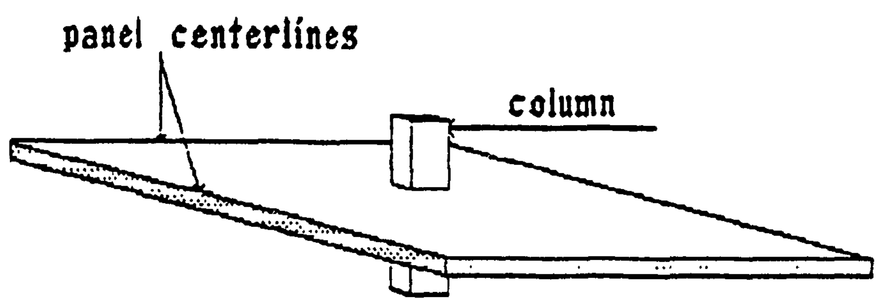


Fig 2.1: Typical interior panel for calculation of quantities.

Selecting the job-built forming system for this study would account for a major proportion of the forming practices in developing countries. This may be dictated by a shortage of materials necessary for other systems to be used. Typical job-built forms for slabs in multistorey buildings would use plywood for good finishing and lumber for structural support. This forming system has been used to estimate the volume of formwork materials required to cast the slab systems. The cost of the additional materials such as nails and oils can be added as percentage costs.

Lumber for formwork should be at least grade 2 or better. It consists of the softwoods with species used being that available in the local area. A low strength lumber was assumed to be idealy available in any area at the least. The type of lumber selected is however not very crucial since comparative costs will allow for higher or lower costs. Plywood was also assumed to be class II plywood both in accordance with the classification of the ACI standard 347-78(6). Expressions for material requirements for formwork were found to be as given in Table (2.2).

TABLE 2.2 : INTERIOR PANEL FORMWORK QUANTITIES

SLAB TYPE	LOC. IN PANEL	MATERIAL TYPE	QUANTITY (mm^3)
fp/fb	slab	plywood	$1.9(L^2 - C^2)$
		lumber	$(0.0784W1^3 + 3.875) \times W1^{1/3} \times (L^2 - C^2)$
fs/sb	slab & drop	plywood	$1.9(L^2 - C^2)$
		lumber	$\frac{8L^2}{9}(0.0784W3^3 + 3.875) \times W3^{1/3} + \frac{L^2}{9}(0.0784W4^3 + 3.875) \times W4^{1/3} + 200/3L(h_d - h)$
pb	slab & beam	plywood	$1.9[(L-b)^2 - 4(C-b)^2 + 4(L-b)(a-h)]$
	slab	lumber	$(0.0784W1^3 + 3.875)W1^{1/3} \times [(L-b)^2 - 4(C-b)^2 + 4(L-b)(a-h)]$
	beam	lumber	$[18614 + 10.85b + 6.36(a-h)] \times (L-C)W2^{1/3}$
ws	slab & drop	plywood	$1.9(L^2 - C^2)$
		lumber	$\frac{8L^2}{9}(0.0784W3^3 + 3.875) \times W3^{1/3} + \frac{L^2}{9}(0.0784W4^3 + 3.875) \times W4^{1/3}$
	slab	no. of plastic molds	$0.89L^2/S^2$

Where loads are....$W1 = 2.4 + 0.024xh$; $W2 = 2.4 + 0.024xa$

$W3 = 2.4 + 0.024xt_w$; $W4 = 2.4 + 0.024xh_d$

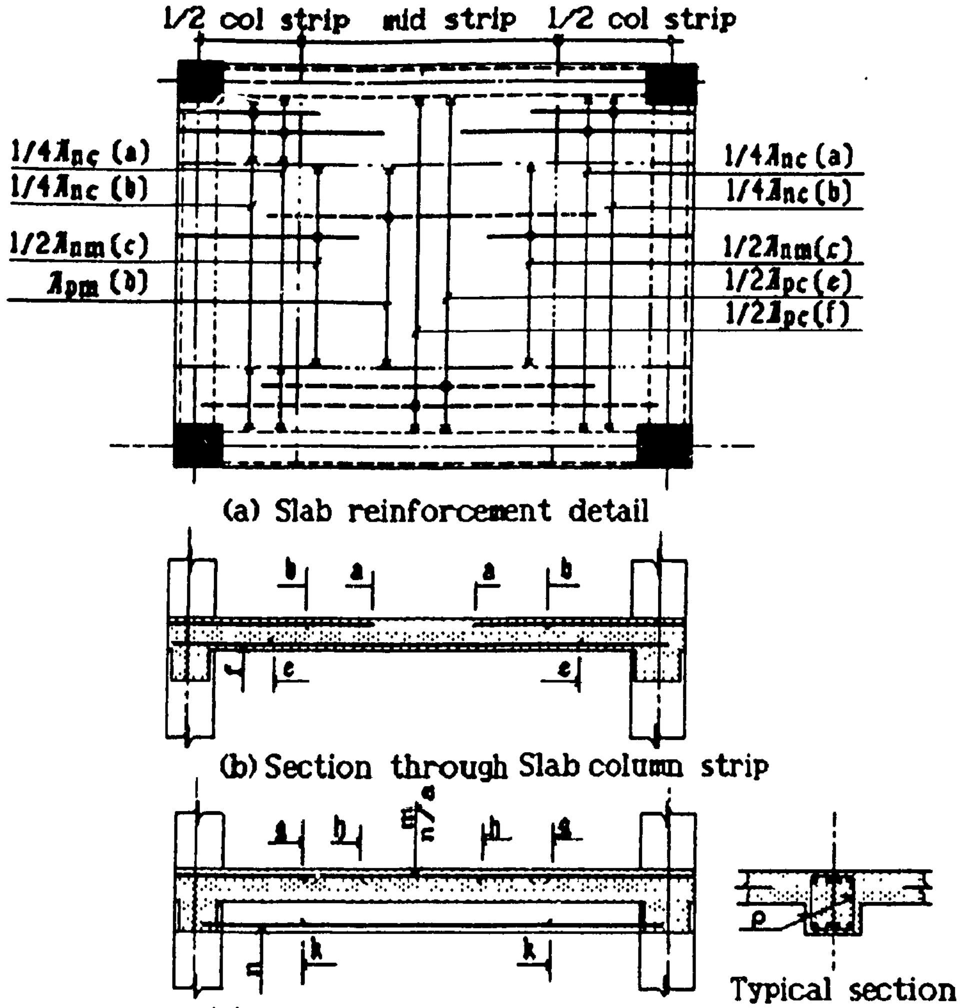
All in KN/M^2

2.2.3 TYPICAL INTERIOR PANEL REINFORCEMENT QUANTITIES

Reinforcement quantities can only be expressed with due regard to the moments they resist. However, knowing the moments alone is not adequate for total volumes of reinforcements in a slab system to be estimated. Steel comes in specific sizes from which an appropriate combination is selected to adequately resist calculated moments. Building codes impose limits on placement spacing so that, with only certain bar diameters available, there are numerous ways reinforcements can be detailed. This is compounded by the fact that the lengths of reinforcement bars for a given moment will depend on location of inflection points and the resulting anchorage required. It is imperative that some assumptions are made.

The difference between the area of reinforcement required by analysis and that detailed was assumed to be minor. The code provides guidelines on cut off points for slabs without beams. Based on general guidelines and results of preliminary analysis, cut off points for reinforcements in slabs with beams were also generalized as illustrated in fig 2.2. The detailing method assumed was that using the loose bar or splice bar method. Hence in an interior panel all bars would be straight bars except for the beam stirrups.

Reinforcement volumes were obtained by multipliying each area required by analysis with the corresponding



(a) Slab reinforcement detail

(b) Section through Slab column strip

(c) Beam reinforcement detail

BAR MARK	C/S AREA	BAR LENGTH	VOL OF BARS	VOL OF BARS by section
a	$0.5A_{nc}$	$.6L_n+C$	$A_{nc}(0.3L+0.2C)$	$A_{nc}(L+C)0.5$
b	$0.5A_{nc}$	$0.4L_n+C$	$A_{nc}(0.2L+0.3C)$	
c	A_{nm}	$.44L_n+C$	$A_{nm}(.44L+.56C)$	$A_{nm}(.44L+.56C)$
d	A_{pm}	$.85L+C$	$A_{pm}(.85L+C)$	$A_{pm}(.85L+C)$
e	$0.5A_{pc}$	$.75L$	$A_{pc}(0.375L)$	$A_{pc}(.875L+75)$
f	$0.5A_{pc}$	$L+150$	$A_{pc}(0.5L+75)$	
g	$0.67A_{ncb}$	$.6L_n+C$	$A_{ncb}(.4L_n+.67C)$	$A_{ncb}(.535L+.46C)$
h	$0.33A_{ncb}$	$.5L_n+C$	$A_{ncb}(L+C)0.165$	
k	$0.75A_{pcb}$	L_n+300	$A_{pcb}(.75L_n+225)$	$A_{pcb}(3.5L+2.5C+900)/4$
n	$0.25A_{pcb}$	$.5L_n$	$A_{pcb}(L+C)0.125$	
p	A_vN_s	$2(a+b)$	$2A_vN_s(a+b)$	

FIG 2.2: Slab with beams generalised bar details (Interior panel)

T. .E 2.3: TYPICAL INTERIOR PANEL REINFORCEMENT QUANTITIES

SLAB TYPE	CRITICAL SECTION	MOMENT TYPE	REINF. QUANTITIES(MM^3)
fp/fb	col strip	Neg	A_{nc} (L+C)
		Pos	A_{pc} 1.75L+150)
	mid strip	Neg	A_{nm} (0.88L+1.12C)
		Pos	A_{pm} (1.7L+150)
fs/sb	col strip	Neg	A_{nc} (1.06L+.94C)
		Pos	A_{pc} (1.33L+1.2)
	mid strip	Neg	A_{nm} (.88L+1.12C)
		Pos	A_{pm} (1.7L+300)
pb	col strip	Neg	A_{nc} (L+C)
		Pos	A_{pc} (1.75L+150)
	mid strip	Neg	A_{nm} (0.88L+1.12C)
		Pos	A_{pm} (1.7L+150)
	beam	Neg	A_{ncb} (1.07L+0.93C)
		Pos	A_{pcb} (1.25L+1.25C+150)
	beam shear		A_{fv} N_{s} (2a+2b)
ws	col strip	Neg	A_{nc} (1,06L+0.94C)
		Pos	A_{pc} (1.33L+1.2)
	col strip	Neg	A_{nm} (0.88L+1.12C)
		Pos	A_{pm} (1.7L+300)

length for a given moment and totaled until all the moments are accounted for. A system of subscripts was developed to distinguish the steel areas from one to the other. This required three subscripts, the first one for negative or positive moment reinforcements, the second for column or middle strip and the third applicable to beams only. The labelling system may be clear from table 2.3 which summarizes expressions for reinforcement quantities.

2.3 CONSTRUCTION METHOD STATEMENTS

Concrete costs for building construction may be broken down into labour costs and material costs. Material costs are the costs incurred in obtaining materials necessary for the concrete to be cast. Included would be the steel reinforcements, lumber, shoring, sheathing and other support systems, and concrete which may be ready-mix or mixed at site. Overheads and supervision costs should also be included in this category of material costs. This item can be added as a percentage of total costs. It will, however, be assumed to be constant in this study. The other materials were broken down into their constituent elements and priced in accordance with the rates in Spon's International Construction Costs Handbook (7). The rates are those applicable in Zambia of Central Africa for 1987.

Labour rates were also obtained from Spon's costs handbooks. Labour quantities in terms of total manhours necessary to complete a project vary markedly from one area to another. Rates have been defined to aid in comparing construction efficiencies on different sites and other countries. The ratio of the labour output to labour quantity inputed defines the labour productivity. Trends in the variations of these values can be established and used to predict the expected labour output at any site. Revay (8) reports that the authorities of the province of Alberta wondered how productivity recorded at one place can be compared with that achieved in a different country or region. In 1980-81 a study was made which recognized that there was a need to provide a standard against which productivity of individual operations could be monitored. The study concluded that the page and nation estimating manual(9) would be an appropriate such standard. Production rates at various locations could then be obtained by applying indices to the standard values. The use of indices or correction factors for productivity based on standard rates are put to good use by international contractors. These multi-national cooperations have developed (and continuosly up-date) broad ranging correction factors for their private use.

Estimators in developing countries often guess the outputs of building tradesmen. Efforts are being made to evaluate outputs from a scientific standpoint (10). Some preliminary analysis results indicate that production outputs are generally lower than estimated outputs even-though these are achievable. The explanation given for lower production outputs is that there is a lack of incentives for workers and the pay is on per attendance in-lieu of a pay per output basis.

In this report, the purpose being to determine the cost competitiveness of designs under unpredictable cost levels, the page and national estimating manual was used where possible to estimate productivity manhours. Reinforcing steel is usually supplied by subcontractors who charge flat rates for cut, bent and fixing per tonne. This is a common practice in most countries including Zambia(11). The available current rates will be used here. Labour manhours for concrete and form work related work must be considered in detail.

2.3.1 LABOUR MANHOURS FOR CONCRETE

The composition of the concreting labour force depends very much on required completion times and hence the selected equipment to be used. Statistically, average costs of delivering and placing concrete for a particular project may be estimated for the selected construction program. In multi-storey construction for medium height buildings the construction program will be determined mainly by the choice of hoisting equipment, and required delivery rate of fresh concrete. Fresh concrete may be supplied from a manufacturing plant in trucks or prepared at site in mixers that vary greatly in capacities. Therefore, there is no ideal construction model that would intergrate all possible alternatives.

A pre-requisite to achieving the cost comparison of the slab systems is thus to make assumptions similar to those made by Jaafari(12). The stated intent of his study was to establish the inherent cost competetiveness of design options in relative terms. The current study can only achieve such a goal. Clearly, the relative cost of each system under any construction procedure must be assumed constant. The actual cost of the system being a product of the prevailing labour rate. Then any construction method can be selected as a standard to aid in the selection of the optimum slab configuration.

The procedure adopted uses a crane and buckets to hoist concrete. Two buckets are ideal so that while one is being filled the other is being hoisted. The buckets have bottom gates for pouring into position. Labour requirements for hoisting would hence be typically as shown in table A.1 of appendix A. Mixing can be done using mixing plants at site. The raw material being ferried by wheel barrows from temporary storage and tipped into mixer/s. The labour force requirement for a 14/10 (0.4m3) mixer being as given in the same table. The labour force requirement for placing concrete will depend on required placement rates determined by the hoisting rate. Generally, placing concrete in slabs would constitute of the labour force shown in Table A.1 of appendix A.

The composite rate for concreting work can now be calculated using the method suggested in the Page man-hour manual.

Labour hours for mixing, hoisting and placing in slabs, but excluding the multipliers for hoisting to different heights, may be summarized as follows:

i) mixing plant hours - - - - - - - - 0.64

ii) hoisting plant hours - - - - - - 0.64

iii) labourer hours - - - - - - - - - - 5.1

iv) wheeling concrete in barrows of $0.08m^3$ capacity up to 25m distance - - 1.07

Total manhours to concrete $1m^3$ are thus = 7.45hrs.

Let labourer rate be \$U, semi-skilled worker rate be \$S and skilled/foreman's rate be \$K. Then labour rate for mixing hoisting and placing concrete upto a height of 3m is given by:

$$\text{Composite rate} = \frac{(0.64\times\$U+0.64\times\$S+1.07\times\$U+5.1\times(2\$K+15\$U)/17)}{7.45}$$

$$=(0.08\$K+0.086\$S+0.834\$U)\text{cost/hr}$$

$$\text{Cost}/m^3 = 7.45\times\text{Composite rate} = 6.21\$K+0.64\$S+0.6\$K$$

Additional costs for hoisting to heights greater than 3m can be put in a general form as follows at floor height H above ground.

$$\text{Concreting Cost} = (6.21\$K+0.64\$S+0.6\$K)\times(1+0.05\times(H-3)/3)$$

Typical values of \$U, \$S and \$K extracted from the Spons international handbook are 1.97, 2.03 and 2.34 respectively as cost of labour per hour for the year 1987. Current rates are 1.98, 2.08 and 3.08 respectively obtained from Shonga Steel Limited. Using the current rates the composite rate would be found to be 2.08 giving rise to a concreting cost of 15.475 per cubic meter for heights up to 3m.

Similarly, composite rates or rates of mixing, hoisting and placing concrete in beams were determined. The results of these calculations are also given in table A.1.

2.3.2 LABOUR RATES FOR FORMWORK

Forming work constitutes of fabricating formwork inclusive of unloading lumber, fabricating in yard, and oiling forms. The forms are then erected by hauling fabricated forms into place, laying out the field, assembling the form and bracing it, cleaning up and standing by for concreting. Once the concrete has achieved the required strength the forms are stripped and cleaned before hauling into new locations.

Formwork is usually erected by carpenters while labourers carry and haul it about. Crews usually consist of several carpenters with a ratio of 1 to 2 labourers to two carpenters. A typical crew of 3 carpenters, 3 labourers and 1 foreman was assumed. The cost of employing a labourer, carpenter and foreman were set at 1.98, 3.08, 4.58 respectively for unskilled, skilled and foreman rates.

Total carpenter and labour manhours would, by description of the work, be the same. Hence, the composite rate for formwork may be calculated as:

((3\$U + 3\$K) + \$F)/7 = 4.13/hr

Manhours for first use of formwork including a multiplier for re-uses are summarised in table A.2 of appendix A.

Formwork labour costs calculated for first use would thus be a product of the composite rate and the total manhours. Results of these calculations are also shown in the same table. Note that an average formwork labour cost rate for a whole slab system can only be determined when the quantities of formwork to the various members are known.

2.4 CONSTRUCTION COST SUMMARY

Table 2.4 gives a summary of the cost of erecting a slab system based on labour and material costs. Concrete costituent costs assume a 1:2:4 mix. Cost of formwork is on a one use basis. Labour costs assume a height of 3m.

TABLE 2.4 : SUMMARY OF CONSTRUCTION COSTS

(1987 rates for Zambia, unless specified)

SUBJECT	COMPONENT	ITEM	UNIT COST	UNIT
Formwork	material cost	plywood 19mm	70	m^2
		lumber(soft-wood)	2900	m^3
		plastic moulds (hire rate)	20	ea./wk
	labour cost	slab	7.847	m^2
		beams	10.325	m^2
		drop heads	13.22	m^2
		laying moulds	1.65	m^2
concrete	material cost	aggregates	120	m^3
		cement	370	MT
		concrete(estm)	270	m^3
	Labour cost	slab	15.475	m^3
		beams	18.444	m^3
steel bars	(1987)	material cost	5150	MT
		cut, bend and fix	400	MT
	(1989)	material cost	13300	MT
		cut, bend and fix	950	MT

CHAPTER 3

OPTIMUM COST OF INTERIOR PANELS

3.1. INTRODUCTION

Chapter 2 focused on the derivation of expressions that establish the cost of a typical interior panel of a flat plate, flat slab, waffle flat slab and slab or plate with beams. Several options are available by which constraints to the variables formulating costs can be obtained and the costs optimized. More often, mathematical optimization is used. In a majority of structural optimization cases the geometrical programming method is used but rarely has it been applied to two way slabs with beams or drop panels. Barr et al(13) developed an algorithm called GPALL based on the geometric programming theory to solve large engineering design-related optimization problems. It was used successfully to optimize the cost of a typical continous reinforced concrete slab bridge. The conclusions were that with higher steel relative cost, it is economical to use a large concrete section with light reinforcement, while, with higher concrete relative cost, a light section with heavy reinforcement becomes feasible. On another note, Salinas(14) used the standard form of the geometrical programming theory to examine the relation of material costs and strength to the cost of reinforced concrete one-way slabs. He concluded that the premium to be paid for strength remains practically unchanged. His study considered a price increase of fifty percent in the basic materials, namely, concrete, steel and formwork.

Loov and Khalil(15) were apparently dissatisfied with the more difficult geometric programming technique and developed an iterative procedure for design of flat plates. The procedure, however, is inapropriate for slabs where there are various elements for whom the thickness has to be optimized. A general approach to the study presented here was to resort to basic principles. In mathematical terms it may be described as a direct search technique. Essentially, all possible combinations of the variable values are examined for the possibility of providing the minimum cost. Certainly, before one can embark on such a search the design boundaries must be well defined to avoid unnecessary computations. Then the steps that expidite the search can be set out. The BASIC language was used to program the solution algorithms presented in the next paragraphs. Designs are limited to those where the direct design method is applicable. Although in the case of flat plates and plates with beams it was possible to compare their optimum solutions in a single program it would be easier to treat the systems under separate headings.

3.2 FLAT PLATE

The code recommends some emperical equations for the minimum overall thickness of slabs for control of deflections. This provided the requirement that the slab depth ,h, (see fig 3.2) be equal to or larger than a minimum depth. The problem of punching or two way shear has a major effect on the minimum possible depth of flat plates supported on columns. An additional requirement was thus to review the minimum thickness until both limitations are adequately satified. Recommended procedures for satisfying shear requirements are, increasing the column dimensions, increasing concrete strength, increasing slab thickness, use of slab shear reinforcement and use of drop panels or column capitals. In this study the use of shear reinforcement was not considered. Either the column dimensions were adjusted or slab thickness was increased in 5mm increments.

Two way shear in flat plates occurs in the vicinity of slab-column connections. Shear failure may occur by punching through of the area of concretrated loading. The factored shear force Vf, causing punching shear, may be computed as the net upward column reaction less the downward load within the area of slab enclosed by the perimeter of the critical section. The critical section for punching shear being located so that the perimenter is a minimum but not approaching closer than half effective depth to the perimeter of the support. Punching shear is increased by the occurance of unbalanced moments being transferred to columns. The code considers that a portion of the total unbalanced moment is transferred by eccentricity of shear. This portion is denoted as Mfv. The maximum shear stress thus, combines the effects of both Vf and Mfv. The code requires that the combined shear stress does not exceed a maximum shear stress resistance. Where shear reinforcement is not provided the shear stress resistance is a function only of the square root of fc' and the ratio of column sides.

Flat plates must also be checked for one way or beam shear. The factored shear force, causing one way action shear, was computed by considering a typical strip of slab spanning between columns. The critical section for one way shear occurs at a distance d from the face of the support. The code requires that shear stress resulting from one way action does not exceed a maximum shear stress resistance. Slabs are generally safe in one-way shear, but where necessary, slab thickness is increased.

The flat plate of thickness satisfying the above shear criteria was subsequently designed for flexure. In the direct design method the sum of the absolute values of the positive and average negative factored moments in each direction of a panel or equivalent frame span are set at the minimum total statical factored moment MO. Then for interior spans of two way slabs the moment MO is distributed to negative and positive moment sections of panel through distribution factors 0.65 and 0.35 respectively. The negative and positive factored moments are further distributed between column and middle strips. Interior square panel flat plate system strip moments would generally be found to be;

MNC = 0.75 x 0.65 MO ....(3-1)

MPC = 0.60 x 0.35 MO ....(3-2)

MNM = 0.25 x 0.65 MO ....(3-3)

MPM = 0.40 x 0.35 MO ....(3-4)

The flexural reinforcement required to resist the design moments above was then determined and the construction cost associated with the design evaluated as follows;

COST=FMC + CMC + SMC + FLC + CLC + SLC......(3-5)

Where FMC = Formwork material cost

CMC = Concrete material cost

SMC = Steel material cost

FLC = Formwork labour cost

CLC = Concrete labour cost

SLC = Steel labour cost

To study the effect of changes in the cost of the construction cost components the design cost in equation (3-5) was re-evaluated for several unit material and labour price levels. New price levels were obtained by associating cost factors CF, CC, CS, CL to formwork, concrete, steel and labour costs respectively in equation(3-5). Construction costs were calculated iteratively for slab thickness increased in 5mm installments until the cost starts rising.The iterative design evaluation process thus produced the optimum flat plate system configuration for each combination of price levels. This iterative process of selecting optimum cost flat plate system is illustrated in the flow chart fig B.1 of appendix B.

3.3 FLAT SLAB

Where drop panel plan dimensions are at least a third of the centre to centre column spacing and drop panels project no less than a quarter of the plate thickness the code recommends a 10% reduction of the minimum thickness for control of deflections required for flat plates. One way and two way shear checks are necessary and follow the same procedure as for flat plates. However, one and two way shear must also be investigated at edge of drop panels. Starting dimensions for the iterative solution process were results of the slab thickness checks for one and two way shear at the appropriate distance off the drop panel edge. The drop panel projection below slab was a minimum of a quarter of the slab thickness which would often require adjustment when scrutinized for one and two way shear around column.

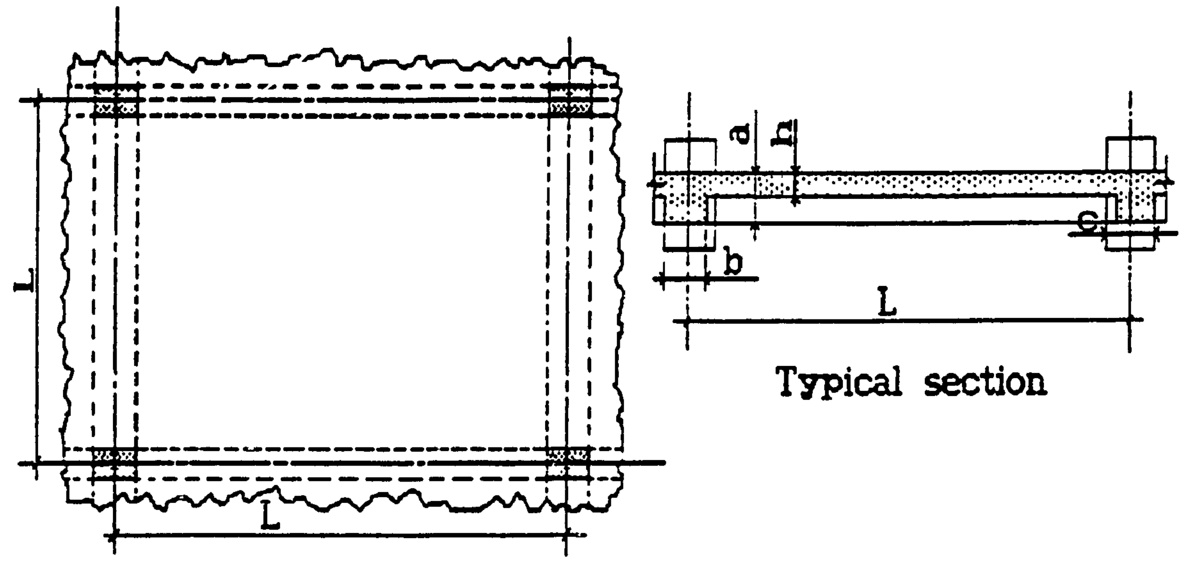


Fig 3.1 Slab-Beam system
typical Interior panel.

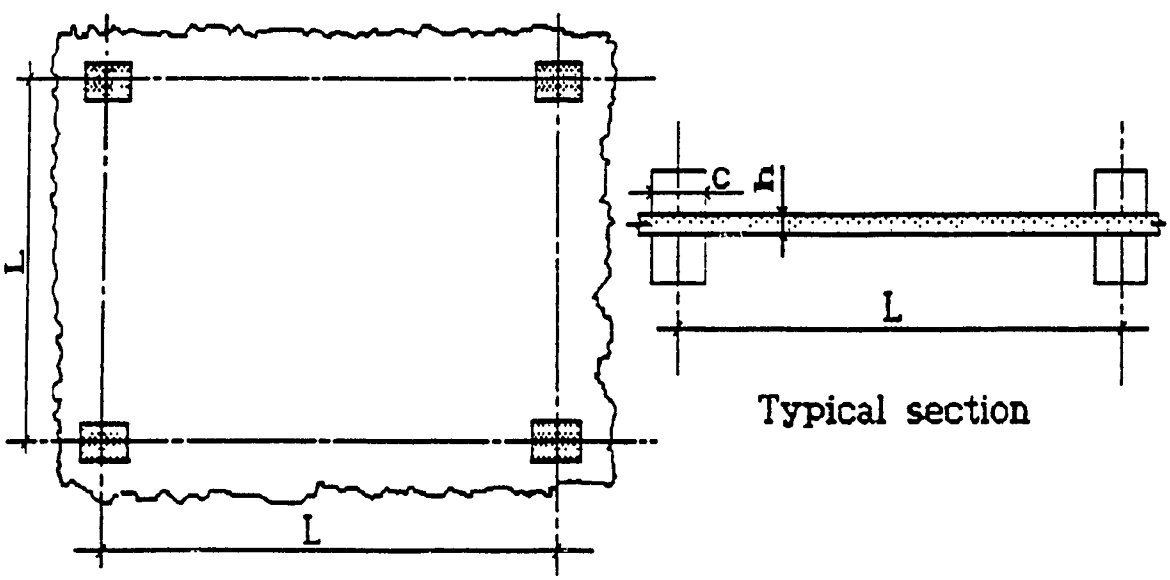
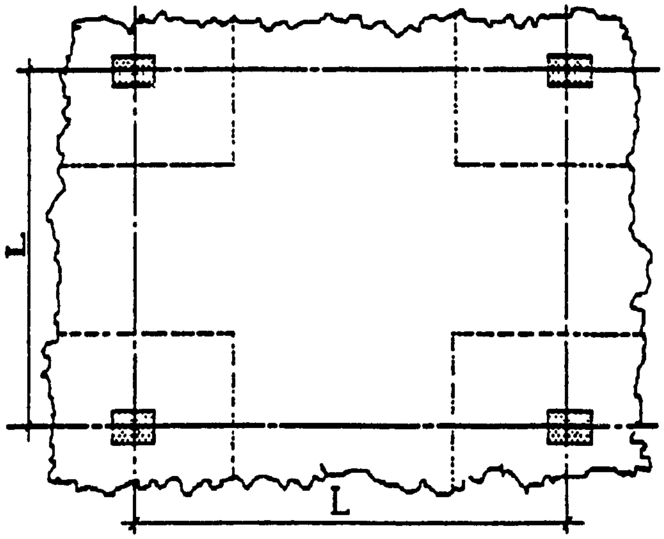
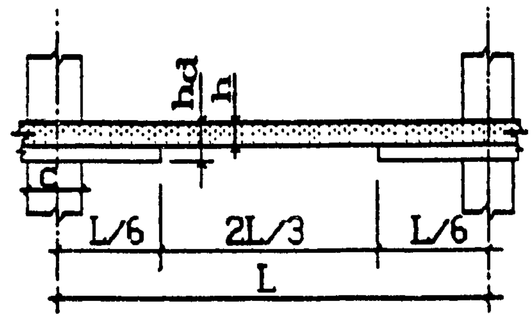


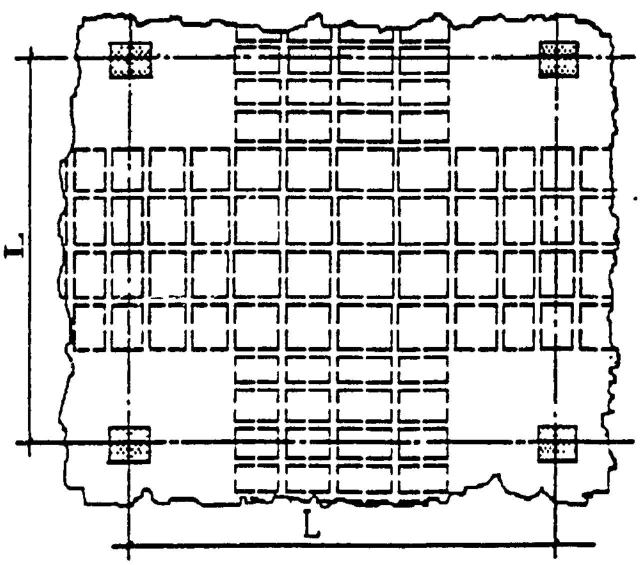
Fig 3.2 : Flat Plate system
typical Interior panel.

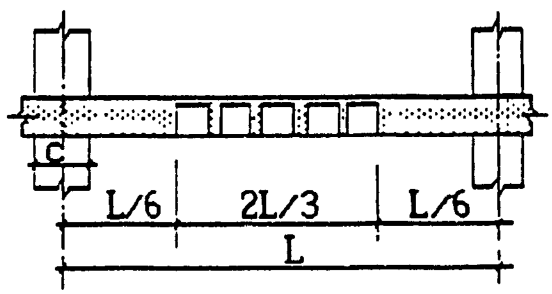




Typical section

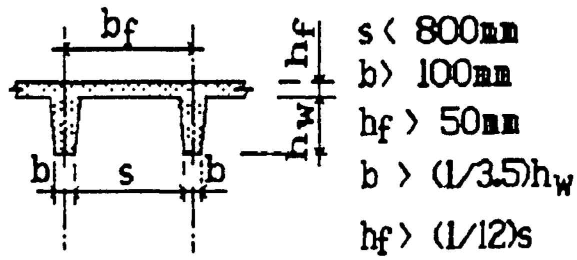
Fig 3.3 Flat Slab system
Typical Interior panel





Typical section

Fig 3.4 Waffle Slab system
typical Interior panel



Dimensional requirements for
Joist Floors

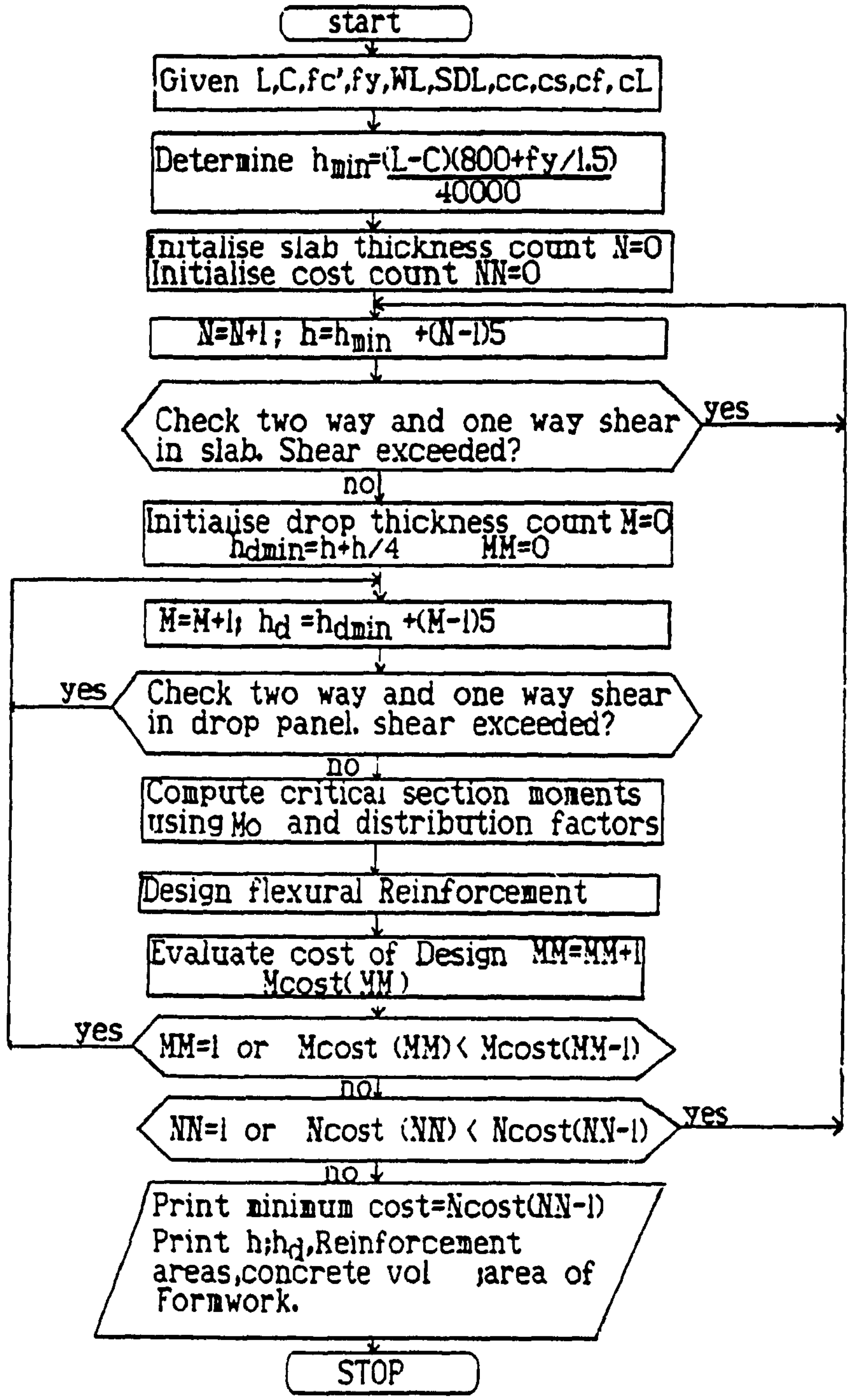


Fig 3.5 : Sequence for cost optimization of Flat Slab

Distribution factors for distributing MO to negative and positive moment sections and finally to column and middle strips are exactly the same as in flat plates. Therefore equations (3-1) to (3-5) still apply here. However, not only was the construction cost calculated iteratively for slab thickness increasing by 5mm, but the drop panel projection was varied from the minimum required to satisfy shear requirements upto some pre-set upper limit. Drop panel projection was also varied in 5mm increments at each level of the slab thickness iteration process. The drop panel projection was increased progressively for as long as the total cost in equation (3-5) was reducing. The slab iteration process is illustrated in the flowchart fig 3.5 (also see fig 3.3).

3.4 PLATE WITH BEAMS

The two way slab-beam system is characterised by the presence of beams between all supports. The flexural stiffness of the beams has a major influence on the expected slab deflections. Code empirical equations for determination of minimum overall thickness of slab for control of deflections incorporate parameters representative of the beam flexural stiffness. The code defines a parameter α, which is the ratio of the beam to slab stiffness. To ensure two way action, limitations are required on the ratio of α for adjacent beams. The code also defines the mean value of α for all beams on the edges of a panel as αm to be used in determination of the minimum overall slab thickness. A high value of αm leads to a thinner depth of slab. Limits are imposed on the values that αm can take by ensuring that all the three control equations are always satisfied. In interior panels with same size beams on all sides $\alpha m=\alpha$ and the upper and lower limits on αm in the control equations correspond to 2.0 and 0 respectively. Where, if $\alpha m = 0$ then slab is a flat plate system, while for $\alpha m > 2.0$ the beam section increases in size but the minimum slab thickness cannot be less than that determined by $\alpha m = 2.0$.

Interior panel slab-beam systems were studied for α values ranging from 0 to 2.0. The slab thickness established by the particular value of α was reviewed by ensuring that one way shear is satisfied at a distance equal to the effective depth off the supports. A two way shear check is necessary where the beam-slab stiffness ratio is less than 1.0. Shear was proportioned to slab and beam by linear interpolation assuming that beams carry no load at zero stiffness ratio.

Where an α value has been selected and the slab thickness selected, an unlimited number of combinations of beam width and overall depth are possible by which the required stiffness ratio can be achieved.

In practice, a trial and error procedure is used to select beam dimensions that satisfy loading and other design requirements. However, where it becomes necessary to search for an optimum cost, all combinations of beam width ,b, and overall depth ,a, (see fig 3.1) must be checked. This was achieved by varying the b/a ratio from 0.25 to 1. Then substituting for the known variables in the expression for the stiffness ratio, a six degree polynomial in h/a can be obtained. Then Newton Raphson techniques were used to determine the root lying between 0 and 1 which was found to be unique. Since h was known, the values of b and a could be uniquely determined.

The proportions of the total static moment MO allocated to negative factored moments and positive factored moments were given in section 3.2 for two way interior panels to be 0.65 and 0.35 respectively. The distribution of these factored moments to middle strip, column strip and beam strip must take the ratio of beam to slab flexural stiffness into account. Since panels are square panels, the distribution is directly related to α resulting in general expressions for strip moments as follows:

MNCB = 0.85 x α x 0.75 x 0.65 x MO ---------------(3-6)

MPCB = 0.85 x α x (0.6+α x 0.15) x 0.35 x MO------(3-7)

MCN = (1-0.85 x α) x 0.75 x 0.65 x MO -----------(3-8)

MPC = (1-0.85 x α) x 0.35 x (0.6+0.15 x α) x MO--(3-9)

MNM = 0.25 x 0.65 x MO -------------------------(3-10)

$$MPM = 0.35 \times (0.4 - 0.15 \times \alpha) \times MO \text{------------------} (3\text{-}11)$$

In the section moment equations above α varies from 0.0 to 1.0 and takes a value of 1.0 for all α greater than 1.0. Consequently, for $\alpha > 1.0$ moments are practically constant while beam moment carrying capacity increases. A high α value will reduce the steel requirements in beams but the same may not suffice for the slab whose thickness may be reducing. That is why it was necessary to study slab-beam systems at selected α values ranging from 0 to 2.

Construction costs associated with each solution were evaluated using equations (3-5). All possible solutions for selected α value were obtained by varying the beam section through the ratios b/a for each slab thickness. Then the slab thickness was varied progressively in 5mm increments until the member proportions corresponding to the minimum cost were determined. This constitutes an iterative process of selecting the optimum cost slab-beam system. The flow chart in fig B.1 of appendix B illustrates this procedure.

3.5 WAFFLE SLAB

Waffle slab systems may combine the waffle portion with solid column head or with solid wide beam sections along column centrelines. Some designs (17) aimed at comparing these two alternatives have shown that the waffle system with wide beam sections has definite advantages for rectangular panels, unequal spans and at large opennings.

A waffle system of constant depth with a solid column head was selected, arbitralilly, for this study although it also appears to have advantages for square panels.

In two way waffle slab systems the waffle portion consists of concrete joists at right angles to each other. The joists are often formed by using standard square void forms. Code lim..tations defining joist construction provide for ribs at least 100mm wide, spaced not more than 800mm clear, and depth not more than three and a half times the width. Based on these limitations standard sizes for re-usable void forms have been established in industry to be 600 x 600mm and 900 x 900mm in 200, 250, 300, 350 and 400mm depths. Ribs are tapered from the bottom to the top to make the removal of void forms much easier. The bottom width of the ribs is usually about 150mm which was used in this study. Hence, in a typical design only the top slab thickness, rib depth and spacing were to be evaluated to satisfy design requirements. the labelling procedure and code limitations for joist construction are shown in fig 3.4.

The code recommends that, for analysis purposes, the waffle part can be assumed to be a solid slab whose moment of inertia equals that of the waffle system. When considered this way, the waffle slab is just like a flat slab with a slab thickness based on the moment of inertia. Flat slab system procedures for evaluating the minimum overall thickness for control of deflections can hence be applied to this equivalent slab system. The study assumed that the overall thickness of slab calculated for flat slab was the required equivalent thickness. By a reverse process the joist section providing such an equivalent slab thickness can be found. At this point rib bottom width is known while the other parameters may take any values. The solution process was simplified by first selecting mimimum values of rib spacing and depth. Top slab thickness became the only unknown which was obtainable by equating moments of inertia. Rib spacing and depth were adjusted as dictated by shear and flexural requirements.

One way and two way shear checks in solid head follow the same procedure as in flat slabs. The shear stresses off the edge of the solid head, however, require special treatment. These shear stresses should be assessed at a distance off solid head equal to half the effective depth of ribs in case of two way shear and a distance equal to the effective depth of ribs for one way shear. The shear stresses cannot exceed the maximum shear stresses as prescribed for flat plates. Two way shear is rarely investigated in ribs off the solid head. One way shear is more critical because the ribs tend to act as numerous small beams. The code recommends that the factored shear stress resistance in ribs may be increased by 10 percent over that required for beams.

To satisfy shear requirements in solid head the overall solid head thickness was adjusted by increasing the top slab thickness. There is a general view that the use of shear reinforcement in ribs should be avoided for economic reasons. An increased rib section is preferable to the difficulties presented by fixing stirrups in the numerous small ribs. Shear requirements were satisfied by doing one or more of the following; reduce rib spacing from 900 to 600mm, increase rib depth or increase top slab thickness. A point to note is that each time the rib spacing or rib depth is changed the top slab thickness was recalculated such that the equivalent thickness does not differ greatly from that being sought.

The moments at critical sections of column and middle strips are the same as those for flat slabs or plates. Reinforcement design for positive flexure assumes that joists act like rectangular beams. Like beams, the minimum percentange reinforment in joists is thus 1.4/Fy. Increasing overall slab depth will reduce the reinforment quantities which also increases concrete and void form requirements. A balance has to be achieved that results in a minimum or optimized cost from equation (3-5) of waffle slab system for a given set of item costs. This calls for an iterative design process already described for other slab systems. An iterative procedure for selecting the minimum cost waffle system is difficult to formulate because most of the dimensional variables do not vary continously since they can only take certain values. Two approaches used in this study were to vary the top slab thickness and then the equivalent solid slab thickness. The variation was in increments of 5mm. The minimum top slab thickness of 75mm was found to suffice in nearly all cases considered. The flow chart in fig B.2 of appendix B shows the iterative procedure.

CHAPTER 4

DESIGN OF TYPICAL FLOOR

4.1. INTRODUCTION

Interior panels are only a part of the floor framing system. The results of their analysis must be interpreted with the overall slab system configuration in mind. In flat plates and flat slabs, for example, the code recommends that the minimum overall slab thickness determined for deflection control shall be increased by 10% in edge panels unless edge beams are provided. The magnitude of unbalanced moments at corner or edge columns is high leading to high punching shear stresses in these regions. The result, edge and corner panel slab thickness will be substantially greater than in the interior panel. In practice a uniform slab thickness is commonly used throughout the slab system as required by the edge conditions. Nevertheless, the relative economics of the structural floor systems may still be dictated by edge conditions. The reliability of comparative costs of interior panels for selecting floor framing systems deserves some investigation. One way of determining the applicability of interior panel results is to study the variation of interior panel material quantities in relation to the slab system material quantities. In this chapter such a study is carried out through the detailed design of a typical floor.

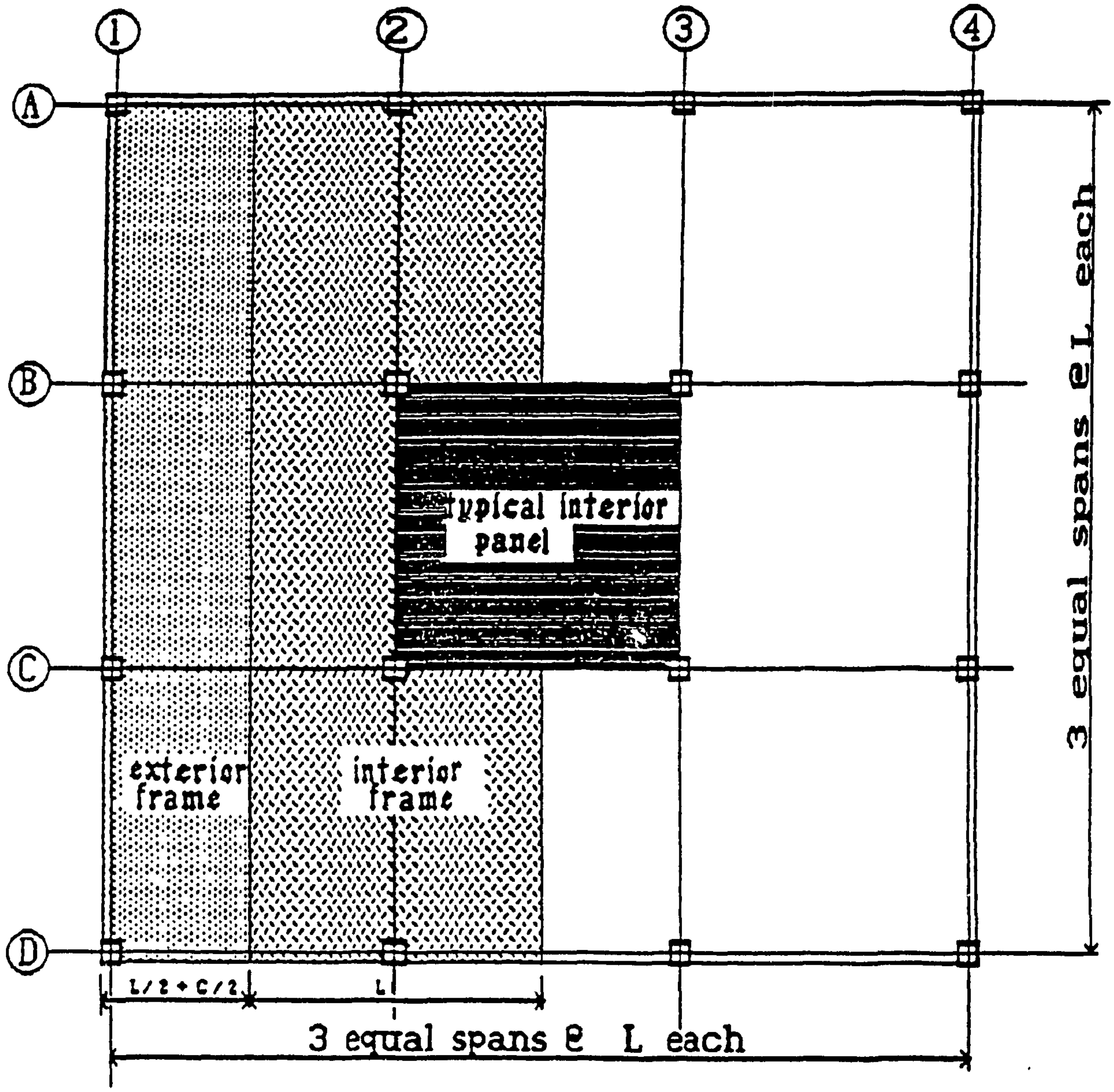


Fig 4.1 : Typical floor layout

The typical floor layout selected to investigate the floor system characteristics had a three bay by three bay configuration as shown in fig 4.1. The design was carried to a further detail by using the equivalent frame method. Results of this analysis also assisted in assessing the viability of the use of the Direct design method as an approximate analysis technique. The detail design would have been tedious without the aid of the Analysis and Design of Slab System (ADOSS) Computer Program(18). The following sections are a summary of the theory and data preparation pertaining to the design of the typical floor.

4.2 THEORY

All design studies, presented here, were in accordance with the canadian standard design code CAN3-A23.3-M84. The code presents a limit state design philosophy which aims at avoiding the attainment of limit states. The first category of limit states are the ultimate strength limit states.

Ultimate strength limit states (concerned with safety) are satified by designing for sufficient strength and stability. This is achieved through the use of load factors and resistance factors. The other category of limit states are the serviceability limit states generally relating to control of cracks and deflections. Servicebility limits are normally checked after designs for strength and stability have been made.

The Canadian standards design code CAN3-A23.2-M84, (referred here-in as 'the code') provides minimum slab thickness requirements for control of deflections in two way slab systems. These provisions are found on a statistical basis to be satisfactory for control of deflections. The code, however, permits the use of slab thicknesses less than the minimum recommended provided the computed deflections do not exceed certain limits given by ratios of the support span. To obtain comparable designs of the two way slab systems it was necessary to provide the same performance criteria in all cases. This was achieved by controlling deflections using the strict provisions of the code.

Cracks arising from volume changes due to shrinkage, thermal effects etc can be controlled by good construction techniques and provision of minimum reinforcement. The effect of flexural cracking of concrete is to induce increases in deflections which are already controlled by code provisions as stated above. A minimum area of reinforcement of 0.002Ag(Ag is gross cross-section area) was the only requirement in both directions for all slabs as recommended by code to account for shrinkage and thermal effects.

Part of the code conditions necessary for control of deflections based on minimum slab thicknesses is that slabs will be designed by the Direct design method or the equivalent frame method. However, even here the code is flexible in allowing for the designer to use any other procedure satisfying equilibrium and geometrical compatibility while ensuring safety and serviceability conditions are met. The direct design method is limited to particular loadings, supports and span conditions of two way slab systems. The Equivalent frame method uses an elastic analysis of equivalent plane frames and will be used in the detailed design.

4.3 DESIGN AID

The equivalent frame method is the basis for design of slab systems by a computer program distributed by the Canadian Portland Cement Association (CPCA). Several alternative slab systems were to be analyzed for this study. Apart from the inherent errors, performing designs by hand would have been extremely time consuming.

The CPCA program, known as ADOSS (Analysis & Design of Slab Systems), is applicable to two way slabs, in particular, flat plates, flat slabs, plates or slabs with beams, waffle slabs and continous beams. The slabs are those selected in this study and so ADOSS offered a common means of analysis for them all.

ADOSS analyses slabs frame by frame. The input to the program is for one frame composed of the slab-beam system at each storey with their columns considered fixed at their remote ends. Results of lateral load analyses on frame-bents (frame through full height of building) may be input to ADOSS. However, lateral loads were not considered in this study.

ADOSS also offers options to design according to the American Concrete Institute Building Code (ACI 318-83) or others the Canadian Code issues. The options include the choice of strength design, limit state design or alternate design philosophies. As pointed out earlier, the limit state design option of the Canadian Code was adopted.

The program however, does not check for beam shear in slabs or for torsion in spandrel beams, both of which must be checked by hand. Torsion was not considered but beam shear had to be checked by hand.

ADOSS also calculates quantities of concrete, steel and formwork although they are usually gross approximations. These approximations offered a check on results of hand calculations.

4.4 DESIGN PARAMETERS

Initially, gross sectional dimensions of system components must be assumed before they can be analyzed whether by computer or otherwise. Based on the data available preliminary sections were computed for input to ADOSS such that they met the basic criteria for performance of that type of slab. The data at hand was an imposed live load of 1.9kN/m^2 assumed for typical residential building based on the National Building Code of Canada. A super imposed dead load of 1.7kN/m^2 was also assumed composed of 0.5kN/m^2 for floor finishes and an average surface weight of 1.2kN/m^2 for all partitioning and curtain walling. The clear headroom from top of slab below and unobstructed by beam or solid head projections was set at 2.6m. Concrete was assumed to be normal density concrete with a 28 day compressive strength of 25MPA. The reinforcing bars were the most frequently used type of reinforcement, ie, Grade 400 (fy=400MPA) with the numbering and nominal dimension as set out in the appropriate Canadian standards.

To ensure uniformity in the design of all slab systems the preliminary member sizing followed a step by step procedure. The procedure is illustrated in flow chart form in fig 4.2. For proper understanding of the flow chart some points need to be noted.

In plates with beams systems, interior and exterior beams were to be of equal dimensions. This proposition was somewhat met when selecting the minimum slab thickness relations from deflection control equations. It was found that for minimum thickness to be limited by all panels (corner, edge and interior) the average ratio of flexural stiffnesses for all beams on the edges of panel were to be equal to 1.5. Consequently the ratio of flexural stiffness of an interior beam should be 1.5 while for an exterior

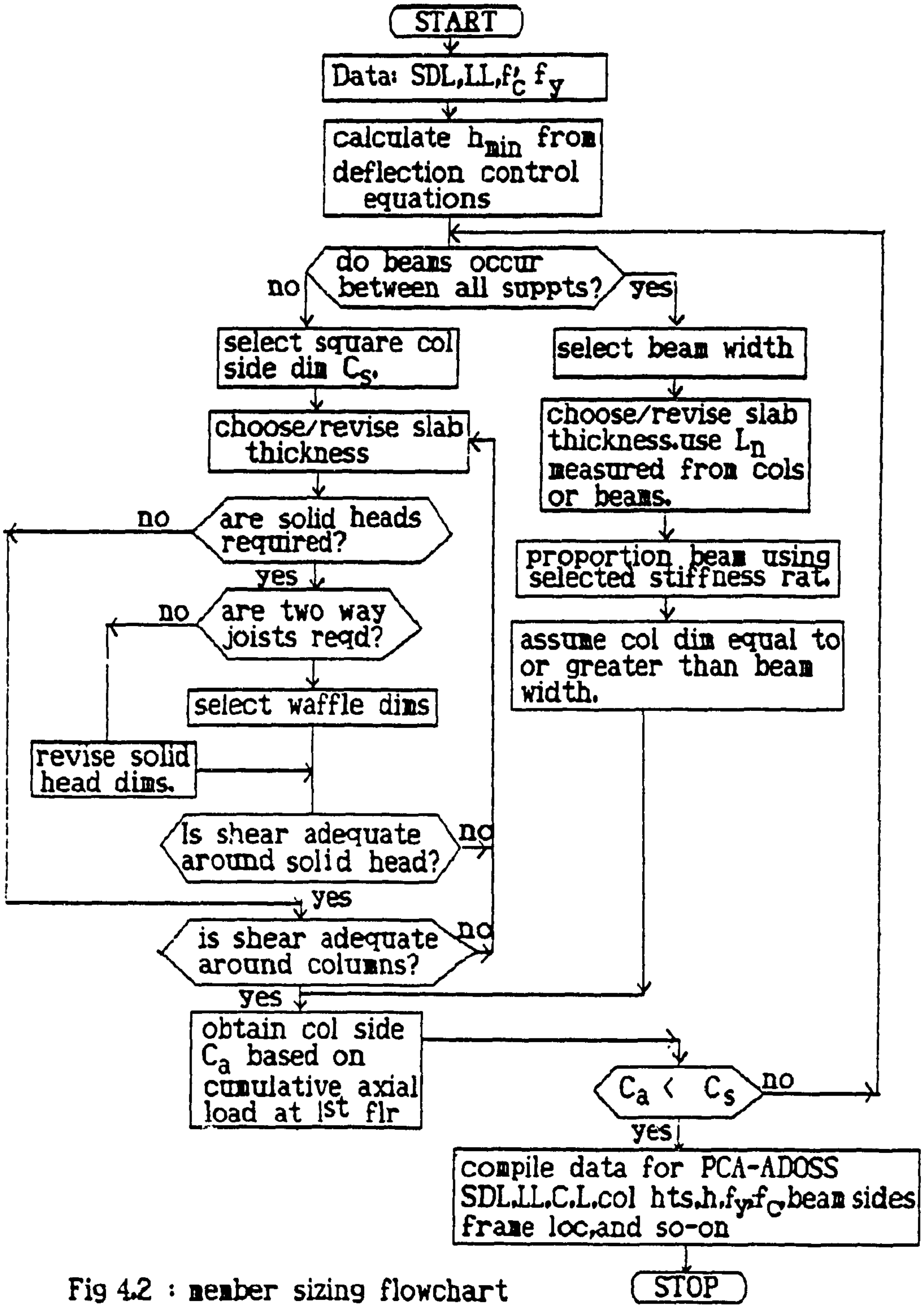


Fig 4.2 : member sizing flowchart

beam it should be 2.5. It turned out that for every interior beam selected such that beam-slab stiffness ratio is 1.5, the same proportion of beam used on an edge beam results in the required stiffness ratio of 2.5. Hence the use of equal beam dimensions was inherent in the preliminary sizing process. Note that in the code the clear span to be used in determining the minimum slab thickness is defined as the clear distance between supports. The designer may choose to measure distance between beams as opposed to columns. This freedom was excercised here especially in the first trial dimensions.

When checking the punching shear at distance of half effective depth from columns and solid heads , the shear resistance was to exceed estimated punching shear by a good margin. This is necessary to account for the additional shear due to moment transfer. The CPCA suggests that, to allow for moment transters, the shear resistance should exceed punching shear by 1.2 at interior columns, 1.6 at edge columns and 2.0 at corner columns. However, it is only applicable to flat plates without beams. Use of these recommended excess capacities in flat slabs and slabs without interior beams proved impractical. It was found necessary to use values slightly different but on the same basis as those suggested by the Canadian Portland Cement Association. These values are 1.2 at interior columns, 1.4 at edge columns and 1.6 at corner columns.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1. INTERIOR PANEL VS SLAB SYSTEM MATERIAL QUANTITIES

Floor formwork quantities are square meter averages of surfaces. Reinforcement steel quantities are inclusive of bar hooks and anchorage embedment. Concrete quantities include the column section within slab, beam or drop panel depth. Plots of these quantities against span ranging from 5 - 11m were made. The graphs obtained were smooth curves, proving that designs were trully uniform. One of the olots is depicted in fig 5.1 for reinforcement quantities representing a quarter of the floor area. Waffle slab reinforcement is minimum for the whole range of spans. The flat slab with standard drop panel dimensions provides lower reinforcements for spans upto about 8.1m where it exceeds that in the slab with beams. The flat plate steel requirements are just a little more than that in flat slabs upto a span of about 7m when it exceeds that in the slab with beams. Reinforcement quantities in flat plate with spandrel beam and flat slab with spandrel beam are higher than in the corresponding slab without spandrel at all spans. This can be attributed to the reduced slab thickness made possible by the presence of spandrel beams.

A similar plot of reinforcement quantities versus span is illustrated in fig 5.2 for interior panel only. At a glance there appears to be good correspondance between the

FIG 5.1: TYPICAL FLOOR REINFT VS SPAN

QUANTITIES FOR QUARTER OF FLOOR ONLY

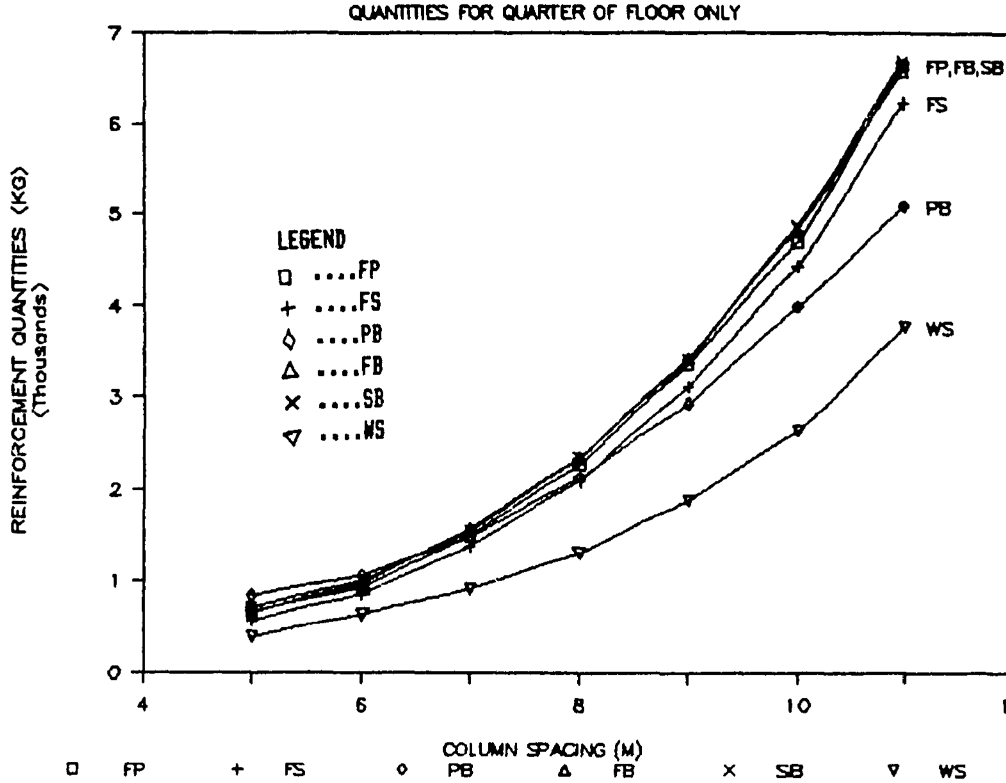
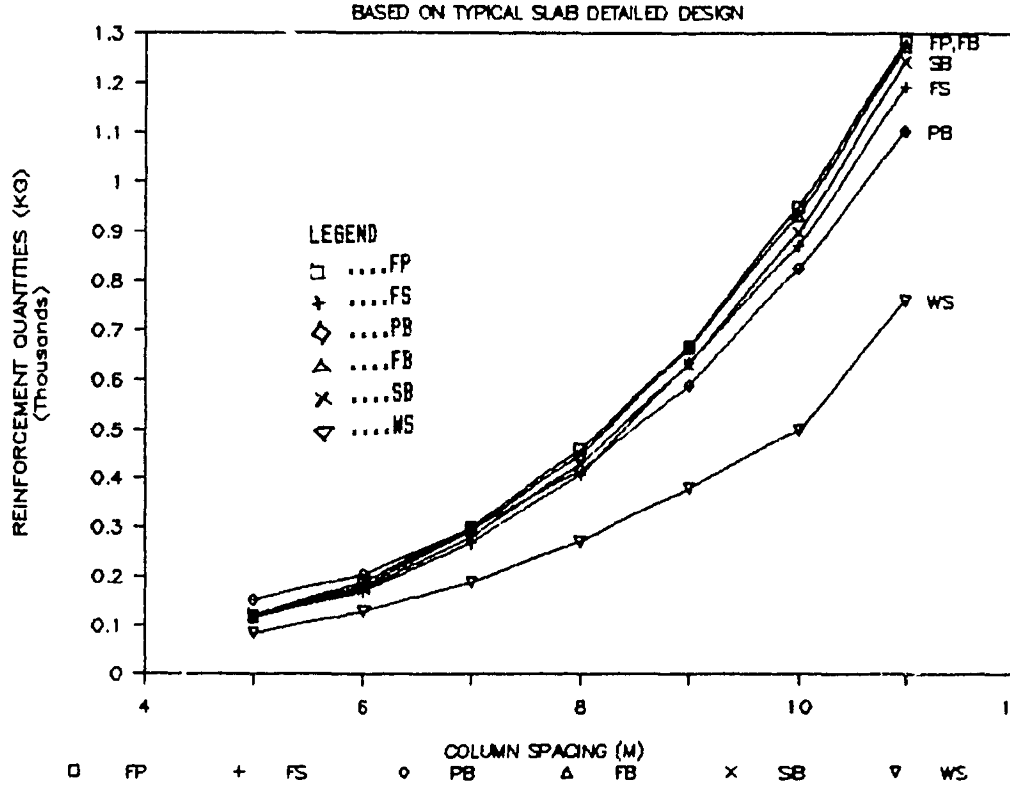


FIG 5.2: INT PAN REINFT VS SPAN

BASED ON TYPICAL SLAB DETAILED DESIGN



two sets of curves. A closer look, however, reveals some major differences. Specifically, the variation of waffle slab reinforcement quantity takes a sudden turn at the 10m span, deviating away from the trend established in the lower spans. Infact this phenomenon is also present in the total slab steel quantity variation but it was dampened by the larger quantities involved. To explain this occurance one needs only to understand how bottom reinforcement in the ribs is placed. Normally, two bars are always placed in this position. Up to the 10m span 2-No.10 bars were adequate. Beyond this span, the 10 dia bar was no longer adequate and the next bar size is 15 dia which explains the giant leap in quantity variations.

Other curves in the interior panel plot may not be as easily explained. Anyhow comparisons of typical floor and interior panel quantity plots were prepared for formwork and concrete. In both cases the interior panel variation of quantities matched those of the entire slab at least in as far as the relative curve positions were concerned.

Where there is good correlation between plots of quantities for entire slab and interior panel, the ratio of interior panel quantity to the corresponding quantity in the entire slab for any span should be a constant. Hence, plots of this ratio against span would be horizontal lines. There was good agreement to this effect for concrete and formwork quantities as may be seen from fig 5.3 and fig 5.4

FIG 5.3: INT PAN/SLAB CONCRETE VS SPAN

BASED ON TYPICAL SLAB DETAILED DESIGN

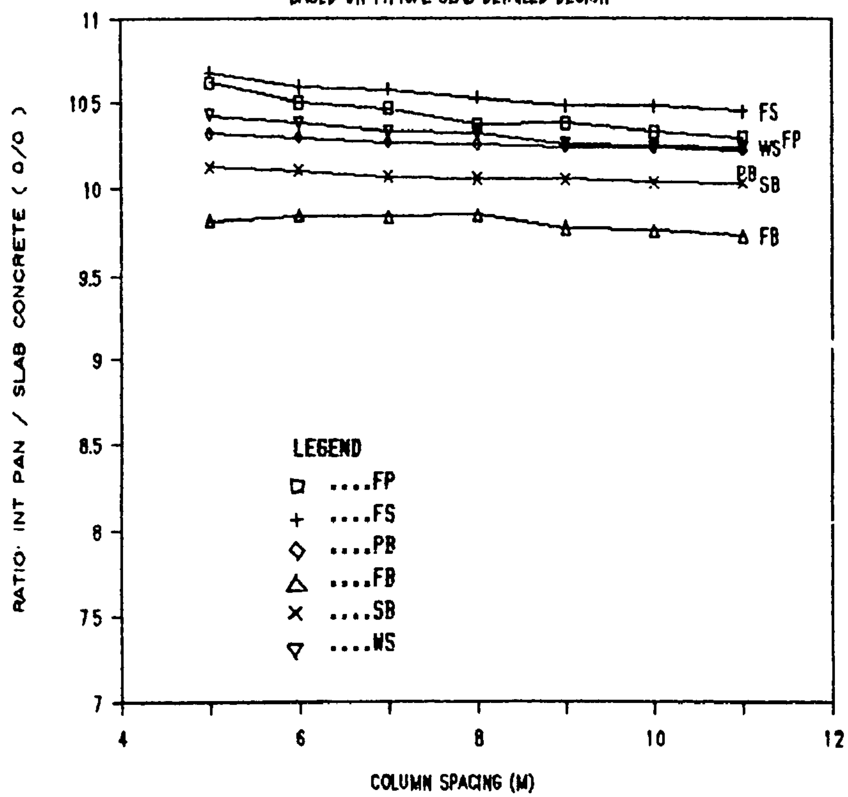
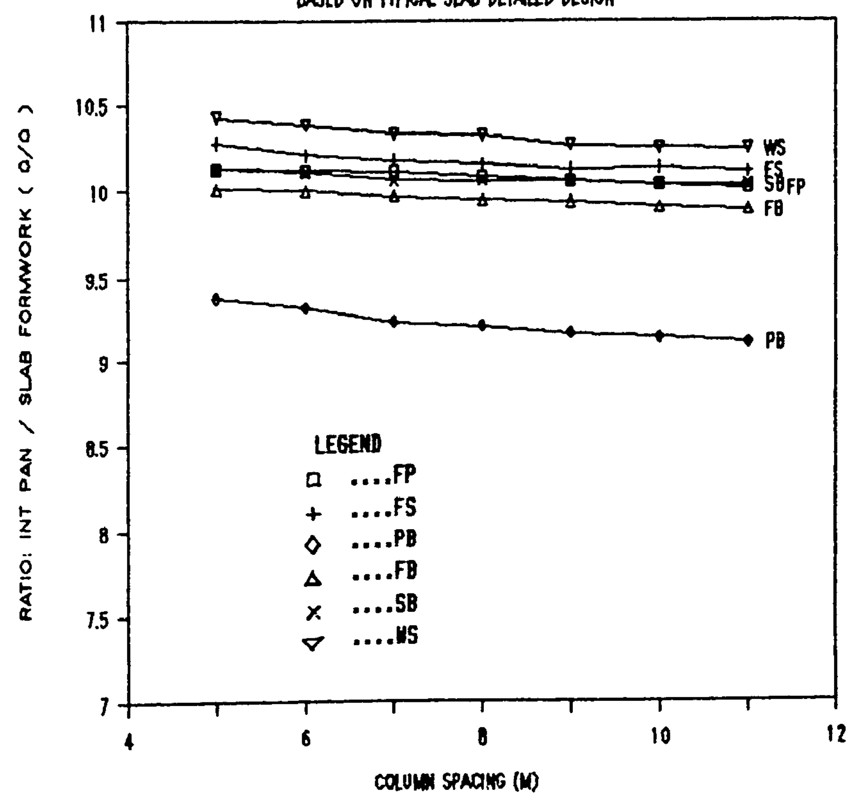


FIG 5.4: INT PAN/SLAB FORMWORK VS SPAN

BASED ON TYPICAL SLAB DETAILED DESIGN



respectively. Ratios are fairly constant albeit the slight decrease with increasing span. The slight decrease may be explained by considering a typical flat plate system.

Entire slab system concrete $=(3L + C)^2 h$

Interior panel concrete $= L^2 h$

$$\text{Ratio int pan/slab is } \frac{L^2}{(3L + C)^2} = \frac{1}{9+6C/L+(C/L)^2} \text{------(5-1)}$$

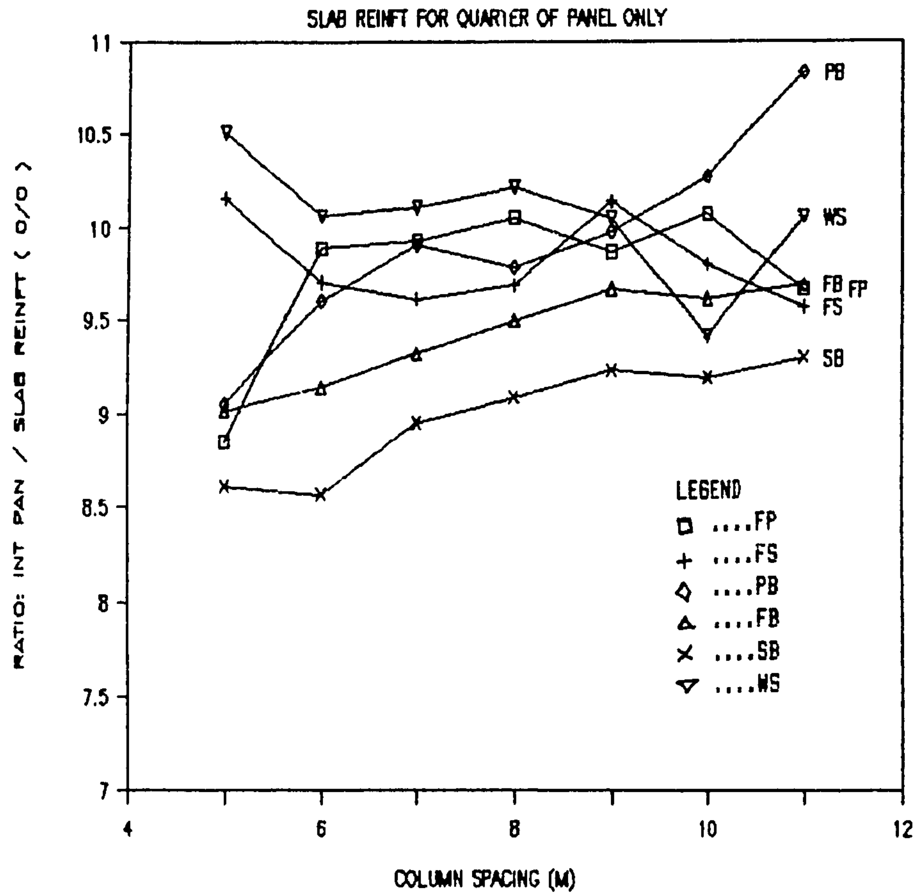
$$\text{Similarly formwork ratio } = \frac{1-(C/L)^2}{9.33+6.11C/L-8(C/L)^2} \text{----------(5-2)}$$

($h = L/36$ assumed)

Graphic representation of equations 5-1 and 5-2 for C/L varying from 0.08 to 0.15 (practical range suggested in american concrete institute design codes) produced nearly linear graphs with negative slopes. The ratio of concrete quantities varied from 0.106 down to 0.101 for C/L equal to 0.08 to 0.15 respectively. While the ratio of formwork quantities varied from 0.102 down to 0.0969 respectively. The decreasing ratio of interior panel to entire slab concrete and formwork quantities depicted in graphs fig 5.3 and fig 5.4 are therefore due to the increasing ratio of column side to column spacing dimensions. The greater the slope the more drastic the changes in C/L have been. In general the graphic representations show that interior panel formwork and concrete quantities may be utilized to compare the material consumption of slab systems to within about 1% of accuracy.

Reinforcement quantities for the typical floor, however, do not compare well with the corresponding interior panel reinforcement quantities. Apart from the flat plate with edge beam and flat slab with edge beam, graphs of the ratio of interior panel to entire slab steel quantities depict consistency only in the spans ranging from 6 to 10m. Graphic representations of interior panel to slab system steel quantities are presented in fig 5.5. The lack of linearity, though drastic in some cases, is confined. The maximum range of ratios occurs in the slab with beams where the percentage ratio varies from 9.05 to 10.83, a difference of 1.78. Whether the ratio should steadily increase or decrease or remain constant with increasing span poses a big theoretical puzzle. Many factors play a role in establishing steel quantities making a theoretical generalization quite difficult. Based on plots of the ratio of required volumes of steel in lieu of provided volumes of steel, the linear relationship was found to be quiet evident. The ratio of interior panel to whole slab steel should be decreasing with increasing span, however , in practice the trend is determined by how steel is provided. Conclusions can be drawn from the graph Fig 5.5 by assuming that it represents the practical range of ratios. That is, interior panel reinforcement quantities may be utilized to compare the material consumption of slab systems to within about 4% of accuracy.

FIG 5.5: INT PAN/SLAB REINFT VS SPAN



5.2 COST SENSITIVITY OF INTERIOR PANELS

5.2.1 VARIATION OF TOTAL COSTS

Little can be envisaged from a comparison of actual costs of various designs at respective cost levels. However, for the purpose of providing a dimensional perspective, the iterated costs of slab systems are presented in fig 5.6. The costs are at the current material cost levels or case 1 in table C.1 of appendix C. The first notable fact is that the comparative costs of the slab systems may be derived from the variation of material quantities. Fig 5.6 resembles fig 5.2 in a remarkable way inspite of the different items they represent and the different design approaches used. The flat plate system is clearly too costly for large spans, as expected, while at spans lower than 7m it approaches the absolute minimum cost. The waffle slab system may be costly in lower spans but with increased span the associated cost almost starts to drop. Costs of slab with beams systems generally lie between that of flat plates on one extreme and the waffle slabs on the other, with more favorable costs at higher beam to slab stiffness ratio. Suprisingly, the flat slab was found to be of high cost for spans upto, and including, the 10m span. This probably, just shows that flat slabs are not very economical for residential loadings.

Considering typical residential spans of 5 to 8m the slab with beams of high stiffness will be economically

FIG 5.6: TOTAL ITERATED COST VS SPAN

AT CURRENT RATES

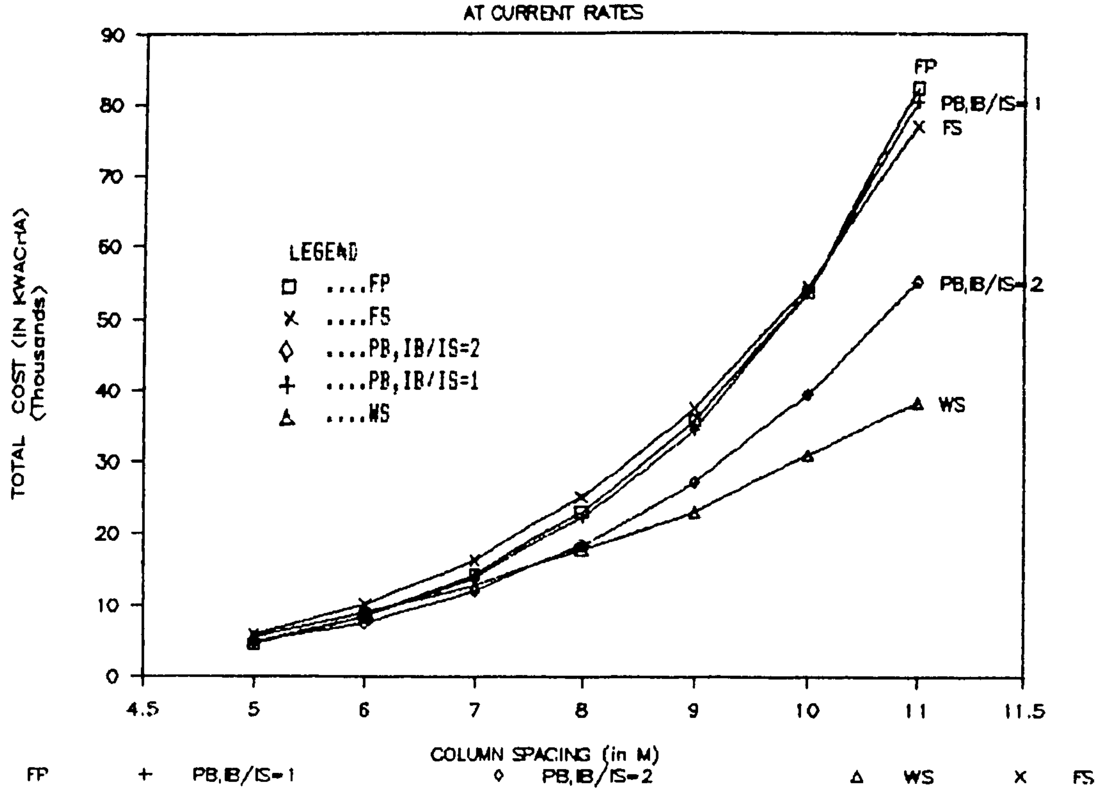
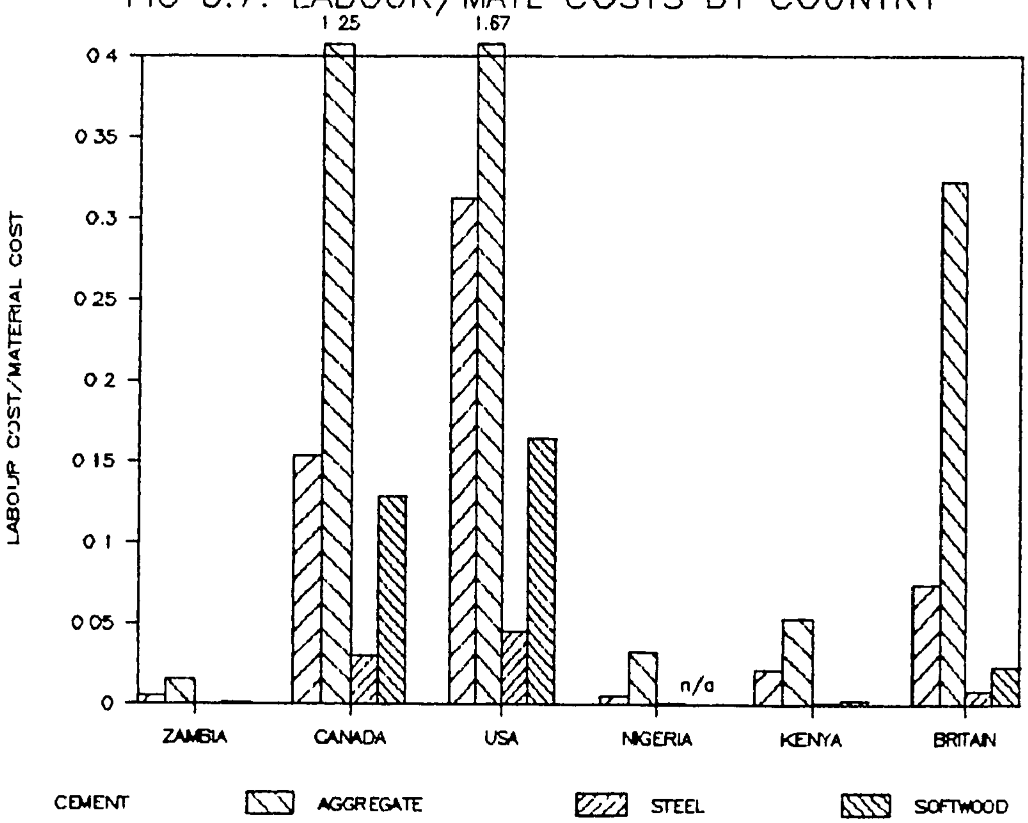


FIG 5.7: LABOUR/MATL COSTS BY COUNTRY



competitive at current rates. This is in contrast to the flat plate system, which is considered to be most economical, under these loading and span conditions, in Europe and America, according to references(1) and (19) respectively. The economy of the flat plate system is attributed to the reduced labour cost. The slab with beams economy is due to steel economy although other factors must take effect, such as reduced concrete quantities . Most authors suggest that the real cost parameter in choosing the type of slab is the ratio of labour to material cost. Fig 5.7, above, illustrates this point based on unskilled labour costs as ratios of costs of unit quantities of materials all using rates from reference(7). Labour costs, in relation to material costs are considerably higher in Britain, Canada and the United States of America. Accordingly, the flat plate system is recommendable. Although labour costs are low in Zambia and the other countries, material cost differentials, alone, may show that the flat plate system is just as economical.

5.2.2 EFFECTS OF CONSTRUCTION ITEM COSTS

Rapid changes in costs of materials are a common phenomena in Zambia. One case is cited in table 2.4 where the steel reinforcement and placement costs increased by more than 200 percent over a two year period. The frequent shortages of materials and subsequent instability in material prices has created even more problems.

Construction costs have increased tremendously on large projects of most developing economies to incorporate uncertainties in pricing. Ofori (20), in a 1984 article, expressed great concern at the depreciating construction activities in Africa. He noted that the major causative factors were the shortages of materials and lack of a skilled workforce. Sometimes, material costs have decreased, either, because the governments exercised control of prices, or, materials are imported and have stockpiled as a result of changed demands. The unpredict ble nature of costs of materials could lead to complete reversals in the comparative economics of two way slab systems. Unless there are adequate resources for comparative cost analyses to be made in every preliminary design, the relation between material costs and comparative economics should be examined to establish trends.

Cost increases of 100 and 300 percent were considered in this study. The cost increases were applied, first to each of the four construction items, then secondly, to combinations of these. Each cost increase ratio, corresponding to a particular material, or item, was denoted by the symbols, CF, CC, CS and CL for formwork, concrete, steel and labour cost factors respectively. A 100 percent increase in formwork costs would correspond to a CF value of 2 being multiplied to the current cost of formwork materials.

The relation between increase in cost of a material and the total cost of a slab sytem would be linear only if cost parameters have not been optimized to minimize the effects of the increase. Inspite of the optimization concept utilized in this study, the relationship was linear. Fig. 5.8 and 5.9 illustrate the perfectly linear relationship between cost of slab (plotted as ratio of new cost/current cost) and increase in cost of one or two materials(cost increase factor plotted). Graphs prepared for other cost increases were also linear but with considerably different slopes. Significant differences in slopes were also apparent at different column spacings. Perhaps the most unique representation of the cost sensitivity was the relation between cost of slab (shown as ratio of new cost/current cost) and span. Fig 5.10 to Fig 5.13 are a sample of such graphs for single item cost increases of 100 percent. In all slab systems, the impact of an increase in the cost of concrete and labour reduced with span. The effect of increased steel costs reduced with span in the case of flat plates but increased for all other slab types. The effect of an increase in formwork costs was considerably higher than for other items. Naturally, the effect of a formwork cost increase varies linearly with span in flat plates, reduces then increases for flat slabs and slabs with beams, and generally reduces in waffle slabs.

FIG 5.8: COST VS FORMWORK COST FACTOR

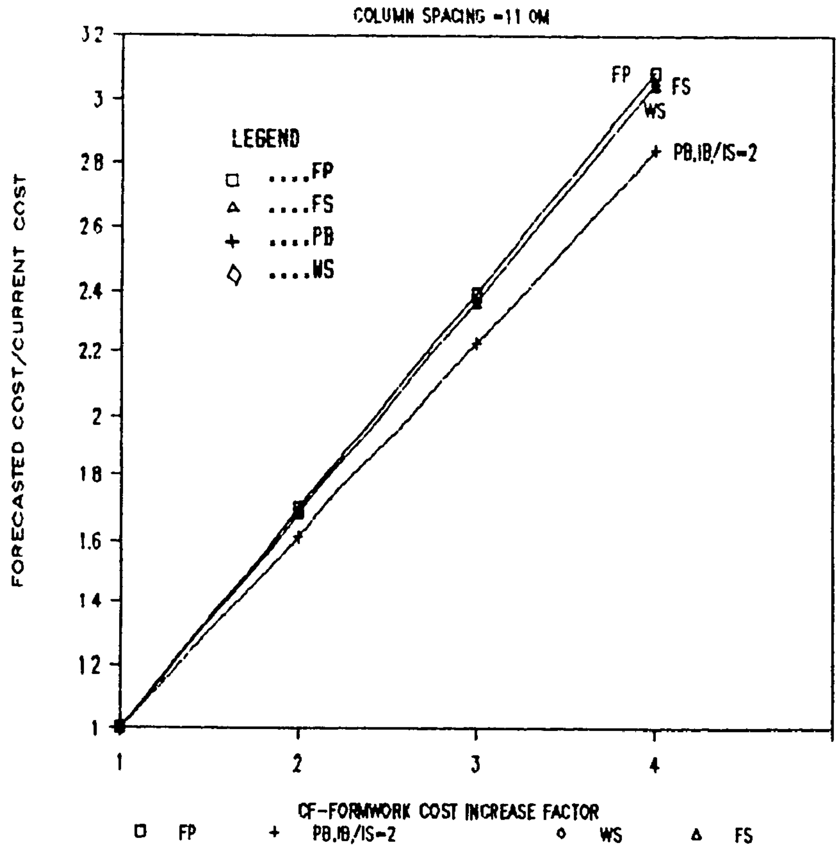


FIG 5.8: COST VS STEEL COST FACTOR

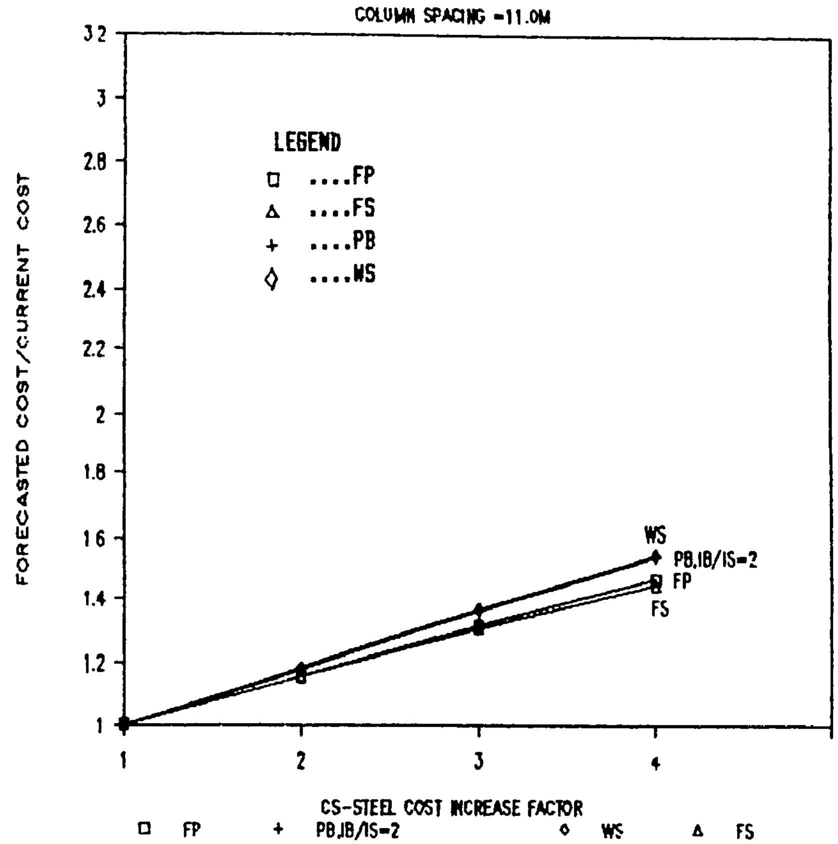


FIG 5.10: COST SENSITIVITY-FLAT PLATE

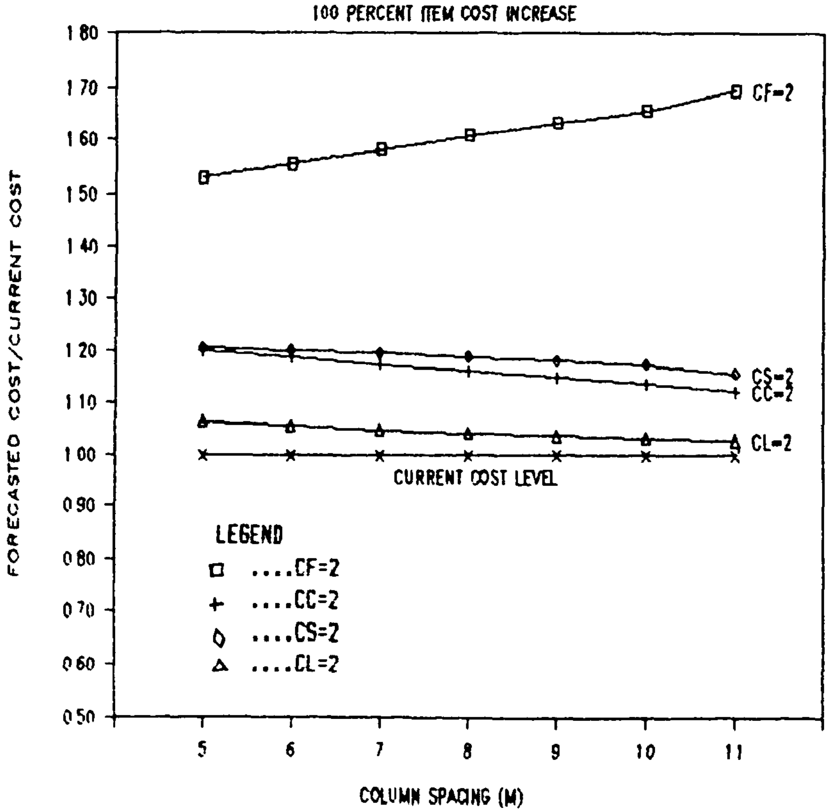


FIG 5.11: COST SENSITIVITY- SLAB-BEAM

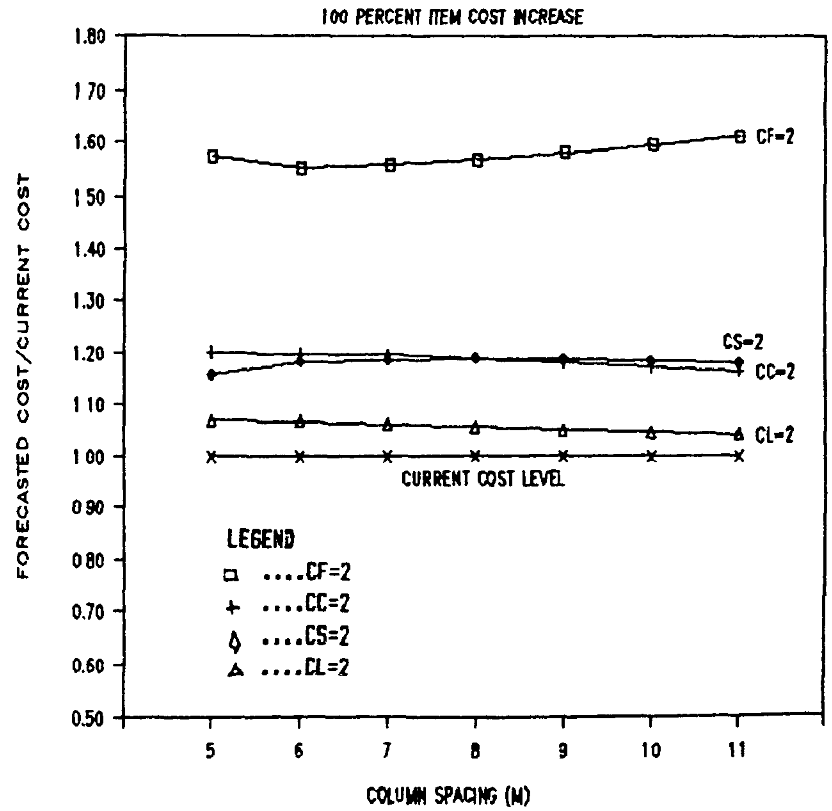


FIG 5.12: COST SENSITIVITY-FLAT SLAB

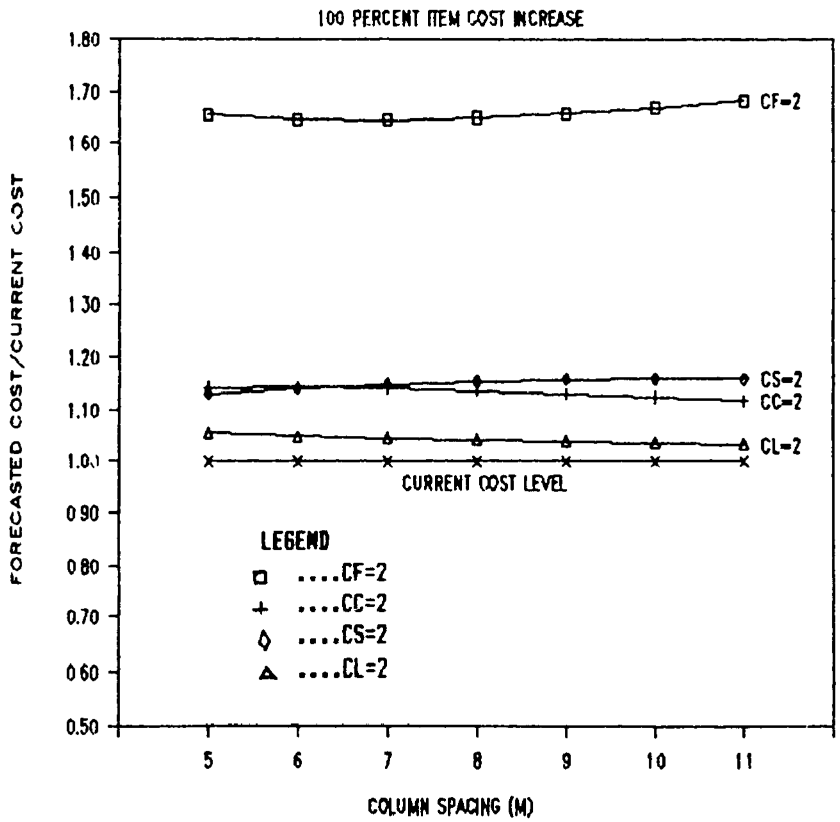
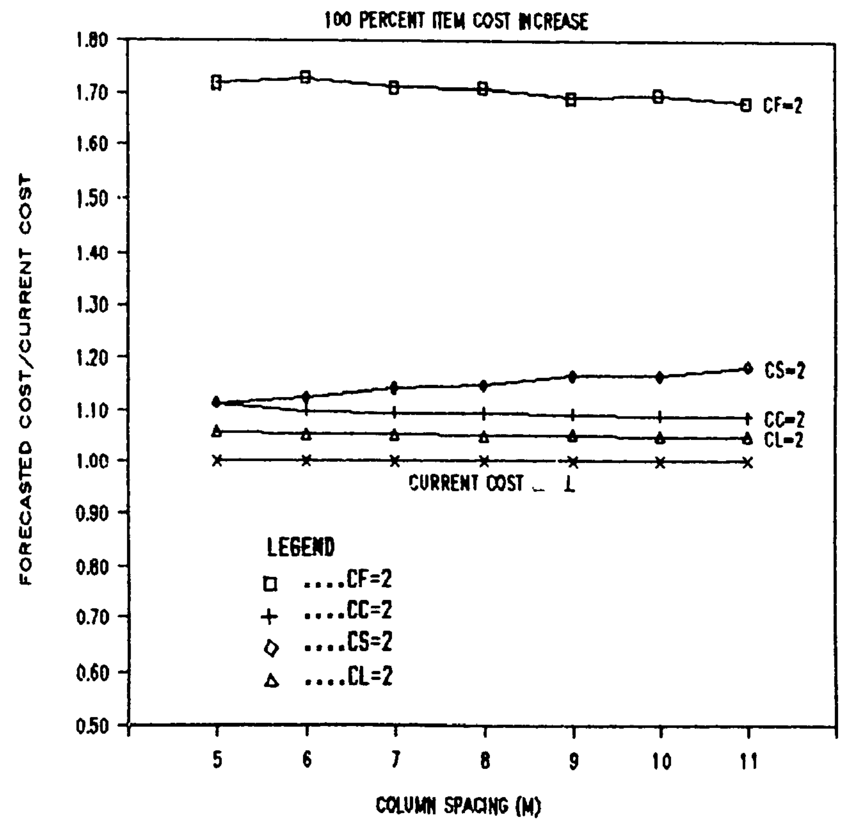


TABLE 5.13: COST SENSITIVITY-WAFFLE SLB



Little difference can be expected in the curvature of these graphs when higher cost increases are considered since the material cost-total cost relationship is linear.

The cost competetiveness of the slab systems at current cost was discussed in section 5.2.1 and illustrated through figure 5.6. The effect of changes on cost of construction items with respect to cost competetiveness of the slab systems can be illustrated by plotting the ratio, calculated cost/selected standard cost, against span on the X-axis. For cost increases of 300 percent, such plots were made and are given in appendix D. These plots clearly show that, at current cost, the flat plate system is economical up to 5.5m span. However, considering the 4 percent tolerance, the slab with beam system is more economical for typical residential spans of 5-7.5m. The slab-beam system retains this economic advantage at increased formwork, labour, or reinforcement costs. Higher steel costs consolidated this advantage at all spans less than 6.5m.

Increases in concrete costs make, the waffle slab system more attractive. For the 300 percent increase in concrete costs it was the most economical system in the entire range of spans, 5-11m.

The same set of plots show that increases in formwork and labour costs improve the competetiveness of the flat plate system at lower spans. It would appear, though , that extremely high increases in labour and/or formwork costs would have to occur before the flat plate system can receive equal acceptability as an economical alternative in Zambia as is the case in Europe and America. This confirms the suggestion by some authors (see section 5.2.1) that the ratio of labour cost to cost of material is the real cost parameter for selecting the slab type.

At current rates, formwork accounts for between 50 and 70 percent of total cost of slab, notable from figures 5.10 to 5.13. The actual percentage in practice may be much less because of re-use factors, and additional construction costs not considered in this study. Labour costs are, by far, the least cost construction item. They account for only about 5 percent of the total construction cost. Steel and concrete costs share nearly equal proportions of the total, cost, with each one averaging 15 to 20 percent. Considering current costs and the fact that the flat plate system is economical at the 5m span, a general conclusion may be drawn. Labour costs exceed 6 percent at this span. Most probably the flat plate system will be economical when this occurs. The influence of the material cost of formwork must also be incorporated in this reasoning but this cannot easily be achieved.

5.2.3 VARIATION OF COST PARAMETERS

There was little or no variation in the section parameters of the slab systems on account of construction cost increases. This implies that cost variations of upto

300 percent in materials or labour do not necessitate a reassessment of the comparative economics of two way slab systems in Zambia. In the slab-beam system the same slab thickness and beam section sufficed at all cost increases whether it be in formwork, steel, concrete, or labour. The minimum cost section in all cases was for a beam to slab stiffness of 2. The beam section being of equal web width and overall depth dimensions.

The plate thickness is the only cost variable in flat plates. This too was not optimized. Similarly, in waffle slab system, designs were satisfied by requirements of flexure and shear. For the waffle slab system, the search for an optimum section could only proceed in finite steps. As a result one minimum cost solution suited as a solution for a wide range of cost levels.

Some iteration did occur in flat slabs but in the drop panel only. It was found that when the cost of steel increased the depth of the drop increased. An increase in the cost of formwork registered a dramatic decrease in the drop panel depth. Increases in the cost of concrete and labour did not have much effect. Increases in drop panel depth due to increases in cost of steel minimized the effect of this increase by increasing the moment arm for column strip negative reinforcements. Similarly, since drop panel formwork costs are quite high, an increase in cost of forms can only be suppressed by a decrease in drop panel depth. Apart from these variations the other parameters(slab thickness) remained constant.

CONCLUSIONS AND RECOMMENDATIONS

If it is established that total construction costs constitute 5-10 percent in labour costs, 15-20% in concrete and steel cost and 50 - 70% in forming costs then the following conclusions can be drawn:-

1) The slab-beam is the most economical floor system for residential or light office construction within the normal span range.
2) The waffle slab system is suitable for all spans beyond the normal span range for residential construction
3) There is no significant difference in the cost competetiveness of the slab systems considered even if construction item costs quadruple in relation to the current situation.
4) Cost optimization of designs does not produce new designs of slab systems considered even when construction item costs quadruple in relation to the current situation.

These conclusions are subject to the assumptions that:-

1) Canadian standards design codes are applicable.
2) The effect of column costs is constant for all slab systems.
3) Loading conditions are as specified in this report.

While cost optimizations did not achieve the desired results, some general recommendations for design of slab systems may be stated:

1) Use of wide shallow beams as edge beams or for all beams in slab-beam is greatly encouraged.

2) Where flat slab systems are used, the depth of the drop panel should be investigated for economy in steel or formwork.

The cost optimization method used in this study leaves much to be desired in relation to the objective of this study. Further research should be directed toward incorporating standard structural optimization techniques in the optimization of two way slab systems. Such a study should also determine the influence of building height, foundation system, and wind resistance.

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A P P E N D I X   A

APPENDIX A

TABLE A.1: CONCRETING LABOUR COSTS

NO.	JOB DESCRIPTION	CLASS OF LABOUR	RATE (in K/hr)
MIXING PLANT LABOUR			
1	wheel cement	unskilled labour	1.98
2	wheel sand	unskilled labour	1.98
4	wheel stone	unskilled labour	1.98
1	mixer driver	unskilled labour	1.98
1	charge hand	skilled labour	3.08
HOISTING PLANT LABOUR			
1	crane operator	semi-skilled	2.08
1	crane oiler	unskilled labour	1.98
1	oversee filling buckets	unskilled labour	1.98
2	receive and dump buckets with one acting as signalman	unskilled labour	1.98
PLACING CONCRETE			
1	vibrate concrete	unskilled labour	1.98
1	spread/vibrate (take turns)	unskilled labour	1.98
3	spreading only	unskilled labour	1.98
2	spreading/levelling	unskilled labour	1.98
1	charge hand	skilled labour	3.08

Summary: 19 labourers

1 semi-skilled worker

2 charge hands

Composite labour rate for slabs/beams = 2.08/hr

Concreting cost rate for slabs = 15.475(1+0.05(H-3)/3)/m3

<u>Concreting cost rate for beams = 18.444(1+0.05(H-3)/3)/m3</u>

TABLE A.2 : FORMWORK ERECTION COSTS

LABOUR HOURS

ITEM per m2	CARPENTER hours	LABOURER hours	TOTAL hours	RE-USE MULTIPLIER (R is Re-use No.)
Beams	1.25	1.25	2.5	0.85-0.025(R-1)
Slab	0.95	0.95	1.9	0.80-0.035(R-1)
Col drop heads	1.6	1.6	3.2	0.80-0.035(R-1)
Plastic moulds	0.2	0.2	0.4	0.8-0.035(R-1)

FORMWORK LABOUR COST/M2

COMPOSITE RATE = (3$U+3$K+$F)/7 = 4.13/HR

ITEM	COST
Beams	(7.5$U +7.5$K +2.5$F)* FB = 10.325FB
Slab	1.9(3$U + 3$K +$F)* FS = 7.847FS
Col drop heads	3.2(3$U +3$K + $F)* FD = 13.22FD
Plastic moulds	0.4(3$U + 3$K + $F)* FM -1.65FM

Where FB.... is formwork to beams/m2

FS.... is formwork to slab/m2

FD.... is formwork to drop heads/m2

FM.... is plastic moulds /m2

A P P E N D I X B

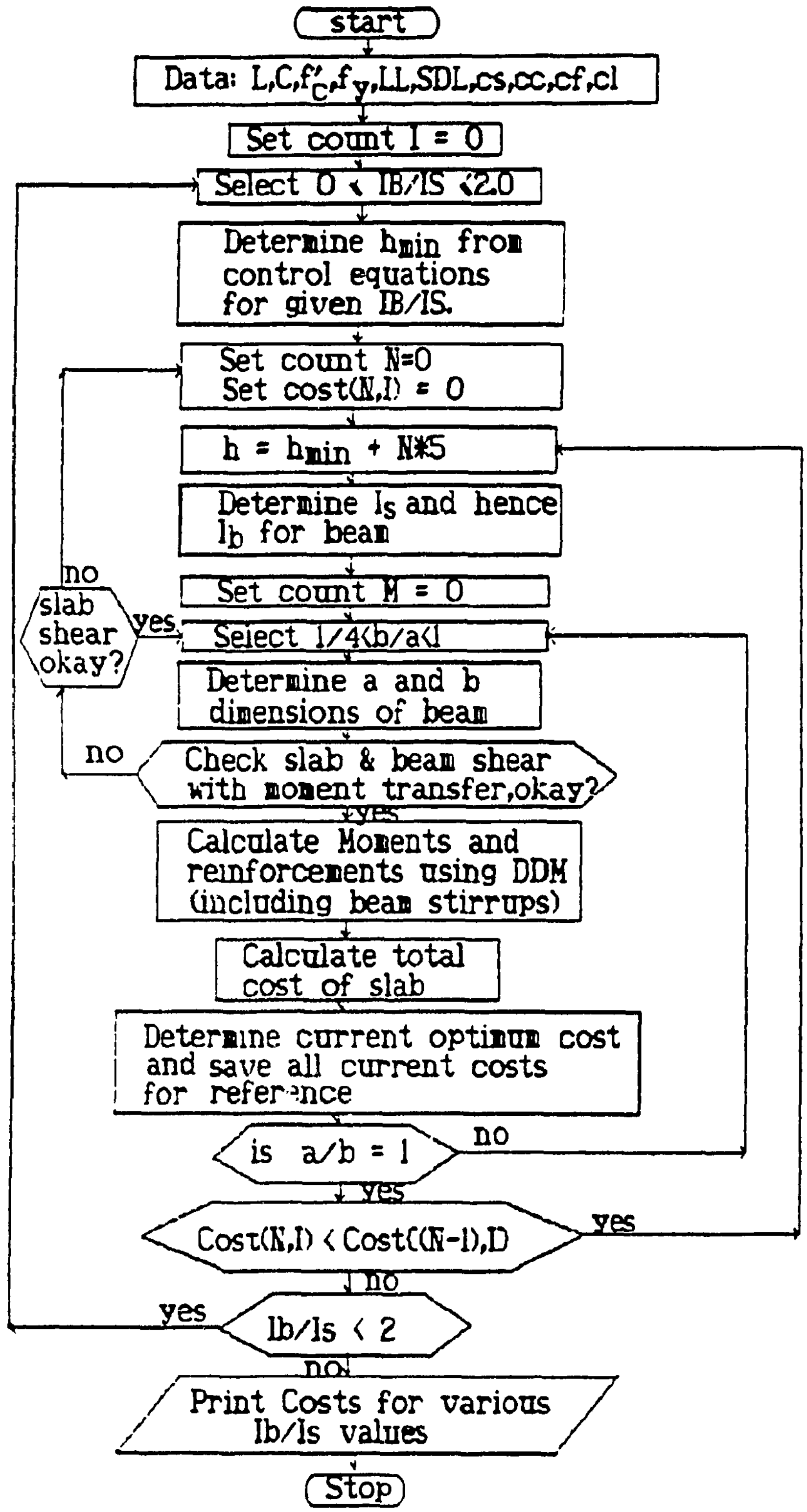

FIG B.1 : Sequence for optimization of flate plate & slab-beam

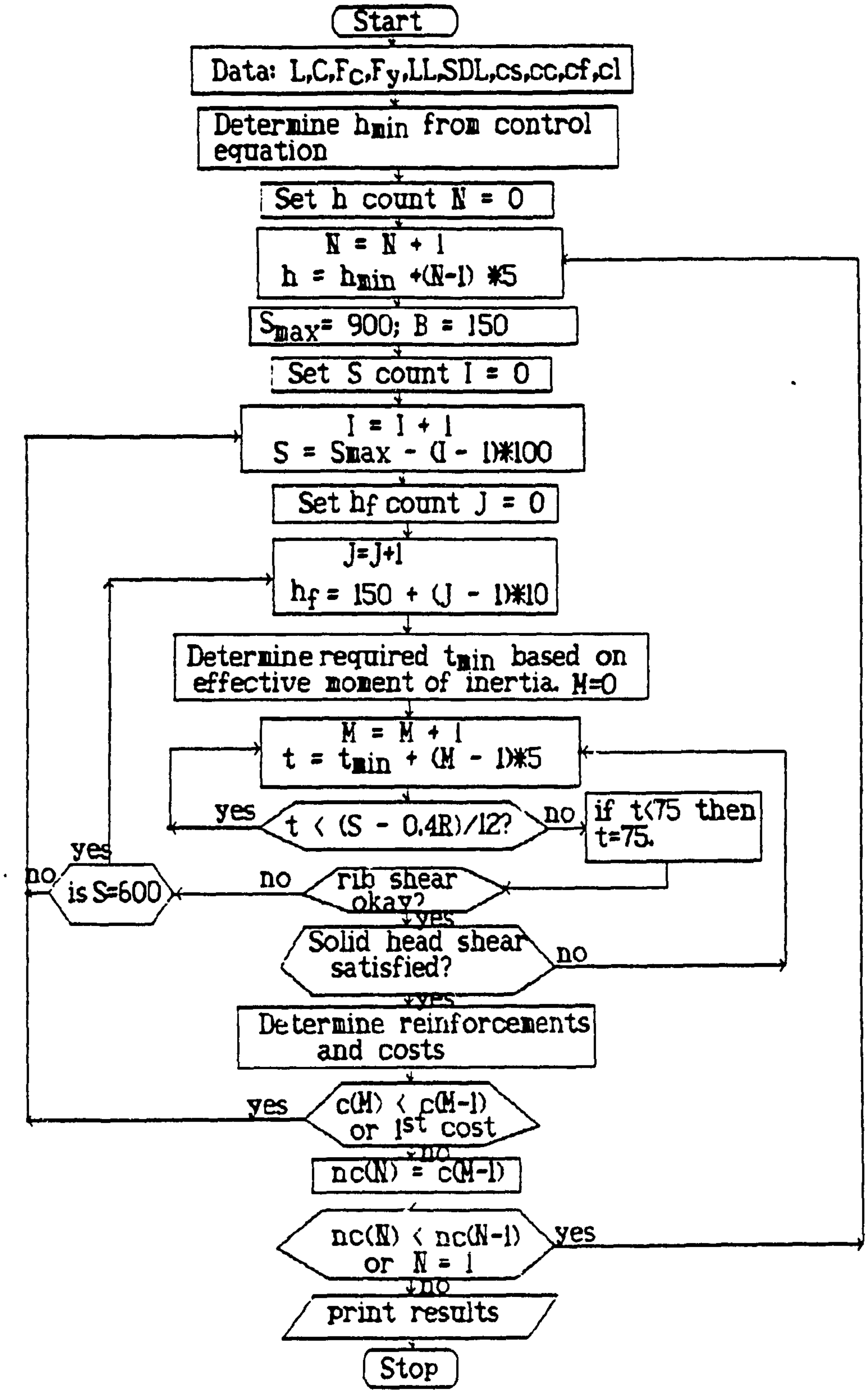

Fig B.2: Sequence for optimization of waffle slabs

APPENDIX C

APPENDIX C

TABLE C. 1 : COST INCREASE FACTORS

CASE NO.	CF	CC	CS	CI
1	1	1	1	1
2	2	1	1	1
3	1	2	1	1
4	1	1	2	1
5	1	1	1	2
6	4	1	1	1
7	1	4	1	1
8	1	1	4	1
9	1	1	1	4
10	2	2	1	1
11	2	1	1	2
12	1	1	2	2
13	1	2	2	1
14	2	1	2	1
15	1	2	1	2
16	4	4	1	1
17	4	1	1	4
18	1	1	4	4
19	1	4	4	1
20	4	1	4	1
21	1	4	1	4

CL = LABOUR COST FACTOR　　CC = CONCRETE COST FACTOR

CF = FORMWORK COST FACTOR　　CS = STEEL COST FACTOR

APPENDIX D

FIG D.1 : RELATIVE COSTS–CURRENT RATES

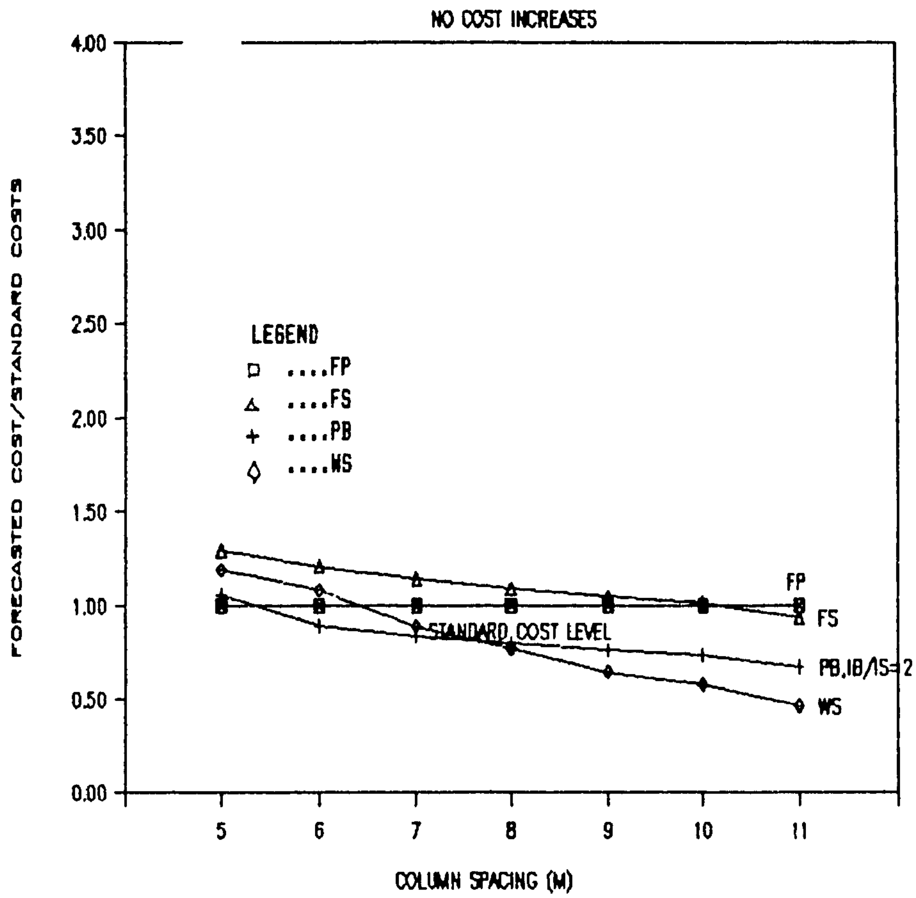

FIG D.2 : RELATIVE COSTS-CONCRETE CC=4

300 PERCENT CONCRETE COST INCREASE

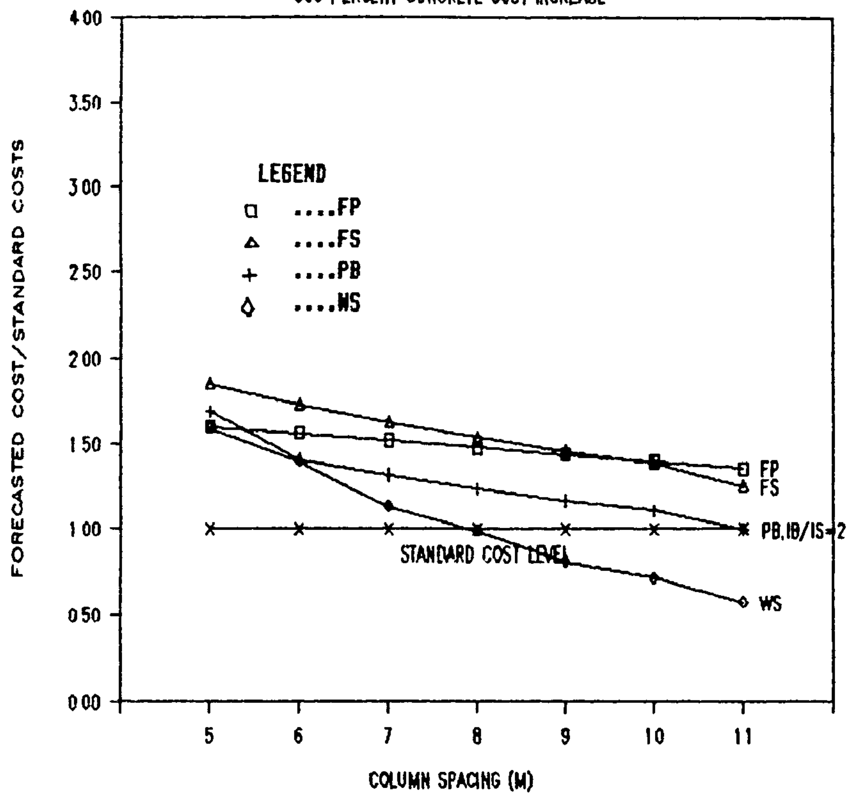

FIG D.3 : RELATIVE COSTS-FORMWORK CF=4

300 PERCENT FORMWORK COST INCREASES

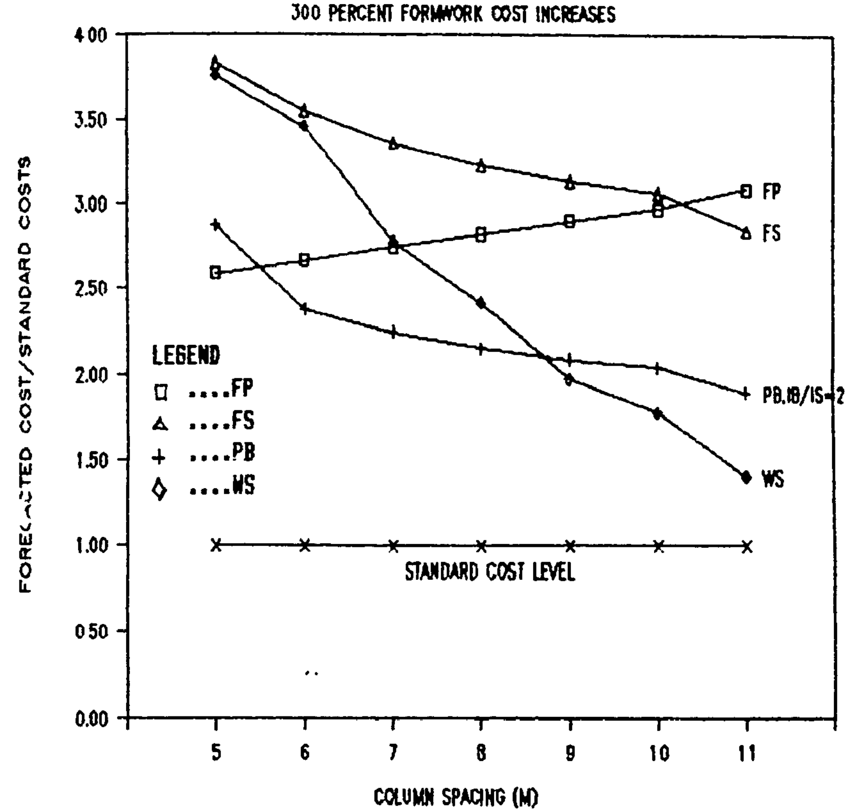

FIG D.4 : RELATIVE COSTS–LABOUR CL=4

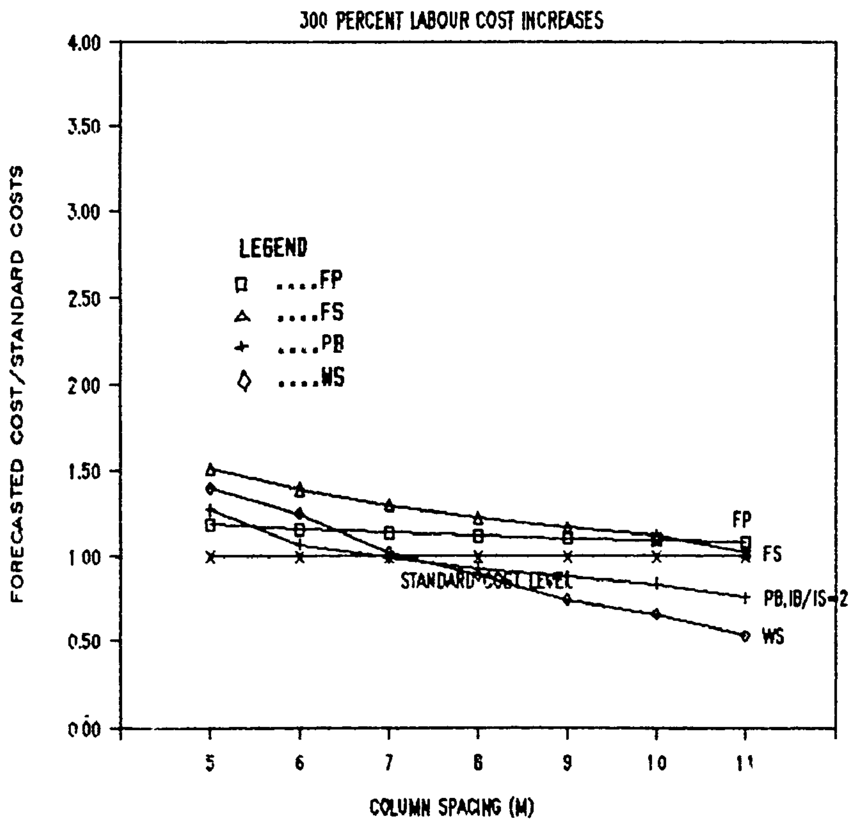

FIG D.5 : RELATIVE COSTS–STEEL CS=4