THE STUDY ON THE STANDARDIZATION OF BRIDGE DESIGN IN MALAYSIA FINAL REPORT VOLUME III DESIGN MANUAL AUGUST 1996

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JAPAN INTERNATIONAL COOPERATION AGENCY
PUBLIC WORKS DEPARTMENT
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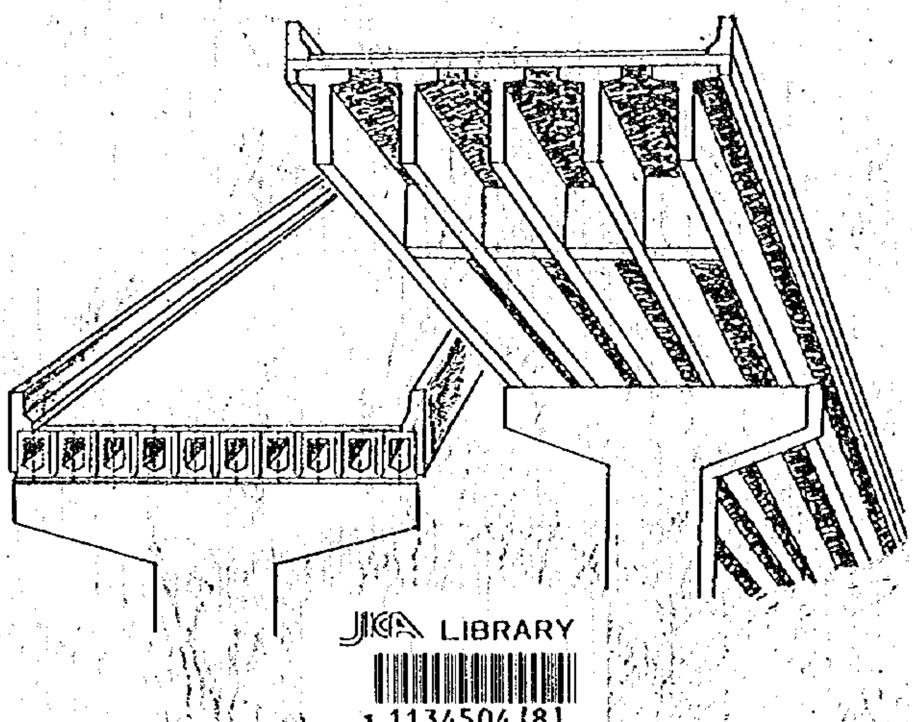
FINAL REPORT

VOLUME III

DESIGN MANUAL



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JAPAN BRIDGE & STRUCTURE INSTITUTE, INC., TOKYO
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INTRODUCTION

The design manual was prepared for an aim of technology transfer to the JKR engineers who are to undertake design of the standard bridges that were established in the Study on the Standardization of Bridge Design in Malaysia implemented by JICA (Japan International Cooperation Agency).

The manual consists of three major subjects, namely, bridge planning, bridge design and bridge construction and cost estimates, and it also includes two operational manuals for computer aided design and drawing programme.

In Division I : Manual for Bridge Planning, although it is difficult to systematize all of the bridge planning procedures, it will provide mainly with the procedures for structural side of bridge planning for the engineers who are not always well experienced. Particularly, the provisions for the waterway crossing requirements, which is the extraction of the Japan's river management ordinance, will be a good help to understand how to take hydrological condition into the bridge planning to cross rivers.

In Division II : Manual for Bridge Design Analysis, it describes the basic concept of the bridge structural analysis and the procedures of design program regarding the standard bridge design produced by the Study. One of the most important objective is to allow JKR engineers to carry out designs by themselves after the Study.

In Division III : Manual for Bridge Construction Plan and Cost Estimates, a primary objective is to assist the JKR engineers in understanding the construction plan, method and estimating procedure of the standard bridge construction project. The construction plan presents mainly current practices and problems on construction method and work approach of bridge construction, and it will relate to the cost estimate. The cost estimate presents fundamental method and procedure which are broadly applicable.

In Division IV : Operational Manual for Computer-Aided Design Programme and Division V : Operational Manual for Computer-Aided Drawing Programme, both explain on respective data files, input operation and basics for key operation for input. They also present the sequence of operation displays and the list of input data for design programme and drawing programme respectively.

Generally, the manuals cover all fundamentals needed for the JKR engineers to carry out the design work from planning stage up to cost estimate and also they include practical operational guidelines for computer aided design and drawing programmes.

Each of the divisions can be used independently of the other but is actually inter-related with each other in the process of design work.

It is hoped that the manuals are useful for the JKR engineers not only to carry out design work, but also to improve the knowledge of bridge engineering.

DIVISION I

MANUAL FOR BRIDGE PLANNING

DIVISION I MANUAL FOR BRIDGE PLANNING

CONTENTS

DIVISION I MANUAL FOR BRIDGE PLANNING

CHAPTER 1 GENERAL

1.1 Introduction

"Bridge Planning" is rather a composite art to involve not only bridge design but also traffic, highway, waterway and other related knowledges. In broad sense, it even includes administration duties such as financing measures, land acquisition, consultations with other authorities for re-location of public utilities, etc. to assure its implementation. Thus, the job of bridge planning, for it requires a wide territory of engineerings, cannot usually be carried out but by the experienced engineers who are of course bridge specialists and further must have other related profound knowledges.

Although it is difficult to systemize all of the bridge planning procedures, this manual will provide mainly with the procedures for the structural side of bridge planning for the engineers who are not always well experienced. Particularly, the provisions for the waterway crossing requirements given in chapter 3.3.2, which is the extraction of the Japan's river act, will be a good help to understand how to take hydrological condition into the bridge planning to cross rivers. However, the manual does not detail about traffic demand and highway studies nor administration matters.

1.2 General Bridge Structure

The general features and terms of the bridge structure explained in the manual are illustrated as follows:

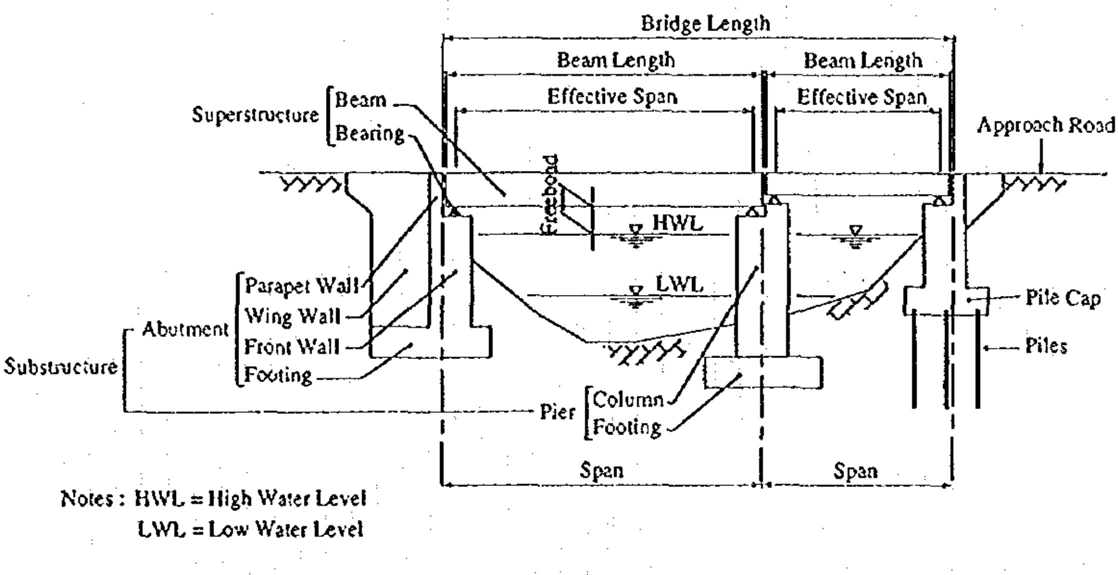


Fig. 1.1 General Bridge Structure

1.3 Bridge Life Cycle

In general, the life of bridge is considered to take a management cycle namely planned, designed, built and used as shown in the following flowchart.

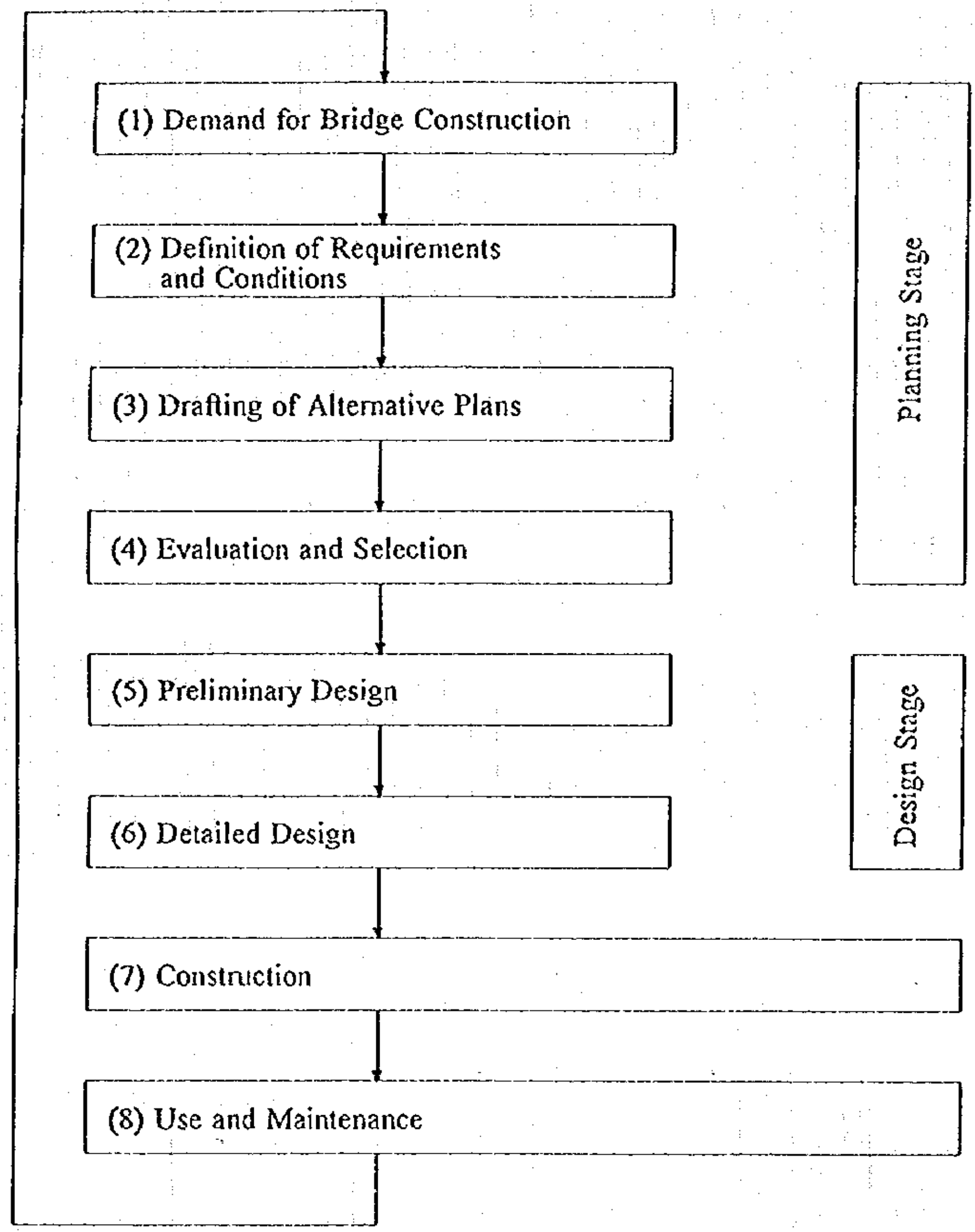


Fig. 1.2 Bridge Life Cycle

In the above cycle, the steps (1) to (4) are defined as the planning stage, and which are the very process to be discussed in this manual.

The steps (5) and (6) are recognized as the design stage to cover the activities from structural analysis up to preparation of drawings and specifications normally including construction contract documents. The step (5) - preliminary design is generally performed with less accuracy than that of detailed design to obtain outline of the project and clarify problems, which enable the authority to take early actions and corrections before detailed design sets in.

Bridge plan is materialized through the detailed design and construction. Completed bridge will be used over a long period as a public facility receiving maintenance and when it becomes decrepit, the demand for new bridge construction arises again.

CHAPTER 2 PROCESS OF BRIDGE PLANNING

2.1 General

The work of bridge planning is begun upon seizing the demand for bridge construction and takes the steps as explained in the bridge life cycle toward implementation. However, the process of bridge planning explained in this manual is not always applicable as it is to every case. By cases, different planning process will be taken: some bridges can be planned more simply but some may need another or additional consideration. Practically, the experienced bridge designers tend to perform bridge planning by taking into account various conditions simultaneously.

This chapter of bridge planning process will be useful for less experienced engineer as a check-list not to miss the essentials of planning and for advanced engineers to prepare lucid explanation and report for their planning.

2.2 Demand for Bridge Construction

Bridges are constructed for various purposes to support roads and highways at strategic points along their routes. Bridge structures are required to cross over rivers and valleys, or for grade-separation with other roads and railways.

Bridges are generally classified and separately called by purpose as follows:

(1) Road or highway bridge

General name for any bridge on roads and highways.

(2) Railway bridge

General name for any bridge on railways.

(3) Flyover or overpass bridge

Bridges for grade-separation with other roads, highways, or railways at intersections.

(4) Viaduct

Bridges to support elevated roads, highways, or railways, which are built mainly at where ground space is limited in urban area or embankment is difficult for ground is soft.

2.3 Definition of Requirements and Conditions

2.3.1 General

Upon receiving a demand of bridge construction plan, it is the first step to clarify all the information attending the proposed bridge plan including both of requirements and conditions. Requirements mean functions and capacities intended to the planned bridge. Conditions mean natural environment and surroundings where the bridge is to be constructed.

2.3.2 Requirements

For the planning of road and highway bridges, generally the following information shall be clarified first, the most of which are obtained from the highway design where the bridge is proposed.

(1) Road alignments (vertical and horizontal)

Approximate location and size of bridge are initially defined in the vertical and horizontal alignments of planned road. The road alignment of the proposed bridge section and its vicinity will be sometimes modified if that is not favorable for bridge planning, for example to avoid steep grade, sharp skew, curbed alignment, etc. In case of bridge replacement, the new bridge is generally planned in parallel with the old, and approach roads are required to connect with the existing road.

(2) Cross-section of road

The cross-sectional profile of bridge generally conforms to the standard cross-section of road design. Same traffic lane widths as that of the road section are normally applied to the bridge section, too. As regards shoulder and sidewalk, different designs are sometimes adopted according to the situation of the proposed bridge for example reducing or adding such widths from economical reason.

(3) Design vehicle loads

Design vehicle loads to be imposed on bridge must conform to the JKR specification for bridge live load. Over loading provision may be considered according to the classification of road and the design traffic volume especially of heavy commercial vehicles ratio.

(4) Affixed public utilities

Bridge, although its major function is to support traffic loads, is often requested to affix the following public utilities:

- Tele-communication cables
- Electric cables
- Water main pipes
- Gas main pipes

These public utilities are generally laid along roads under pavement, but at bridges held by bridge structures.

The requests of affixing public utilities should be confirmed to each utility agency and taken into account in bridge planning if requested. The accommodation space for utilities shall be secured in the bridge cross-sectional profile and their additional weights including that of affixing devices be taken into bridge loading. Utilities are usually accommodated under bridge deck and between beams to be concealed from the external view. Affixing of electric lines needs insulated covering against electric shock risk. Affixing of water main pipes sometimes makes the bridge structural layout difficult especially in case that water pipe is too large to be accommodated between beams.

The cost of affixing public utilities is generally beared by the utility agencies in the proportion of the utility weight to the total bridge design loads.

2.3.3 Conditions

Bridges are so planned as not only to meet the prescribed requirements but also to be compatible with the surrounding environment and other site conditions. Normally, the following conditions are involved:

(1) Topography

Bridge structure is planned so as to fit in surrounding topography. Normally, topographic map and profile elevations are prepared by the plane and levelling surveys, and on which bridge structure is planned mainly for determining location, length, and spans.

(2) Geology

Also, the ground condition of bridge site will decide the structural design of bridge. General soil information such as natures and depths of typical soil layers, would be obtained by collecting previous data if available or by performing soil boring, sounding, or geophysical exploration if no previous data is available. All such information is gathered into a soil profile and which should be combined with the topographic profile for the convenience of performing planning work. Geological information is used mainly for selecting bearing stratum, and location and type of substructures in particular of foundations.

(3) Climate

Malaysia is under the tropical monsoon climate and its weather is moderate: there is neither typhoon nor earthquake although a year is divided in dry and wet seasons and heavy rains in wet season. Therefore, the weather condition of the country affects bridge structures little except some local phenomena i.e. strong monsoon wind at the east coast of the peninsula and salt injuries of concrete in marine atmosphere. Careful attention is required to the measures against rains as well as to the hydrological and hydraulic conditions of bridge sites. Depending on the intensity of rains, the constructions in mountainous terrain during the rainy season have a risk of land slide.

(4) Hydrology

Minimum height, depth, and length of bridge structure are generally decided from the hydrological conditions of waterway except the bridges planned far higher above the water level to take navigation clearance, to cross deep valley, or for grade-separation.

The most necessary hydrological information is that of flood such as rate of discharge and high water level, which can be determined by the run-off analysis assuming rainfall intensity. However, this theoretical approach is often not reliable due to complexity and lack of field data. The second-best measure is to trace the past flood marks along river basin for determining the high water level and to survey the past flood damages nearby the planned bridge for deciding the course of flood flow.

The cross-section of waterway (existing water level and river bed elevations) at the planned bridge location should be surveyed and plotted on the topographic profile together with high water level and river improvement plan if there is. After confirming the cross-sectional profile of waterway, bridge structure is planned so as to be positioned above the high water level with appropriate freeboard and over the extent of the high water level. Hydrological conditions prepared for bridge planning are subject to the approval of JPS (Jabatan Pengairan Dan Saliran).

The hydrological survey items and the waterway crossing conditions necessary for bridge planning are detailed later in Chapters 3.2.3 and 3.3.2 respectively.

(5) Grade separation

When bridge is planned for grade separation with other road or railway, the following information of the crossing road or railway at the planned bridge location is necessary:

1) Plan and longitudinal profile

2) Cross-sectional profile

3) Over-head and under-ground public utilities

4) Road or track clearance

5) Overlay and widening plan

The crossing conditions with road and railway are detailed later in Chapters 3.3.3 and 3.3.4 respectively.

(6) Construction

Construction of planned bridge should be practicable. To ensure that, the following basic information about construction is generally required, which sometimes will be a decisive factor to select bridge type.

1) Access to construction site

For such bridge sites as the access of heavy equipment is difficult due to steep terrain or densely built up, the selection of bridge type will be limited by construction method and availability of equipment.

2) Transportation to bridge site

Modern bridge construction uses many of large precast or factory-made members such as for beams and piles, and accordingly needs more heavy equipment to erect them. The transportable size and route will be a main factor to decide the maximum span for the bridges remote from existing road.

3) Traffic diversion

In case of re-construction plan of existing bridge, traffic diversion is usually necessary. If there is no detour way in the vicinity of the bridge, temporary bridge or the stage construction scheme is required and that raises the project cost significantly.

4) Construction pollution

The piling work by the use of diesel hammer has been prohibited according to the area and hours in Kuala Lumpur. These years, the construction work in urban area is becoming restricted for the environment conservation. Bridge plan has to consider it in selecting construction method.

2.4 Drafting of Bridge Plan

2.4.1 Preparation of Base Map

Bridge plan is drafted on the base map which is prepared as follows:

1) Base map is a drawing having a topographical plan lower and a profile upper corresponding in position. Base map shall cover an area sufficient for planning both of bridge and its approach roads. Depending on the terrain and the size of the planned bridge, an area of around 100m wide by the bridge length plus 50m for each approach is a minimum idea.

2) Add the geological information to the base map: soil profile and boring data on profile, and boring location on plan.

3) Add the hydrological information to the base map: high water level, river bed elevations and freeboard on profile, and water flow course and banks on plan. If there is any river improvement plan, plot its finished elevations and alignments.

4) In case of the grade-separation bridge, enter the information of the crossing road or the railway on the base map: cross-sectional elevations and clearances on profile and horizontal alignments on plan. If there is any widening or overlay plan, plot its finished alignments and elevations.

5) Plot the planned road alignments on the base map: finished road elevations on profile, and center and other lane lines on plan. In addition, enter the designed cross-sectional profile of bridge on the upper right of the base map.

2.4.2 Drafting of Bridge Plan

Bridge plan will be drafted on the base map normally taking the following procedures:

(1) *Location and length of bridge*

Location and length of bridge are determined on the profile of the base map by selecting the abutment location at both bridge ends.

For the bridges planned with minimum height on the high water level, the abutment location is generally so decided, according to the crossing conditions, as to minimize the bridge length from economic reason. Crossing conditions are detailed later in Chapter 3.3.

However, for the bridges planned high from the ground or from the high water level and so the abutment location is discretionary, it can be decided economically by the cost comparison between bridge and embankment. A sample of the cost comparison is demonstrated as follows:

1) Assume minimum three points of tentative abutment location with different earth-fill and abutment heights along an approach road. For example, suppose three earth-fill heights 4, 6 and 8m along the approach road of 4% slope and so each point is 25m apart.

2) Estimate roughly and sum up the hypothetical costs for each point, and compare them to find the most economical location. Refer to Fig. 1.3 - Economical Abutment Location and Table 1.1 - Hypothetical Costs.

3) Regarding the estimate of the superstructure cost in Table 1.1, although type and span will not have been decided yet in this stage, assume them tentatively and estimate the average unit cost per bridge surface area from the past records of project costs. For instance, PC (prestressed concrete) beam with 20~30m span may be an appropriate first assumption. If there is a possibility of steel beam likely in Sabah and Sarawak, estimate the cost in same manner.

Irrespective of the above economic study, the limit of the safe embankment height will be an decisive factor to decide the abutment location where the ground cannot support high fills. After all such studies, abutment is generally planned of its height ranging from around 4 to 15m.

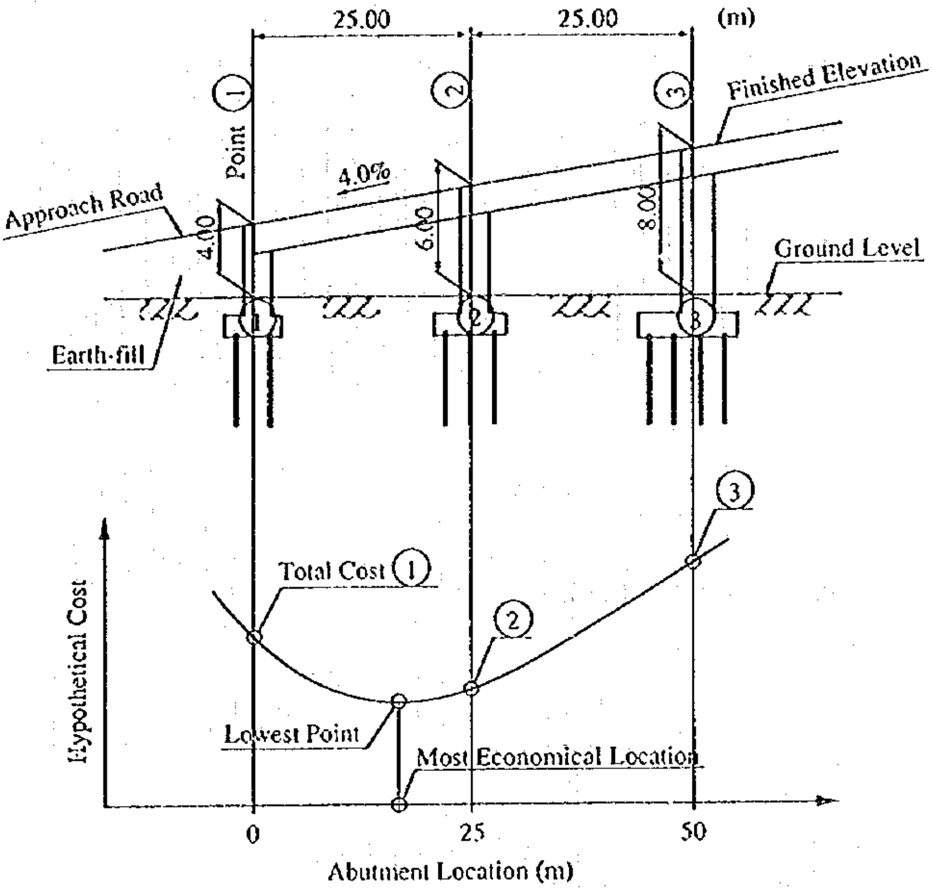


Fig. 1.3 Economical Abutment Location

Point	①	②	③
Abutment Cost	for Abutment ①	for Abutment ②	for Abutment ③
Superstructure Cost	for Section ①-③, 50m	for Section ②-③, 25m	-
Earth-fill Cost	-	for Section ①-②, 25m	for Section ①-③, 50m
Total Cost	①	②	③

Table 1.1 Hypothetical Costs

(2) Pier location, span, and bridge type

After defining the bridge location and length, the next is to select pier location and spans as well as applicable bridge types (of superstructure) by studying the ground (topographical and geological) and crossing (of waterway, road, or railway) conditions on the base map. As spans are decided, the choice of bridge types will be narrowed down, that is, span is generally selected within the applicable range for each bridge type. The reference of bridge types and suitable span range is given later in Tables 1.5 and 1.6 of Chapter 3.4.2.

For the bridges where pier location is restricted by crossing conditions, it is so selected, according to the recommendations detailed in Chapter 3.3 -- Crossing Conditions, as to minimize the interference with the crossing river, road, or railway.

However, for the bridges where pier location is discretionary, it is selected economically by comparing the costs for several span alternatives. This study holds true more for longer bridges. A sample of the cost comparison is demonstrated as follows:

1) Propose minimum three alternatives of different span arrangements and their corresponding bridge types for a certain bridge length. For example, suppose the following three arrangement cases as shown in Fig. 1.4 - Economical Span Arrangement.

2) Estimate roughly the total construction cost for each case including all the superstructure and substructure costs and compare them to find the most economical span arrangement and bridge type.

3) In order to hold the accuracy of the cost comparison, bride type should be selected appropriate for the proposed span and be practicable in construction. The superstructure type is comparatively quick to be looked up from the standard design compared to the substructure which is usually not easy and owes much to the engineer's experience and the past similar designs. Simplified stability calculation is sometimes required to define the size of substructure especially the size and number of piles which affect the substructure cost significantly.

4) In conclusion, the economy of selecting span and bridge type is understood as the cost balance between superstructure and substructure. That is, the cost of superstructure is generally much the higher as the longer span is. On the other hand, the cost of substructure depends more on ground condition than span and bridge type. Therefore, for such bridges as the substructure cost is small because, for instance, no pile is required owing to shallow bearing stratum, the choice of shorter span with many piers is economical, and vice versa.

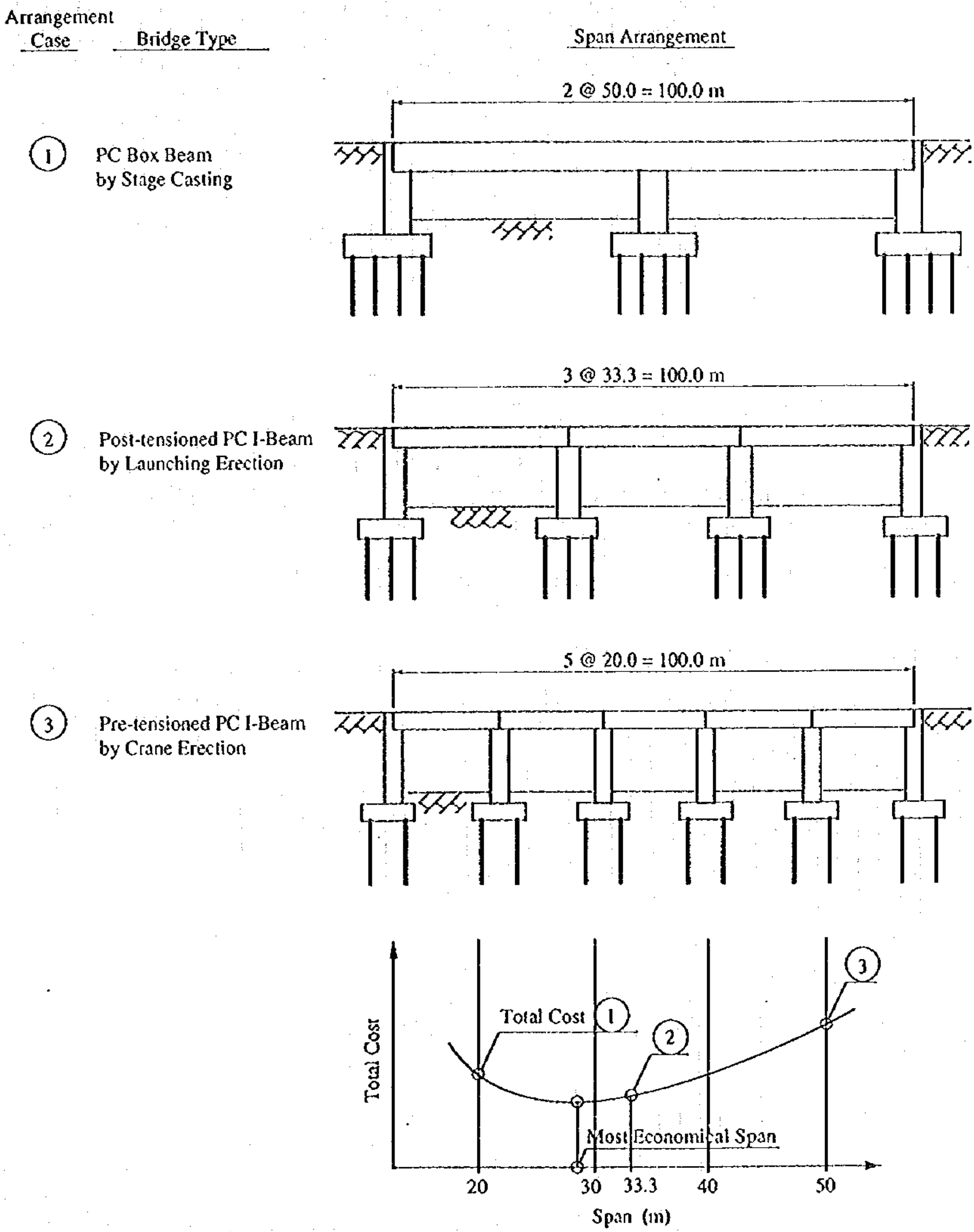


Fig. 1.4 Economical Span Arrangement

5) The above sample of cost comparison was performed among the different bridge types proposed for each span arrangement respectively. On the other hand, the most economical span for a certain bridge type can be known by performing the cost comparison under the same bridge type. For instance, PC post-tension beam is generally used in a span range of 20 to 40m and the most economical span of this type will be derived from the following comparison:

Arrangement Case	Number of Spans		Span(m)		Bridge Length(m)
①	5	×	20	=	100
②	4	×	25	=	100
③	3	×	33.3	=	100

(3) Practice of span arrangement

Multiple span bridge should be arranged in regular spans with uniform beam height as much as possible to create the view of straight, clean horizontal lines, and which also helps labor-and cost-savings in design and construction.

However, for the bridges spanning deep valley, the pier location is controlled by terrain and accordingly regular span arrangement is difficult. In Fig. 1.5, for instance, Option ① intends same span but the construction of high pier is difficult. Therefore, the irregular spans or one span option shown in Option ② and ③ will be considered as alternatives. In general, when pier height is over around 30m, the options of longer span bridges will be economically more competitive than shorter span types.

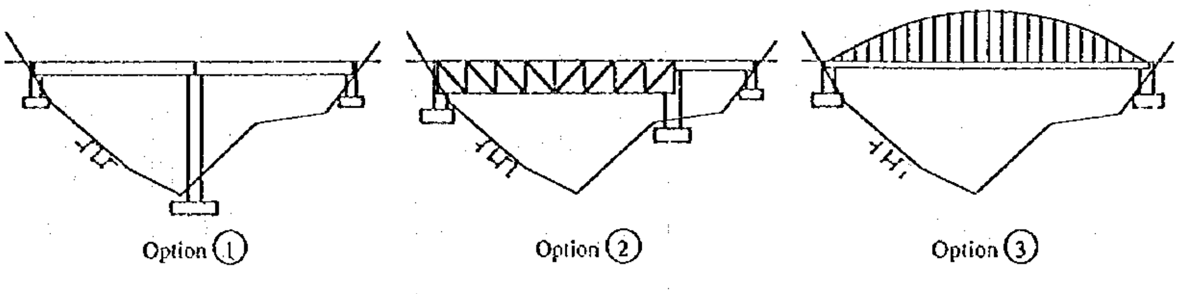


Fig.1.5 Span Arrangement for Deep Valley

2.5 Evaluation and Selection

2.5.1 General

Chapter 2.4 explained how to draft a bridge plan to meet the requirements in the given conditions. As a matter of course, drafting work is always performed to seek better plan and through this process various judgments and reasonings as well as selections will have been developed. However, such developed information will not have been logically compiled yet. For the final conclusion of bridge plan, the process of evaluation is required to justify it objectively.

The work of evaluation should start with making clear the objects of evaluation, be followed by the comparison of advantages and disadvantages, and be concluded by rating.

2.5.2 Objects of Evaluation

Bridge is a composite figure made of many components which are roughly divided into superstructure and substructure including foundation. To select such a composite figure, evaluation is required in both ways, partially and totally.

(1) Partial evaluation

This is performed to select type for a certain member part(s) or to give individual solutions of a planned bridge, and the results of that are reflected to the total evaluation.

Take an example for the selection of foundation and suppose that ground is soft alluvial deposits with intermediate sand layers and diluvial formation is about 30m deep. From this assumption it is immediately clear that this site needs some piles, but the problem is to select the type and size of piles. Several options will be suggested with advantages and disadvantages. For instance, PC spun piles may be the cheapest but uncertain in execution for the depth of bearing stratum and the existence of intermediate sand layers. Steel pile piles are reliable in construction but may be the most expensive. Bored cast-insitu concrete piles will be another possibility reliable in execution and advantageous for less noise if the site is in residential area, but the site work will be complex for large facilities and skilled techniques are required. Thus, even taking the problem of piles which is only a part of a bridge, evaluation is required to select the best.

In bridge planning, the followings are the most common objects of partial evaluation:

1) To select location, length, and spans

- Abutment location by cost comparison between bridge and earth-fills, and maximum safe earth-fill.

- Pier and span arrangement by cost comparison among several combinations of superstructures and substructures.

2) To select foundation

- Type of foundation and construction method according to ground condition, cost, and execution reliability.
- Kind and size of piles by cost comparison among several sizes and required numbers of piles, and availability of equipment.

3) To select substructure

- Type of abutment by its required height sometimes including comparison between retaining wall type and earth-pressure-relieved type from economy, aesthetic view and earth-fill stability.
- Type of pier by its required height and situation.

4) To select superstructure

- Type and construction method according to span, terrain and accessibility, material and equipment (for erection) availability, cost, aesthetic view, etc.
- Construction method of deck slabs: cast-insitu or pre-cast according to accessibility and time schedule.

(2) Total evaluation

This is performed to finally select the best bridge plan as a whole. The information obtained through the partial evaluations will be combined in various ways to produce several complete bridge plans for final selection. These final plans shall be of minimum two alternatives in order to compare them but not so many suggesting that five may be maximum to clearly understand the differences among them.

1) Evaluation factors

Bridge plans should be evaluated from various aspects such as economy, construction, maintenance, aesthetics, etc. As regards safety and crossing conditions, which are usually not included in the evaluation factors, because bridges undoubtedly ought to be planned safe and to satisfy conditions.

2) Rating

The simplest technique to assess the overall alternative plans would be to use a ranking method. A sample rating by using the method is demonstrated as follows:

i) Two kinds of rating factors are used, ranking and importance. The product of the two factors is defined as the score for each alternative relative to each evaluation factor. A total score adding the scores of all evaluation factors is the conclusion of the rating, and normally the alternative plan scored the highest will be the most desirable.

ii) Ranking factors will be given by whole number with respect to the order of desirability among the alternatives. The rank of 1 is assigned to the least desirable alternative and a rank of n (equals the number of alternatives) is assigned to the alternative that is the most desirable.

iii) Importance factors will be given by weighing priorities among the evaluation factors. The assignment of importance factors is voluntary, and with which the policy of bridge plan can be considered in the result of evaluation.

iv) Supposing a sample rating of three alternatives, where the first priority is given to economy and the other importance factors are weighed equal, the following result is obtained. In this assumption, the highest total score will be 30 and the lowest 10.

Evaluation Factor	Ranking Factor	Importance Factor	Score
Economy	1, 2, ③	4	12
Construction	1, ②, 3	2	4
Maintenance	1, ②, 3	2	4
Aesthetics	①, 2, 3	2	2
		10	22

v) It is noted that the above ranking method cannot distinguish incremental differences among alternatives. One way to remedy this is to establish the ranking scale on the basis of relative difference, but that is too hypothetical to give factors properly.

3) Evaluation table

Total evaluation is the summary of all the information obtained through the planning process and which should be clearly explained to the public or to the parties concerned for approval. Evaluation results should be understandable to third party, so that the use of table is recommended for better presentation. Fig. 1.6 shows the example.

Alternative Plan		1 (Naming)			2			3		
Illustration (Profile)		(Figure)								
Evaluation Factor	Importance Factor	Comment	Ranking Factor	Score	Comment	Ranking Factor	Score	Comment	Ranking Factor	Score
Economy	4	(Comments)	3	12		1	4		2	8
Construction	2		2	4		3	6		1	2
Maintenance	2		2	4		1	2		3	6
Aesthetics	2		1	2		3	6		2	4
Conclusion & Total Score	10			22			18			20

Fig. 1.6 Evaluation Table

Note: Scores shown are sample only.

2.5.3 Scope of Evaluation

Evaluation is performed to figure out the best bridge plan under a certain situation or scope, and within the scope all alternative plans come out and conclusion is made. The scope depends on purpose, location, and site condition of bridge plan, but it is generally established conforming to the policy of the client (government department). If bridge plan is a part of highway project, the scope will be decided in the coordination with the highway plan.

(1) Evaluation under fixed bridge length

Bridge planning is started from the basic matters to decide location and length and followed by foundation and substructures. Normally, the comparison of superstructure types as well as spans is highlighted as the main object for the final selection. In such case, most substructures and foundations have been already decided by partial evaluations, accordingly the total evaluation will be played under the fixed bridge length, the differences of substructures and foundations will not be emphasized, and the cost of approach road construction will not be included in economic comparison. This is the simplest and most common evaluation to decide only bridge type and spans.

(2) Evaluation among different bridge length

Bridge location is normally determined along the road alignment. However, in the course of bridge planning, modification of road alignment is sometimes suggested in the vicinity of the bridge location for more favorable bridge plan. Evaluation arrives at the common procedure if road alignment is reviewed regardless of bridge types. However, if road alignment is different in relation to bridge types, total evaluation will become complex: bridge alternative plans must be compared in different location and bridge length, and cost of approach road should be included in economic comparison.

(3) Scope of economic evaluation

It is ideal for economic evaluation to estimate the total life cost covering design, construction, right-of-way acquisition and maintenance costs. However, because design cost is so small against construction cost and maintenance cost is difficult to estimate, economic evaluation is often performed with only the initial cost of construction and right-of-way acquisition.

CHAPTER 3 ENGINEERING FOR BRIDGE PLANNING

3.1 Principles for Bridge Planning

The followings are the common suggestions useful in bridge planning:

(1) Road alignment to minimize bridge cost

In general, bridge cost is much higher than road cost per length. To minimize total project cost, road alignment needs the review from the bridge engineer's view although it is selected normally considering traffic strategy, right-of-way availability, obstacles, ground condition, and other many factors. Attention shall be given to the location of bridge and its approach roads. Even after deciding road alignment, sometimes bridge design can be much improved by minor changes of the bridge location and the alignment of approach roads.

(2) To meet requirements of bridge

Bridge shall be planned to meet the requirements expected to the planned bridge such as of width, length, height, and loading capacity. In particular, the requirements of crossing waterway, highway, and railway shall be carefully determined after due consultation with their administration agencies.

(3) Safety and economy of bridge structure

Bridge structure must be planned safe but at the same time it is required to achieve economical design, construction, and maintenance. The two concepts of safety and economy seems to be conflicting, and taking equilibrium between them is a major problem to be solved in bridge planning. Because bridge is an important public facility, it seems to be a right way that the first priority should be given to safety and the second to economy.

The criteria for safety of structures is generally given by design specifications or codes, which is a necessary condition in bridge design but not a sufficient. To harmonize safety and economy in bridge planning, it requires the profound knowledges of an experienced engineer. However, the drawing or the completed bridge given due such consideration will convince the people's eye that the structure is functional, and safe, too.

(4) Easier and faster construction

When there are more than two prospective alternative plans and they are of alike cost, it is then recommended to study their construction methods and select easier one.

In case that a new technique is proposed, it is recommend to respond such challenge as much as possible after careful study for its reliability for the good of the progress of technology.

(5) Standardization of structures

Bridge planning is in principle performed individually to select the most suitable design according to each site condition. However, when a number of bridges similar in size and type are planned, such as for a series of over-bridges along a highway, the standardization of structures will often save cost and time both in design and construction. For instance, in case that the spans of the overbridges fluctuate a little, it is more economical and practical to apply a standard beam of same size and span to all bridge sites despite accompanying somewhat construction losses.

(6) Traffic safety and comfort

The traffic safety and comfort on bridges depend not only on the road geometries and alignment but also on the existence of structures on the bridge surface. In view of that, the following measures are recommended:

1) Adopt deck type bridge rather than through type except the cases that the under-bridge clearance is limited. However, through type is more economical than deck type for longer span range.

2) Prefer continuous beam bridge to simple beam to reduce expansion joints which may cause the vehicle running shock and become to be a big maintenance burden.

(7) Easier maintenance

The maintenance troubles of bridges are frequent on the secondary members such as expansion joints and bearing shoes rather than on the primary members. Therefore, continuous beam bridge, because it has less such troubles, is more favorable for maintenance. In particular, the reduction and the easy maintenance of expansion joints should have been considered from the planning stage. Repair or replacement work of expansion joints on bridges will become a major cause of traffic interference.

(8) Aesthetic consideration

Bridges must serve for the public to carry traffic safely, and they have to be constructed and maintained with reasonable cost. That is the primary function of bridges and can be designed by applying physical science. On the other hand, bridges exist long time on ground exposing to the public eye, so that bridges cannot help having the ornamental function as well. The bridge design successful in pleasing the people's eye, is not necessarily obtained only by the use of physical techniques. Because bridge design is a process of human creation, it is natural to seek beauty in the view of bridges.

3.2 Field Surveys

3.2.1 Topographic Survey

(1) Survey method

Topographic survey is carried out to prepare topographic map which is necessary as the base map for bridge planning. The survey method is generally to use plane-table surveying or aerial photogrammetry. The area of topographic map for bridge planning is regional and the scale is comparatively large, so the plane-table surveying is normally used. If the existing photographic map covering the proposed bridge site is available, it may be utilized by magnifying the scale but that needs the complementary and correction surveys by plane-table surveying.

(2) Scale

The scale of topographic map suitable for bridge planning ranges 1/100 to 1/500. The scales 1/200, 1/300 or 1/400 are generally used.

(3) Contour

In principle, topographics have to be indicated by contours, the interval of 1m is preferable. The planimetric map having no contours is sometimes used although it is unfavorable, but at least point contours should be shown to find elevations.

(4) Road center line

Road center line, if it is available at site, should be surveyed and shown on the topographic map. It is useful to find the planned bridge location on the map and can be used as a datum line in planning work.

(5) Profiles

Longitudinal and lateral profiles are preferably prepared by the leveling surveying along the road center line. Alternatively, profile can be also prepared from the contours of the topographic map and that may be usable considering less accuracy.

3.2.2 Geological Survey

(1) Survey method

The geological survey for bridge planning is performed putting stress on obtaining the general subsoil information of bridge site rather than detailed soil tests. The survey is normally carried out from data collection and field reconnaissance, and followed by boring, test-pit digging, or geophysical exploration if considered necessary.

(2) Data collection

This is performed at early stage of the survey to collect the existing geological data around the site. Major data sources are the geological map issued by the National Geology Office and the previous boring or geophysical data of the vicinity. If such existing data is available, the general geological condition of site can be often supposed from those data and so special field survey may not be required. Thus, the execution of field survey depends on the data collection.

In addition, the existing bridges and buildings will be another useful data source by studying their foundations about type, size, depth, and whether settlement and tilt are seen or not.

(3) Field reconnaissance

This is a visual survey to judge the subsoil conditions from the outcrops of rocks and strata. The survey is carried out by walking along the existing roads and rivers in the survey area while observing the outcrops to make a field reconnaissance map.

Landslide traces, obstacles, terrain and ground condition which seem to be troubles for construction, will be also surveyed in this occasion.

(4) Soil boring

1) General

Boring survey is the most popular soil investigation method to obtain the underground stratification and the engineering characteristics of strata. The method bores a hole into ground by boring machine, collects samples and carries out in-situ tests through the hole. Soil boring generally performs the following surveys:

- Discrimination of stratification
- Observation of cored soils
- Sampling and laboratory soil test
- Water level in bore
- In-situ test in bore: standard penetration test, vane shear test, lateral loading test, pore water pressure test, geophysical exploration, etc.

Survey results should be carefully compiled by using the prescribed forms such as boring log and soil test data sheets, because these boring data will be the basis and repeatedly referred for designing and construction.

2) Application of Standard Penetration Test

The result of standard penetration test "N-value" is the most important information widely used for the design of foundations. Many design factors as shown in Table 1.2 can be estimated from N--value.

Table 1.2 Design Factors Estimated from Standard Penetration Test

Sandy Soil	Clayey Soil	Construction
- Relative density - Internal frictional angle - Coefficient of bearing capacity - Void ratio	- Consistency - Uniaxial compression strength (undrained)	- Judgment on possibility of piling penetration - Judgment on effect of soil improvement - Study of excavation method - Judgment on land slide
- Bearing capacity of soil - Bearing capacity of piles - Coefficient of ground reaction		

The Meyerhof's suggested relations of N-value to relative density and internal frictional angle is given in Table 1.3 and the Terzaghi-Peck's of N-value to consistency and uniaxial compression strength in Table 1.4.

Table 1.3 Meyerhof's Relation of N-Dr-ϕ for Sandy Soil

N-value N	Density of Sand	Relative Density Dr	Internal Frictional Angle ϕ (degree)	Static Cone Penetration Test Value qc(kg.f/cm^2)
< 4	very loose	< 0.2	< 30	< 20
4~10	loose	0.2~0.4	30~35	20~40
10~30	medium	0.4~0.6	35~40	40~120
30~50	dense	0.6~0.8	40~45	120~200
> 50	very dense	> 0.8	> 45	> 200

Table 1.4 Terzaghi-Peck's Relation of N-Consistency-qu for Clayey Soil

N-value N	Consistency of Clay	Uniaxial Compression Strength qu (kg.f/cm^2)
< 2	very soft	< 0.25
2~4	soft	0.25~0.50
4~8	medium	0.50~1.00
8~15	stiff	1.00~2.00
15~30	very stiff	2.00~4.00
> 30	very hard	> 4.00

For selecting the bearing stratum for bridge foundation according to N-value , although there are many exceptions to design and site condition, the following values are generally suggested as a standard:

Sandy Soil : N > 40
Clayey Soil : N > 30

3.2.3 Hydrological Survey

Hydrological survey is vital to planning the bridges which cross rivers and channels. From bridge site, the following information should be collected and analyzed:

(1) Condition of river course

Condition of river course is a decisive factor to determine the crossing location and direction, the bridge length, and the protection against erosion and scouring. Major survey and analysis items include the followings:

1) Meandering reach, curved reach, or straight reach

2) Historical change of meandering and bank erosion

3) Pattern, size, and movement of sand bars

4) Change of longitudinal river bed elevations such as aggradation or degradation of sediment

5) Area of flood plain and width of main stream during flood

(2) Condition of major floods

It is important for bridge planning not to worsen the capacity of waterway for flood flowing by bridge construction. Flood discharge should be passed smoothly and safely at bridge. To determine the design discharge of flood and the required opening of bridge, it is necessary to survey and analyze the past major flood records on the following points:

1) Flood water level, area, and duration at bridge

2) Flood discharge and velocity of current

3) Debris, sediment, and floating logs

(3) Design discharge

JKR has the guideline for the hydrological design for the bridges crossing rivers and channels, in which the following return periods for the estimation of design discharge are specified:

1) For bridges to cross rivers and drainage channels : 100 years

2) For sewerage culvert : 50 years

(4) Meteo-hydrological condition

Meteo-hydrological condition of bridge site and river basin is also a necessary factor for determining design discharge, high water level, design wind velocity, construction method and schedule. Major items of this survey are as follows;

1) Seasonal variation of temperature, relative humidity, and wind velocity

2) Seasonal variation of rainfall

3) Seasonal variation of water level and discharge

(5) River improvement plans

Any river improvement plan, whether it is on-going or a future plan, should be investigated and entirely taken into account for bridge planning so that the bridge construction should not become any obstruction to the river improvement plan.

3.3 Crossing Conditions

3.3.1 General

Bridges are constructed to cross existing land space, most are public spaces such as rivers, roads, railways, etc. and where the bridge structures must exist long time after being constructed. Therefore, bridge plan is required to have due consultations with the competent authorities of such space for approval.

The crossing conditions required to bridge plans are different depending on their planned location and purpose, and governed by various regulations relevant to land use.

In addition, urban and local development plans are also involved in bridge planning, although these conditions should have been consulted in the road planning stage.

This chapter explains the general crossing conditions for river, road, and railway, because most bridges are planned to cross them.

3.3.2 Waterway Crossing

The figures and equations shown in this section are recommendation based on the River Management Guideline, Japan.

Fig. 1.7 shows the general concept of river cross-section and bridge layout.

(1) Location and direction of crossing

1) Cross river at its straight reach.

In numbers of meandering rivers in Malaysia, bank erosion has occurred more at the curved reaches than the straight. Although small meanderings occur even in straight reaches by the movement of sand bars, it is far better to select bridge site at the straight reach compared to the curved. Further, it is important to investigate the historical change of river course and bridge site should be selected at where the change of river course is small.

2) Cross river in perpendicular to its flow.

The bridges crossing rivers with skew direction often cause erosions and scourings at around bridges. Skew bridge will produce asymmetric turbulence in river flow and that makes the bank protection against erosion very difficult.

Therefore, the crossing at straight reach with right angle is the most recommendable. If it is unavoidable to cross river at curved reach or with skew direction, protection shall be provided not only at around abutments but also to the adjacent river banks with sufficient length.

(2) Waterway width and freeboard

1) Lay abutments outside of waterway.

If bridge opening is shorter than the waterway width, flow will be constricted at the bridge, and that causes backwater effect on upstream. This phenomenon will endanger the bridge by incurring severe erosion and scouring. Therefore, it is necessary to design the bridge opening wider than the waterway width.

Although it varies depending on water depth, the design discharge and the required waterway width have a general relation as follows:

Design Discharge (m3/s)	Waterway Width (m)
300	40 - 60
500	60 - 80
1,000	90 - 120
2,000	160 - 220
5,000	350 - 450

2) Minimum freeboard on high water level (HWL)

The freeboard between HWL and the top of dike for the rivers having compound cross-sections, shall be not less than the following values:

Design Discharge (m3/s)	Freeboard (m)
less than 500	0.5
500~2000	1.0
over 2000	1.5

For the rivers having single cross sections, the freeboard is recommended to take at least 0.6 m.

The clearance between HWL and the soffit of bridge beam shall be decided by adding the allowance of 0.5m to the above-mentioned freeboards. (Fig. 1.8).

(3) Minimum span length

1) Span length has a direct relation to the possibility of clogging the bridge opening with floating logs or debris. The minimum span length in relation to the design discharge is generally given by the following formula.

$$L = 20 + 0.005Q$$

where, L: span length (m), measured in perpendicular to flow
Q: design discharge (m^3/s)

2) However, if it is ensured that there is far less possibility of floating logs and debris so that clogging rarely occurs, the minimum span length can be reduced to the following values:

$Q < 500\ m^3/s$ and $W < 30\ m$: $L = 12.5\ m$
$Q < 500\ m^3/s$ and $W \geqq 30\ m$: $L = 15\ m$
$500 \leqq Q \leqq 2000\ m^3/s$: $L = 20\ m$

where, W: waterway width (m)

3) Pier location close to bank

Pier should not be laid on the slope of bank nor at the foot of bank slope. It is recommended for the piers planned close by bank to take at least the following distance from the toe of bank slope.

Design Discharge (m^3/s)	Distance (m)
less than 500	5
over 500	10

For where it is difficult to take the above distance, sufficient bank protection should be provided on the foot of bank slope and around the pier against possible local scouring.

4) Impediment rate of pier width to waterway

The existence of piers in waterway is the biggest impediment to water flow imposed by bridge construction. The smaller pier width is, the better water flows. There is a guideline to control the total pier widths in a waterway by the impediment rate to the waterway width:

		Impediment Rate
Desirable	:	less than 3 %
Maximum	:	5 %

The pier width shall be measured in perpendicular to flow direction at high water level, and the waterway width shall be the width of high water level.

(4) Abutment design

1) Invert-T type abutment

There are many examples in Malaysia that bank seat type and pile bent type abutments are damaged by local scouring. Many protections provided in front of such abutment are washed away by flood and piles are exposed.

Therefore, it is recommended to adopt the invert-T type abutment instead of bank seat and pile bent types.

2) Embedding depth of footing

Footing shall be embedded into river bed. Where the scouring risk is high, it shall be deepened below the anticipated scour depth. (Fig. 1.8)

3) Parallel to flow

Abutments shall be laid in parallel to flow.

(5) Pier design

1) Oval or round shape for pier column

The existence of piers in waterway unavoidably bridges about turbulence in water flow and which is a major cause of the local scouring around piers. To lighten this effect of piers, it is recommended for the cross-sectional shape of the pier columns to be oval or round which disturbs water flow much less compared to rectangle. It is also recommended that a pier have only a single column, but do not have double or multiple columns which rather induce severer turbulence in closely standing columns.

Where flow is not stable or curved, round shape is more adaptable to the change of flow than oval.

2) Embedding depth of footing

Footing shall be embedded into river bed deeper than the anticipated scour depth. A guideline of the embedding depth of footing is given as follows (Fig. 1.8):

Location of Pier	Embedding Depth
i) Low water channel and the part of high water channel within 20m from the top of the slope of low water channel:	More than 2m below the river bed of low water channel
ii) High water channel beyond 20m from the top of the slope of low water channel:	More than 1m below the river bed of high water channel

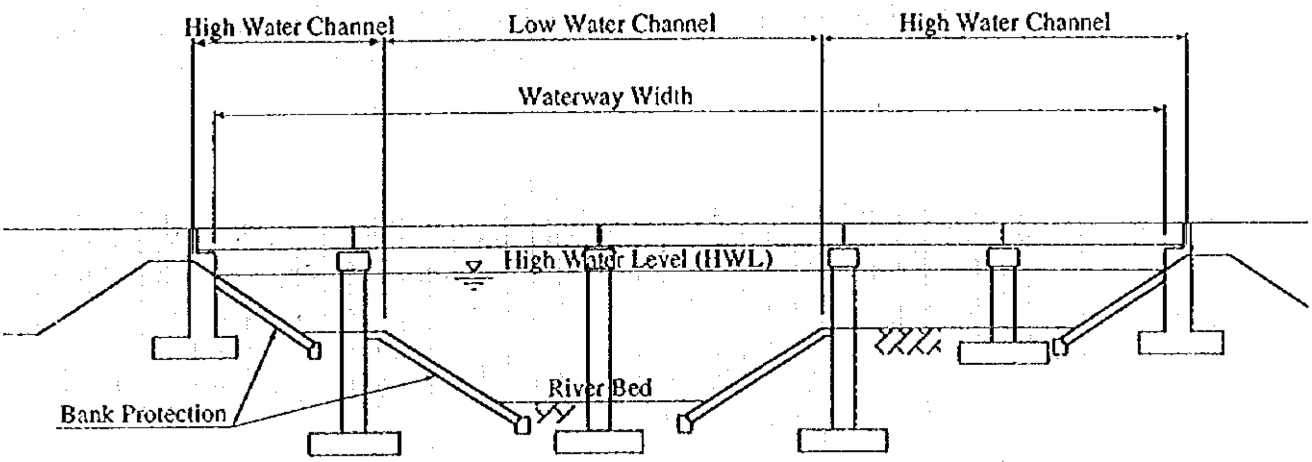


Fig. 1.7 General River (Compound) Cross-section and Bridge Layout

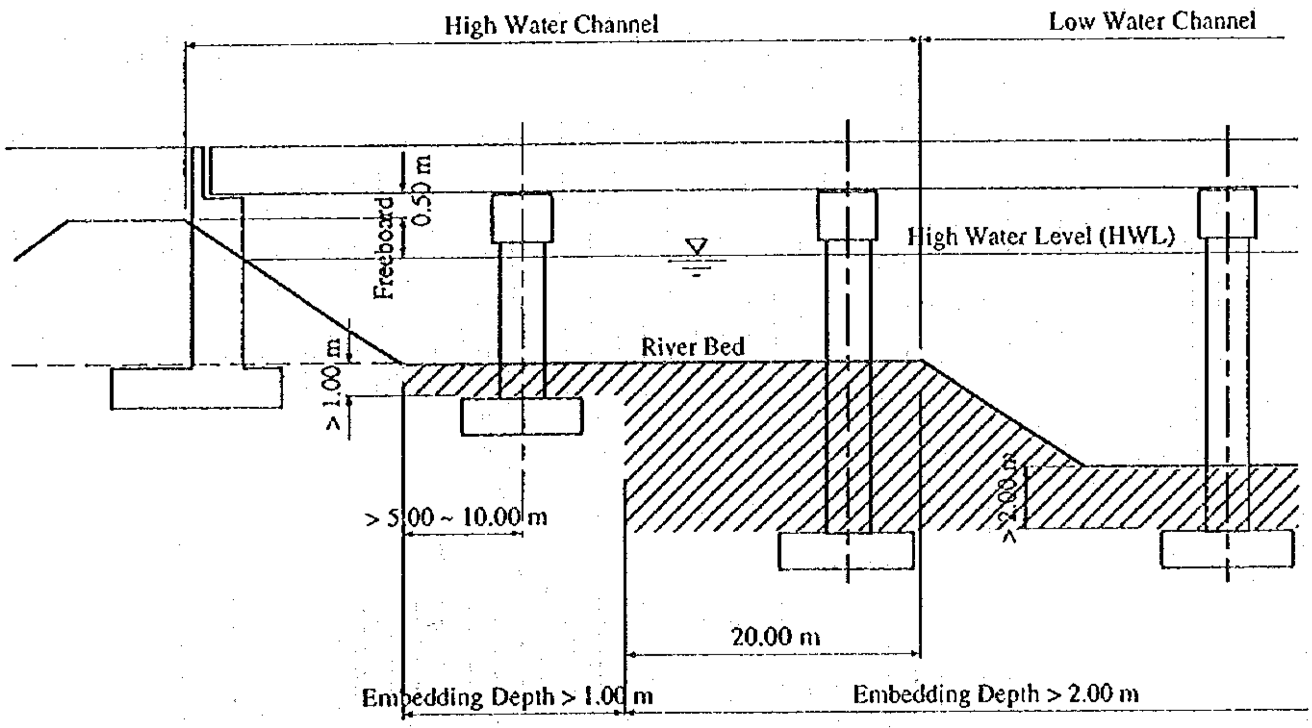


Fig. 1.8 Freeboard and Embedding Depth

(6) Bank protection

Bank protection is required to protect the slope of bank from the erosion which may be caused by the turbulent water flow induced by the construction of piers.

1) Covering area

Bank protection shall be provided both on high water bank and low water bank, from the top to the toe of bank slope, for the extent of 10m up - and down - stream from the side of bridge including the underneath of bridge.

For skew bridges, additional covering area is required as shown in Fig. 1.9 to cope with asymmetric flow turbulence.

2) Embedding depth

Bank protection shall be embedded into river bed not less than 0.5 ~ 1.0m for small rivers and 1.0m for large rivers. Where scouring risk is high, it shall be deepened below the anticipated scour depth.

3) Foot protection

The toe of bank protection shall be protected against scouring with gabion packs or stones.

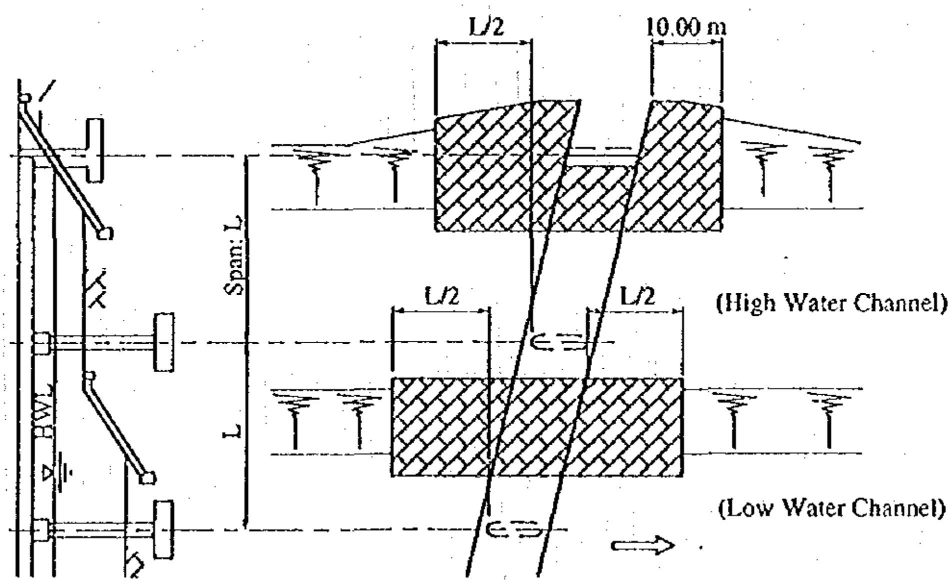


Fig. 1.9 Area of Bank Protection

3.3.3 Roadway Crossing

(1) Information of crossing road

As previously mentioned in Item(5)-Grade separation of Chapter 2.3.3, the following information of the crossing road is required for bridge planning:

1) About existing road:

- class and grade
- cross-sectional profile
- right-of-way
- clearance limit
- longitudinal profile

2) About future plan

- designated, or not designated to the roads of city planning
- sidewalk plan, or not
- overlay and widening plan, or not

3) About public utilities

(2) Consultation items

The following items are to be consulted with the competent authority of the crossing road:

1) Bridge length and spans

2) Location of abutments and piers

3) Embedding depth of foundations

4) Under-bridge clearance

5) Diversion road

6) Construction method (includes protection of existing road and traffic)

(3) Clearance limit (a minimum clearance to meet current JKR practice)

In the case of Japan, a clearance height of 5.0m above the existing road surface under the soffit of the planned bridge beam is recommended from the following reasons.

Reason 1: The road geometric design act specifies the clearance to be 4.70m. In addition this, an allowance of 0.3m is considered for future overlay.

Reason 2: The legal vehicle size is 3.80m. On the other hand, steel bridge needs a space of minimum 1.0m under bridge beam for repainting work as well as a margin of 0.2m.

(4) Location of abutments and piers

1) General

Abutments and piers are prohibited inside of roadway. It is favorable for the traffic of the crossing road to have sufficient lateral margins between roadway and abutment, and not to have a pier on median strip.

However, the following cases are technically and economically very difficult to avoid a pier on median strip:

i) Crossing road is very wide having six lanes or over.

ii) Bridge is skewed to crossing road with over about 50 degrees even if it has only four lanes or less.

iii) Crossing road is separated into up and down lanes.

iv) Frontage road and/or waterway run in parallel to crossing road.

When a pier is designed on median strip, it is recommended to consider collision load of vehicles for the design of pier.

2) Lateral margin

If a pier is put in median strip, the median needs to be widened at least for the pier width so as to maintain the original lateral space. Even in case that median cannot be widened sufficient, a minimum lateral margin of 0.5m (Japanese Standard) is required between the pier and the clearance limit of the crossing road as shown in Fig. 1.10. Guardrail or autoguard will be installed in the lateral margin.

Footing of pier, as shown also in Fig. 1.10, shall be preferably not extended beyond the median width to avoid uneven settlement on roadway, and embedded more than 1.0m to secure the space for underground public utilities.

3) Special lateral margin for expressway

The crossing with expressway, where vehicles can run in high speed, needs greater lateral margin for abutments and piers not to be visual oppression against drivers. It is recommended to take minimum 3.0m for massive structure like abutment and thick pier and 1.5m for slender structure like thin pier (pier width is less than about 1.0m).

3.3.4 Railway Crossing

(1) Information of crossing railway

Like the road crossing, the following information of the crossing railway is required beforehand:

1) About existing railway:

- class and grade
- rail-gauge and cross-sectional profile
- right-of-way
- clearance limit
- electrified or not

2) About future plan

- electrification plan, or not
- double-tracking plan, or not
- elevating plan, or not

(2) Consultation items

The following items are to be consulted with the competent authority of the crossing railway:

1) Bridge structural type

2) Bridge length and spans

3) Embedding depth of foundation

4) Location of abutments and piers

5) Under-bridge clearance

6) Construction method (includes relocation and protection of existing railway facilities)

7) Consignment construction, or delegation of supervisors

8) Installation of guardfence

(3) Clearance limit

The clearance limit of railway is different depending on the type and kind of railways. The railway of Malaysia has been developed based on the British gauge and is now in progress of electrification. Fig. 1.11 shows the clearance of the Malaysian National Railway both for the electrified and not electrified.

However, in recent years new commuter railway system is going to be constructed in urban area. To cross with such new system, clearance limit should be confirmed by individual consultation.

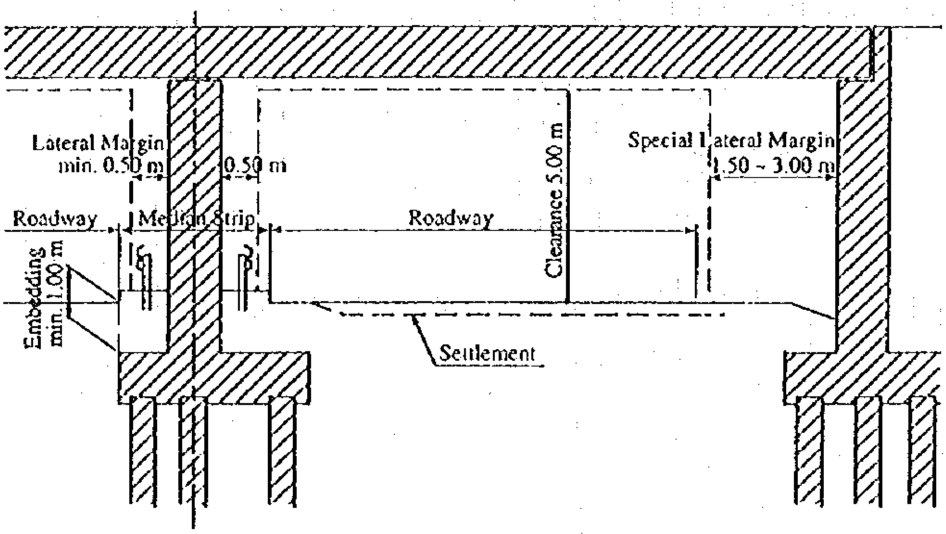


Fig. 1.10 Clearance for Road Crossing
(Note : Dimensions are tentative, which shall be checked. Minimum clearance to meet current JKR practice.)

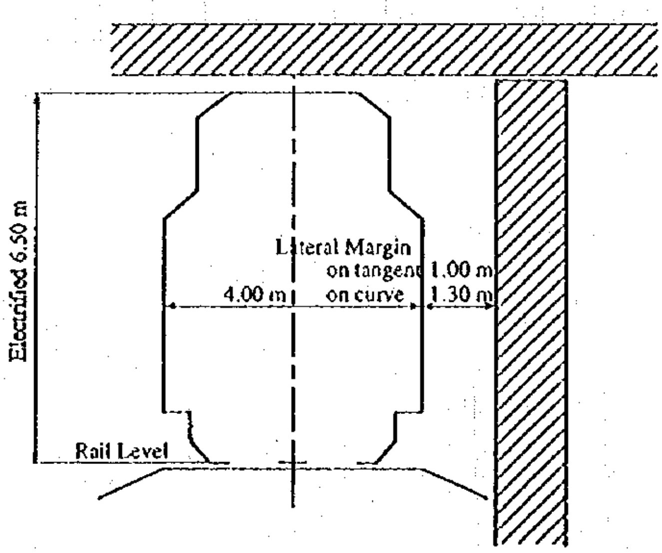


Fig. 1.11 Clearance for Railway Crossing
(Note : Dimensions are tentative, which shall be checked. Minimum clearance to meet current JKR practice.)

3.4 Bridge Structure Type

3.4.1 General

There are many factors to be considered in the selection of a structure type. One of the most important is the region of the country in which the bridge will be built. For various reasons there seem to be preferred bridge types in certain regions of the country. Proximity to the steel mills and the cost of freight may well control the choice between a concrete or a steel bridge. The experience of the local contractors is a big factor, and offering a structure type which is strange to them can only result in higher prices. If the area is remote, every effort should be made to minimize the necessary labor on the job so that a large number of men will not have to be imported at a very high cost.

It should be obvious that most of the discussion of this chapter is concerned with moderate-sized bridges. When a truly large structure is to be built, there are many other influences which come into play and its design becomes a very special exercise. The truly monumental bridge will dominate its environment. It will cost a great deal of money and take a great deal of time to design and build.

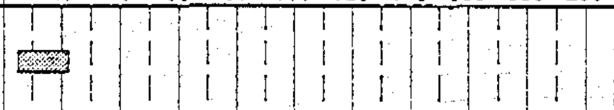
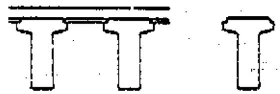
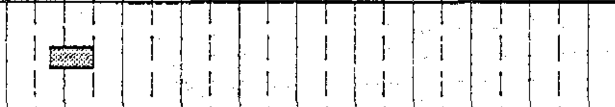
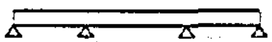
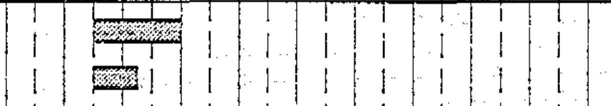
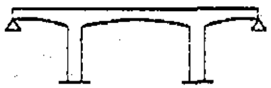
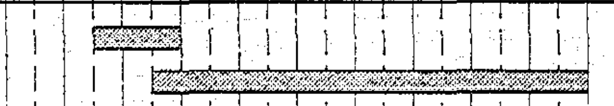
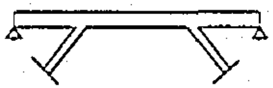
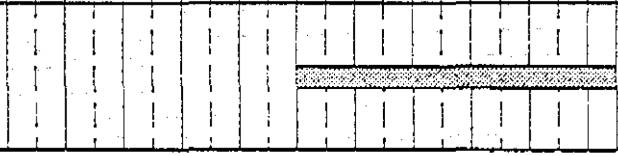
3.4.2 Superstructure

(1) Concrete structure

Concrete is a very versatile material and lends itself well to complicated configurations. It can be formed into smooth sweeping curves or the intricate details of statuary. It may be precast and prestressed (decreasing the weight) and made into large beams which may be set across long gaps without falsework supports. Or it may be cast in place and post-tensioned. Concrete is very heavy and its use ensures a high dead load factor. When poured in place, it requires forms and falsework which are often inconvenient to traffic. The size and weight of beams which may be transported on the highways is limited. This curtails plant production and often forces the use of forms and falsework which may be undesirable.

Concrete (prestressed) bridge types and their normal span range are given in Table 1.5.

Table 1.5 Prestressed Concrete Bridges and Suitable Span Range

Bridge Type	Structural Shape	Construction Method	Span Range (m) 20 40 60 80 100 120 140 160 180 200	Beam Hight -Span Ratio	Curved Structure
Pre-tensioned Hollow Slab		Crane Erection		1/24	×
Pre-tensioned I-Beam		Crane Erection		1/18	×
Post-tensioned I-Beam		Crane Erection Launching Erection		1/15	×
Continuous Hollow Slab		Fixed Staging Travelling Staging		1/22	○
Continuous Box Girder 		Fixed Staging Travelling Staging		1/20	○
		Canti-lever Erection		1/18 ~ 35	○
Rigid Frame 		Fixed Staging Canti-lever Erection		1/17 ~ 49	×
		Fixed Staging Canti-lever Erection		—	×
Cable Stayed 		Canti-lever Erection		1/40 ~ 100	×

(2) Steel structure

In the late 1960s, the fabrication of structural steel underwent some very great change. Welding took over and almost completely displaced the long-serving rivet. Welding has many quickly recognized advantages in saving weight, labor, and simplifying details – resulting in much more economical steel structures.

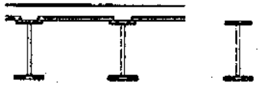
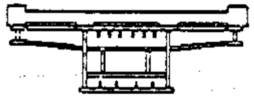
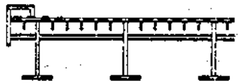
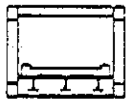
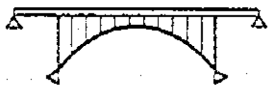
With the rise in use of welding, there has also been an increase in the use of high-strength bolts, usually for field splices. Bolted joints are often used to connect welded members. This is to simplify the field work and make the erection quicker and easier.

Welding has made the hybrid girder possible. This is a girder which combines a number of different strengths of steel to match the stress levels in a member. This can result in very trim, clean-looking girders without the changes of flange thickness throughout the span length.

Steel bridges which need to be painted to prevent corrosion are natural subjects for the use of varied colors. Bridge color should conform to the general desire for compatibility. The color or colors selected should harmonize well with the surroundings. Soft greens, tans, and browns are natural earth colors and fit well. Where the structure is to be minimized, gray colors against the sky and dark colors against deep shadows serve to swallow a structure.

Steel bridge types and their normal span range are given in Table 1.6.

Table 1.6 Steel Bridges and Suitable Span Range

Bridge Type	Structural Shape	Structural Form	Span Range (m) 20 40 60 80 100 120 140 160 180 200	Beam Hight -Span Ratio	Curved Structure
Rolled H-Beam				1/20	×
Welded Plate I-Girder		Simple Girder Continuous Girder		1/17 1/18	○
Welded Plate Box-Girder		Simple Girder Continuous Girder		1/22 1/23	○
Orthotropic-Deck Plate Girder		Simple Girder Continuous Girder		—	○
Truss		Simple Truss Continuous Truss		1/9 1/10	×
Arch		Langer Girder		1/6.5	×
		Lohse Girder		1/6.5	×
		Tied Arch		1/6.5	×
Cable Stayed				1/40 ~ 100	×

3.4.3 Substructure

(1) Abutments

Abutments, mostly made of concrete, are located at both ends of a bridge not only to support superstructure but also to withstand the earth-pressure from the back. In general, stability of abutment is secured by the combination of own weight and backfill on rear footing. As abutment become high, wall thickness will be reduced to save concrete volume in compensation for reinforcing with steel bars.

(2) Piers

Piers, mostly made of concrete, are located intermediately between both abutments to support superstructures. Type and shape of piers are selected according to the conditions of bridge site in particular of the crossing conditions.

3.4.4 Foundation

(1) General

Foundation is a structure made of concrete, steel or timber, and built into ground as a part of substructure to transmit loads from superstructure to ground. In broad meaning, surrounding ground (bearing stratum) is regarded as a part of foundation. Foundation is classified by construction method and depth as shown in Table 1.7.

(2) Spread foundation

Spread foundation is generally constructed with shallow and comparatively wide concrete footings which are supported directly on ground.

Spread footing is generally used where the bridge site meets the following ground conditions (Handbook of Civil Engineering, JSCE.):

1) Supporting ground lies shallow within about 5m from ground surface.

2) Bearing strength of the supporting ground is appraised more than 30 of N-value by standard penetration test in case of sandy soil and more than 20 for clay.

3) Such supporting ground develops in a depth more than 1.5 times the designed footing width (shorter side) under the bottom of footing.

4) When ground water level is high, draining and cutting off measurers are available.

Table 1.7 Foundation Types and Construction Depth

Foundation Type		Construction Depth (m) 10 20 30 40 50 60 70 80 90	Diameter or Side (m)	Ground Condition: Soft and Weak	Ground Condition: Intermediate Hard Layer	Ground Condition: Intermediate Gravel Layer	Ground Water: High Level	Ground Water: Artesian over 2 m from Ground	Frictional Support	Construction in Water	Batter Pile	Environmental Condition: Low Vibration & Noise	Environmental Condition: Less Effect on Neighboring Structures	Environmental Condition: Existence of Poisonous Gas
Spread Footing			> 1.50	×	○	○	△	×	×	△	—	○	○	△
Driven Piles	RC Spun Pile		0.25 ~ 0.50	○	×	×	○	○	○	△	○	×	×	○
	PC Spun Pile		0.30 ~ 1.00	○	△	×	○	○	○	△	○	×	×	○
	Steel Pipe Pile		0.30 ~ 1.50	○	○	△	○	○	○	△	○	×	△	○
	Inside Drilling Method		0.40 ~ 1.20	△	○	△	○	×	△	△	△	○	○	○
Cast-in-Place Concrete Piles	Reverse Circulation Method		0.80 ~ 1.50	○	○	×	○	×	△	△	×	○	○	○
	All Casing Method		0.80 ~ 1.50	○	○	△	○	×	△	×	△	△	○	○
	Earth Drilling Method		0.80 ~ 1.50	×	○	×	○	×	△	×	×	○	○	○
Caisson	Open Caisson		> 2.00	○	△	△	○	△	×	△	—	○	△	○
	Pneumatic Caisson		> 4.00	△	○	○	○	○	×	△	—	○	△	×
Steel Pipe Sheet Pile Foundation			> 3.00	○	○	△	○	○	×	○	—	×	△	○

○ : practicable △ : difficult × : unpracticable

5) For foundations in waterway, there is less possibility of scouring, or countermeasures against scouring are available.

(3) Pile foundation

1) General

Where spread footing cannot be found on rock, or on dense granular or stiff cohesive soils within a reasonable depth, pile foundation is the most often used. For locations where the scouring risk is high or unacceptable settlement is anticipated by the use of spread footings although soil condition would permit the use, pile foundation may also be used as a countermeasure.

2) Pile types by supporting manner

Pile foundation is classified as end-bearing, friction, or a combination of both according to the load transferring manner. End-bearing pile derives major portion of support capacity from the resistance of bearing stratum. Friction pile derives major portion of support capacity from the friction resistance along the side of the embedded pile. The bearing capacity of combination end-bearing and friction pile is derived as the sum of the resistance from the pile tip and from the friction of embedded shaft.

It is recommended to use end-bearing piles as much as possible. In case that the use of friction pile is unavoidable because bearing stratum exists deep for instance more than 60m below, the long-term consolidation settlement of the pile group should be considered in design. In this regard, friction piles are not recommended for the statically undetermined structures like continuous beam and rigid frame bridges because such structures are easily affected by uneven support settlements.

3) Pile types by material and construction method

Pile foundation is also classified by construction method as driven and bore piles. Driven piles have various kinds made of timber, precast concrete, or structural steel sections.

Timber piles are limited in length and rarely used nowadays except small, less important constructions.

Precast concrete piles, the early type of which was RC (reinforced concrete) square pile cast at field, are nowadays a factory-made product and the most commonly used for the foundations up to around 30 m in depth. The piles are reinforced with reinforcing bars or prestressing steel, centrifugally compacted to form circular cross-section, and so called commonly RC or PC spun piles. PC spun piles are more used than RC piles because PC piles are more durable against cracks and can be driven deeper than RC piles despite small cost difference between them. The market size of PC piles varies 30~80 cm in diameter.

Steel piles may be pipe or H-section. Steel pile piles offer higher resistance against driving impact and accordingly can be penetrated deeper than PC piles. The pile, for they are expensive, are generally used for where PC piles cannot be constructed because bearing stratum is deep or hard intermediate layers exist. For corrosion of steel, the margin of 0.02 mm/year is generally considered to steel thickness under normal environment.

Steel H-piles are conveniently used for temporary construction and often pulled out to re-use several times.

Bore piles are constructed by placing reinforcing steel cage and concrete into pre-drilled holes. According to the soil condition and the desired pile depth, the methods of drilling and maintaining hole are selected. Generally either wet or dry drilling, water or slurry, temporary or permanent metal casing will be used as necessary to produce sound concrete foundation shafts free of defects. Bore piles are advantageous in urban and neighboring construction owing to their characteristics of less noise and vibration compared to the driven piles, but the method needs skilled techniques and high quality control for the complexity of construction operation. Bore piles will be studied as an alternative to steel pile piles for planning deep foundation.

4) Batter piles

Where the lateral resistance of soil is considered not to be adequate against horizontal loads, or when increased rigidity of the entire structure is required, batter piles are often used to save piles. However, batter piles are not recommended where settlement of compressible soil and so negative skin friction loads are expected, and for bore piles due to their difficulty of construction except all casing method. Instead, it is recommended to increase the number or the diameter of piles.

5) Spacing of piles

The following minimum center-to-center pile spacings are recommended to decide the size of foundation.

For end-bearing piles:	2.5 times pile diameter/width.
For friction piles :	3.0 times pile diameter/width preferably, but not less than 2.5 times.

3.5 Preliminary Cost Estimate and Construction Plan

3.5.1 Preliminary Cost Estimate

(1) General

Cost estimating is a procedure to break down an object of work into its component parts such as number of spans, substructures, or a cubic meter of concrete thereby making each part more sensitive to accurate measurement of quantity and estimate of its cost.

For a government department (JKR), the cost of a bridge project will include the costs of administration, surveys, design, right-of-way acquisition, construction, supervision, and financing. For a consulting engineer, the cost will depend on its definition in his contract with the client. For a contractor, the cost will include the actual cost of construction to which an allowance for contingencies and profit is added to produce the contractor's bid.

Cost estimate may be classified by purpose as:

- Preliminary estimate,
- Comparative estimate, and
- Detailed contractor type estimate.

The preliminary estimate will be performed at the planning stage to establish the viability of a project and to earmark funds. The comparative estimate serves at the design stage of the project development and it is used for the engineer's estimate. The detailed or the contractor type estimate is used by contractors. The more detailed and accurate the estimate, the more costly and time consuming the estimation process becomes.

(2) Preliminary cost estimates

These estimates are the first made for a project in the planning stage and are used to bracket the probable cost within a rather wide range. They must be based on limited and only general definition as to scope and detail. For example, a bridge plan would be compared as to terrain, length, height, width, soil condition, etc. with the previous projects having similar characteristics, from which an approximation of quantities and cost could be made. This level of estimation may be prepared for the accuracy within 20~30 percent of final cost.

However, if additional accuracy is required for more detailed cost comparison of alternative plans or financing plan by the owner, the simplified unit cost method is recommended. The method involves the use of unit prices from previous projects applied to the quantities for a new project by grouping cost items into several major items. Here the quantities for the new project are known (at least approximately) and the previous unit costs are taken from a project or projects selected because of their general similarity of cost-controlling conditions, where a certain degree of accuracy can be obtained without extensive adjustments or detailed analyses. The unit costs for estimating bridge project cost are commonly grouped into the following major items:

Cost Item	Quantity Measurement Unit
1) Superstructure	
- Pavement (include curbs and rails)	surface area (m^2)
- Expansion joints	size and length (m)
- Main beams (include erection work and diaphragms)	
PC precast beam	type, size, and number of beams from standard design (pieces)
PC box girder	concrete volume per bridge surface area from previous project data (m^3/m^2)
Steel girder	steel weight per bridge surface area from previous project data (ton/m^2)
- Deck slab	concrete volume (m^3)
- Bearing shoes	capacity and number of shoes (pieces)
2) Substructure	
- Abutments and piers	concrete volume (m^3)
- Excavation and temporary cofferdam	excavation volume (m^3)
- Foundation	
Spread footing	be included in abutment and pier concrete volume.
Piles	type, size, depth, and number of piles (total pile length : m)

3) Approach road

- Road earth-fills earth volume (m^3)
- Pavement surface area (m^2)

4) Other works

- Temporary staging surface area (m^2)

- Traffic detour-way length or surface area (m or m^2)

- Bank protection surface area (m^2)

The unit costs used in this method are considered to include all direct, indirect, and corporate indirect costs. With this method, accuracy is expected to be within 10~20 percent range.

Significant error may occur if unusual construction techniques and temporary facilities are involved and are not properly compensated for in the unit costs applied. This is a reason why the preliminary cost is sometimes far below the contractor's bid cost.

3.5.2 Preliminary Construction Plan

(1) Objectives of planning

The main objectives of construction plan are:

1) to envisage how the construction will be carried out, in what order and with what method and resources; reducing the construction to a number of manageable activities.

2) to anticipate potential difficulties and risks to overcome them, so that their effects can be minimized. This is the major objective of construction planning, because civil engineering is a high risk business and the planning is fraught with uncertainty.

3) to schedule resources (men, equipment, materials and money) to enable optimum use.

4) to provide a basis for predicting and controlling time and cost.

(2) Planning process

Planning is the mental activity of working out what has to be done, how, by when, by whom, and with what. Planning techniques assist in the analysis of plan organizing information, and in which the plan is communicated to others. Taken together these two elements of planning produce the plan a strategy and tactics for the execution of the project in terms of activities, time, quantities, resources, and perhaps costs. The plan is expressed as charts and reports.

Planning depends on data. Without reliable data, planning can only process best guesses. As each construction project is different, all construction projects are a learning process and this learning process enables the plan more accurate as the project progresses. New data can be used to refine or revise the plan.

(3) Planning hierarchy

Before starting to prepare any plan, it is vital to decide who the plan is for and what level of detail is required. Table 1.8 lists the people who may require or prepare a construction plan, summarizes what they will need to know, and gives appropriate time–scales.

(4) Project duration

There are two ways to determine how long the project will take:

1) It is imposed by external considerations of the time available, and designer or contractor then has to devise a plan to meet this requirement, or

2) It is built up from a detailed analysis of the work to be done and the resources available, using estimates of the time required for each activity.

The examples of externally imposed considerations are: (1) it is difficult to attempt building piers in deep waterway in high water season, (2) asphalt paving work will be avoided in rainy season, or (3) the client may often have an economic or administrative need for fast construction, or require a project to be constructed in stages for budgetary reason.

The examples of detailed analysis of the work and the resources are: (1) the output of a construction operation is determined by the capacity of a key plant or equipment or by the work sequences; this is most common in bridge construction, or (2) it is also common that contractors tend to assess the combination of resources most likely to complete the work at minimum direct cost; then the duration of the operation is calculated from the volume of work in this way.

(5) Planning tools

Four techniques are commonly used in construction planning: bar chart, line–of–balance, linear programme, and network analysis.

Table 1.8 Planning Hierarchy

	Plan drawn up		Primary	Scope of plan	Scope of	Time-	Unit	Level of
	For	By	purpose of plan		programme	scale		detail
Clients	Government department	Project director	Administrative planning	A project overview from identification of need through feasibility study, preliminary design, detail design, land acquisition, and construction period.	Outline project programme	Entire project	Month	Low
Clients	Government department	Project director	Financial planning	A project overview from project conception to implementation including appointment of consultant and contractor, design and documentation periods, and construction.	Outline project programme	Entire project	Month	Low
Managers	Project manager	Project manager	Co-ordination of design and construction	Design period, documentation, letting contracts, and construction period.	Outline and broad details	Project design and construction	Week	Low/medium
Managers	Contractor	Staff	Tender plan	All activities within construction period in sufficient detail to enable contractor to prepare the tender.	Construction programme	Construction period	Week	Medium
Managers	Contractor's or Engineer's Representatives	Staff	Resource planning	Every activity, major items of plant, dates of key material deliveries and site movement.	Short-term programme	Months	Day	Medium/high
Managers	Foreman	Staff	Disposition of of plant and manpower	Every operation with the actual plant, manpower and supervisors employed.	Weekly programme	Weeks	Half-day	High

Bar Chart

The bar chart is easy to draw, easy to understand, and best used for straightforward, well-understood construction work with simple relationships between the activities. Main disadvantages are that it neither show relationships between activities nor relate activities to location.

Line-of-balance

Line-of-balance was derived from manufacturing industry, and has been found to be effective in planning work that is truly repetitive. Line-of-balance has been found to be difficult to use on projects which require a large number of operations to construct each identical unit. The problems arise from the difficulty of showing all the information on one chart, especially when using the technique to monitor progress. However, it is an excellent means of relating resources, activity durations and the general pace of work.

Linear programme (or time-chainage chart)

Linear programming is a specialized technique for linear work. This is a basic tool of the construction of a large canal and it is especially useful in tunnelling. Like line-of-balance, this is a simple two-dimensional graphical technique and can show clearly only a limited amount of information and a limited degree of complexity.

Network analysis

Network analysis is a logical and analytical technique. It is most effective when used for complicated projects, especially those with external constraints and complex interrelationships. The technique is based on drawing the logical relationships between construction operations, and establishing which operations have the most crucial effect on the project duration. The technique is known as the critical path method (CPM), and a version which incorporates a statistical method for calculating the probability that a project will be completed on a specific date is called the programme evaluation and review technique (PERT). Network analysis has a good and comprehensive logical basis, lends itself easily to computer processing, and can be used as an effective control tool.

It is of fundamental importance to note that the level of detail of the plan and the choice of technique are related. For example, the overall programme for a large and complex project should be drawn by a network. However, for an simple activity "piling work", obviously network analysis does not work well. Instead line-of-balance programme should be used.

(6) Planning components

Planning tools aim to express the work to be done to a time-scale; some also include resources and perhaps cost. The other major factor in construction control is "quality" which is undertaken by separate techniques from the planning tools. However, quality is related to time and cost through the skill and judgment of construction management.

The major components of planning techniques are:

- activities: this means a work to be done for example preparing a drawing, materials to be ordered, piles to be driven, or concrete to be placed.

- activity durations: the time required for the completion of each activity.

- project time-scale: the time structure of the project; it is usual to give each week or month a number (this makes calculation easier).

- event: an occurrence at a specific point in time; for example, the start and end of construction.

- work method: the plan must be expressed in some logical way, indicating the sequence of operations, and which activities and events are interrelated; this may be implicit (as with bar charts) or explicit (in network analysis, where work method is usually called logic).

- resources: generally include men, machines, materials, and money, and even such essentials as managerial skill.

- costs: what the work has or will cost, often derived directly from the unit costs of the individual resources.

3.6 Environmental Impact and Aesthetic Consideration

3.6.1 General

The term "environment" is meant to be interpreted broadly as the whole complex of physical, social, cultural, economic, and aesthetic factors which affect individuals and communities and ultimately determine their form, character, relationship, and survival. The definition "environmental impact" is any alteration of environmental conditions or creation of a new set of environmental conditions, adverse or beneficial, caused or induced by the action or set of actions under consideration. The attention given to environmental conditions will vary according to the nature, scale, and location of the proposed action or actions. Attention would be given to those factors most evidently affected, such as the effects on the resource base, including land, water quality and quantity, air quality, public services and energy supply, as well as other environmentally critical areas.

Generally, impacts can be categorized as either primary or secondary. This distinction is important for consideration of alternatives and ways to minimize adverse impacts in performing impact analysis. One way to describe the distinction is that project "inputs" generally cause primary impacts and project "outputs" generally cause secondary impacts. Primary impacts are generally easier to analyze and measure, while secondary impacts are usually more difficult to measure. Secondary impacts may, in fact, be more significant than primary impacts.

3.6.2 Impacts of Highway and Bridge Project

(1) General

Bridges are a part of highway, accordingly the environmental impacts of the bridge construction should be discussed in the environmental problem of the highway construction.

Highway construction has impacts in a number of areas, the most noteworthy of which are aesthetics, air quality, circulation and traffic patterns, noise, socioeconomics, water quality, and wildlife. Highway may stimulate or induce other actions (secondary impacts), such as more rapid land development or changed patterns of social and economic activities. Impacts associated with secondary action may often be even more substantial than the primary impacts associated with construction. For example, the effect on population and area growth associated with the construction of new highways may be among the more significant impacts.

(2) Aesthetics impacts

Of general concern relative to aesthetics are such impacts as: (1) blocking viewlines along visual corridors (such as valleys, stream courses, and streets); (2) blocking viewlines to landmarks in the community from residential, recreation, and commercial areas that benefit from view; (3) bridges or elevated highway out of scale with adjacent urban development; (4) visual distraction and displeasing glare visible in recreational and residential areas; and (5) unattractive contrast between existing vegetation and revegetated or landscaped areas, between natural landforms and engineering features, and between urban or existing development patterns and highway features.

(3) Air quality impacts

Air quality impacts include: (1) dust and/or particulate matter on vegetation and structures surrounding the construction site or along roads: (2) tire and exhaust particles coating roadside vegetation and structures; (3) increase in severity of existing smog conditions due to an increase in automobiles traveling through the area; and (4) generation of vehicle fumes and odors (such as from exhaust emissions, or tire and brake rubber).

(4) Noise impacts

Noise impacts generally involve the area within sound of the traffic such as:

(1) disturbance of surrounding passive recreational activities requiring quiet and serene conditions for their enjoyment; (2) disturbance of educational, health care, and cultural activities or institutions particularly sensitive to noise, such as schools, churches, hospitals, sanitariums, auditoriums, and theaters; (3) disturbance to operation or patronage of commercial activities requiring or benefiting from quiet surroundings; and (4) disturbance to surrounding residential development.

(5) Socioeconomic impacts

Socioeconomic impacts include: (1) removal of residential, commercial, and industrial land uses and displacement of both residents and jobs; (2) removal of structures or sites of scenic, architectural, archaeological, or historic significance; (3) loss of site having unique potential or suitability for commercial or industrial activities; (4) loss of taxable private land revenues; (5) relocation costs to displaced residents greater than compensation paid; and (6) severance of interpersonal ties of displaced residents to former neighborhood/community (family ties, ethnic bonds, or neighborhood friendships).

(6) Water quality impacts

Water quality impacts involve one or more of the following: (1) turbidity and silting of adjacent streams and reservoirs caused, for the most part, by the erosion of the raw soils exposed during construction and maintenance operations (the primary impact of these effects generally involve increased operating costs or shortened life of affected reservoirs and channels; damage or elimination of fish and other aquatic life; and possible damage to buildings, roads, and bridge foundations); (2) watershed modification caused by the impingement of the road system and its construction on estuaries, marshes, wooded swamps, and streams – in particular, in estuaries disturbance of natural flows can affect ecological determinants such as sedimentation patterns, mixing of fresh and salt waters, nutrient flows, shellfish beds, fish and wildlife, and local vegetative patterns; (3) highway runoff contamination caused by runoff containing oil, fuel, tar, pesticides, fertilizer, deicing salts, animal and human wasted, and the products of combustion which can affect water quality, wildlife, and roadside vegetation; (4) sanitary wastes from temporary and permanent waste disposal facilities (Note: Waste disposal is accomplished through portable toilets during construction and permanent rest areas after construction); in either case, raw or inadequately treated discharges can have an impact on local water systems; and (5) contamination of surface and ground water supplies and recharge areas by polluted fill material, where the use of polluted fill material can affect the concentrations of biological, physical, chemical, and radiological contaminants in water supplies.

(7) Wildlife impacts

Wildlife impacts would generally include: (1) loss or degradation of unique or highly productive wildlife, fish or shellfish habitats; (2) division of wildlife ranges and migratory patterns; (3) displacement of wildlife to other ranges; (4) impairing or blocking migration and/or movement of aquatic biota; and (5) visual disturbance of wildlife on adjoining lands.

(8) Circulation impacts

Circulation impacts include: (1) blocking or impairing access along existing street patterns crossed by the highway, such as access to public and private services of residents and patrons within the service area, reinforcing or creating physical barriers between social groups, congesting through-street traffic by diverting traffic from dead-end or rerouted streets, and disrupting public transit routes; (2) dividing single land uses or resource areas such as agricultural operations, recreation areas, wildlife ranges or habitats; (3) increasing truck and construction equipment traffic on public roads during construction; (4) providing new or improved access to previously inaccessible or relatively inaccessible public and private lands; (5) providing or improving access to relatively undeveloped areas outside urban centers, thus inducing commercial and industrial operations to locate outside urban centers; and (6) increasing traffic traveling through the area and thus causing an increased demand for travel related services.

3.6.3 Aesthetic Consideration

(1) General

A bridge should never usurp its role as a part of the highway. It should always look as though it thoroughly belonged to the rest of the highway. It should be completely compatible with its surroundings. The bridge belongs in that location.

Giving a bridge a retiring personality is not always easy. A designer must subdue any impulse to make his structure stand out as a monument to his design prowess. A bridge which stands out when it should be merely a part of the highway often develops into an eyesore. The designer must be careful lest he be carried away by transient public fads.

Designs, which rely for their beauty on good proportions, clean lines, and an honest approach to function, have worn better with time. So will the structure built in the future if they have basic excellence built into them: good balance, trim proportions, clean lines, interesting but not fancy forms. The bridge designer should never forget the projected 100-year life for his structure. If he wants his critics of eight or ten decades hence to admire his work, he had better make it a basically excellent design. He must walk the fine line between uninteresting starkness and overdone decoration.

High fills have blocked the view of many attractive canyons, when the beauty of many deep canyons would be enhanced by the slim framework of a well-designed bridge. Fills and culverts have their place when they will not destroy natural beauty. But, where there is such beauty, consider a bridge rather than a culvert and fill. As a practical matter, the apparent economy of a culvert at the base of a huge fill may prove false if debris plugs the culvert and the water backs up and floods the country above the road. Such short-sighted economy can generate lawsuits.

(2) Aesthetics of structures

It is important that the bridges be completely compatible with their surroundings. They should look like they belonged there. As soon as they have had a chance to weather a little, they should look like they had always been there a completely natural and acceptable part of the landscape. This often means that attention should be given to the characteristics of prominent buildings nearby or other landmarks with which the bridge must fit. Compatibility is the key.

Beauty in a structure comes from its basic design. Beauty starts with the first concepts of the structure and from there proportion, form, and general design must follow to achieve a pleasing result. Beauty in this context, coming from well-proportioned, carefully planned design, is not expensive. Even though the owner may not wish to spend any extra money for aesthetics, the designer still may create a good-looking structure by his careful handling of proportion, shape, and light and shadow.

(3) Number of spans

The number of spans is the first important decision that has critical aesthetic effects. Unless a structure is part of a grade-separation where the span arrangement is dictated by the intersecting roadways, the number of spans to be used in a bridge is one of the first determinations for the designer to make. If the structures is easily visible as a whole, an odd number of spans will be found to be more pleasing than an even number.

(4) Balance of span and height

The length of spans depends upon the length of the bridge and its height above ground. The ground, the piers, and the bridge deck create a series of generally rectangular shapes. If the piers are close together, these rectangles may be higher than they are wide and look crowded. The arrangement is generally better when the rectangular areas are longer in a horizontal direction – generally, the longer the better. The cost of longer spans is a factor, so the designer must balance the aesthetic tastes against the money available to build the bridge.

Structures which are high above the ground look thinner and more graceful. It is possible to have a deep, heavy structure that looks delicate and graceful if it is high enough above the ground. For structures which cannot be high in the air, every effort must be made to make the superstructure as thin as possible or at least to create an illusion of thinness. Concrete superstructures can be made thinner by prestressing. Rounding the corners, sloping the outside faces of girders, building a longitudinal ledge to develop a shadow area on the lower part of the girder – all of these are useful tools in creating the illusion of a thin superstructure.

It is often difficult to design a thin steel superstructure economically for a reasonably low short span. Shallow steel girders become uneconomically heavy. Problems of erection or traffic may override cost to make short steel spans desirable. Some steel box designs produce attractive, thin superstructures for long spans.

(5) Artistic techniques

1) An attractive structure is produced by the harmony of all of its elements. The use of a number of details which are attractive in themselves does not guarantee a pleasing overall effect. This is what makes aesthetics an art rather than a science. The satisfactory end result depends upon the taste and innate ability of the designer to combine all of the details artistically.

2) There are some details which may help the overall effect. Sloping the exposed area of the front faces of the abutments in toward the roadway will produce a pleasing dynamic effect in a short structure which seems to launch it across its span. Tapering the piers so they are smaller at the bottom than at the top will tend to decrease the feeling of heavy attachment to the ground and make the structure appear to float. On the other hand, for very tall piers, tapering from the ground to a thinner neck high in the air seems to release the superstructure from its solid ties with the ground, and this also makes it appear to float. Modern highways are avenues of speed and flowing movement. The bridges should carry out that feeling, emphasizing the horizontal lines and playing down the verticals which tend to interrupt the flow.

3) Piers have been made round, square, or rectangular. There are many interesting shapes which can be developed out of these basic forms which, with some of the tapering effects, can add considerable interest.

4) Attractive designs are not necessarily fancy designs. There is a strong appeal to very simple plain shapes and forms. As noted earlier, there is also an economic appeal in that they are cheaper to build. Contractors are notoriously unaesthetic when they come to build forms or fabricate shapes which vary from the straight, square, and simple. Nevertheless, fancy shapes do cost more money and the designer should be constantly

seeking the plain, easier-to-build solutions.

5) It cannot be stated too often that the successful and artistic bridge must fit its site, and be completely compatible with its surroundings. It is imperative that the designer be familiar with the site and have a feeling for the environment of his structure. There have been examples of designs which were totally unsuited for their locations because the designer did not take the trouble to visit the site and see just how and where his structure would fit into the landscape.

References

1. Japan Society of Civil Engineering, "Handbook of Civil Engineering, 4th Edition.", Giho-do, 1989

2. Japan Highway Public Corporation, "Design Manual, Vol. II." 1983

3. Japan River Associations, "Commentary of River Works Ordinance", Sankai-do, 1978

4. Japan River Associations, supervised by River Bureau, Ministry of Construction, "Manual (draft) for River Works in Japan - Planning." Sankai-do, 1985

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8. Keretapi Tanah Melayu Berhad., "Permohonan Laluan - laluan Dalam Rezab KTM Berhad." November 1994

DIVISION II

MANUAL FOR BRIDGE ANALYSIS

DIVISION II MANUAL FOR BRIDGE DESIGN ANALYSIS

CONTENTS

CHAPTER 3 ANALYSIS FOR SUBSTRUCTURE

DIVISION II : MANUAL FOR BRIDGE DESIGN ANALYSIS

CHAPTER 1 General

1.1 Introduction

This chapter describes the basic concept of the bridge structural analysis and procedures of a design program regarding the standard design produced by "the Study on the Standardization of Bridge Design in Malaysia". One of the most important objectives of the manual is to allow JKR engineers to do design by themselves after the Study. In future, this manual would be useful tool for the engineers .

The manual is divided into superstructure in Chapter 2 and substructure in Chapter 3. Each Chapter starts with the notations used for analysis and it is followed by the design conditions to specify the characteristics of materials and loads. Then, analysis procedures are explained for the designs of bridge structure.

1.2 Outline of Bridge Structural Analysis System

The analysis system is shown in the Fig.2.1, 2.2 and 2.3 on the following pages. A full automatic design system, with personal computer, is applied for superstructure, whereas a partially computerized system is used for substructure design.

"Bridge Structural Analysis" is the techniques required to assess the adequacy of the structural strength or resistance of the bridge under the reasonably conceived severe loads. The analysis techniques are entirely based on the theories and formulas stipulated in the authorized publications.

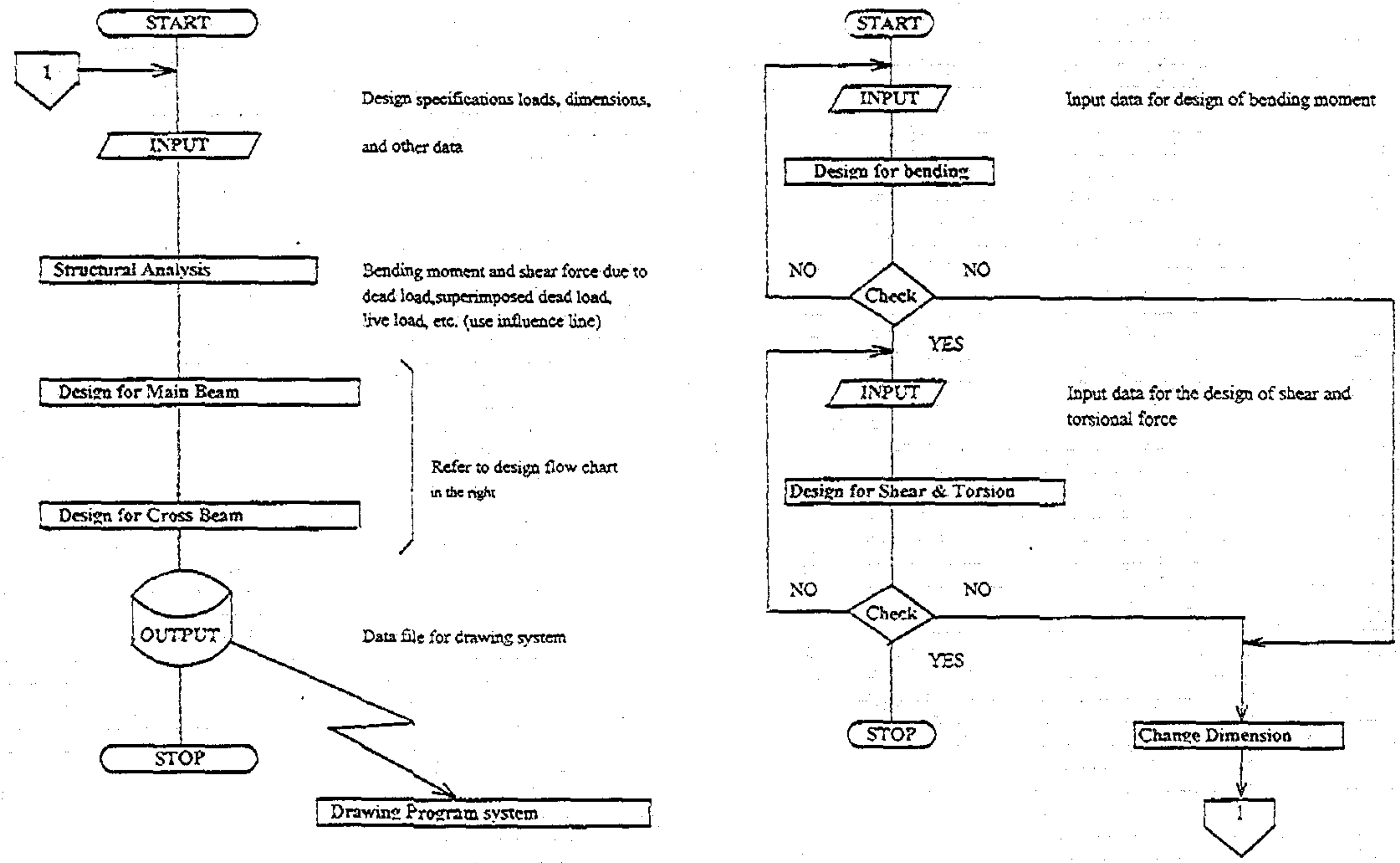


Fig. 2.1 Bridge Structural Analysis System

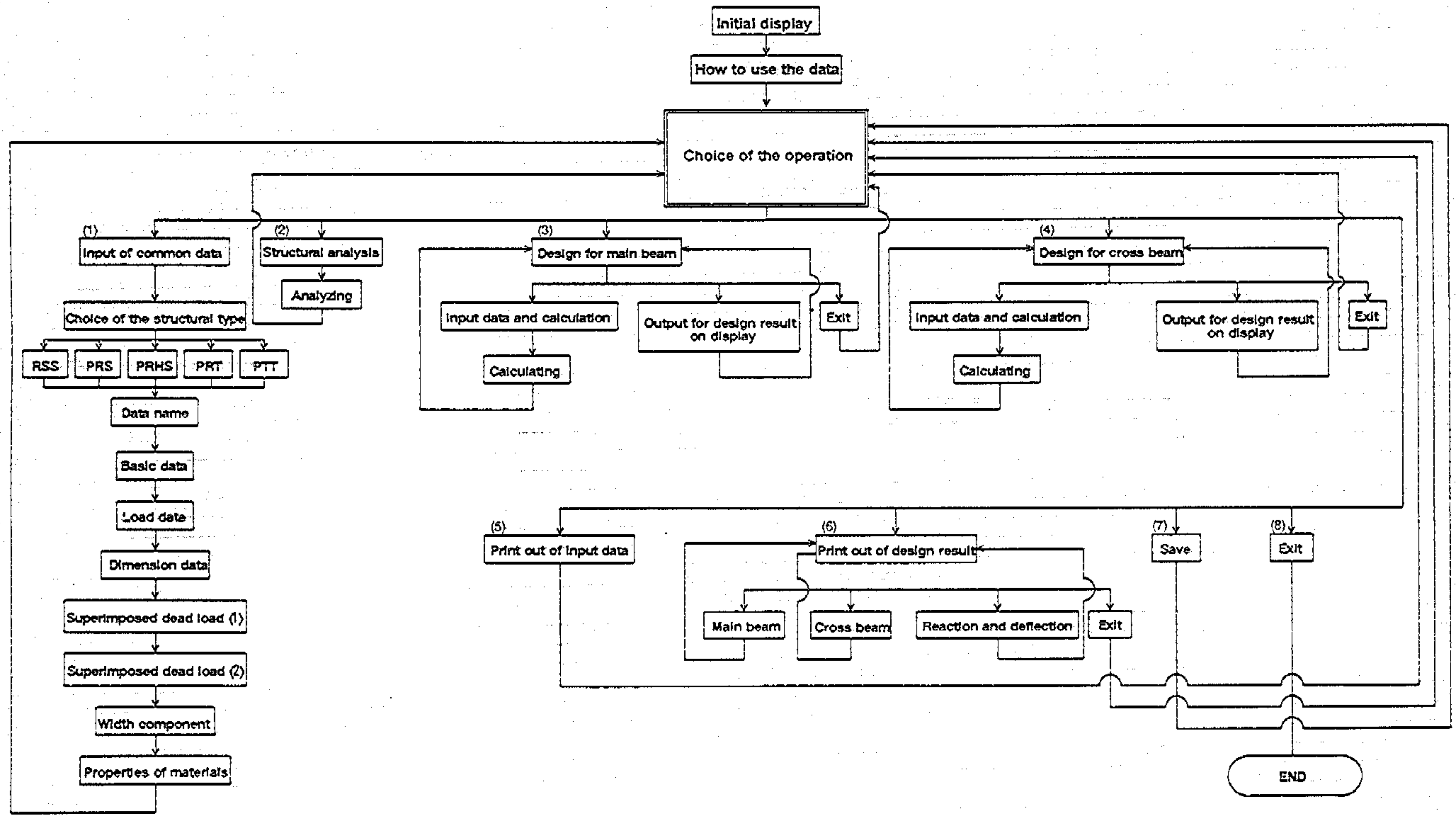


Fig. 2.2 Flowchart for Superstructure Design Programme

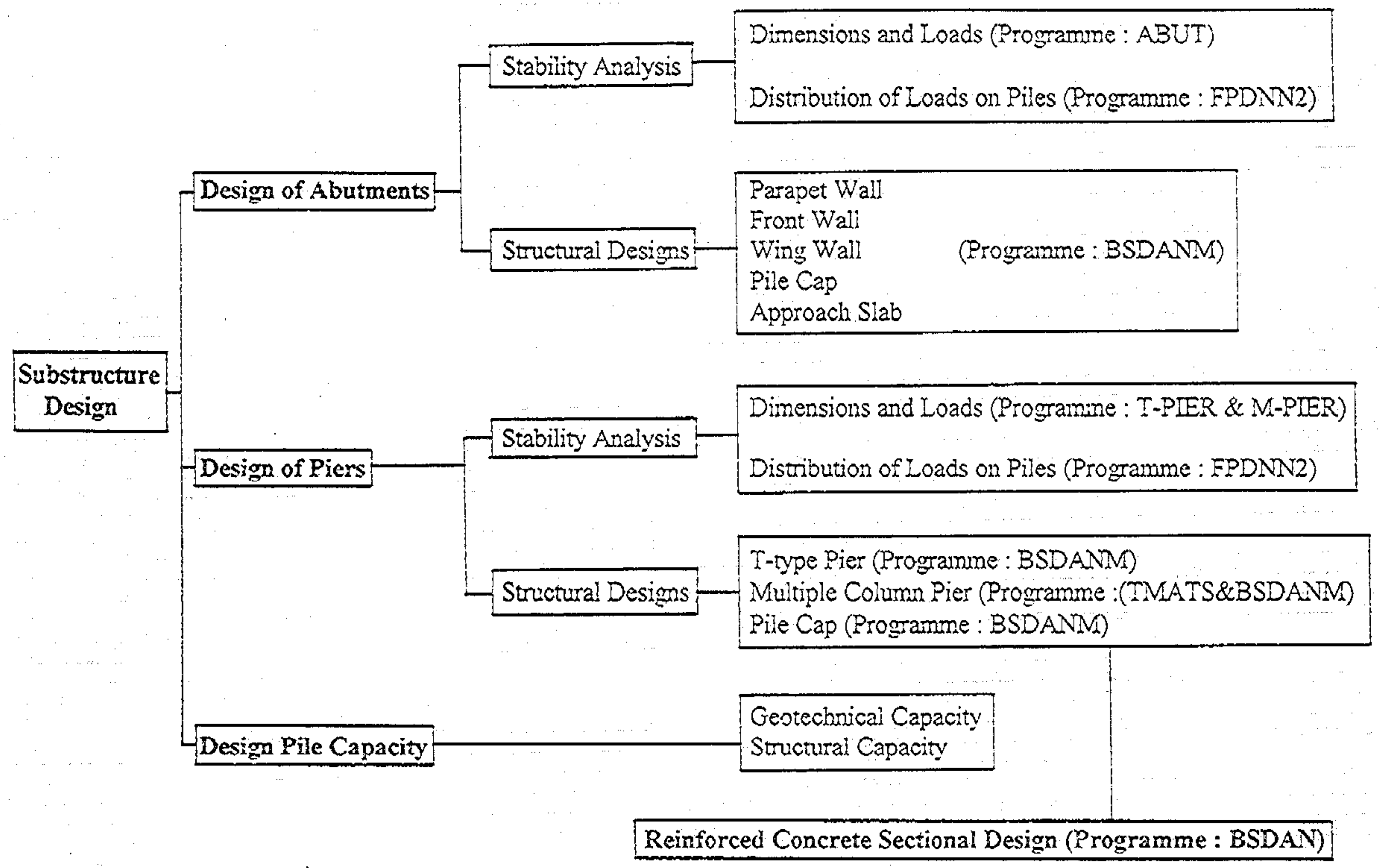


Fig.2.3 Flowchart for Substructure Design Programme

1.3 Design Standard

In accordance with JKR's request, the following British standards were adopted to the main parts of the analysis:

(1) BS 5400

Part 1	:	1988 - General Statement
Part 2	:	1978 - Specification for Loads
Part 4	:	1990 - Code of Practice for design of concrete bridges
Part 7	:	1978 - Specification for materials, concrete reinforcement and prestressing tendons
Part 8	:	1978 - Specification for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 9	:	1983 - Bridge Bearings

(2) BD 37/38 : 1988

Revision of BS 5400 - part 2 : 1978 - Specification for Loads

(3) BS 8004

Code of Practice for foundations

(4) BS 8110 - Structural use of concrete

Part 1	:	1985 - Code of Practice for design and construction
Part 2	:	1985 - Code of Practice for special circumstances

In addition, the following Japanese standards are also adopted in instances whereby it is impossible to comply with the above mentioned standards.

- Specifications for Highway Bridges (Japan Road Association)

Part I	:	1994 - General Specifications
Part II	:	1994 - Specifications for Concrete Bridges
Part III	:	1994 - Specifications for Substructure

1.4 How to Use the Manual

(1) General Steps

The analysis step usually starts with setting design conditions first, the analysis for main frame or major members of bridge in the next, and then that for minor members follows. Since manual is prepared along with this steps, it is recommended to study the steps that described in the manual.

(2) Numerical Explanation

All analysis in the manual are demonstrated by sample computation to enable to trace them numerically.

(3) Use of Computer Programs

Throughout the manual, many computer programs are prepared to save time and labour for computations. The analysis using computer programs display only input and output data. To see the theories and foumulas on which the programs are based, it is required to refer to the design standards invidivually with the references given in the manual. However, such theories and foumulas as considered ambiguous to apply are described in the each section of the manual.

(4) Pursuit of Subtle Input and Output Data

The goal of bridge analysis is to decide sizes and reinforcement of bridge members with appropriate strength against the loads set up by design conditions. At first, the sizes and reinforcement shall be assumed from the past similar design results to prepare the initial input data, and then those are to be put into analysis to obtain the output data. That is, effects of the design loads in various forms such as overturning moments, displacemnts, stresses, strength, etc.

If the output data exceed or come far below the limitations imposed by the materials and the design conditions, the input data shall be modified repeatedly until the output data become proportioned to the limitations to conclude the sizes and reinforcement of bridge members finally.

It is, however, noted that bridge structure shall not be designed only by structural analysis but also take account of structural compatibility and harmony between bridge members, aesthetic appearance and construction easiness. In this regard, strength redundancies of bridge members are generally unavoidable in bridge structural design except at critical sections.

The design manual is quite sufficient to assist a designing of a new bridge, and it is recommendable to use the design result which have been made in the Study as the standard design.

1.5 Limitation in use

1.5.1 Superstructure

The superstructure type dealing in this manual is limited to the following five types with recommendable span ranges:

	Name of Type	Span Length (m)
- RCSS :	Reinforced Concrete Solid Slab	6 ~ 10
- PRSS :	Pre-tensioned Concrete Solid Slab	6 ~ 10
- PRHS :	Pre-tensioned Concrete Hollow Slab	10 ~ 16
- PRT :	Pre-tensioned Concrete Composite T-Beam	16 ~ 22
- PTT :	Post-tensioned Concrete Composite T-Beam	22 ~ 45

Table 2.1 Type of Superstructure

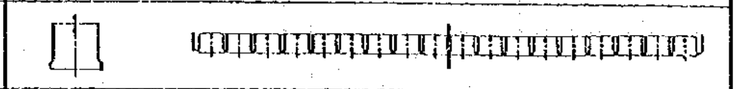
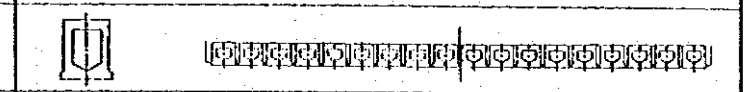
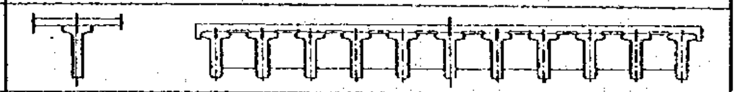
Beam Type	Span Range	Cross Section
RCSS	6m - 10m	
PRSS	6m - 10m	
PRHS	12m - 20m	
PRT	18m - 22m	
PTT	25m - 45m	

Table 2.2 Span Range of Standard Beams

Type	5m	10m	15m	20m	25m	30m	35m	40m	45m
Reinforced Concrete Solid Slab									
Pre-tensioned Concrete Solid Slab									
Pre-tensioned Concrete Hollow Slab									
Pre-tensioned Concrete T-beam									
Post-tensioned Concrete T-beam									

Even though the use of the standard design is limited as mentioned above, the automatic design analysis program itself has a wide range of applications. The main application and restriction for the program are as follows:

- A span length can be freely selected up to 45m span length. However, it is quite important to be careful when deciding on its application for the span range shorter or longer than the standard design, because such application may cost more, or the fabrication and construction of a beam may become technically difficult.

- A width component can be freely selected, and a footway can also be designed.
- The dimension is fixed to the structural types, thus an optional dimension cannot be applicable.
- An angle of skew can be freely selected if not exceeding 30 degrees.

1.5.2 Substructure

The substructure type dealing in this manual is limited to the following types.

Table 2.3 Type of Substructure

(Number of Cases)

(a) Standard Design					
Type		Span x	Foundation x	Skew =	Total
Abutment	H = 6m	5	2	2	20
	H = 8m	2	2	2	8
	H = 10m	2	2	2	8
				subtotal	36
T-type Pier	H = 10m	6	2	2	24
	H = 15m	2	2	2	8
	H = 20m	2	2	2	8
				subtotal	40
Multiple Column Pier	H = 10m	6	2	2	24
	H = 15m	2	2	2	8
	H = 20m	2	2	2	8
				subtotal	40
				Total	116

(b) ample Design	(Number of Cases)
Spread foundation for 8m high Inverted-T Abutment	1
High Inverted-T Abutment (12m)	1
High T type Pier (30m) with 3 types of Foundation	3
Bored Pile Foundation for Multiple Column Pier	1
Total	6

However, the pile foundation (PC spun pile, diameter 0.6m) proposed in the Standard Design is only a sample design. Although the pile was selected from its recent popular use and the ground condition was assumed to represent the typical geological feature in Malaysia, the Standard Design can not answer every local problems. In general, for the design of foundation, the design parameters should be decided individually according to each bridge construction site, and the Standard Design will help the design process as a reference.

CHAPTER 2 Analysis for Superstructure

2.1 Explanation of Notations

The notations used are basically the same with BS5400, so only important and new ones used in this Manual and the calculation sheets are in the following.

(1) Notations for Load

D	:	Dead load except superimposed dead load
D1	:	Main beam load
D2	:	Cross beam load
D3	:	slab load only for composite T- beam
SD1	:	Superimposed dead load except premix dead load
SD2	:	Premix dead load
HA	:	Normal live load
HA+HB	:	Combined load with abnormal live load 30 units
HB*	:	Abnormal live load 45 units

(2) Notations for Loading Condition

Stage 1	:	under (D1)
Stage 2	:	under (D1+D2+D3)
Stage 3	:	under (D1+D2+D3+SD1+SD2)
Stage 4	:	under Serviceability Limit States
Lc.1	:	Load combination with permanent load and HA
Lc.2	:	Load combination with permanent load and (HA+HB)
Lc.3	:	Load combination with permanent load and HB*
S.L.S.	:	Serviceability Limit States
U.L.S.	:	Ultimate Limit States

(3) Notations for Sectional Force

M	:	Bending moment
S	:	Shear force
T	:	Torsional moment

(4) Notations for Prestress

fpti	:	Initial prestress
fpt1	:	Prestress due to (fpti + instanteneous deformation)
fpt2	:	Prestress at the immediately after anchoring
fpth	:	Horizontal prestress at the immediately after anchoring
fptv	:	Vertical prestress at the immediately after anchoring
fcpt	:	Concrete stress due to (fpt) at the level of PC tendon
fcp1	:	Concrete stress due to (D1+fpt) at the level of PC tendon

fcp2	:	Concrete stress due to (D2+D3+SD1+SD2) at the level of PC tendon
fcpg1	:	Concrete stress due to D1 at the level of PC tendon
fcpg2	:	Concrete stress due to (D2+D3) at the level of PC tendon
fcpg3	:	Concrete stress due to (SD1+SD2) at the level of PC tendon
Loss1	:	Loss of prestress due to creep and shrinkage at the composite time
Loss2	:	(Loss of prestress due to creep and shrinkage at the end time for creep) - (Loss1)
Loss(relax)	:	Loss of prestress due to relaxation of PC tendon
fpe(comp.)	:	Effective prestress at the composite time
fpe(end)	:	Effective prestress at the end time for creep
Fai(comp.)	:	Creep coefficient at the composite time
Fai(end)	:	Creep coefficient at the end time for creep
Shr.(comp.)	:	Shrinkage at the composite time
Shr.(end)	:	Shrinkage at the end time for shrinkage

(5) Notation for concrete stress

Cr-diff	:	Concrete stress due to creep difference for composite beam
Shr-diff	:	Concrete stress due to shrinkage difference for composite beam
Limit comp.	:	Allowable compressive stresses
Tens1	:	Allowable tensile stresses at the immediately after anchoring
Tens2	:	Allowable tensile stresses under S.L.S.

(6) Notations for location of cross section

(bu)	:	Upper fibre of beam
(bl)	:	Lower fibre of beam
(su)	:	Upper fibre of slab
(bw)	:	web of beam
(bf)	:	flange of beam
(s)	:	slab

(7) Notations for dimension

H	:	Beam height
Yu	:	Distance of the upper fibre from the centroid of the concrete section
Yl	:	Distance of the lower fibre from the centroid of the concrete section
Yp	:	Distance of the level of PC tendon from the centroid of the concrete section
A	:	Area of concrete
I	:	Second moment of area
Zu	:	I / Yu

Zl	:	I / Yl
Zp	:	I / Yp
J	:	Second moment for torsion of area

2.2 Design Conditions

This section describes procedure of the input of common data which is shown in Fig. 2.2 "Flowchart for Superstructure Design Programme". The display's number for input data of design programme and the clause's number of Part 4 of BS5400 are shown in the title of each section as references. The diplay is attached in " DIVISION IV."

2.2.1 Common Conditions

In order to proceed the structural analysis, requisite data such as span length, width, the number of main beams and actual cross beams, loads, dimension and properties of material shall be prepared. The data shall be used, the analytical data for design and the basic data for drawings.

2.2.1.1 Structure Type [D.1]

- Reinforcec concrete solid slab (RCSS)
- Pre-tensioned concrete solid slab (PRSS)
- Pre-tensioned concrete hollow slab (PRHS)
- Pre-tensioned concrete composite T-beam (PRT)
- Post-tensioned concrete composite T-beam (PTT)

2.2.1.2 Beam Length and Span Length [D.3]

Table 2.4 Basic Dimension of Beams

Structural type	Beam (m) length	Span (m) length	Beam (m) height	No.of(nos) main beam	Space of(m) main beam	No.of(nos) cross beam
	6.3	6.0	0.45			---
RCSS	8.4	8.0	0.60	10	1.31 (1.11)	---
	10.5	10.0	0.75			---
	6.3	6.0	0.40			
PRSS	8.4	8.0	0.50	18 (15)	0.76 (0.78)	2
	10.5	10.0	0.65			
	12.5	12.0	0.60			
PRHS	14.5	14.0	0.70	18 (15)	0.76 (0.78)	3
	16.6	16.0	0.80			
	18.6	18.0	1.25			
PRT	20.7	20.0	1.35	11 (10)	1.28 (1.20)	1
	22.7	22.0	1.40			
	25.7	25.0	1.80			
	28.7	28.0	1.90			
	30.7	30.0	2.00			1
PTT	32.8	32.0	2.10	7 (6)	2.10	
	35.8	35.0	2.30			
	40.9	40.0	2.70			2
	45.9	45.0	2.85			

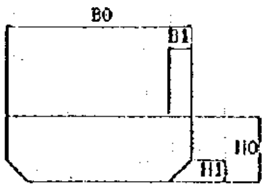
Note : Value in ( ) is for R3 raod class.

2.2.1.3 Dimensions

The dimension for each structural types in the standard design is as follows.

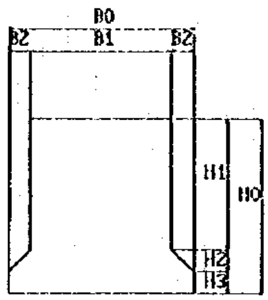
In any of the types , it was aimed that simplification in the cross-section and uniformity in cross sectional shape for whole span of a bridge for ease in construction.

(1) Reinforced Concrete Solid Slab [RCSS] [D6.1]

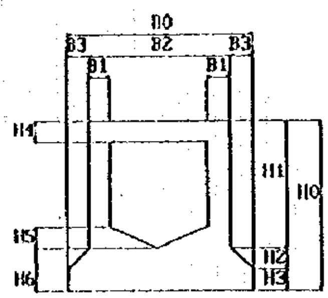


The RCSS dimensions is provided with small haunches at the lower edge of the beam so as the form can be removed easily.

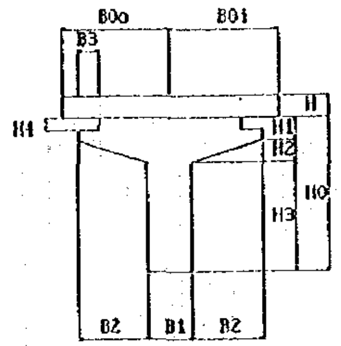
(2) Pre-tensioned Concrete Solid Slab [PRSS] [D6.2]



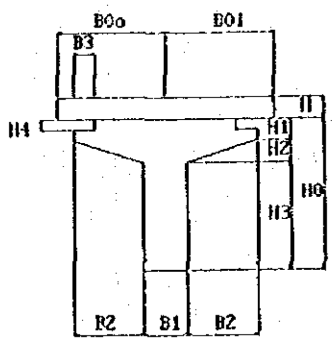
(3) Pre-tensioned Concrete Hollow Slab [PRHS] [D6.3]



(4) Pre-tensioned Concrete Composite T-beam [PRT] [D6.4]



(5) Post-tensioned Concrete Composite T-beam [PTT] [D6.5]



The PRT and PTT dimensions are provided with a space to allow the permanent forms for the slab to be installed.

Thickness of a part of the web for PTT is designed thicker at the both ends of beam for setting of rubber bearings.

2.2.1.4 Other Conditions

(1) Crossfall

Cross-fall shall set to be 2.5%. For details, refer to the section 2.2.2.3 "Superimposed Dead Load."

(2) Longitudinal Slope

Although a longitudinal slope is decided by the geographical conditions at where bridges are to be located, in the study which must be regarded as the standard design, the longitudinal slope is ignored, i.e. 0%.

(3) Class of Road

Two types of R5/U5 and R3/U3 are adopted according to "JKR's Guide on Geometric Design of Roads".

(4) Design Life [cl.6, Part 1]

A design life of 120 years has been assumed throughout BS5400 (unless otherwise stated).

(5) Environmental Condition [cl. 5.8.2, Table 13]

In the discussion with JKR, the environment condition for coverage and crack width of reinforced concrete member have been decided as mentioned below.

	Environmental Condition	Limitation
Coverage	very severe environment	50mm
Crack width	severe environment for RC beam	0.25mm
	very severe environment for slab of composite T-beam	0.15mm

2.2.2 Load Conditions

2.2.2.1 Load to be considered

D : Dead load
SD1 : Superimposed dead load except premix dead load
SD2 : Premix dead load
HA : Normal live load
HB : Abnormal live load 30 units
HB* : Abnormal live load 45 units
FL : Footway load

2.2.2.2 Dead Load

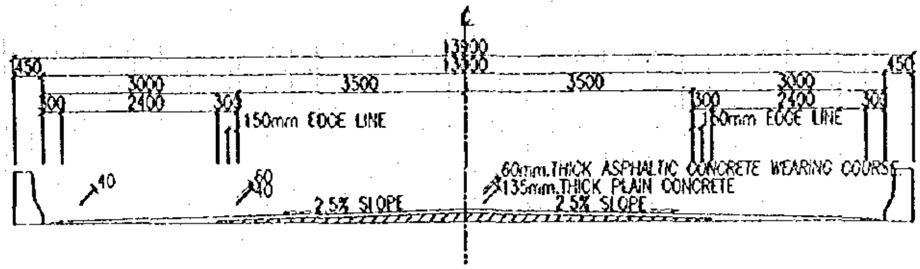
The unit weight applied as dead loads are as follows.

- Reinforced and precast concrete	25.0 (kN/m3)
- Mass concrete	23.5 (kN/m3)
- Premix	23.0 (kN/m3)

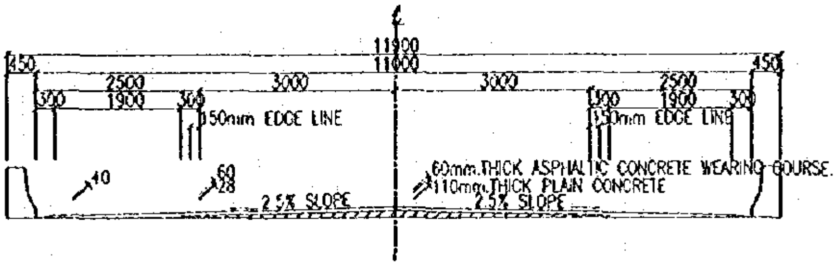
2.2.2.3 Superimposed Dead Load [D.7, D.8, D9]

The superimposed dead loads in the standard design are as follows.

(1) For R5/U5

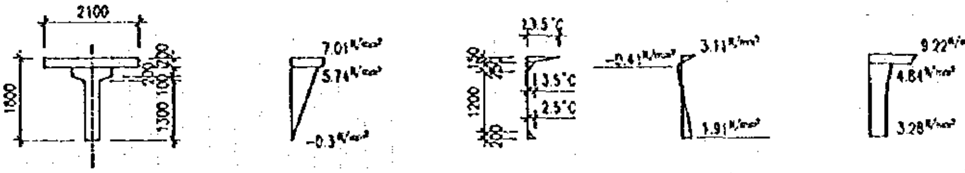


(2) For R3/U3



2.2.2.4 Temperature

As shown in the followings, the temperature load will not become a critical factor in the construction. Therefore, it is disregarded. However, the design of rubber bearing or expansion joint shall yet to be considered.



T beam cross section — Stress distribution (D+1.2SD+1.2HA) — Teperature difference — Stress distribution (1.0TD) — Stress distribution (D+1.2SD+HA+0.87TD)

The results of calculation for temperature difference derived from BD37/38 are as below (reference with the above figures).

- Tensile stress due to temperature difference is arising only on the top of beam, however, this tensile stress is not severe effect, because the top is usually compression zone in the simple beam.

- Compressive stress due to temperature difference on the bottom of beam is also not severe effect, because the compressive stress on the bottom is small and does not exceed the allowable compressive stresses.

- Compressive stress on the top of slab does not exceed the allowable stress, because compressive stress under S.L.S excluding temperature difference is usually not severe, and has a remainded stress against the allowable stress.

2.2.3 Design Properties of Materials

(1) Concrete [D.10]

Item		Unit	Reinforced Concrete	Prestressed Concrete	
				Main beam	Cross Beam / Slab
Grade		(N/mm2)	40	50	40
Characteristic strength		(N/mm2)	40	50	40
Modulus of elasticity		(KN/mm2)	31.0	34.0	31.0
Compressive stress		(N/mm2)	16.0	20.0	16.0
Tensile stress	At transfer	(N/mm2)	-----	1.00	1.00
	Class 2 pre-tensioned	(N/mm2)	-----	3.20	2.90
	Class 2 post-tensioned	(N/mm2)	-----	2.55	2.30
Design crack width		(mm)	0.25	-----	-----
V, Vtu (maximum shear & tortional stress)		(N/mm2)	4.75	4.75	4.75
Vtmin (reinforcement required stress)		(N/mm2)	0.42	0.42	0.42

Note: Class 1: HA only, HA+HB(30)
Class 2: HB(45) only

(2) PC Tendon [D.11]

Item		Unit	Longitudinal PC tendon	Trnasversal PC tendon
Class and Symbol		-----	T12.7 or T15.2	4K15 or 7k13
Characteristic strength		(N/mm2)	1860	1860
Modulus of elasticity		(KN/mm2)	196	196
During stressing		(N/mm2)	1488	1488
Immediately	Pre-tensioned	(N/mm2)	1395	1395
after anchoring	Post-tensioned	(N/mm2)	1302	1302

(3) Reinforcement Bar [D.12]

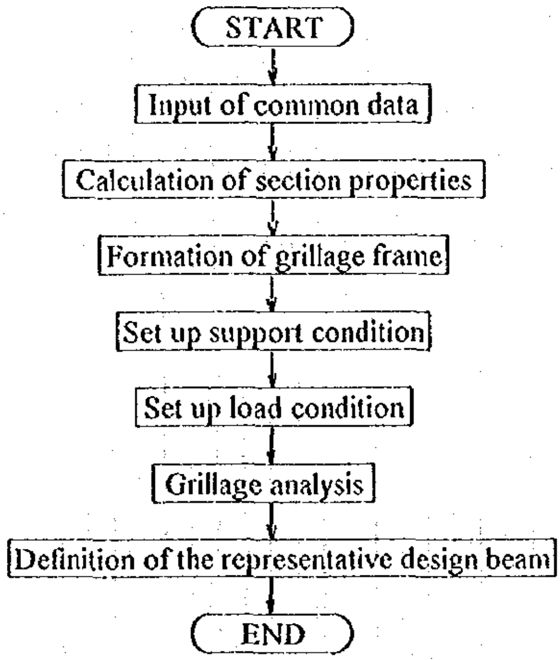
Item	Unit	T460
Characteristic strength	(N/mm2)	460
Modulus of elasticity	(KN/mm2)	200
Compression	(N/mm2)	345
Tension	(N/mm2)	345

2.3 Method of Structural Analysis

This section describes procedure of the structural analysis which is shown in Fig. 2.2 "Flowchart for Superstructure Design Programme". The clauses's number of Part 4 of BS 5400 is shown in the title of each section as references.

2.3.1 Calculation of Sectional Force

Calculation of sectional force shall be carried out according to the Analysis Flowchart undermentioned.



The sectional force shall be calculated by the grillage analysis. Elastic methods of analysis shall be used to determine internal forces and deformations. The support condition shall be fixed for vertical, and free for horizontal and rotation. The grillage frame is set as follows by the computer. The entire member cross section shall be used as the constants for the calculation of sectional force.

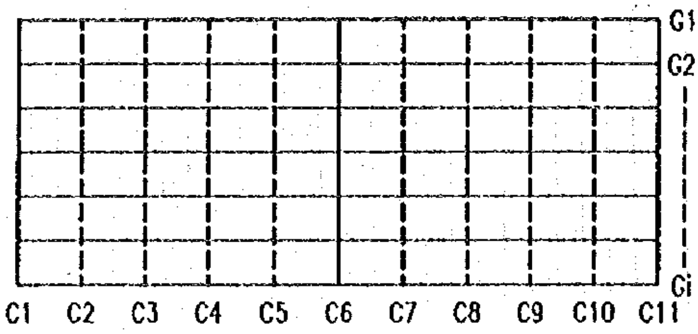


Fig. 2.4 Formation of Grillage Frame

In the Grillage Analysis, the numbers of main beams (Gi) shall not exceed 20 and the numbers of cross beams(Ci) shall be always 11. Cross beams shall be composed of actual cross beams and the analytical cross beams only for the design of main beams.

Actual number of intermediate cross beams shall not exceed 5 beams. The analytical cross beams for the main beam design shall be set with the numbers corresponding to that of intermediate cross beams as shown in the following chart.

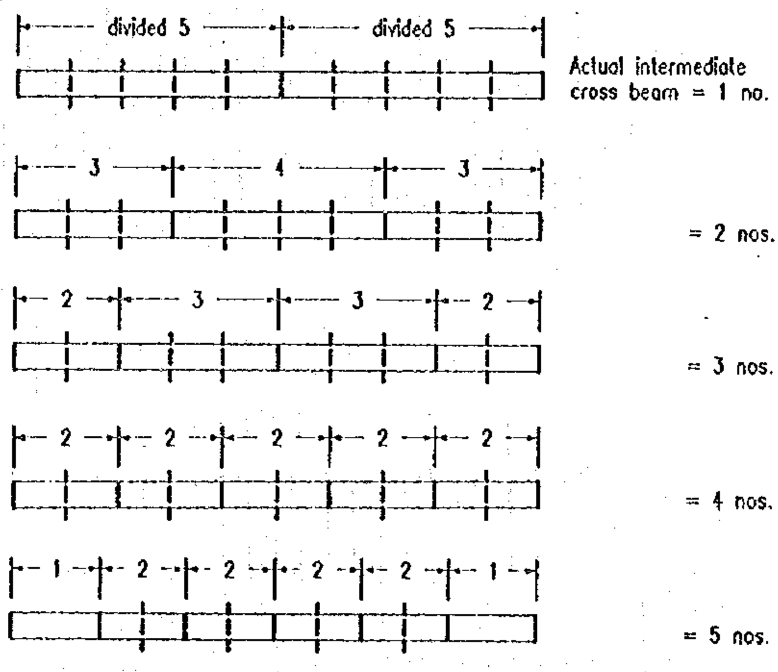


Fig. 2.5 Setting of Intermediate Cross Beam

Formation of grillage frame shall be carried out for each discrete stage of erection. For example, the grillage frame for the composite T-beam shall be formed three types as follows.

- Simple beam
- Grillage frame composed of main beam and cross beam
- Grillage frame composed of main beam, cross beam and slab

2.3.2 Effective Width

The section properties in structural analysis shall be calculated as a gross concrete section in consideration of effective width of flanged beam.
In particular, the end cross beam is not effective to the whole section, but main beam and intermediate cross beams are effective whole section in this standard bridges.

2.3.3 Distribution of Live Load [cl. 6.1 ~ 6.4]

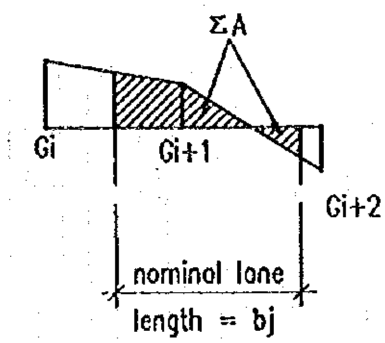
Based on the following method, distribution of live load shall be carried out according to the grillage analysis.

2.3.3.1 HA Load

(1) Calculation of the Transversal Direction.

In BD 37/88, the position of a nominal traffic lane to a transversal direction is fixed, thus, live loads shall be distributed equally to each lane. Therefore, the influence value at the cross beams for each traffic lane shall be calculated by using the influence lines for each cross beam.

(a) The Influence Value at the Position of Each Cross Beam (j: traffic lane, m: cross beam)



$g_{j,m} = \sum A_{j,m} / b_j$

(2) Calculation of the Longitudinal Direction

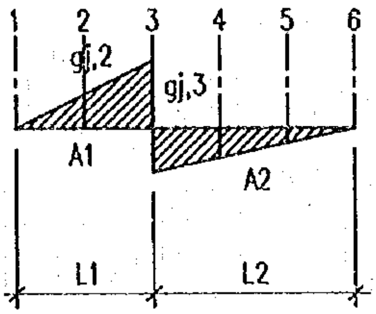
(a) Concentrated Load Pj

The sectional force due to concentrated load shall be calculated by multipling a maximum influence value among g j,m at each cross beam position by the load strength.

P j,m(-) = g j,m (min) x P(KEL)
P j,m(+) = g j,m (max) x P(KEL)
Where, P (KEL): concentrated load strength (kN)

(b) Distributed Load Qj

For the distributed load, that the load strength is varied by the load length of the influence lines. Accordingly an area in the longitudinal direction is calculated first, the load strength for the length of the influence line is calculated in the next, and then sectional force due to distributed load shall be obtained by multipling the area by the load strength.



(c) Lane Coefficient ß

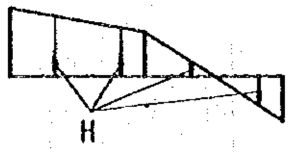
In BD 37/88, the coefficient ß must be considered, and it is varied by each traffic lane, while ß itself is differentiated by the lane width and the length of the influence line. The final sectional force considered ß shall be obtained as a maximum value among Pj and Qj of each traffic lane multiplied by ß as below.

Qj (-) = {Σß (Lj (-),bj) x [Qj (-) + Pj (-)]}min
Qj (+) = {Σß (Lj (+),bj) x [Qj (+) + Pj (+)]}max

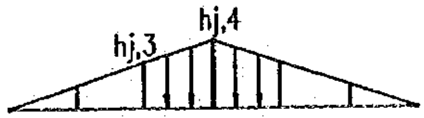
2.3.3.2 HB Load

HB load shall also be regarded same as HA load, therefore it shall be considered that the load shall not move in the transversal direction as it is moving in the longitudinal direction.

(a) Transversal Direction



(b) Longitudinal Direction



2.3.4 Design Section

2.3.4.1 Main Beam

At the end of main beam, the design section shall be at the point which is apart about one half of the main beam height from the support. In the middle of main beam, the design section shall be at the point of cross beam.

2.3.4.2 Definition of the Representative Main Beam

The calculation of the sectional force is carried out at the position shown in the above section, but for the design of main beam, the sectional force as for the flexural , shear and torsion shall be compared by the computer itself. The representative main beams shall be derived from the four critical forces calculated by the structural analysis as follows.

- The beam causing the maximum moment under the U.L.S.
- The beam causing the maximum shear under the U.L.S.
- The beam causing the maximum torsion under the U.L.S.
- The beam causing the minimum torsion under the U.L.S.

2.3.4.3 Cross Beam

The cross beam shall be designed at the actual cross beam.

2.3.4.4 Definition of the Representative Cross Beam

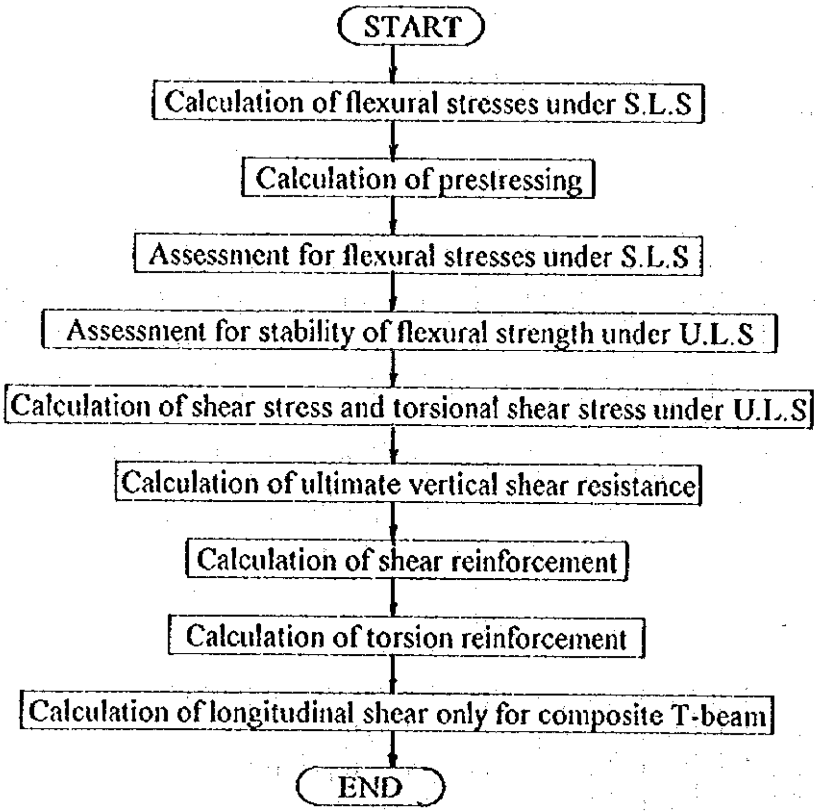
One end cross beam and one intermediate cross beam shall be selected as the representative cross beam for design, in the same way as the design main beam is chosen.

2.4 Design of Main Beam

This section describes procedure of the design for main beam which is shown in Fig. 2.2 "Flowchart for Superstructure Design Programme". The display's number for input data of design programme and the clauses's number of Part 4 of BS 5400 are shown in the title of each section as references. The display is attached in "DIVISION IV."

2.4.1 Design Flowchart

Design of main beam shall be carried out according to the Design flowchart below.



2.4.2 Input Data

2.4.2.1 Creep Coefficient and Shrinkage [E 3.1 and cl. 4.3.2.1]

According to the CEB manual, the creep coefficient and shrinkage are determined by setting humidity, effective thickness of the members and cement to be used.

In the study, the following data shall be adopted.

- Humidity : 80 %
- Effective thickness : 30 cm
- Cement to be used : Rapid hardening Portland cement
- Composite time : 200 days after main beam concrete is harden

Table 2.5 Creep coefficient and shrinkage

Structural type		Creep coefficient		Shrinkage	
		t = 0 - t = 200 days At composite time	t = 0 - t = ∞ At creep end time	t = 0 - t = 200 days At composite time	t = 0 - t = ∞ At creep end time
PRSS PRHS		——	2.0	——	25.0
PRT	Beam	0.5	2.0	5.0	20.0
PTT	Slab	——	2.2	——	20.0

2.4.2.2 Data Relating to Prestressing Tendons [E 3.2]

After selecting the kind of PC tendons, various constants for the PC tendons shall be set based on the Malaysian standard of its kind. The various constants shall be described in detail in Clause 2.4.4 " Calculation of Prestress". The PC tendon which will be used shall be determined after judging how it can be arranged in a balance in the cross section. The followings are the general examples in the Study.

(1) Pre-tensioned slab type Span 10 m or less T12.7
Span 10 m or more T15.2

(2) Pre-tensioned T beam type T15.2

(3) Post-tensioned T beam type Span 35 m or less 12T12.7
Span 35 m or more 12T15.2

2.4.2.3 Arrangement of PC Tendons [E 3.3]

After inputting the data shown below, the arrangement of PC tendons shall be determined as the basic data for the calculation of the prestress.

(1) Anchoring Position of Main Beam End

General anchoring position of the main beam end is 100 mm with the pre-tensioned type, and 150 mm for the post-pretensioned type. For the debonded type of pre-tensioned beam , it shall be determined after carrying out trial calculations.

(2) Height

A minimum value of covering and space for the PC steel is set as follows.

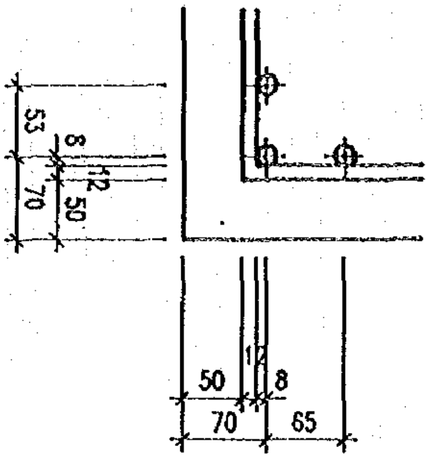
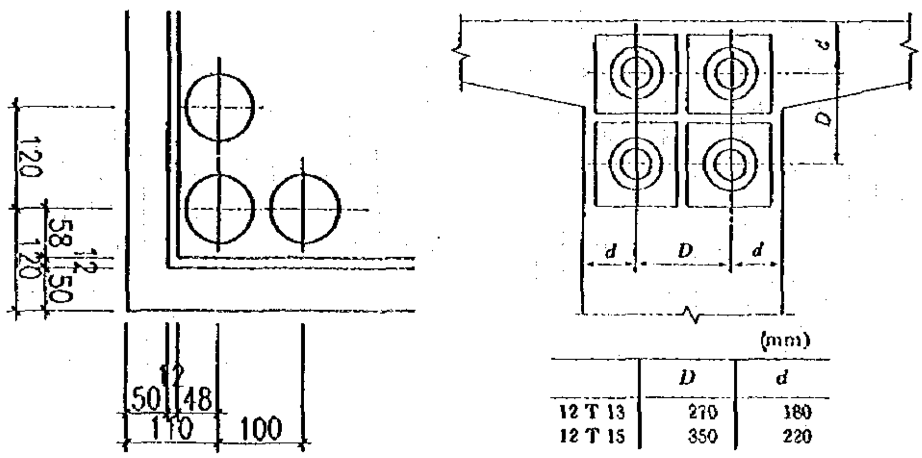


Fig. 2.6 Minimum Cover and Space for PC Tendons of Pre-tensioned Beam



	D	d
12 T 13	270	180
12 T 15	350	220

Fig. 2.7 Minimum Cover and Space for PC Tendons of Post-tensioned Beam

(3) Bending Up Angle

The bending up angle shall be adopted only for the T-beam type.

(4) PC Tendons Curvature Radius

The curvature radius of PC tendons for the PTT type shall be adopted 10 m as the standard value.

(5) Number of PC Tendons

These data for PC tendons shall be determined after carrying out trial calculations.

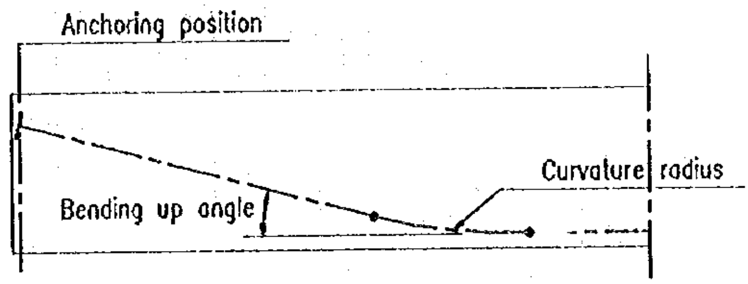


Fig. 2.8 Side View of Arrangement of PC Tendons

Height of anchoring position shall be determined based on the balanced arrangement with the number of PC tendon.

2.4.2.4 Design Data for Shearing Force and Torsional Moment

The reinforcing bar covering shall be input for calculation of the link of the main beam. Also the slab reinforcing bar covering and the types of surface of beam shall be input for calculating the slab link and the shear connector of the composite T-beam respectively. The cover shall be taken as the distance from the fibre of beam to the center of reinforcing bar.

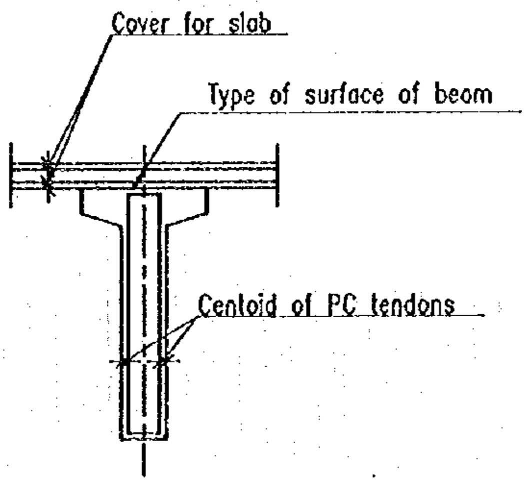


Fig. 2.9 Cover of Reinforcement

2.4.2.5 Combination of Loads [E.4 and cl. 4.4, BD37/88]

Load Combination Case		D	SD1	SD2	HA	HA+HB	HB*	FL
Serviceability	1	1.00	1.00	1.20	1.20	----	----	1.00
Limit	2	1.00	1.00	1.20	----	1.10	----	1.00
State	3	1.00	1.00	1.20	----	----	1.10	1.00
Ultimate	1	1.20	1.20	1.75	1.50	----	----	1.50
Limit	2	1.20	1.20	1.75	----	1.30	----	1.50
State	3	1.20	1.20	1.75	----	----	1.30	1.50

2.4.3 Stages for Calculation of Sectional Force

(1) Stages for Calculation

The sectional force shall be calculated at the following conditions, as in the method described in the section 2.2.5.

Stage 1 : Immediately after anchoring
rfL x D1

Stage 2 : Under own dead load
rfL x (D1 + D2 +D3)

Stage 3 : Under superimposed dead load
rfL x (D1 + D2 +D3 + SD1 + SD2)

Stage 4 : Under live load (= S.L.S)
rfL x (D1 + D2 +D3 + SD1 + SD2 + LL)

where
LL : Live load

2.4.4 Calculation of Prestress [cl. 6.7]

2.4.4.1 Initial Prestress at Jack

The initial prestress shall be set at the jack end, and shall be taken as the initial value of the prestress calculation. In the design stage, the initial prestress for the pre-tensioned beam is set as the same value as is applied immediately after anchoring.

Therefore, the loss of prestressing such as the anchorage pull-in, steam curing or relaxation before anchoring, which may all occur immediately after the anchoring, shall be managed by the manufacturers.

2.4.4.2 Prestress at Immediately after Anchoring

(1) Losses of Prestress [cl. 6.7.2]

The factors for the losses of prestressing at the immediately after anchoring shall be limited as follows for pre-tensioned or post-tensioned beams.

(a) For the Pre-tensioned Beams

- Steam curing
- Relaxation before anchoring
- Anchorage pull-in
- Instantaneous (Elastic) deformation

(b) For the Post-tensioned Beams

- Friction in duct
- Anchorage pull-in
- Instantaneous (Elastic) deformation

(2) Maximum Initial Prestress [cl. 6.7.1]

The maximum initial prestress shall be limited as follows for pre-tensioned or post-tensioned beams.

(a) For the Pre-tensioned Beams

- During stressing : 0.8fpu = 0.8x1860 = 1488 N/mm2
- Immediately after anchoring : 0.75fpu = 0.75x1860 = 1395 N/mm2

(b) For the Post-tensioned Beams

- During stressing : 0.8fpu = 0.8x1860 = 1488 N/mm2
- Immediately after anchoring : 0.70fpu = 0.70x1860 = 1302 N/mm2

2.4.4.3 Effective Prestress

The loss factors of the prestress from immediately after anchoring to the serviceability limit state shall be as follows for both pre-tensioned and post-tensioned beams.

- Creep and shrinkage
- Relaxation after anchoring

2.4.4.4 Example of Calculation of Prestress

(1) For Pre-tensioned Beams

(a) At the Immediately After Anchoring

(i) Losses due to anchorage pull-in ($\triangle$ fpo)

$\triangle$ fpo = Ep x ($\triangle$ l / l)
= 196,000 x (3 / 20,000) = 29.4N/mm2 → 30N/mm2
Ep : Modulas of elasticity
$\triangle$ l : Anchorage pull-in
l : Distance between anchorages

(ii) Losses due to steam curing ($\triangle$ fpsc)

$\triangle$ fpsc = α x Ep x T x C
= 10 x 10 x 196,000 x 60 x 0.22 = 25.9N/mm2 → 30N/mm2
α : Coefficient of linear expansion (/°C)
T : Temperature different (°C)
C : Supplementary coefficient

(iii) Losses due to relaxation before anchoring ($\triangle$ fpr1)

$\triangle$ fpr1 = fpio x r1
fpio : Initial prestress
r1 : Relaxation before anchoring (= 3%)
$\triangle$ fpr1 = 1488 x 0.03 = 45 → 50N/mm2

(iv) Prestress at the immediately after anchoring (fpi)
fpt = 0.8fpu - $\triangle$ fpo - $\triangle$ fpsc - $\triangle$ fpr1 = 1488 - 30 - 30 - 50
= 1378

In the design stage, the prestress that considered above losses at the immediately after anchoring shall be given as the initial prestress for calculation of prestress.

The losses due to anchorage pull-in, steam curing and relaxation before anchoring should be controlled by manufacturers.

Therefore, the prestress at the immediately after anchoring shall not exceed 1350N/mm2.

(b) Effective Prestress

(i) Losses due to instanteneous deformation ($\triangle$ fpid)

$\triangle$ fpid = n f cpg

n : Ep/Ec
fcpg : Concrete stress due to prestress and permanent load at the level of PC tendon

(ii) Losses due to creep and shrinkage [$\triangle$ fp(CR + SH)]

$\triangle$ fp(CR + SH)
$= \{ n \times \varphi \times (fcpt + fdg) + Ep \times \varepsilon s \} / \{ 1 + n \times fcpt / fpt \times (1 + \varphi / 2) \}$

φ : Creep coefficient
fcpt : Concrete stress due to prestress at the level of PC tendon at the immediately after anchoring
fdg : Concrete stress due to permanent load at the level of PC tendon at the immediately after anchoring.
εs : Shrinkage
fpt : Prestressing steel stress at the immediately after anchoring

(iii) Losses due to relaxation after anchoring [$\triangle$ fpr2]

$\triangle$ fpr2 = fpt x r2
r2 : relaxation after anchoring (= 3%)

(iv) Effective prestress

fpe = fpt - $\triangle$ fpid - $\triangle$ fp(CR + SH) - $\triangle$ fpr2

(2) For Post-tensioned Beam

(a) Initial Prestress at Jack

The prestress at the immediately after anchoring shall not exceed 1,302 N/mm2 of a limitation of prestress. In the trial calculation, the losses of prestress until the immediatly after anchoring will cause 100 N/mm2 approximately. The initial prestress at jack can be given about 1,400 N/mm2, but in the Study it shall not be exceed 1,350 N/mm2 in order to keep within the limitation as same as that of pre-tensioned beam

(b) At the Immediately After Anchoring

(i) Losses due to friction in duct ($\triangle$fpf)

$\triangle fpf = fpi \times e^{-(kx+\mu\alpha)}$

fpi : initial prestress
k : coefficient of friction for length (=0.0033 /m)
x : distance from the jack
μ : coefficient of friction for curvature (=0.3 /rad)
α : angle of deflected tendon

(ii) Losses due to anchorage pull-in ($\triangle fpa$)

$\triangle fpa = Ep \times (\triangle l / l)$

(iii) Losses due to instanteneous deformation ($\triangle fpid$)

$\triangle fpid = 1 / 2\ n \times fcpg \times (N-1) / N$
N : frequency of prestressing

(iv) Prestress at the immediately after anchoring

$fpt = fpi - \triangle fpf - \triangle fpa - \triangle fpid$

(c) Effective Prestress

The effective prestress should be calculated the same low as the case of pre-tensioned beam.

$fpe = fpt - \triangle fp(CR+SH) - \triangle fpr2$

The losses of prestress resulting from subsequent shrinkage and creep of the concrete shall be calculated based on the CEB/FIP international Recommendations using 80% humidity of Malaysian meteorological data.

2.4.5 Assessment for Flexural Stresses under Serviceability Limit States

2.4.5.1 Crack width for Reinforced Concrete Member

In reinforced concrete under Serviceability Limit State,compressive concrete stress,reinforcement stress and crack width shall be checked as mentioned below.

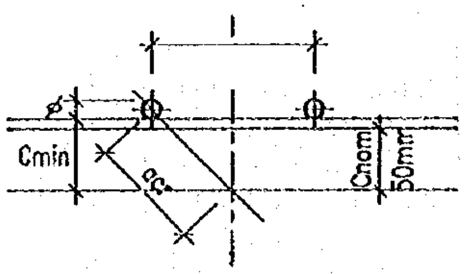
(1) Design Crack Width [cl. 5.8.8.2]

Design crack width = $(3acr\ \varepsilon m) / (1+2(acr-Cmin) / (h-dc)$

whereas

$\varepsilon m = \varepsilon 1 - \{ (3.8bth(a'-dc)) / \varepsilon sAs\ (h-dc)\}\{1-(Mq)/(Mg)\}10^{-9}$.

acr is the distance from the point considered to the surface of the nearest bar which controls the crack width.



The design crack width shall be limited to the value of 0.25mm.

(2) Compressive Concrete Stress [cl. 4.1.1.3]

The compressive concrete stress shall be calculated from the equation:

$fcu = (Mg+Mq) \times dc / I$

where:

$I = 1/3 \times bt / dc^3 + Es/Ec \times As \times (ds-dc)^2.$

$dc = (-nAs + \sqrt{\{(nAs) + 4 \times 1/2 \times bt \times nAs \times ds/bt\}}$

$Ec = \{1 / (Mg+Mq)\}(Eg \times Mg + Ec \times Mq)$

$Eg = 1/2\ Ec$

The compressive stress shall not exceed the value of 16.0 N/mm^2.

(3) Reinforcement Stress [cl. 4.1.1.3]

$fs = Es/Ec \times (Mg+Mq) \times (ds-dc) / I$

The reinforcement stress shall not exceed the value of 345 N/mm^2.

2.4.5.2 Flexural Stresss for Prestressed Concrete Member

(1) Stages of Examination

For the S.L.S., the examination of flexural stress shall be carried out for prestressed concrete members at four stages as mentioned below :

Stage 1 : Immediately after anchoring
Stage 2 : Under own dead load
Stage 3 : Under superimposed dead load
Stage 4 : Under live load (= S.L.S)

(2) Section Properties

The concrete stresses caused by the load and prestress shall be calculated by using the cross section constants corresponding to each load combination. For example, the bending stress of the composite T-beam shall be calculated by the following three cross section constants.

- Net cross section (exclude the sheath for PC tendon)
  [Prestress at transfer and Beam self-weight]

- Gross transformed section 1 (transformed main beam only)
  [Dead load excluding beam self-weight and slab for composite T-beam

type]
[Superimposed dead load and live load for Slab beam type]

- Gross transformed section 2 (transformed main beam and slab)
  [Superimposed dead load and live load only for composite T-beam type]

(3) Stress Limination [cl. 6.3.2.4]

The concrete stresses under Stage 1 shall be limited to the undermentioned value.

$-1.0N/mm^2 \leqq fc \leqq 20N/mm^2$.

The concrete stresses under Stage 1 and Stage 2 shall be calculated to know the condition of the stresses under permanent dead load which will be loaded frequently on the beam.

The concrete stresses under Stage 4 shall be calculated under 3 load combination with live load as described in clause 2.4.2.5, and the limitation shall be within the undermentioned value.

Lc.1 (with HA)	$0 \leqq fc \leqq 20$	class 1 members
Lc.2 (with HA+HB)	$0 \leqq fc \leqq 20$	class 1 members
Lc.3 (with HB*)	$-3.20 \leqq fc \leqq 20$	class 2 pre-tensioned members
Lc.3 (with HB*)	$-2.55 \leqq fc \leqq 20$	class 2 post-tensioned members

(4) Calculation of Concrete Stress

Concrete stress due to load

$fcu = M / Zu$

$fcl = M / Zl$

Concrete stress due to prestress

$fcpu = \{(fp \times Ap) / Ac\} + \{(fp \times Ap \times (Yu - Yp)) / Zu\}$

$fcpl = \{(fp \times Ap) / Ac\} + \{(fp \times Ap \times (Yu - Yp)) / Zl\}$

Composite concrete stresses

$\Sigma fcu = fcu + fcpu$

$\Sigma fcl = fcl + fcpl$

The composite concrete stresses shall not exceed the limitation of respective load combination.

2.4.5.3 Judgment for Design Result [E.6]

The design results shall be within the limitation summarized as follows.

(1) Reinforced Concrete [cl.4.1.1]

- Flexural crack width   below 0.25 mm
- Concrete stress   below 16.0 N/mm2
- Reinforcing steel stress   below 345 N/mm2

(2) Prestressed Concrete [cl. 6.3.2.4]

- Concrete compressive stress at the immediately after anchoring
  below 20.0 N/mm2
- Concrete tensile stress at the immediately after anchoring
  below -1.0 mm2
- Concrete compressive stress under S.L.S.
  below 20.0 N/mm2
- Concrete tensile stress under S.L.S
  below 0.0 N/mm2 (Class 1)
- Concrete tensile stress under S.L.S
  below -3.2 N/mm2 (Class 2) for pre-tensioned beam
  below -2.55 N/mm2 (Class 2) for post-tensioned beam

2.4.6 Assessment of Stability under the Ultimate Limit State

2.4.6.1 Resistance Moment [cl. 5.3.2 and cl. 6.3.3]

The resistance moment for the Reinforced concrete shall be calculated according to Clause 5.3,2. Part 4, BS5400.

The resistance moment for the Prestressed concrete shall be calculated according to Clause 6.3.3. Part 4 BS5400.

When analysing a cross section in determination of the ultimate strength, the following assumptions shall be made.

- Plane sections remain plane
- The strain at the outermost compression fibre is taken as 0.0035
- The tensile strength of the concrete is ignored.
- The stress-strain curves for concrete, reinforcement and prestressing tendons are given in figure 1, 2, and 3 respectively in Part 4, BS 5400.

The ultimate moment of resistance shall be larger than 1.15 times the required value, if it is less than 1.15 times, the section shall be proportioned such that the strain is not less than:

$0.002 + fy / (Es \cdot \gamma m)$ .....at the centroid of the reinforcement
for reinforced concrete
$0.005 + fpu / (Ep \cdot \gamma m)$....at the outermost tendon for prestressed concrete

On pre-tensioned solid slab and hollow slab, assessment shall be made on main beam section ignoring the filling area.

On composite T-beam, assessment shall be made on composite section with slab.

2.4.6.2 Shear Resistance [cl. 5.3.3 and cl. 6.3.4]

Calculation for shear and torsion is only required for the ultimate limit state.

(1) Calculation of Shear Stress and Torsional Stress

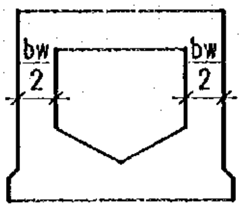
(a) Shear Stress

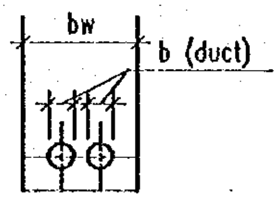
The shear stress, v, at any cross section shall be caculated from:

$$v = V / bd$$

where

V : the shear force due to ultimate loads
b : the breath of the section.
For the reinforced concrete and pre-tensioned beam, b shall be taken as the rib width (bw), and for the post-tensioned beam, b shall be taken as (bw - $\sum$ b(duct))





d : the effective depth to the tension reinforcement.
For the reinforced concrete and pre-tensioned slab type, d shall be taken as the depth from the extreme compression fiber to the centroid of the tendons, and for the composite T-beam type, d shall be taken as the composite depth with due allowance for the different grades of concrete where appropriate.

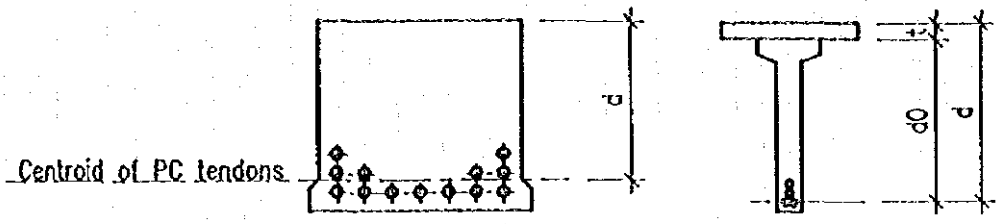


Fig. 2.10 Effective Depth

(b) Torsional Stress [cl. 5.3.4 and cl. 6.3.5]

The torsional stress, vt, shall be calculated by respective cross section as below.

For the reinforced concrete and the pre-tensioned solid slab, vt shall be calculated by the rectangular section.

$$vt = 2T / \{ h^2_{min} (h_{max} - h_{min} /3) \}$$

For the pre-tensioned hollow slab, vt shall be calculated by the box section.

$$vt = T / (2h_{wo}A_o)$$

For the composite T-beam, vt shall be calculated by T section.

$$vt = T (h_{max}\, h^3_{min}) / \sum (h_{max}\, h^3_{min})$$

The section of properties shall be based on those of the composite section, with due allowance for the different grades of concrete where appropriate.

Where the torsional shear stress, vt, exceeds the value vtmin (=0.42N/mm2 for concrete grade 40 or more), reinforcement shall be provided.

(c) Maximum Shear Stress [cl. 5.3.3 and cl. 6.3.4]

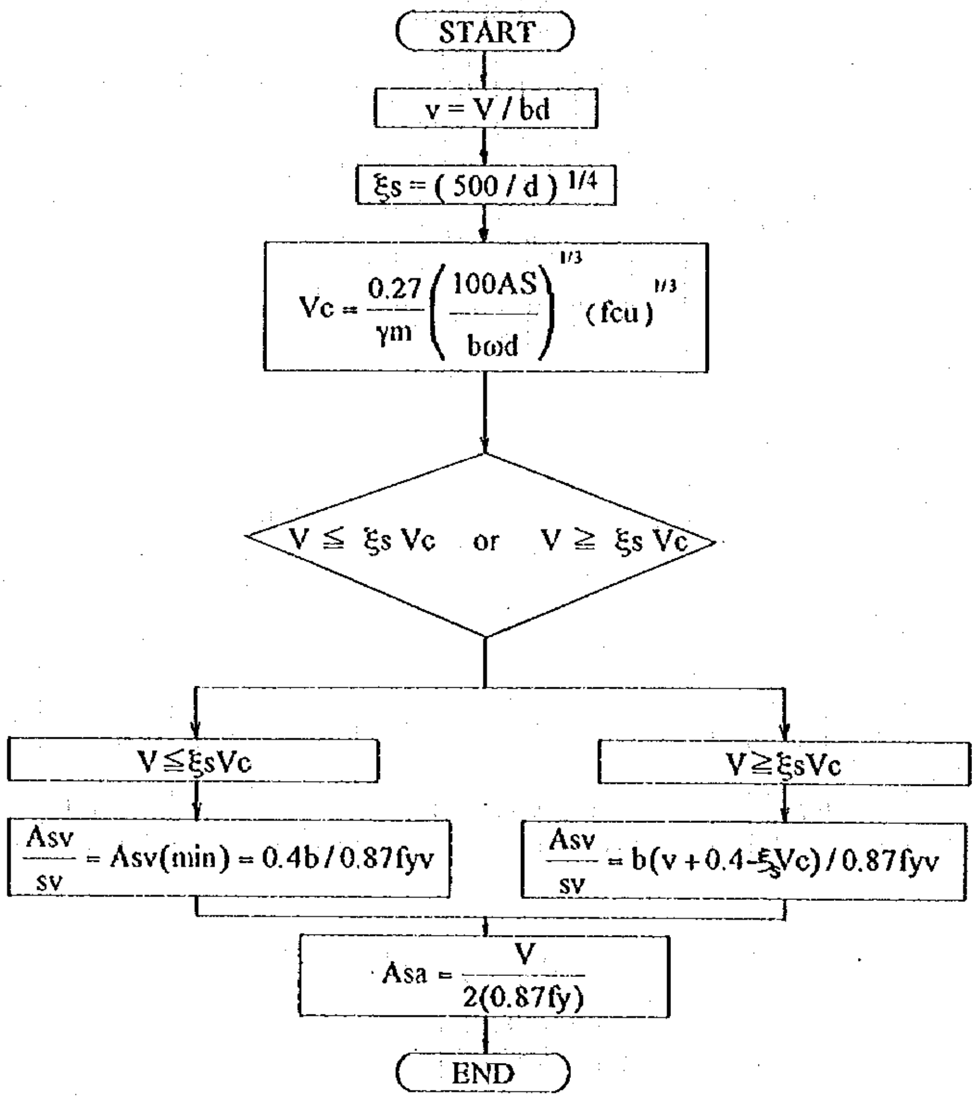
Shear stress (v+vt) due to shear force and torsional force under ultimate limit state shall not exceed the appropriate value given in Part 4, BS 5400:

For reinforced concrete (v+vt)max = 4.75 N/mm2
For prestressed concrete (v+vt)max = 5.30 N/mm2

(2) Shear reinforcement

(a) Reinforced concrete member [cl. 5.3.3]

Shear reinforcement for reinforced concrete member shall be calculated based on the Part 4, BS 5400 as undermentioned flow chart.

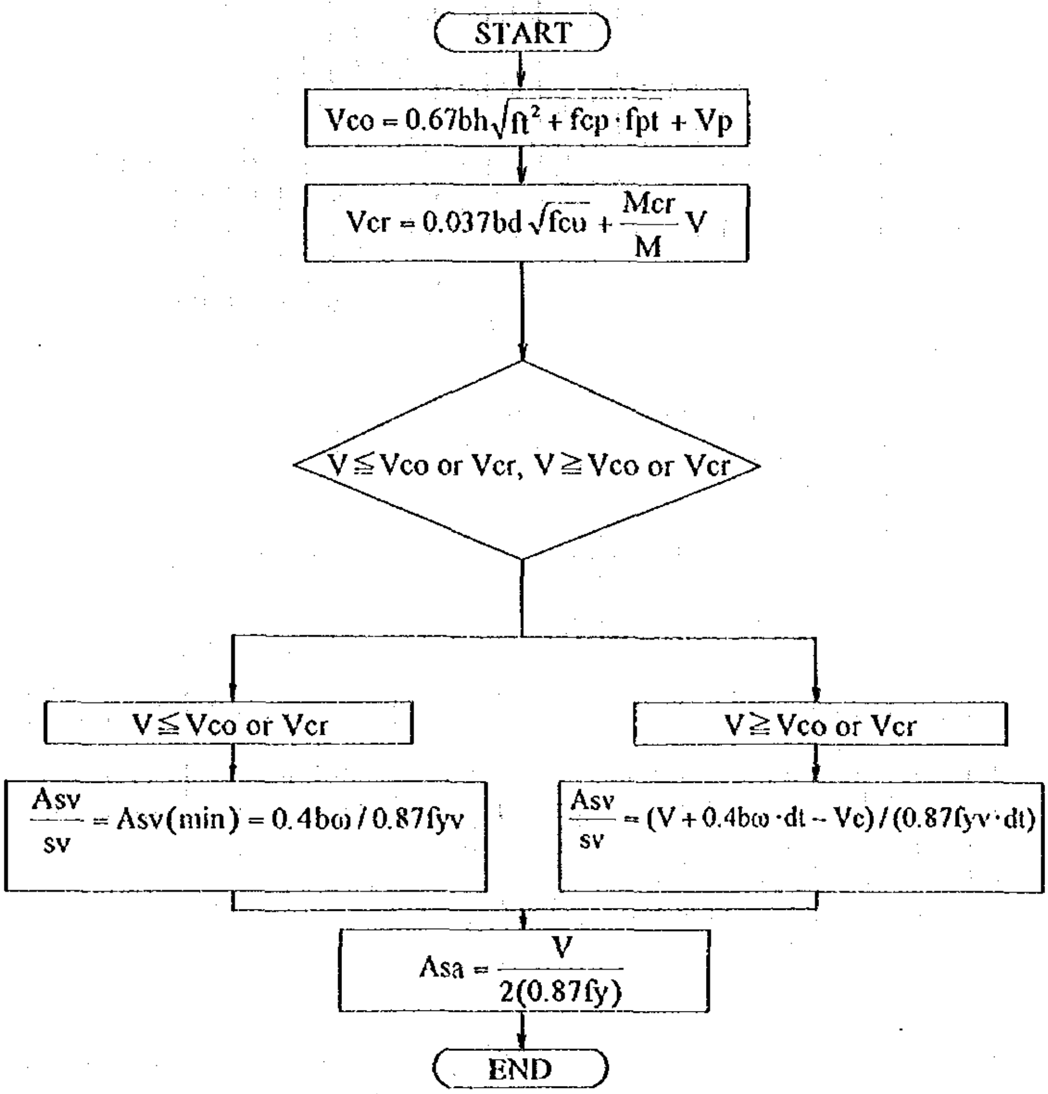


where

$\gamma m = 1.25$, $fcu \leqq 40$

(b) Prestressed Concrete Member [cl. 6.3.4]

Shear reinforcement for prestressed concrete member shall be calculated based on the Part 4, BS 5400 as undermentioned flow chart.



where

$ft = 0.24\sqrt{fcu}$

$Mcr = 0.37\sqrt{fcu} + fpt\ )\ I / y$

The value of 0.87 shall be applied as the partial safety factor for the fcp・Vp and fpt.

"dt" shall be taken as the depth from the outermost compression fiber to the lowest PC tendion, and for the composite T-beam, dt shall be taken as the additional depth with slab thickness.

In the calculations of Vco and Vcr for the composite T-beam, the precast beam shall be only resisted alone.

In the Study, shear reinforcement, Asv, shall be calculated the required cross-sectional area of all the legs of the links per 1.0m length.

2.4.6.3 Torsional Reinforcement [cl. 5.3.4 and cl. 6.3.5]

The calculation of torsional reinforcement shall be carried out only where the torsional shear stress in a minor rectangle is larger than minimum ultimate torsional shear stress (vtmin). (refer to clause (b), (1), 2.4.6.2 in this manual)

(1) For the rectangular section, the torsional reinforcement shall be calculated from:

$$Ast / sv = T / 1.6X_1Y_1 (0.87 fyv)$$

$$As_L / s_L = Ast / sv (fyv / fy_L)$$

(2) For the box section, the torsional reinforcement shall be calculated from:

$$Ast / sv = T / 2Ao (0.87 fyv)$$

$$As_L / s_L = Ast / sv (fyv / fy_L)$$

(3) For the T section, the torsional reinforcement shall be calculated by the same way with the rectangular section to the divided component rectangules for purposes of torsional design.

$$Ast / sv = T / 1.6X_1Y_1 (0.87 fyv)$$

$$As_L / s_L = Ast / sv (fyv / fy_L)$$

In the study, torsional reinforcement, Ast, shall be calculated the required area of one leg of a closed link at a section per 1.0m length.

2.4.6.4 Arrangement Shear Reinforcement and Torsion Reinforcement

Shear reinforcement and torsion reinforcement required shall be calculated by summing up the area of one leg of a closed link at a section per 1.0m length.

$$\sum Av = Asv / 2 + Ast$$

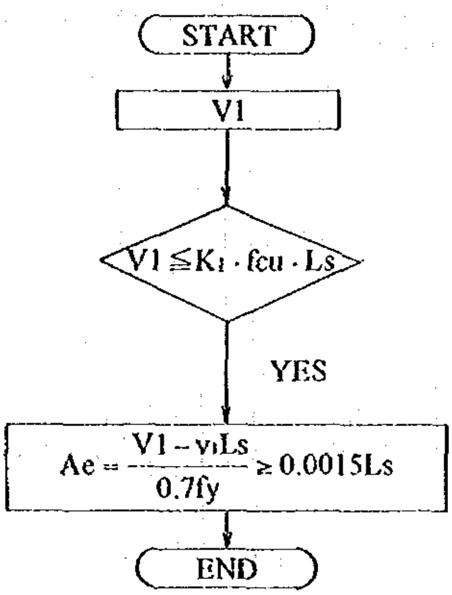
On the other hand, the area of additional longitudinal reinforcement required shall be calculated by summing up the required amount for shear force in the tensile zone, and for torsion in the half surrounding length of link.

$\sum Asa = Asa + As_L / 2$

The area remainded of the required tensile reinforcement and prestressing tendons at the ultimate limit state for the bending moment shall be effective to the longitudinal reinforcement $\sum Asa$.

2.4.6.5 Longitudinal Shear [cl. 7.4.2.3]

The calculation of longitudinal shear shall be carried out only for the composite T-beam.

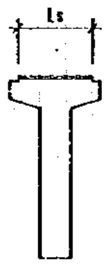


where

k1 and v1 shall be taken as the undermentioned values based on the Table 31, Part 4, BS 5400.

Type of shear plane	fcu = 40 N/mm2 k1	v1
Type 1	0.15	0.80
Type 2	0.09	0.05

where,
Ls : Length of the shear plane:



For pre-tensioned T-beam
Ls=620mm
For post-tensioned T-beam
Ls=820mm

2.4.6.6 Judgment for Design Result [E.7]

(1) Resistance Moment

- The ultimate moment of resistance shall be 1.15 times more than the required value.

(2) Maximum Shear Stress

- The sum of stress by shearing force and torsional moment (v+vt) shall be below the limitation.

(3) Shear and Torsion Reinforcement

The reinforcement for shear and torsion shall be arranged larger than required area of one leg of a closed link per 1.0m length which be obtained from the design result.

The arrangement of reinforcement shall be determined in consideration of the diameter and the spacing, and when the reinforcement with big diameter and the narrow spacing will be required, it is better to change the web thickness The standard arrangement of link and the determination procedure are shown below as reference.

Table 2.6 Shear and Torsion Reinforcement

Dia.	Normal Area	Spacing	Reinforcement (mm2/m · 1 leg)
10	78.5	100	785.0
		150	523.3
12	113.0	100	1130.0
		150	753.3
16	201.0	100	2010.0
		150	1340.0
20	314.1	100	3141.0
		150	2094.0

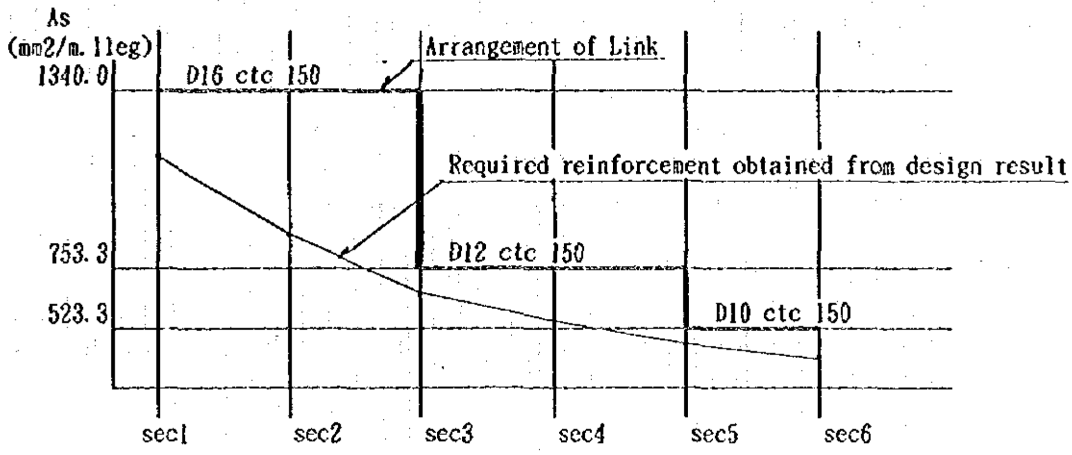


Fig. 2.11 Procedure of determination of arrangement

(4) Longitudinal Shear Reinforcement

The arrangement for longitudinal shear reinforcement shall be determined by the same way as the clause (3) abovementioned, but the required area shall be calculated per 1.0m length crossing the shear plane.

Table 2.7 Longitudinal Shear Reinforcement

Dia.	Nominal Area	Spacing	No. of bar crossing the shear plane		Reinforcement (mm2/m)
				Bar shape	
10	78.5	150	2		1046.7
12	113.0	150	2	- do -	1506.7
16	201.0	150	2	- do -	2680.0
			4		5360.0
20	314.1	150	2		4188.0
			4		8376.0