REPORT

OF

HIGHWAY BRIDGE CONSTRUCTION

IN

COLOMBIA

SURVEY TEAM OF JAPAN

DISPATCHED BY

OVERSEAS TECHNICAL COOPERATION AGENCY OF JAPAN

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PREFACE

We, as the members of Survey Team of the Japanese Government (which was under the name of "the Japanese Bridges Mission to the Republic of Colombia" during our work in the Republic) dispatched by the Overseas Technical Cooperation Agency to the Republic of Colombia, take now the liberty of presenting our suggestions on the bridge construction in the Republic as the conclusive opinion of our studies and investigations not only of bridges existing as well as under construction but also of the plan and sites of bridges scheduled to be constructed in the Republic, where we stayed from December 1962 to January 1963.

We should be very much pleased if our suggestions would be of some help to the future planning and construction of new bridges in the Republic.

On this occasion, we should like to extend our deepest thanks to the Government of the Republic of Colombia and all those who have assisted us in our stay in the Republic.

March 1963

Takeo Fukuda
Head, Survey Team of Japan

Organization of the Survey Team

The Survey Team was composed of the following members.

Chief ;	TAKEO FUKUDA Professor, University of Tokyo
Member ;	MASAJI SAGARA Ministry of Construction.
Member ;	KINZO YASUNAMI Mitsubishi Shipbuilding & Engineering Co., Ltd.
Member ;	TOMOJI MATSUI Japan Engineering Consultant Co., Ltd.
Member ;	SHUJI KOIKE Miyaji Iron Works Co., Ltd.
Associate Member;	EITARO ITO Miyaji Iron Works Co., Ltd.
Associate Member;	MICHIHIKO SUGIYAMA Yawata Iron & Steel Co., Ltd.

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Chapter 1. On Toll Bridge System

1-1. General remarks

One of the solutions which ensure a rapid development for an outdated traffic network with a limited source of fund is the concurrent comployment of the toll way system. This goes to signify that while the bridges are built as the objects of the continued general public project, part of the fund is set aside for investment on the toll way system, so that a resular re-investment may be kept on for the bridge construction from the resulting returns of annual revenues of toll. This solution has been employed in Japan for the past 10 years with remarkable success.

In Japan, a special high-way account has been set up for construction of a high-way system, capitalizing mainly on the revenues from gasoline tax, from which part of the investment is diverted to the toll-way system. On the other hand, some public corporations (half-state and half-civilian enterprise) have been created to deal with planning and execution of the business related with toll bridges and toll ways.

The toll system finds its priority sites at such places where a significant amount of traffic and hence a promising outlook for investment efficiency is present, or/and which may constitute an important part of a scenic network. Sharing the majority of the capital funds for these public corporations ,the Japanese government also provides the guarantee for the business balance and at the same time forbids any profit-seeking activities on the part of such corporations.

It is considered highly advisable that Colombia takes up the similar policy of toll system as an auxiliary means of promoting the development of her high-way bridges.

In connection with the proposal regarding the employment of the toll system, it is also considered advisable that some related factors, as described below, must be taken into consideration.

i. Even though the toll business should be handled on the basis of commercial management participated by the civilian capital, it is desirable for the government to subsidize part of the capital fund. This policy will help open ways to such favorable by-effects as participation of foreign capital and prolongation of the refund period, also enabling the government to exercise a real voice in the management of the corporation.

ii. In case the revenue exceeds the anticipated rate and thus earlier completion of refund is made possible, it is necessary to impose a new restriction on the profit level and place the high-way utility for free use by the public by transferring the control of business to the government responsibility in an earlier date, for such is the most desirable outcome any public utility such as a high-way bridge is destined to.

iii. Planning of a toll bridge is based on the estimated volume of traffic, and therefore strongly subject to uncertainties as compared to the common civilian enterprises at large. Consequently, allowances must be made to cover the possible deficit with an extra government subsidy or extend the period of toll until it is deemed necessary, in case the toll revenue should fall short of the anticipated rate.

1-2. Tentative estimation regarding Barranquilla Bridge

This investigation mission has found out that the ferry transport at the estuary of River Magdalena presents the gravest deterrent to the general flow of traffic in this region, thus calling for the most pressing measure of innovation. Described below is a study on the possibility of a toll bridge at the ferry site, dealing categorically with its design assessments, the traffic estimaticn and the balance prospect.

The site under study occupies a strategic midway point connecting the Barranquilla port, most important port in Colombia, and the Cartagena district lying in the southwest, with the Santa Marta district, principal agricultural region and famed scenic spots. It is anticipated that as the Pan American High-Way Project proceeds, a good use will be made of this site for a considerable period of time as part of the significant trunk route proceeding all the way from Panama, via Colombia, to Venezuela.

A. General description

a. Bridge site. Two possible sites are conceivable, one at the present position of the ferry (about 680 m long across the river surface) and another at about 1km upstream where an ait can be utilized as the intermediate foot-hold. Comparison of their cost assessments shows a greater expense for the latter which calls for a larger bridge length than the former, and accordingly the choice has been made in favor of the former.

b. Span length and clearance. Assuming that the river boats, likely of the comparable size to the present one, will continue to operate in the future here, it is proposed to provide roughly 3 spans with clearance of 25 m.

c. Overall plan of toll. On the right-bank side, a longitudinal gradient shall be provided at the approach way in conformity with the central curvature of the existing high-way, which continues out into the seciton of the embanked high-way. The side lanes shall be provided on both sides of the existing high-way as a substitute.

On the left-bank side of the river, there shall be a special section of about 500 m in length built as the approach-bridge structure, which leads away to the ordinary high-way and is finally connected to the CRA 38, the artery route entering into the port sector. The office buildings, the toll gate, the parking lots, and other necessary installations shall be located in the vicinity of this conjunction.

Summing up, the general layout comprises:

Bridge structures	1,560 m
Approach ways	2.550 m (including 100 m of bridge structure)
Total	4,100 m

d. Width. If it is assumed that the automobile traffic rises to 10,000 vehicles per day about 25 years later, the peak intensity of traffic will then mark statistically about 10% of this value, i.e. 1,000 vehicles per hour. Following the accepted criterion that the practical traffic efficiency is confined to about 950-1,000 vehicles/hr

Fig. 1-1 P

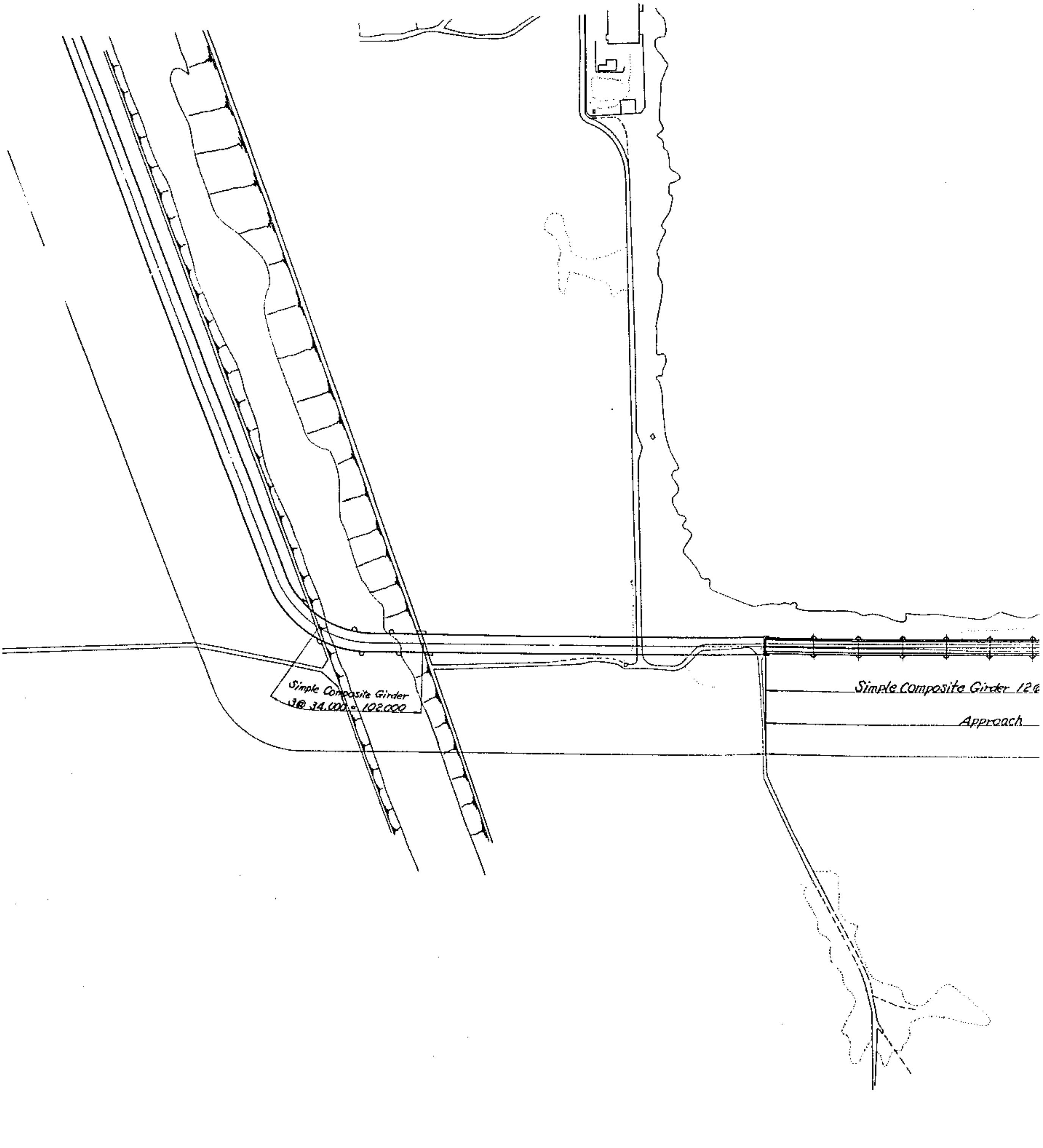

Simple Composite Girder
3@ 34,000 = 102,000
Simple Composite Girder 12@
Approach

Fig. 1-1 Planc of Barranquilla-Bridge (No.1)

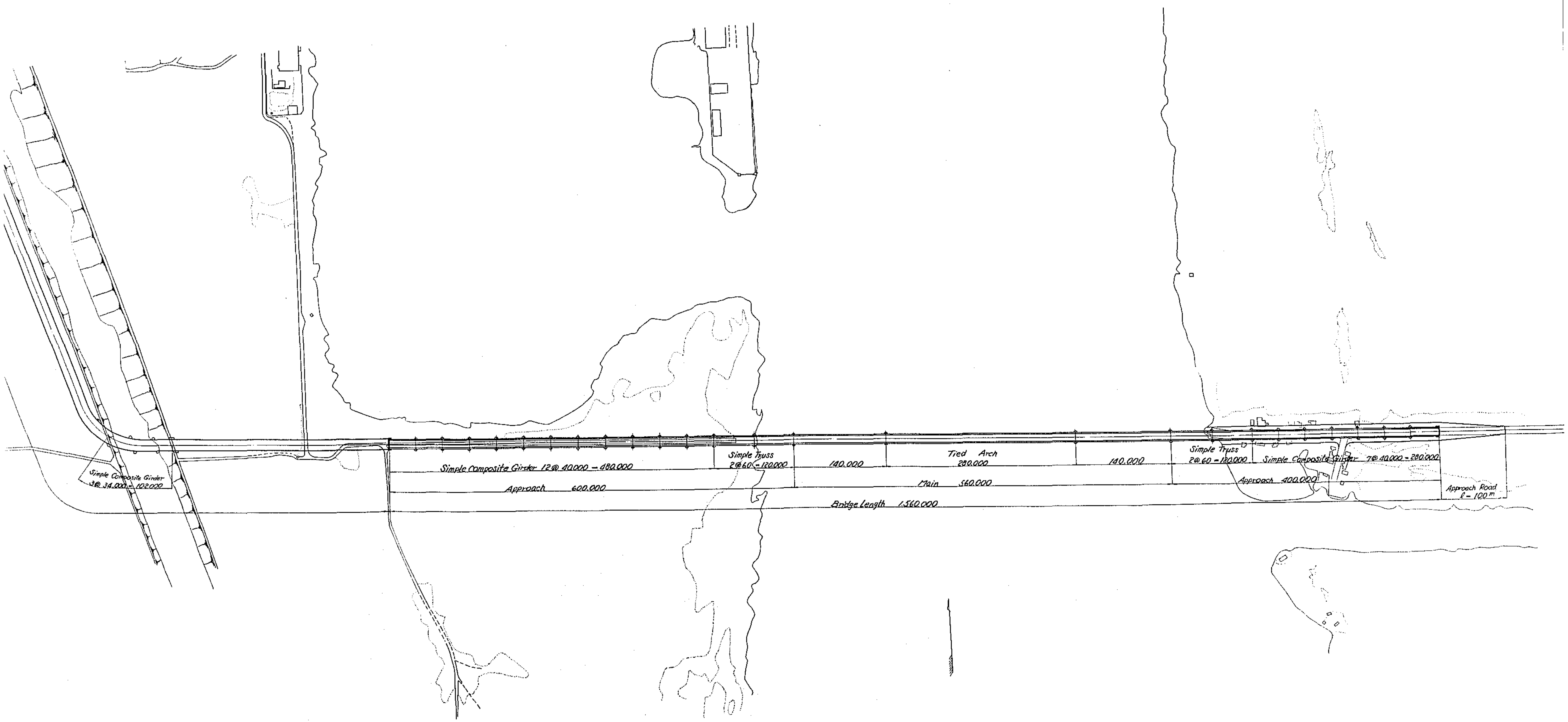


Fig.1-2 Planc of Barranquilla-Bridge (No.2)

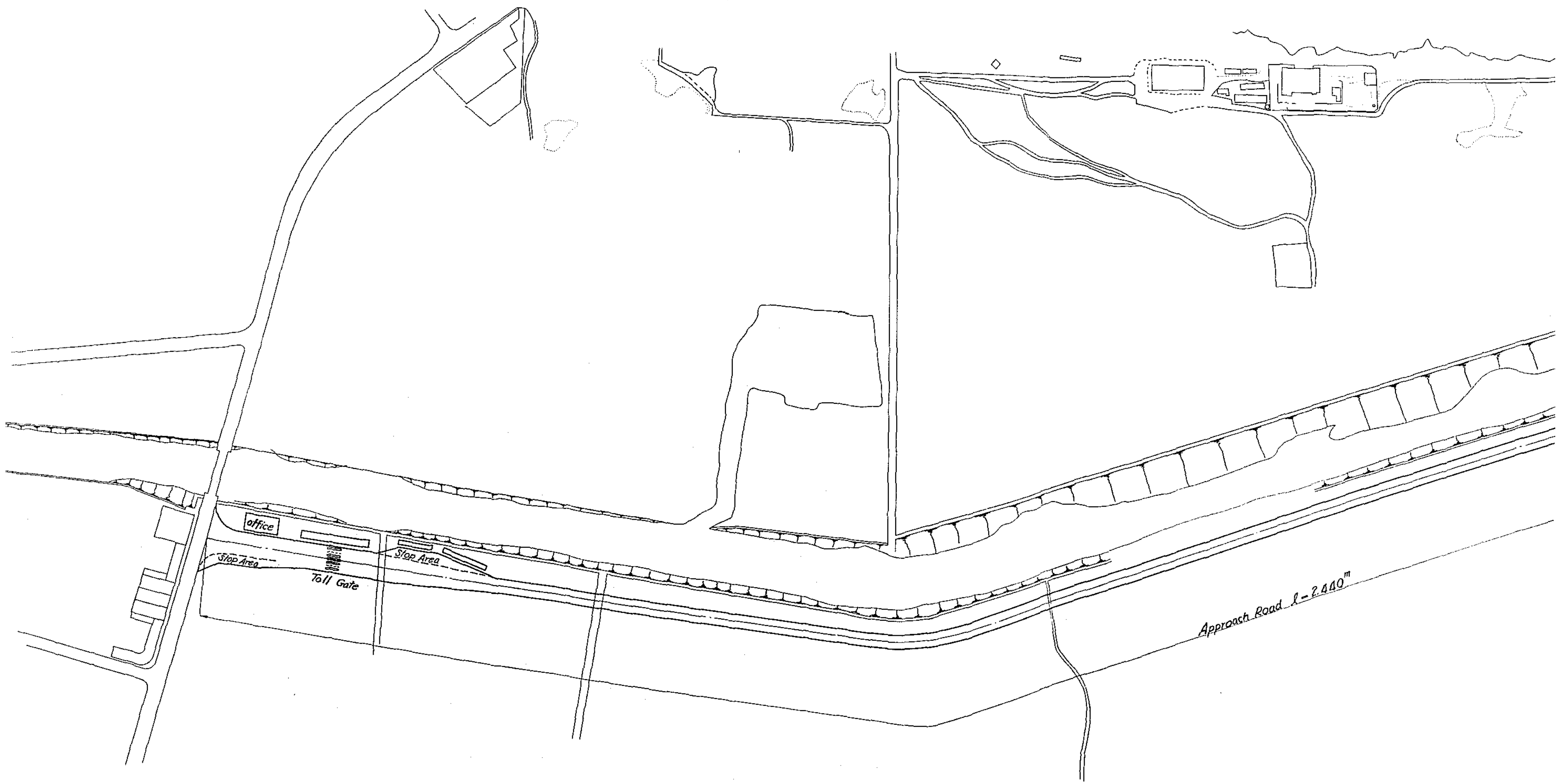


Fig. 1-2 Planc of Barranquilla-Bridge (No. 2)

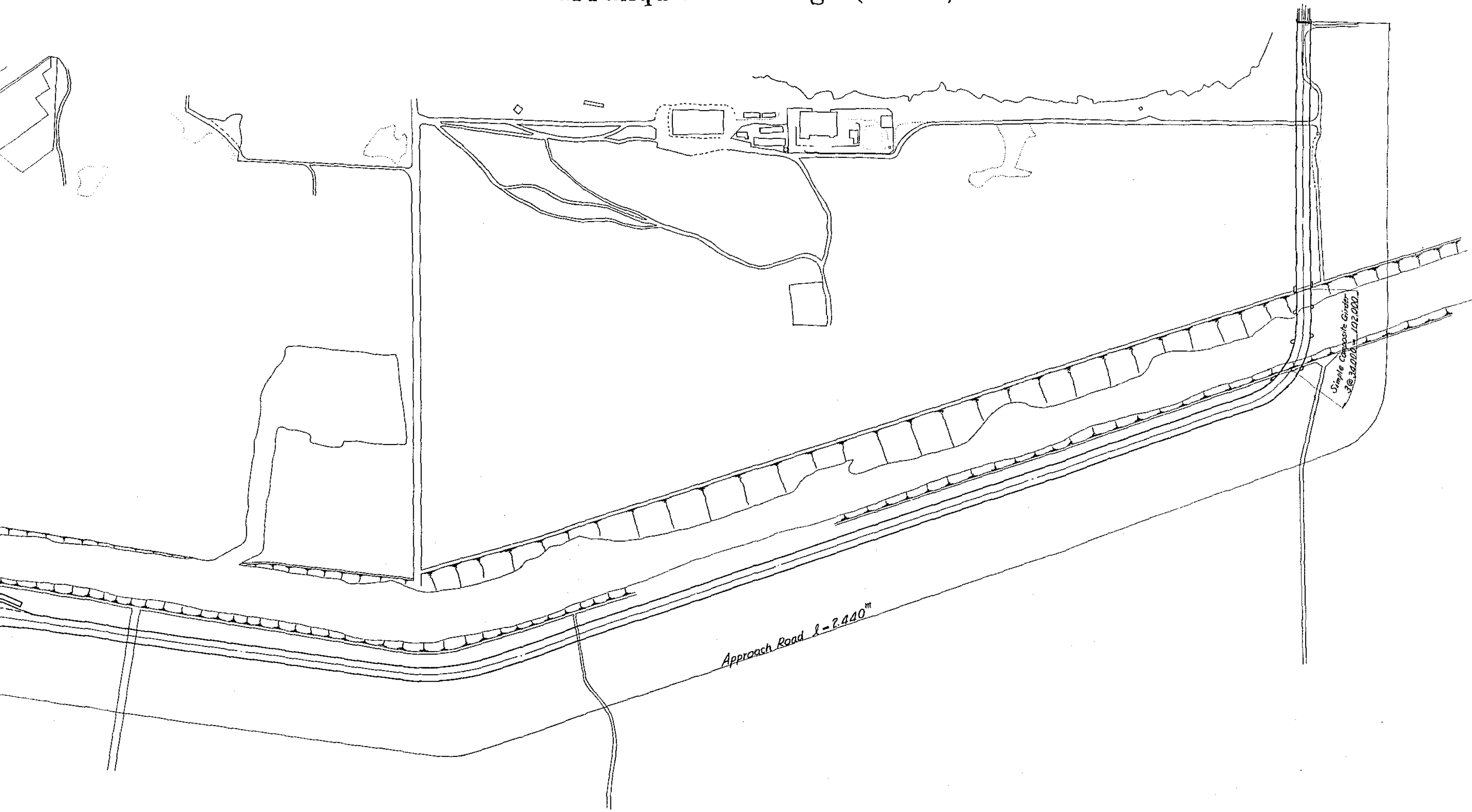
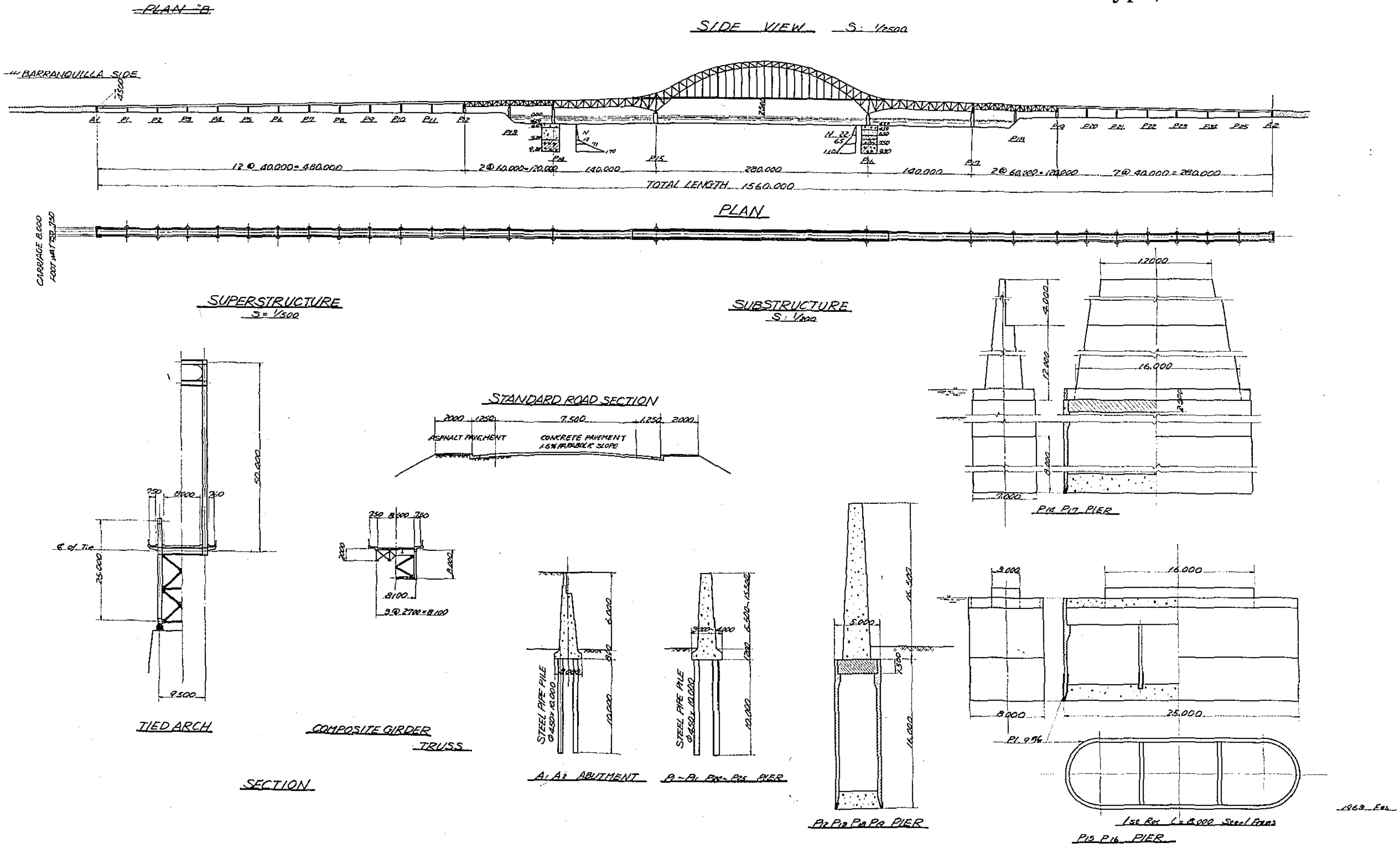
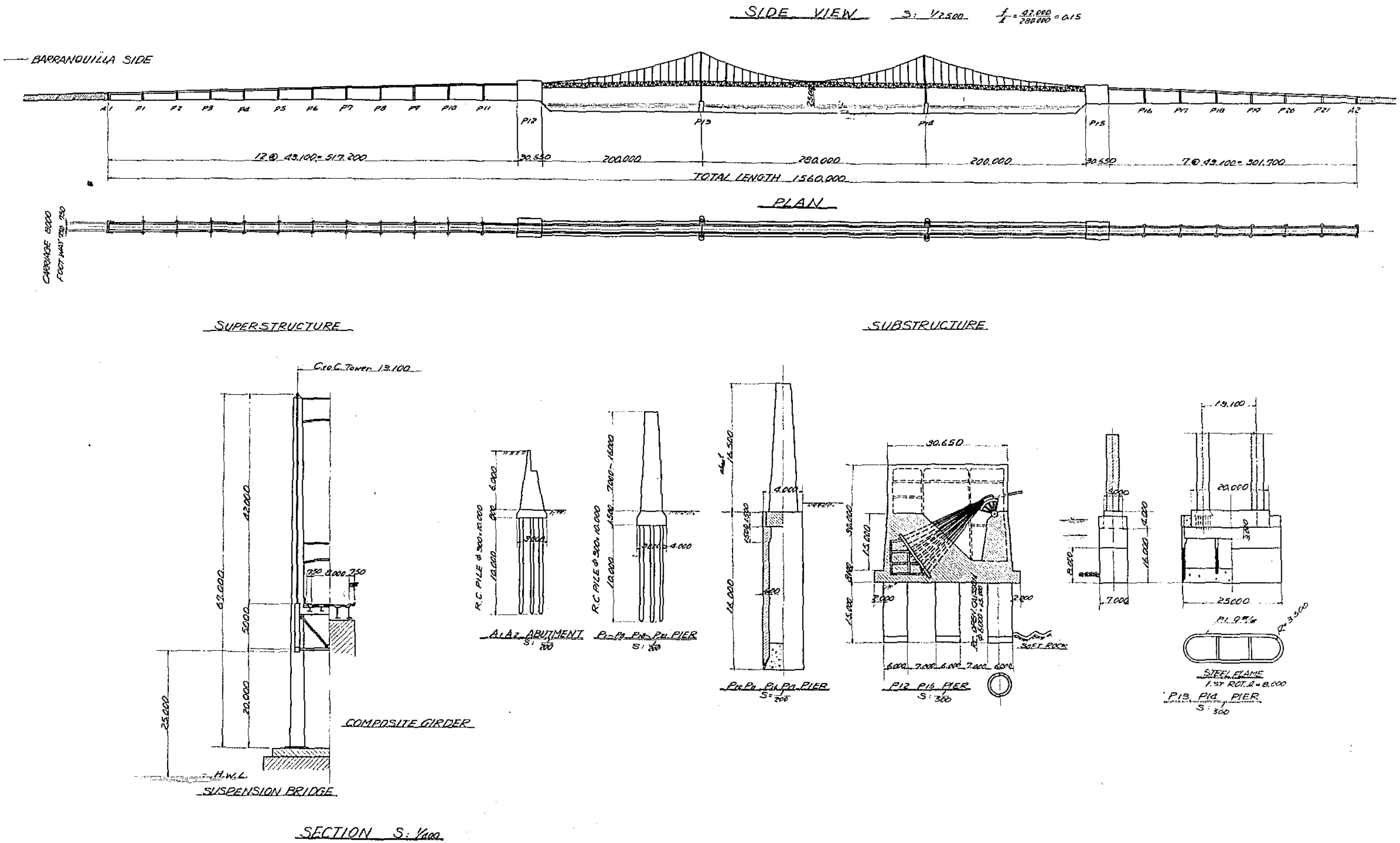


Fig. 1-3 General View of Barranquilla-Bridge (Tied Arch-Type)



~~BARRANQUILLA-BRIDGE~~
~~PLAN-A~~

Fig. 1-4 General View of Barranquilla-Bridge (Suspension-Type)



for the lane width of 7.5 m, and also taking into account some allowance of width, we arrive at 8.0 m as the optimum bridge width apart from the pedestrian alleys to be installed on both sides.

The basic width of the approach way will be 14 m in all, comprising the pedestrian alleys of 2 m wide running on both hems of the vehicle route, the vehicle lane of 7.5 m wide, and the treated shoulders.

e. Structure and construction. The foundation of the sub structure shall be biult with open caissons and pile foundations. For the super structure, a comparative assessment has been made between the tied-arch type based on cantilever erection and the suspension type based on the cable erection. The result shows that the tied arch type is preferable, because, as shown in the separate table, this ensures a better economy for the total construction of the super and sub structures. For the approach structures, it has been found out that the trusses and composite beams represent the most economical type in view of the necessary sub structures corresponding to the span lengths.

B. *Eestimation of cost*

a. Super structure. The following conditions are assumed to affect our estimation of the cost.

i. SM41, SM50, and SM60 shall be used for principal steel material.

ii. Coating shall be done in three layers, comprising the base, medium and surface layers.

iii. Erection shall be done by the cantilever procedure for the main structure and by the staging procedure for the approach structure.

iv. The factory products shall be fabricated primarily by welding, while the field assembly by revetting.

The resulting estimation of the cost of the super structure is shown categorically in Table 1-1.

Table 1-1 Cost of Super-structure (Tied Arch)

Description	Weight in ton	Unit Price in Yen	Integral Price in 1,000 Yen
① Main Structure			
main materials	3,192	60,000	191,520
auxiliary materials	3,192	7,400	23,620
woking	3,192	46,500	148,430
test and packing	3,192	6,000	19,150
transport (by land)	3,192	2,600	8,300
transport (by sea) Japan→Barranquilla	3,192	21,600	68,950
erectin, concrete works	3,192	90,000	287,280
quarantee 0,5%	3,192	1,170	3,750
expenses 10,0%	3,192	23,500	75,100
Total			826,100
② Approach Structure			
main materials	2,272	59,000	134,050
auxiliary materials	2,272	7,400	16,810
working	2,272	45,000	102,240
test and parking	2,272	6,000	13,630
transport (by land)	2,272	2,600	5,910
tranport (by sea) Japan→Barranquilla	2,272	21,600	49,100
election concrete works	2,272	45,000	102,240
quarantee 0,5%	2,272	1,150	2,110
expenses 10,0%	2,272	18,800	42,610
Total			468,700
Grand Total			≒1,295,000

b. Sub structure. The following conditions are assumed to affect our cost *estimation of the sub structure.*

i. The foundation of the piers which stand in the river stream shall be built by the open-caisson procedure. However, the wall of its first-lot, ranging over 8 m in length, shall be built by placing pre-packed cement concrete into the exterior cylinder of steel which will be assembled in advance on the land and sunk into position from a boat.

ii. Open-caisson procedure shall be applied to the pier foundations which rest on the land sides lying close to the stream, in order to allow for the future possibility that the stream width may expand at this point due to progress or river control works or reclamation of the flood plains for other utility purposes.

iii. Pile-foundation technique shall be applied to other foundations standing on the land, since it is justifiable and economical in view of geological factors and environments. A steel cylindrical pile having reliable joints and easy for driving, is adopted as preferable to a reinforced concrete pile, because the latter is liable to cracks due to driving impacts and whose joints are unreliable. However, a jointless reinforced concrete pile may be used.

iv. The fenders are necessary for the bridge piers in the stream against a hazard of collision by boats. They are so designed as to allow least damage to a colliding boat as well as the pier.

The resulting estimation of the cost of the sub structure is shown categorically in Table 1-2.

Table 1-2 Cost of Sub-Structure

Description	Unit Price in 1,000 Yen	Quantity	Integral Price in 1,000 Yen	Remarks
A_1 A_2	4,000	2	8,000	steel pipe pile 450ϕ×10m 10 pieces
P_1~P_{11}, P_{20}~P_{25}	5,000	17	85,000	〃 〃
P_{12}, P_{13}, P_{18}, P_{19}	20,000	4	80,000	open caisson 18m×5m×16m
P_{14}· P_{17}	75,000	2	150,000	open caisson 20m×7m×16m 1-lot 8m steel frame
P_{15}, P_{16}	93,000	2	186,000	open caisson 25m×8m×16m 〃 〃
Protection	15,000	4	60,000	
Beacon			16,000	
Total			585,000	

c. Approach way and other related facilities. The following conditions are assumed to affect our cost estimation of the approach way and other related facilities.

i. 8 lanes shall be provided at and in-and-out of the toll gate, comprising 2 lanes for automobiles, one lane for the trucks, and another lane for buses and other special types of vehicles, for each direction.

ii. Beneath the bridge shall be provided an inspection way for surveillance and maintenance operations.

iii. The measures for traffic safety shall comprise guard-rails, illuminations and other counter-accident installations.

iv. Such measuring instruments as a wind meter and vibration meter shall be installed to provide data to aid control measures.

The cost covering these installations is shown in table 1-3.

Table 1-3 Cost of Approach Way and other Equipments

Description	Quantity	Cost in 1,000 Yen	Remarks
approach road	2,440 m	195,000	W=2.0+1.25+7.5+1.25+2.0=14
bridge	100 m	105,000	curved bridge
across bridge	3	30,000	
safety equipments		10,000	
check road	300 ton	40,000	
measuring instruments		15,000	
buildings		65,000	office, toll gate, warehouse tavatories, etc.
Total		460,000	

d. Total cost. The totatl cost is summarized as follows.

Super structure	$1,295,000 \times 10^3$ Yen	
Sub structure	585,000 "	
Approach way, etc.	460,000 "	
Total	2,340,000 "	= US$6,500,000

e. Comparative design. A comparative design has been done tentatively (as shown in Table 1-4), by assuming a suspension-type for the main part of the bridge; however, it has been found out that the cost of the suspension bridge exceeds that cf the proposed tied-arch type. (Fig. 1-2)

Table 1-4 Cost of Comparative design (Suspension)
(Part 1) Supper structure

Description	Weight in ton	Unit Price in Yen	Integral Price in 1,000 Yen
① Main Structure			
main materials	3,535	80, 000	282,800
auxiliary materials	3,535	9, 000	31,810
working	3,535	45, 000	159,075
test and packing	3,535	6, 000	21,210
transport (by land)	3,535	2, 000	7,070
transport (by sea) Japan→Barranquilla	3,535	20, 000	70,700
erection, concrete works	3,535	60, 000	212,100
quarantee 0,5%	3,535	1, 110	3,923
expenses 10,0%	3,535	22, 220	78,477
Total			867,165
② Approach Structure			
main materials	1,945	59, 000	114,760
auxiliary materials	1,945	7, 400	14,390
working	1,945	45, 000	87,525
test and packing	1,945	6, 000	11,670
transport (by laed)	1,945	2, 600	5,060
transport (by sea) Japan→Barranquilla	1,945	21, 600	42,010
erection, concrete works	1,945	40, 000	77,800
quarantee 0,5%	1,945	910	1,770
expenses 10,0%	1,945	18, 250	35,500
Total			390,485
Grand Total			1,257,650

(Part 2) Sub structure

Description	Unit Price in 1,000 Yen	Quantity	Integral Price in 1,000 Yen	Remarks
A_1, A_2	4,000	2	8,000	
$P_{[illegible]}$~$P_{[illegible]}$, $P_{[illegible]}$~$P_{[illegible]}$	4,000	18	72,000	
$P_{[illegible]}$, $P_{[illegible]}$, $P_{[illegible]}$, $P_{[illegible]}$	15,000	4	60,000	
$P_{[illegible]}$, $P_{[illegible]}$	65,000	2	130,000	
$P_{[illegible]}$, $P_{[illegible]}$	310,000	2	620,000	anchor ·
protection beacon			40,000	protection 15,000×2 beacon 10,000
Total			930,000	

(Part 3) Approach way and other equipments

Description	Quantity	Cost in 1,000 Yen	Remarks
approach way	2,440m	195,000	
bridge	100m	105,000	
across bridge	3	30,000	
safety equipments		10,000	
check road	300ton	40,000	
measuring instraments		15,000	
buildings		45,000	using anchor pier
Total		440,000	

(part 4) Total cost

Description	Cost in 1,000 Yen	Remarks
supper structare	1,257,000	
sub structure	930,000	
approach and others	440,000	
Grand Total	2,627,000	> 2,340,000

C. Estimation of traffic volume

a. General trend of increasing traffic. The increase of traffic depends upon the population, direct and indirect potentialities of consumption, development of industrial and economical factors, and the sociological way of life inside the region in question. However, it is acknowledged that the traffic will generally grow through 4 discernible phases of evolution as described below.

1. phase of trial;
2. phase of growth or phase of morphosis during which the traffic is transformed into the essential part of social functions;
3. phase in which the rate of growth declines; and
4. phase of stability.

Thus, it is understood that the traffic volume grows steadily by year, nearing eventually to the state of ultimate limit, as the regional economical growth approaches its final maturity or as the number of vehicles comes into saturation or due to the less flexible limitations resulting from the regional accommodations of traffic.

Such a long-term evolution of traffic will be expressed in general by a differential equation as follows:

$$\frac{dy}{dt} = Y \cdot g(t) \cdot F\left(\frac{Y}{K}\right)$$

where Y = traffic volume (vehicles/day),
t = time, and

K = limit volume of traffic (vehicles/day).

In solving this equation, a frequent use is made of the numerical computation procedure, after substituting it with a logistic curve or Gompertz curve. However, there arises a difficulty on how to determine the integral constants.

In determining the constants, a macroscopic procedure tries to estimate the cruising vehicle-kilometers on the basis of the relationship between the national product and the traffic volume, while a cumulative procedure capitalizes on analyzing the characteristics of a region in question and taking into consideration other related environments. These two procedures have merits and shortcomings of their own, and it is most desirable that the results obtained independently from these two procedures should agree well with each other.

b. Traffic estimation at Barranquilla Bridge. The basic data is too scant to allow a theoretical estimation of the future probable volume of traffic at the proposed position of the bridge, and neither macroscopic nor cumulative procedures may give but an inaccurate conclusion. On the other hand, it seems that we may be subject to a lesser degree of errors if use is made of the experiences associated with similar conditions which have been already analyzed and confirmed in practices in Japan.

The following Fig. 1-5 and 1-6 show a procedure currently in use in Japan, which have been tried by both macroscopic and regional checkings and confirmed as statistically justifiable. This procedure of analysis employs the Gompertz curve.

Precautions must be exercised in utilizing these diagrams. Classification must be made into the ranges A, B, and C by considering the regional characteristics and other conditions, and then an appropriate value determined within each region.

Another subject of precaution is that some adquate modification must be introduced if the current volume of traffic fails to give an appropriate criterion for estimating the future trend. In other words, in cases where a passing obstruction affects temporarily the traffic volume, or where a sudden change of trend is foreseen in the near future due to transformation of traffic patterns which might result from, namely, abolition of railways, it is necessary to introduce some hypothetical modification into the existing data of traffic volume to obtain an adequate basis of trend analysis.

As far as the Barranquilla Bridge is concerned, the impact of the planned Pan American Express High-Way and hence a considerable increase of traffic is expected in terms of trucks or other hauling vehicles. Therefore, it is advisable to adopt the upper grade of the range A for trucks, the median grade of the range A for automobiles, and the lower grade of the range A for buses.

On the other hand, the number of pedestrians does not seem to increase so rapidly as the buses and automobiles. Therefore, the increment shall be assumed to amount to 1/2 the basic number of pedestrians, with the rate of increase resting at the lower limit of the range C.

It shall be further assumed that construction of the bridge takes about three years, and that it is to be inaugurated for public use in 1966. Thus, our estimation shall extend to the times 10 and 20 years after the reference year of 1966.

Assuming that the rate of increase of the traffic volume from 1962 to 1966 is 1.6~1.7, the annual average of the daily traffic volumes is estimated, as shown in Fig. 1-5, and 1-6.

(Notice) **Characters of Highway**

range A: Primarily used by the secondary industries such as the manufacturing industries, or where a remarkable increase of traffic volume is anticipated over a long period of time due to some other reasons.

range B: Highways not falling in the categories A and C.

range C: Primarily used by the primary industries such as agriculture and forestry.

Modification of current volume of traffic

In case there is a definite outlook that the innovated highway may attract a transferred volume of traffic from other transportation routes, the current volume is to be modified by adding the optimum amount of the transferred volume.

Table 1-5 Estimated volume of traffic (vehicle/day)

Types of vehicles	Traffic volume in 1962	1966 (reference)		1976		1986	
		Rate of increase	Traffic volume	Rate of increase	Traffic volume	Rate of increase	Traffic volume
Bus, Special vehicle	170	1.7	290	2.5	725	5.8	1,682
Trucks	150	1.6	240	3.6	864	9.0	2,160
Automobiles	380	1.7	640	3.0	1,920	7.0	4,480
Total	700		1,170		3,509		8,322
Pedestrians	600	1.4	840	1.7	1,428	2.5	2,100

Fig. 1-5 Traffic volume, 10years later (estimated rate of increase)

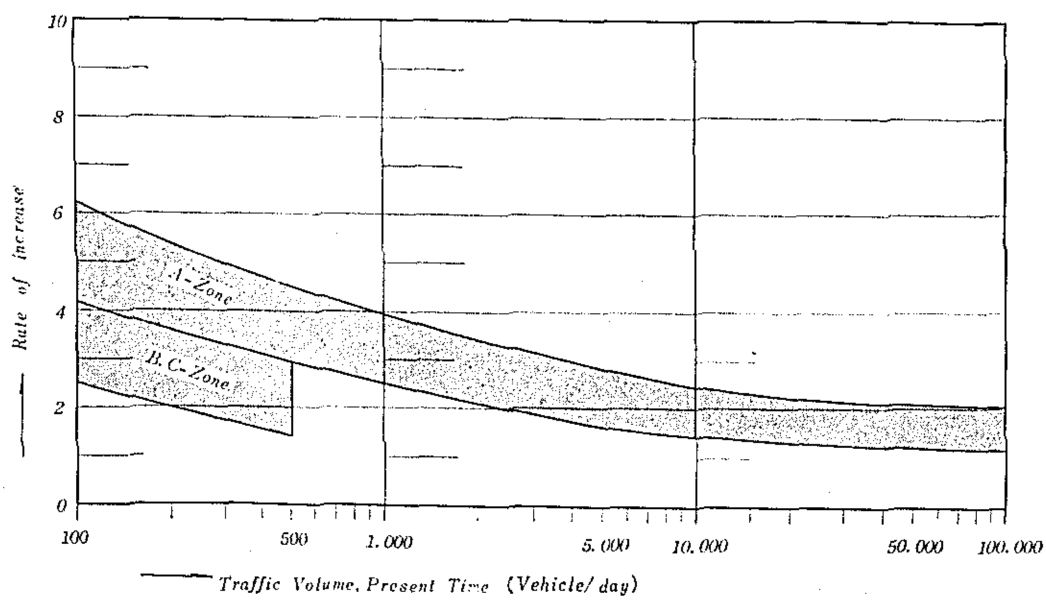
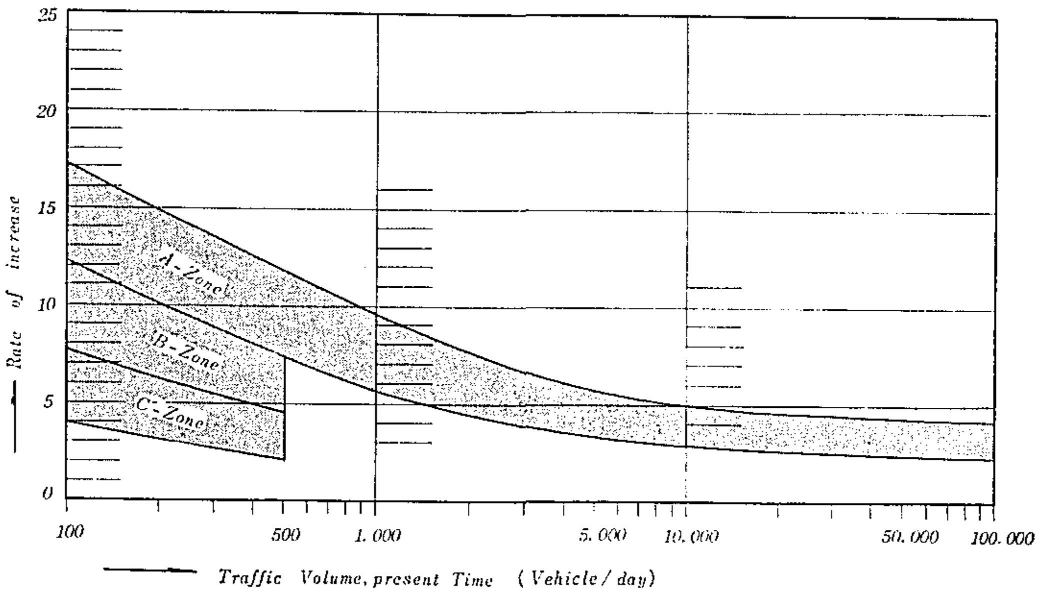


Fig. 1-6 Traffic volume, 20years later (estimated rate of increase)



D. Toll charges and balance prospect

The toll charges must be specified, primarily, within the extent of the benefit which the public may draw by using this utility. However, also to be taken into consideration are other related factors and policies. No sufficient data is available now to determine the optimum charge for this bridge. Accordingly, it shall be only attempted here to give a rough estimation of the balance prospect for the case where the toll charge is to be strictly equal to the present ferry charge (Table 1-6A), and the case where the former isgreater by 50% than the latter (Table 1-6B).

Table 1-6 Toll Charges

Types of vehicles	Current ferry charges	Case A, equal to ferry charges	Case B, 50% higher than ferry charges
Bus, Special vehicles	12 Peso	12 Peso	18 Peso
Trucks	10 〃	10 〃	15 〃
Automobiles	5 〃	5 〃	8 〃

In the computation of the revenue, it shall be assumed that it undergoes a linear change during the intermediate periods between the rference years of 1966, 1977 and 1986. It shall be further assumed that the requisite fund for construction, amounting to US$6,500,000 in total, comprises the investment of US$1,620,000 (about 25%) which may be refunded from the dividened of the annual toll revenue of 10-12% after inauguration of the toll way, and the remaining US$4,880,000 which may be redeemed at a rate compatible with the annual revenue by compound-interests of 7.5 and 8% per year. No charge shall be demanded from a pedestrian.

Thus, the estimation leads to the results shown in Tables 1-7 and 1-8.

Table 1-7 Balance sheet (in US$ 1,000)
Case A. Toll charges assumed to be equal to ferry charges
(12 peso for buses and special vehicles; 10 peso for trucks; 5 peso for automobiles.)

	Year	Capital investment	Loans			Revenue	Expenditures			Remarks
			Debt to refund	Interest (7.5%)	Total		Maintenance	Investment dividend	Refund payment	
	1963	1,620	880	66						
	64		1,500	183						
	65		2,500	385						
Total		1,620	4,800	634	5,514					
1	1966		5,514	414	5,928	368	200	10% 162	6	
2	67		5,922	444	6,366	440	200	162	78	
3	68		6,288	472	6,760	513	200	162	151	
4	69		6,609	496	7,105	585	200	162	223	
5	70		6,882	516	7,398	658	320	162	176	
6	71		7,220	542	7,764	730	200	162	368	
7	72		7,396	555	7,751	802	200	162	440	
8	73		7,511	563	8,074	875	200	162	513	
9	74		7,561	567	8,128	947	200	162	585	
10	75		7,543	566	8,109	1,024	320	162	542	
11	76		7,567	568	8,135	1,092	200	12% 194	698	
12	77		7,437	558	7,995	1,243	200	194	849	
13	78		7,146	536	7,682	1,394	200	194	1,000	
14	79		6,682	501	7,183	1,545	200	194	1,151	
15	80		6,032	452	6,484	1,696	320	194	1,182	
16	81		5,302	398	5,700	1,847	200	194	1,453	
17	82		4,247	319	4,566	1,998	200	194	1,604	
18	83		2,962	222	3,184	2,149	200	194	1,755	Residue
19	84		1,429	107	1,536	2,300	200	194	1,536	370
20	85	1,620	0	0	0	2,451	320	194	1,620	317
									Total	682
					682 = 42% of capital fund					

Table 1-8 Balance sheet (in US $ 1,000)
Case B. Toll charges assumed to be 50% higher than ferry charges
(18 peso for buses and special vehicles, 15 peso for trucks, 8 peso for automobiles.)

	Year	Capital fund	Loans			Revenue	Expenditures			Remarks
			Debt to refund	Interest (8%)	Total		Maintenance	Capital dividend	Refund payment	
	1963	1,620	880	70						
	64		2,000	236						
	65		2,000	415						
Total		1,620	4,880	721	5,601					
1	1966		5,601	448	6,049	565	200	10% 162	204	
2	67		5,845	468	6,313	676	200	162	314	
3	68		5,999	480	6,479	787	200	162	425	
4	69		6,054	484	6,538	899	200	162	537	
5	70		6,001	480	6,481	1,010	320	162	528	
6	71		5,953	476	6,429	1,121	200	162	759	
7	72		5,670	454	6,124	1,232	200	162	870	
8	73		5,254	420	5,674	1,343	200	12% 194	949	
9	74		4,725	378	5,103	1,455	200	194	1,061	
10	75		4,042	323	4,365	1,566	320	194	1,052	
11	76		3,313	265	3,578	1,677	200	194	1,283	
12	77		2,249	184	2,433	1,909	200	194	1,515	Residue
13	78		918	73	991	2,141	200	194	991	756
14	79	1,620	0	0	0	2,372	320	194	1,620	38
									Total	749
					749 = 49% of capital fund					

The maintenance cost may be specified as follows.

a. Operating cost of office

personnel	(for about 50 people)	\$92,000 per year
operation		58,000 "
miscellaneous		20,000 "
total		170,000 "

b. Maintenance cost

pavement	\$0.25/m² × 50,000	\$13,000 per year
illumination, etc.		17,000 "
total		30,000 "

c. Coating cost

Coating shall be done every 5 years. Partial repair shall be entered into the maintenance category. 6,000 ton of steel is allocated for construction of the inspection way and the security installations for traffic safety.

Coating cost \$20/ton × 6,000 ton = \$120,000

Chapter 2. Rational Type and Erection of Bridge

2-1. Types of Bridge and these characteristics

A. Types of Superstructure

Of a variety of factors which depend upon material and mechanical systems to be adopted, it is necessary to choose the most adequate type and erection technique which are best suited to the topographical, gological and other conditions at this site.

A steel bridge, generally characterized by homegeneity of quality and a high degree of manufacturing precision, is the most reliable type of structure. The merits which are attributed to a steel bridge consist of its light dead load which may impose less burden on the substructure, the facility in actual erection which makes it suitable as a long-span bridge, and the diversity of the possible types of structure which allows free access to appropriate types and erection techniques. On the other hand, a shortcoming is also present with a steel bridge that high maintenance cost is needed because coating must be repeated regularly.

The most economical type of steel bridge having the medium or short spans, is the composite-girder type which compounds in principle the steel girder and the concrete slab. In recent years, an impressive advance has been achieved in the stress analysis technique, and high-tension steel with good weldable properties is made available. Therefore, the applicable span lengths have been increased for each type of steel bridge. By utilizing the steel slabs which relieve the dead load, it is possible today to build a long-span bridge which has so far been the exclusive field of truss.

At such a site where the concrete aggregate is available in abundance in the neighborhood, and where the geological properties of the ground ensures a sound bearing capacity, a reinforced concrete structure is highly economical as a medium or short span bridge. The shortcoming that a bulky substructure having large bearing power is needed due to its heavy dead load, is counter-balanced by the merit that the heavy load itself reduces vibration and noise and thus makes the car driving most pleasant.

As a means of releaving the dead load of the reinforced concrete slab bridge, hollow tubes may be built in, and thus a hollow type bridge is applicable to a span as long as 10~18 m.

Table 2-1 Types and conformable span length

Materials	Types	Span length in m	Materials	Types	Span length in m
Reinforced concrete	Slab	< 10	steel	H-beam	< 15
〃	hollow-slab	10 ~ 18	〃	composite-beam	15 ~ 45
〃	T-beam, rigid-frame	15 ~ 25	〃	continuous	40 ~ 60
〃	cantilever, arch	20 ~ 60	〃	*steel deck plate*	60 ~ 120
Prestressed cncrete	slab	< 15	〃	truss	40 ~ 80
〃	T-beam	15 ~ 45	〃	Langer	60 ~ 140
〃	cantilever, continuous	25 ~ 60	〃	arch	50 ~ 300
〃	Dywidag	40 ~ 80	〃	suspension	> 120

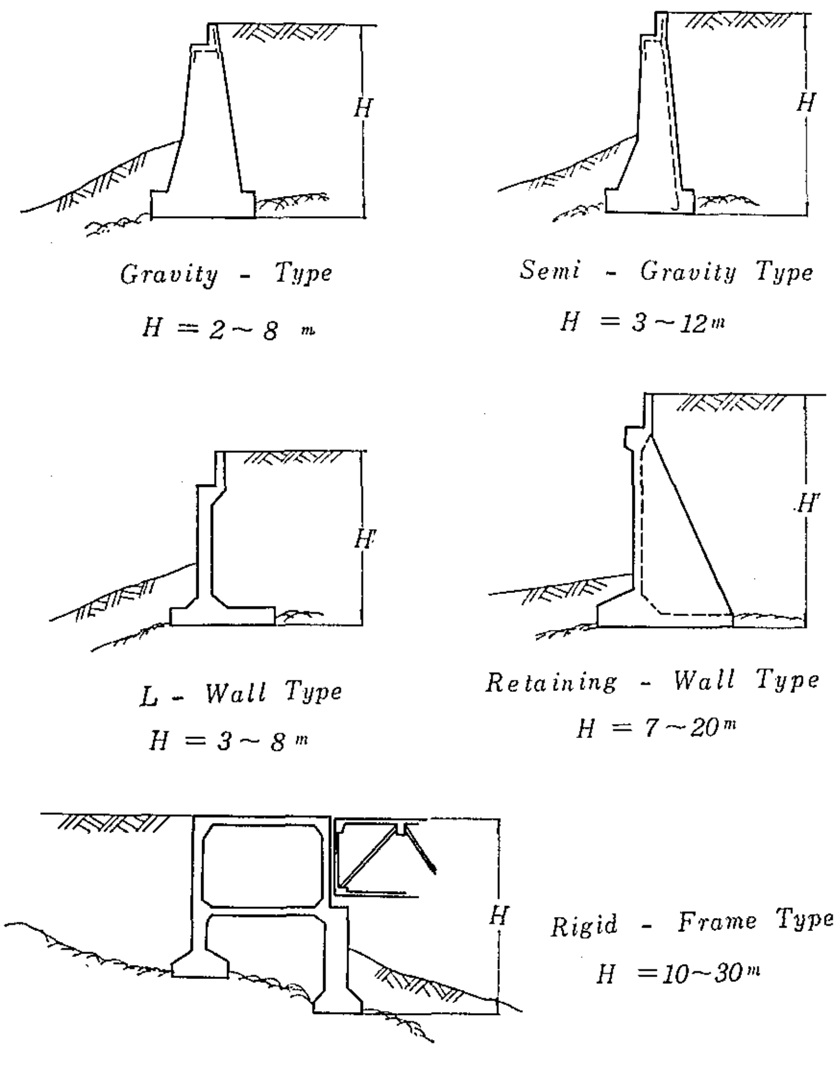
PC concrete has gained a wide popularity. The pre-tensioning technique is applied to a slab bridge, while the post-tensioning technique to a girder bridge. They are accredited with the characteristics which lie between reinforced concrete and steel, and their longs and shorts are also of the medium nature.

Table 2-1 shows the types which are generally in use and their appropriate span lengths.

B. Types of Substructure

Various types of substructure are conceivable depending upon the topographical and geological conditions, but the fundamental types may be classified as shown in Fig. 2-1

Fig. 2-1 Type of abutment



Gravity type seeks stability against overturning by its own dead load alone, and hence is not too high and usually applied to relatively favorable ground conditions. Semigravity type, slightly smaller in substance volume, utilizes the tension reinforcements against an appreciable amount of tensile strengths acting upon a certain portion. For L-type, the weight of back-filling soil provides the principal factor of stability, and, for a higher structure above a certain limit, either retaining-wall type or rigid-frame type is economical.

The substructure is directly affected by the underlying ground conditions. Unless geological exploration is performed to a sufficient degree in advance of actual planning, great loss of expenses may result from the need of design modifications arising in the midst of the construction process.

The foundation work is vital to the substructure and most difficult to execute. It is desirable to adopt an appropriate technique and push forth with the study of the execution procedures. It is believed that such a policy cuold be the most favorable approach toward achieving overall economy of bridge construction in Colombia.

The simplest of the foundation work is to place the foundation on the excavated bottom of the ground. Maximum workable depth is, however, limited to about 5~6 m, beyond which excavation will become difficult.

The most flexible technique may be a pile foundation, whose types are described below:

i. Wooden pile. The pile lies below the ground water level, with the penetration depth resting within 7~8 m.

ii. Reinforced concrete pile. In general, precast products are most frequently used. Although cracks may develop easily due to driving impacts and joints are often unreliable, the precast pile can be executed without a joint down to a depth of about 12 m. The tip may be shaped as a sharp wedge or made to have a hollow, depending upon the nature of the ground conditions. It is usually considered justifiable that a hollow pile with uniform diameter which is produced by centrifugal principle is cheaper and of better quality than a pile having a slenderer diameter toward the tip.

iii. In-placed concrete pile. This technique places concrete in situ into a bore-hole in the ground, and there exist several patent procedures characterized by the structures of earth drill and the processes of concrete placing.

iv. Steel pile. Steel pile comprises steel cylinder and H pile. As far as the penetration depth is concerned, H pile is superior to cylindrical pile, but in case where adequate amount of bending rigidity is required the latter is advantageous because concrete may be placed into the hollow space.

Steel pile is characterized by facility of penetration and weldable property of joints, factors which enable a long pile to be executed, and shall be recommended for popular use for improvement of foundation technique in the future.

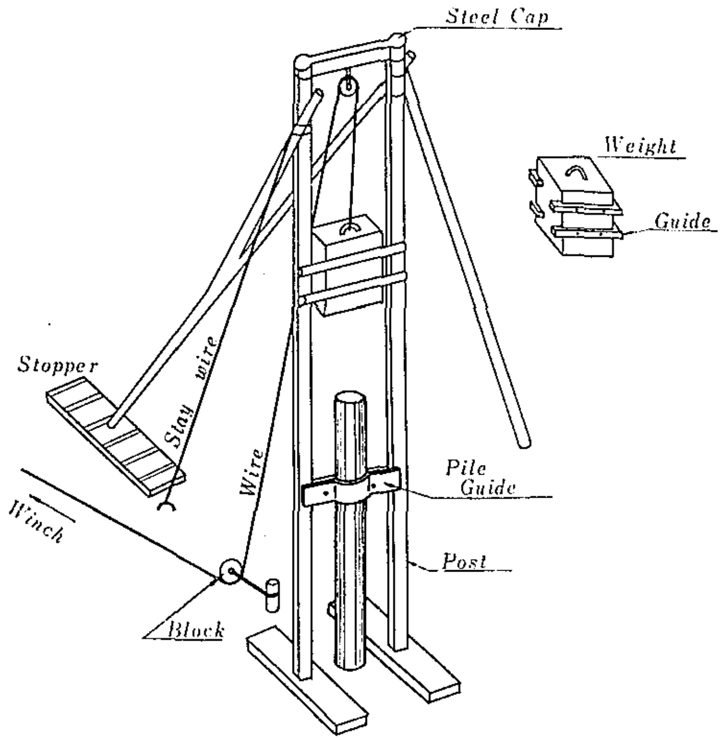
The most efficient type of pile driving machines is Diesel hammer, but there are also vibration type and water-jet process.

Fig. 2-2 shows the simple and reliable techniques which could be employed for a small-scale project of pile foundation. It is necessary to use a larger hammer weight but to refrain from making the drop length too large.

v. Open caisson. Open caisson technique may be applied to an important foundation structure at such a place where clousure or pile-driving is difficult to execute. According to this technique, a reinforced concrete cylinder with circular or elliptical cross-section is sunk into the ground by excavating and removing the interior soil, and as the cylinder penetrates deep into the ground more concrete is added on the upper section of the cylinder wall and open excavation is resumed. This process is repeated until the caisson touches the predetermined depth, and then concrete is placed on the bottom to strengthen the bearing capacity of the foundation. This technique is generally recommendable.

Pneumatic caisson technique, used in unfavorable ground conditions, has the air-tight caisson filled with compressed air, so that seepage of water may be prevented during excavation and sinking operations.

Fig. 2-2 Simple pile-driver



C. Influences of river and determination of economical span lengths.

a. Interrelationship with river. Although considerations must be provided regarding such aspects as river regimen, and control and utility of river water, one statement to follow shall be primarily confined to some precautions which must be taken its account with respect to protection of the bridge. The Japanese engineers have encountered bitter experiences that while the super structure was built as a sufficienctly durable work, the changing river conditions endangered the sub structure necessitating reconstruction of the bridge at great loss of expenses.

i. Scour. A solid body immersed in a running fluid will cause scour to its adjacent boundary of unconsolidated material. This effect is especially noticeable during floods, the scour attaining the maximum intensity with the arrival of the flood peak, only to be followed by accretion during the succeeding abatement. The extent of scour varies depending upon flood discharge, bed slope, bed material and shapes of such extent shall be considered to occur in Colombia.

In Japan, scour depth of about 2~5 m is to be anticipated for a small river, and that of proportionally increasing value for a large river. Field measurements performed in Japan gives an instance of over 10 m in scour depth, and the hazard of such extent shall be considered to occur in Colombia.

ii. Changes of river regimen. At a river where there is no revetment on the lower-water channel, the meander positions may shift, though gradually, with passage of time, and therefore it may be precarious to plan a bridge purely on the basis of the existing river regimen. It is necessary to provide various long-standing measures such as a deep embedment for a foundation which is now to be placed on an elevated river bed. On the other hand, if a meandering river is shortened by a cutoff or the channel capacity for flood accomodation is reduced, the river bed will usually tend to subside, and if further the upstream basin of the river is deteriorated, a need may arise to extend the channel width.

Therefore, it is necessary to provide such far-sighted precausions, in the case of an untamed river, as guaranteeing the foundation embedment on the basis of future estimation, or designing a bridge abutment which may be usuable as a pier if need be.

Meagre economy which is strived for at the onset of construction should not cause a great loss in the future.

iii. Clearance of girder. For the reasons stated above, a large flood discharge may well be expected to occur in an untamed river, and the maximum stage may even exceed the currently admitted peak of flood. On the other hand, if embankment is set up on a flatbottom, transported debris may settle here, raising the river bed and in consequence the waterlevel. A flood often transports washed trees and vegetal growth, piling them up against the bridge piers and thus raising the flood level upstream of the bridge.

These hazards must be studied with good care, and a sufficient height of clearance must be adopted.

In Japan, several classified clearances have been prescrived in accordance with bed gradients and whether or not the river in question is tamed. They fall in the range of 80~150 cm, allowance which is also considered possible for the rivers of Colombia.

b. Determination of Economical Span Lengths.

As far as the river code and other restrictions permit, economical span lengths of a bridge must be determined, as a matter of course. in such a way that the overall cost of the whole structure, including the super and sub structures and the approach ways on both extremeties of the bridge, may ensure the utmost degree of economy.

Setting up some zonal classifications depending upon foundation conditions and topographics, and also assuming that the cost of the supper and sub structures may be expressed as a funnction of the span length l, it is possible to determine the value of l in such away that it may correspond to the minimum value of the overall cost of construction. There are a few litertures which deal with economical values of l in terms of design conditions, and costs of material, processing and labor, regarding the bridges in Japan, whose reliability is justfied statistically in the light of actual instances.

Some results will be quoted briefly in the account to follow, which seem relatively compatible with conditions predominant in Colombia.

If it is intended to determine the most rational and economical types and techniques of the supper and sub structures corresponding to the ground conditicns and spain lengths, the economical span length l will tend to increase in general as difficulty mounts for execution of a substructure due to unfavorable ground conditions or large water depth; in other words, as the cost of the substructure increases. Thus, the proportion of the cost of the substructure against the entire cost of the bridge boils down to the values tabulated in Table 2-2, in which it is assumed that aneconomical span length has been selected and such conditions related to river regimen and navigation have been negelected. For the smilar conditicns of foundation and ground, the cost of the lower structure rises in proportion to the heights above the river bed to the bridge crest.

Table 2-2 Ratio of substructure-cost, adopting economical span length

Kinds of base	Base soil condition	h in m	Ratio of substructure-cost in % steel bridge	P.C bridge
Footing foundation	bearing layer is deep	4,5	28 ~ 30	35 ~ 40
		10	35 ~ 40	45 ~ 50
Pile foundation	closed sand	4,5	30 ~ 35	36 ~ 42
		10	40 ~ 43	48 ~ 52
	bearing layer is deep	4.5	35 ~ 40	42 ~ 48
		10	48 ~ 50	55 ~ 60
Open caisson foundation	closed sand	4.5	30 ~ 34	36 ~ 45
		10	42 ~ 48	48 ~ 50
	bearing layer is deep	4.5	38	50 ~ 52
		10	38	52 ~ 55

Notice; h=distance between the top of pier and the bottom of river

C. Example of Design.

Shown below are the examples of design at three possible sites of bridge, which employ different sets of economical span lngths and types. It must be noted that because the detailed knowledge of the ground conditions is not available at this moment, some modifications must be made in the future.

Fig. 2-3. Example Design of Langer-Girder
Huila, Las Ceibas, Sobre la Quebrada Las Ceibas, en la Carretera Nieva-Aipe.

Fig. 2-4. Example Design of Deck-Truss
Meta, Guatiquía, Sobre el Río Guatiquía, en la carretera Villavicencio-Monfort.

Fig. 2-5. Example Design of Warren-Truss
Bayaca, Mani, Sobre el Río Charte.

2-2. The adoption, in particular, of deck-type construction.

From the point of view that bridges are part of roads, it is desirable that one can pass over them free from care and unconsciously of their existence. In this regard deck bridges are preferable to through bridges, because in the former the floor lies entirely over the bridge structure whereas in the latter the structure stands out over the floor and so may hinder one's free field of vision. Most bridges now existing in Colombia belong to through type. However, as the deck-type construction offers many a technical as well as economical advantage as explained below, we recommend that bridges should as much as possible be designed as the deck type, if there is an ample clearance and unless some particular conditions or reasons require the through-type structure.

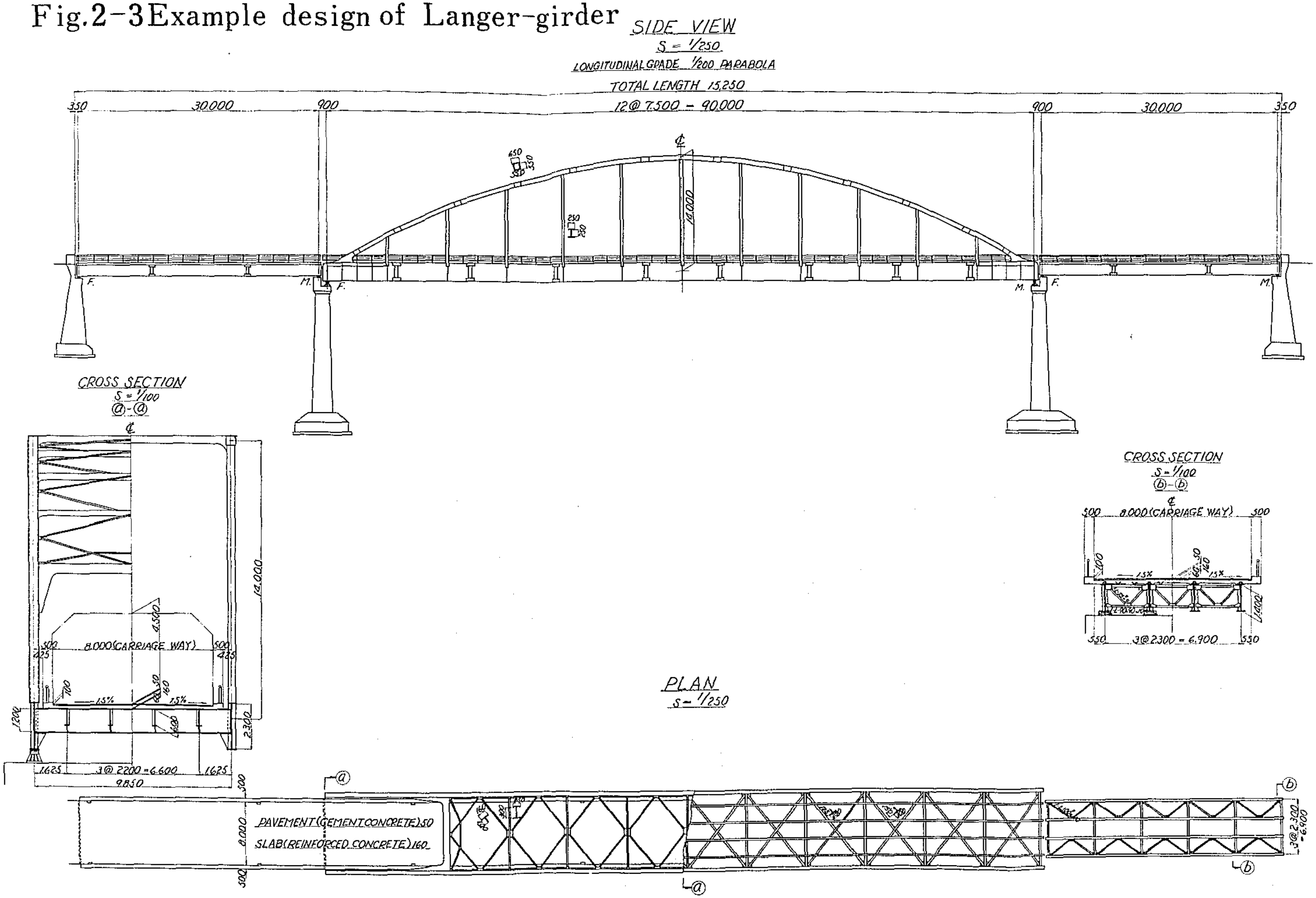
a. In the deck-type construction the distance between centers of main girders or trusses can be made smaller than will be required in the through-type construction, so that the length of floor beams will become smaller and less amount of steel will be required for lateral bracings and sway bracings. Moreover, under certain circumstances we can omit the floor beams entirely by spacing closely more than two main girders under the floor. For the above mentioned reasons the deck-type structure requires far less steel amount of and therefore much more ceonomical than the through-type structure.

b. The so-called composite structure, in which the reinforced concrete floor slab and the underlying steel girders are jointed by shear connectors so that these two parts can jointly resist the bending moment as one body, can be realized only in the deck-type structure. The saving of steel weight by the composite-girder design amounts in general to 20 to 30 percent the steel weight required in non-composite design.

The composite structure can be applied with advantage to deck truss bridges. Several composite truss bridges up to a span length of 50 meters have already been constructed in Japan. The attached drawing shows one of these composite truss bridges, on which various loading tests have been carried out and the efficiency and safety of the composite construction proved. In the composite truss bridges steel members of large cross-section are not required for tuss members, especially for upper chord members, so that steel members of relatively small size, now produced in Colombia, can advantageously be uitlized.

c. Whereas it is almost impossible or extremely difficult to broaden the roadway of through bridges after their completion, the roadway of deck bridges can easily broadened by placing new girders or trusses parallel to the existing ones or by installing cross brackets. Therefore, under certain circumstances, the so-called step-wise investment is possible in the deck-type construction, i.e., we build at first a bridge of small roadway width and afterwards broaden the roadway in accordance with the increase of traffic. One way insist that this can be done also in the through-type construction by placing at first the main trusses wide enough for the future widening of the roadway. In this case, however, the main trusses must be strong enough for the traffic load assumed to increase in the far future and we must use while unnecessary in the beginning. much steel for floor beams and bracings so that the intended step-wise investment can scarcely be realized. The example is shown in

Fig.2-3 Example design of Langer-girder



Main Span	254^t	(SM50 127^t)
Side Span	74^t	(〃 44^t)
Shoe	6^t	
	334^t	(〃 171^t)
Handrail	9^t	
Expansion Joint etc	2^t	
Total Wt	345^t	

Fig. 2-4 Example Design of Deck-Truss

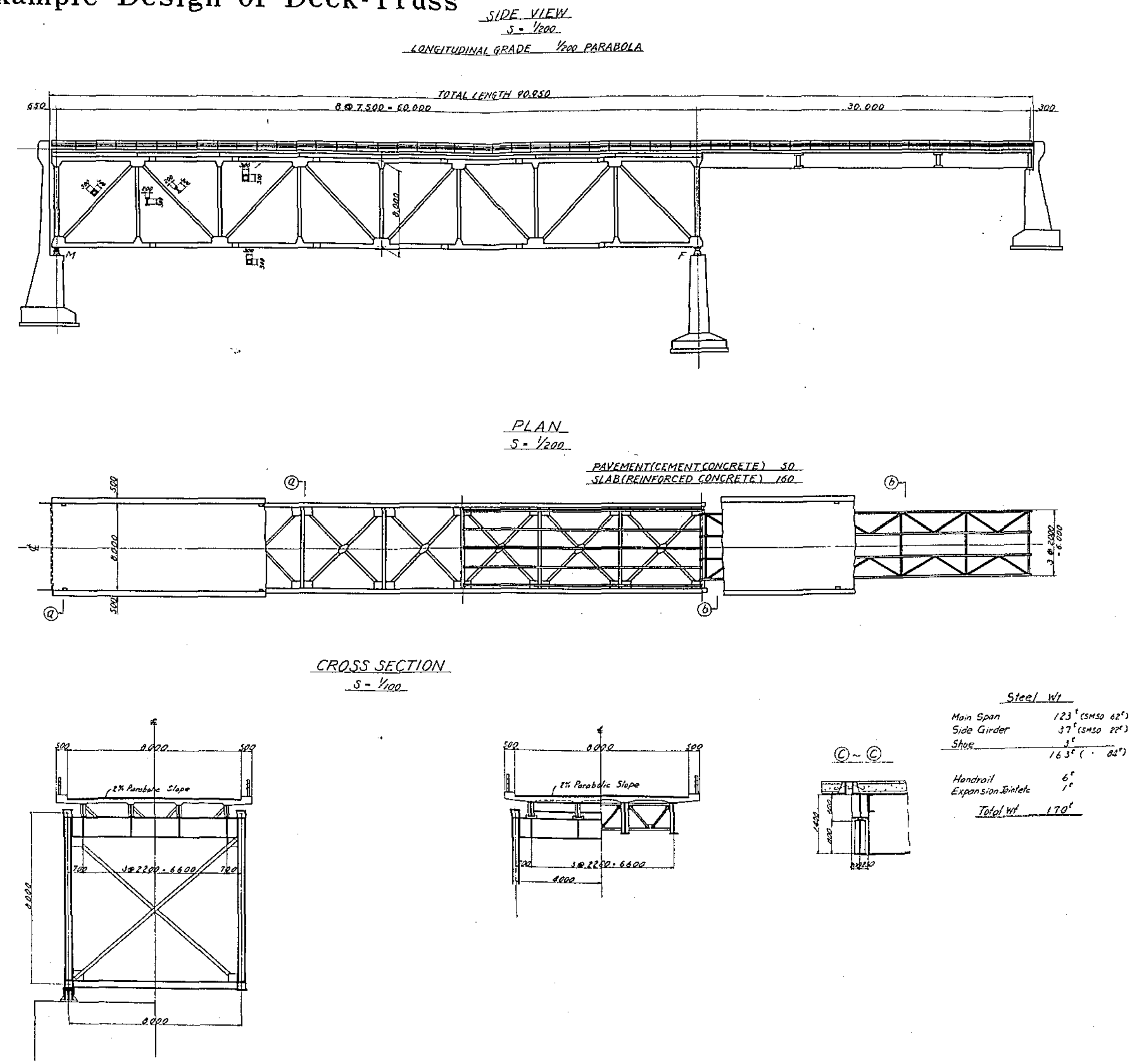


Fig.2-5 Example design of Worren-truss

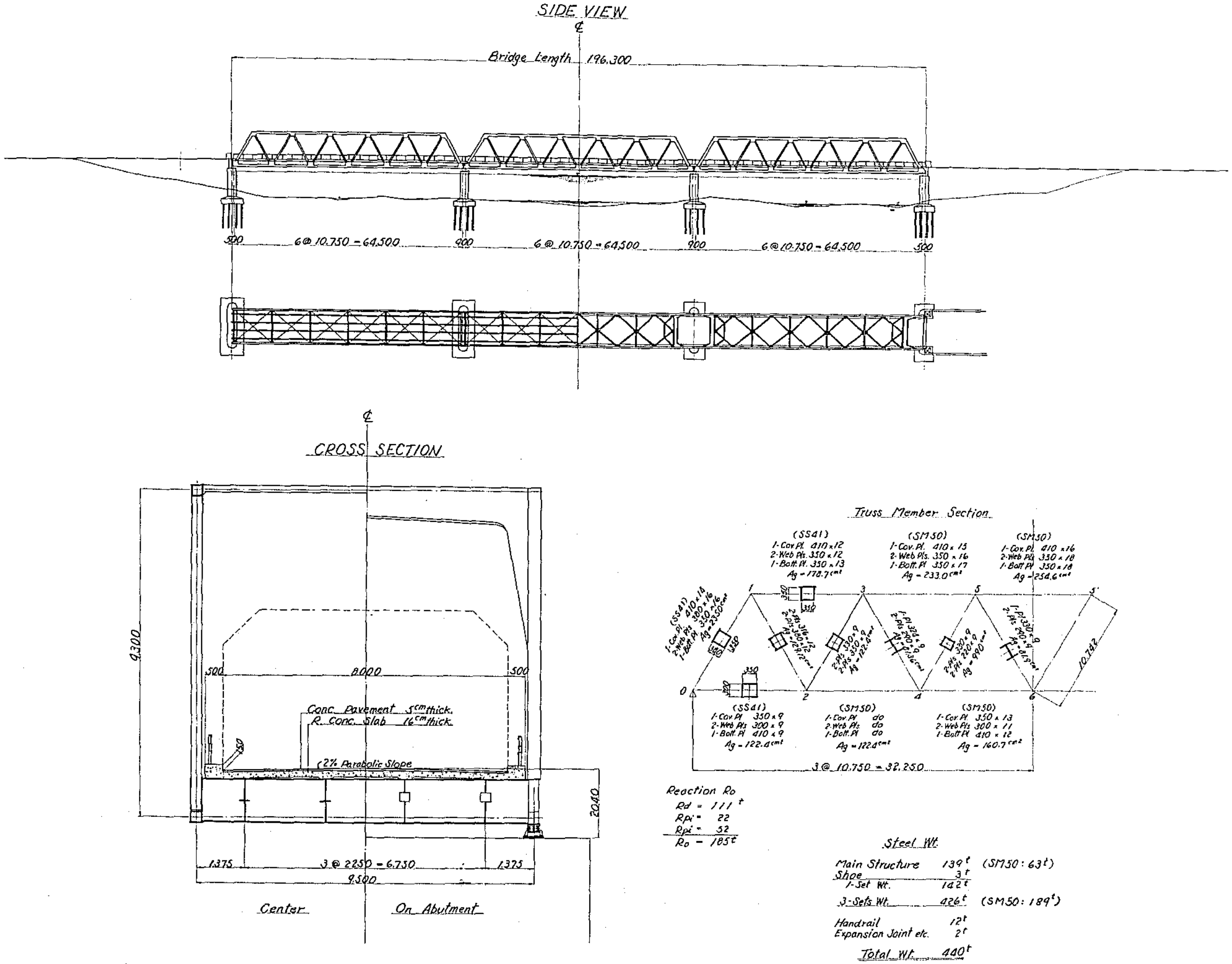


fig. 2-6 Composite-Truss (General View)

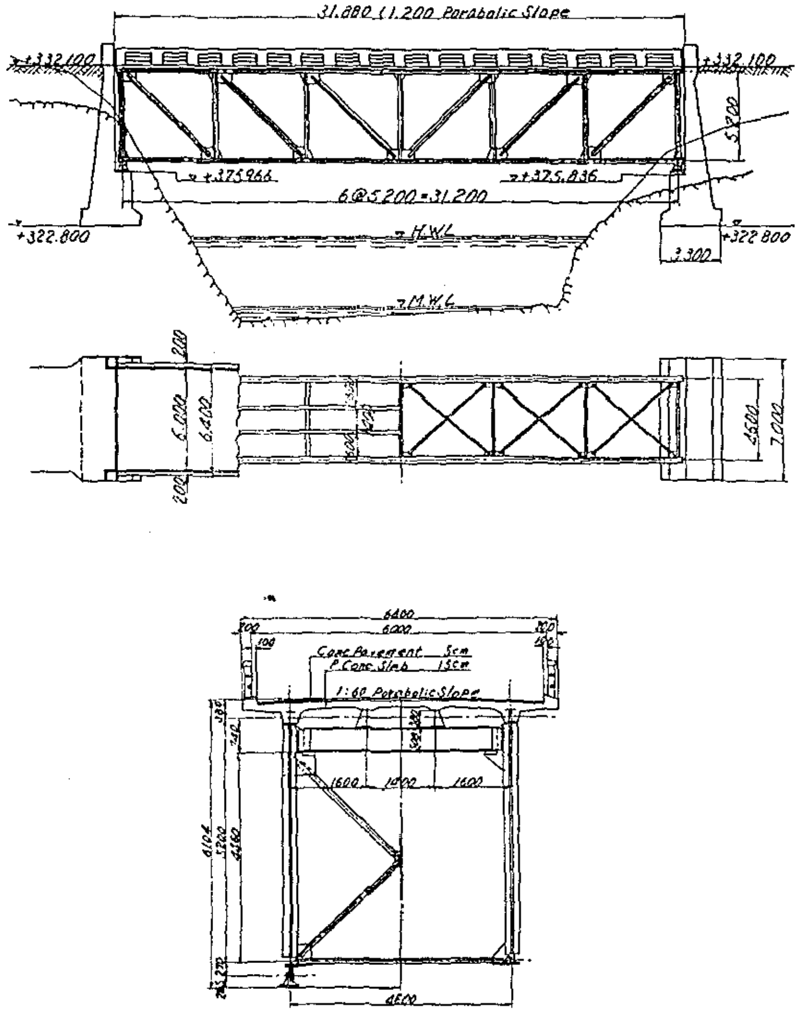


Fig. 2-7 Composite-Truss (Details)

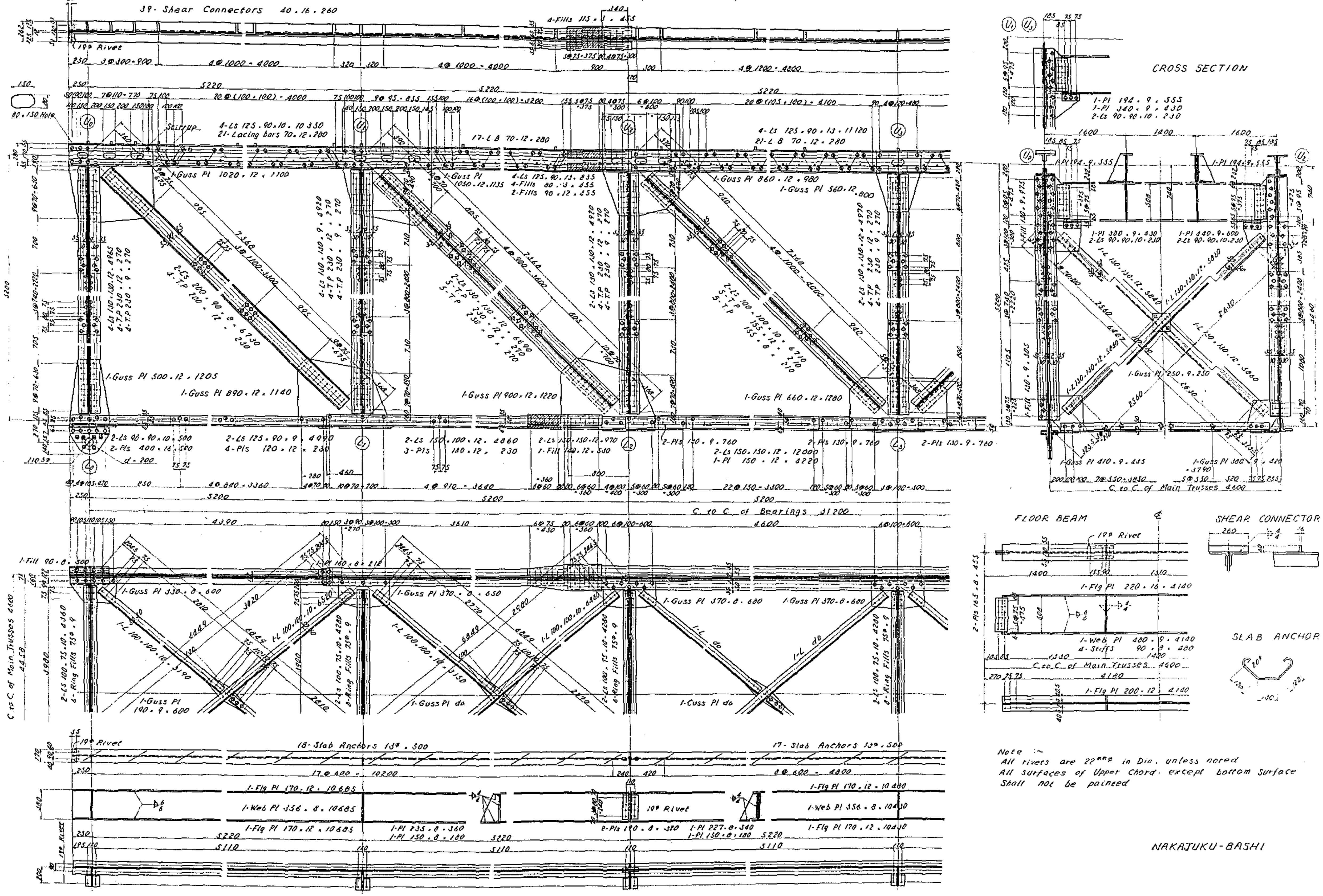


Fig. 2-8 Widening of the Roadway (General View)

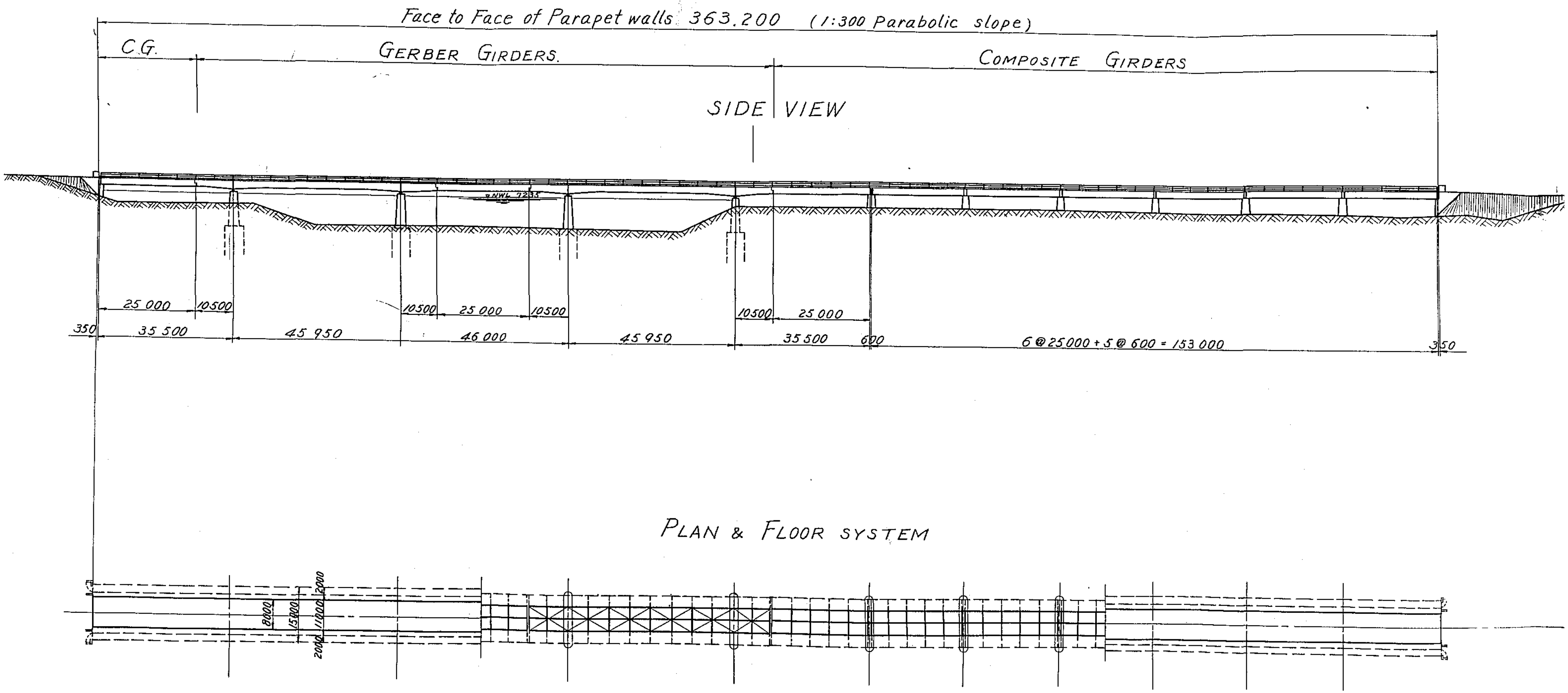


Fig. 2-8 Widening of the Roadway (General View)

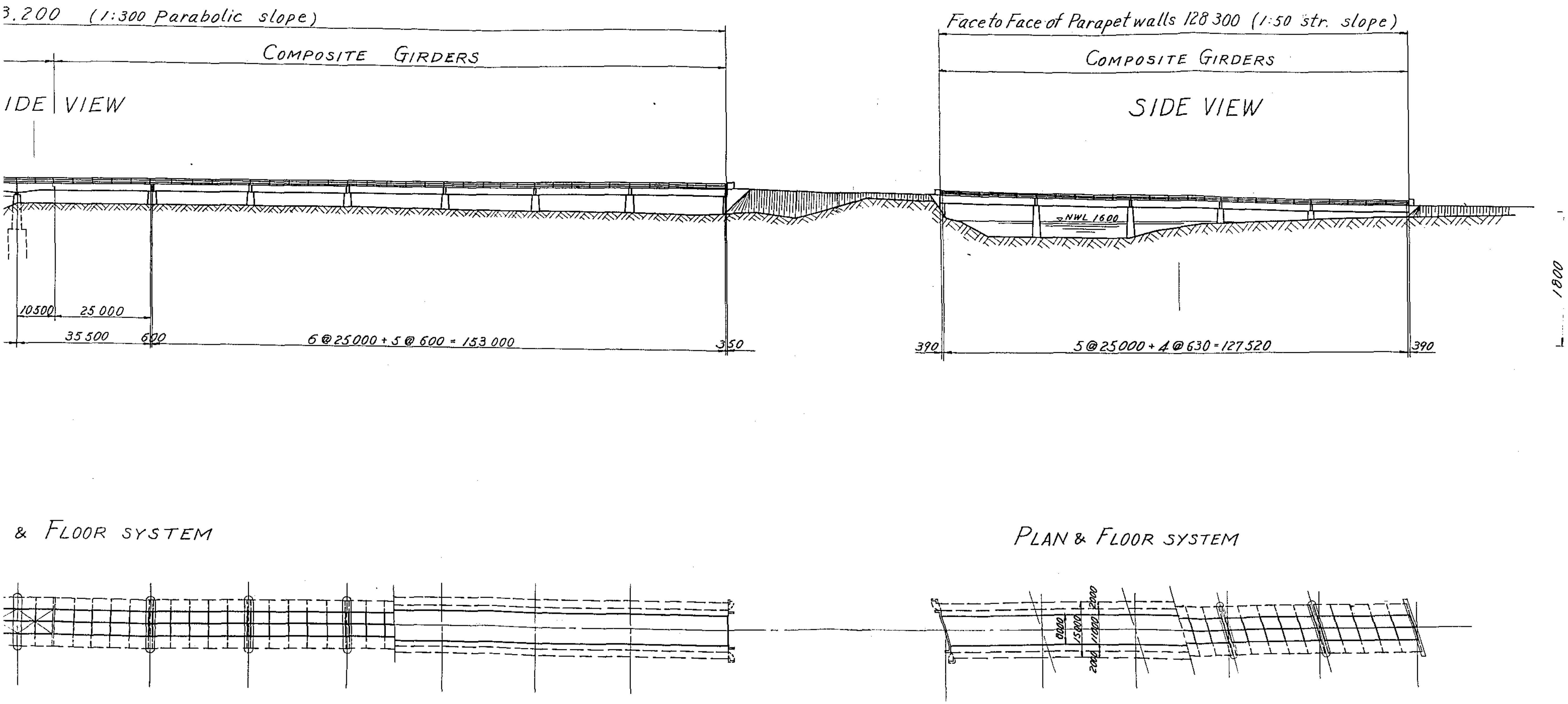


Fig. 2-9 Widening of the Roadway (Cross Section)

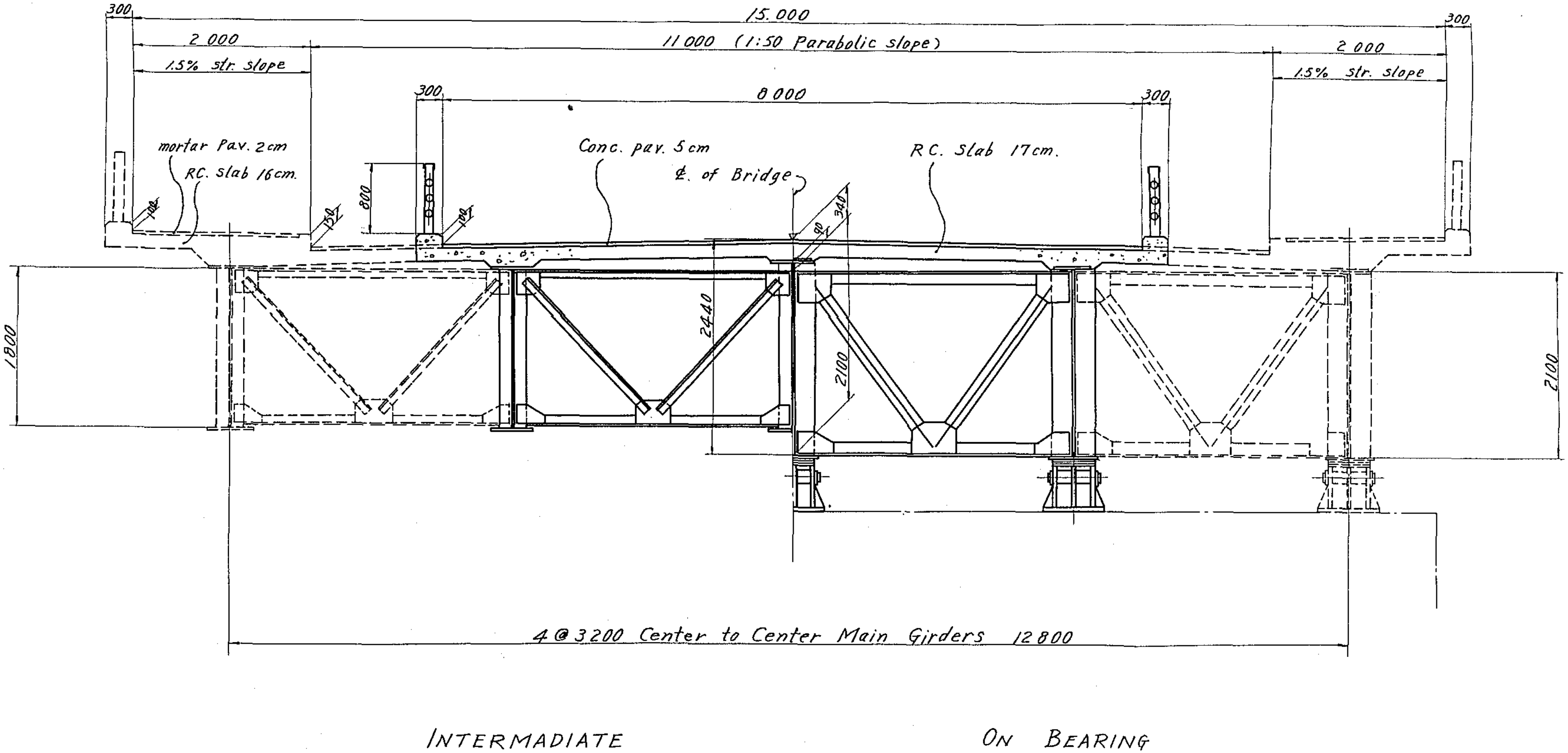


Fig. 2-8 and2-9.

d. The center of gravity of the whole structures and the bearings of deck bridges lie, as a matter of course, lower than those of through bridges. Therefore, the substructures of deck bridges are more stable than those of through bridges for horizontal loads such as wind forces and seismic forces. The substructures of deck bridges are in general easier to build and require less material. Regarding the substructures, we can say from the above reasons that the deck-type bridges are more economical.

Chapter 3. Use of Home-made Products

3-1. Steel Bridge

a General Remarks.

It is most necessary as well as desirable that a home project, even bridge construction, shall provide motives to foster domestic industrial capacities and thus gradually develop the domestic potentialities of supply.

In the light of applicability to bridge construction of the Colombia-made steel products as specified in Table 3-1, the greatest difficulty is that the steel plate is only 4.55 mm in maximum thickness and too small in size to use for the Gusset plate. The channel-section members have a slender web and these and the I-beams, excepting the one with 8" × 4" × 0.27", may not be used as bridge material. Therefore, only usuable material consists of L-section and flat steel.

As far as the chemical contents of steel is concerned, the presence of relatively large proportion of phosphor casts doubt on its adaptibility to welding, and hence the steel members should be connected by riveting.

If it is intended to employ only this limited assortment of usable products, only a truss type of medium and short spans is possible, in which a Gusset plate is not needed. Such an example is shown below.

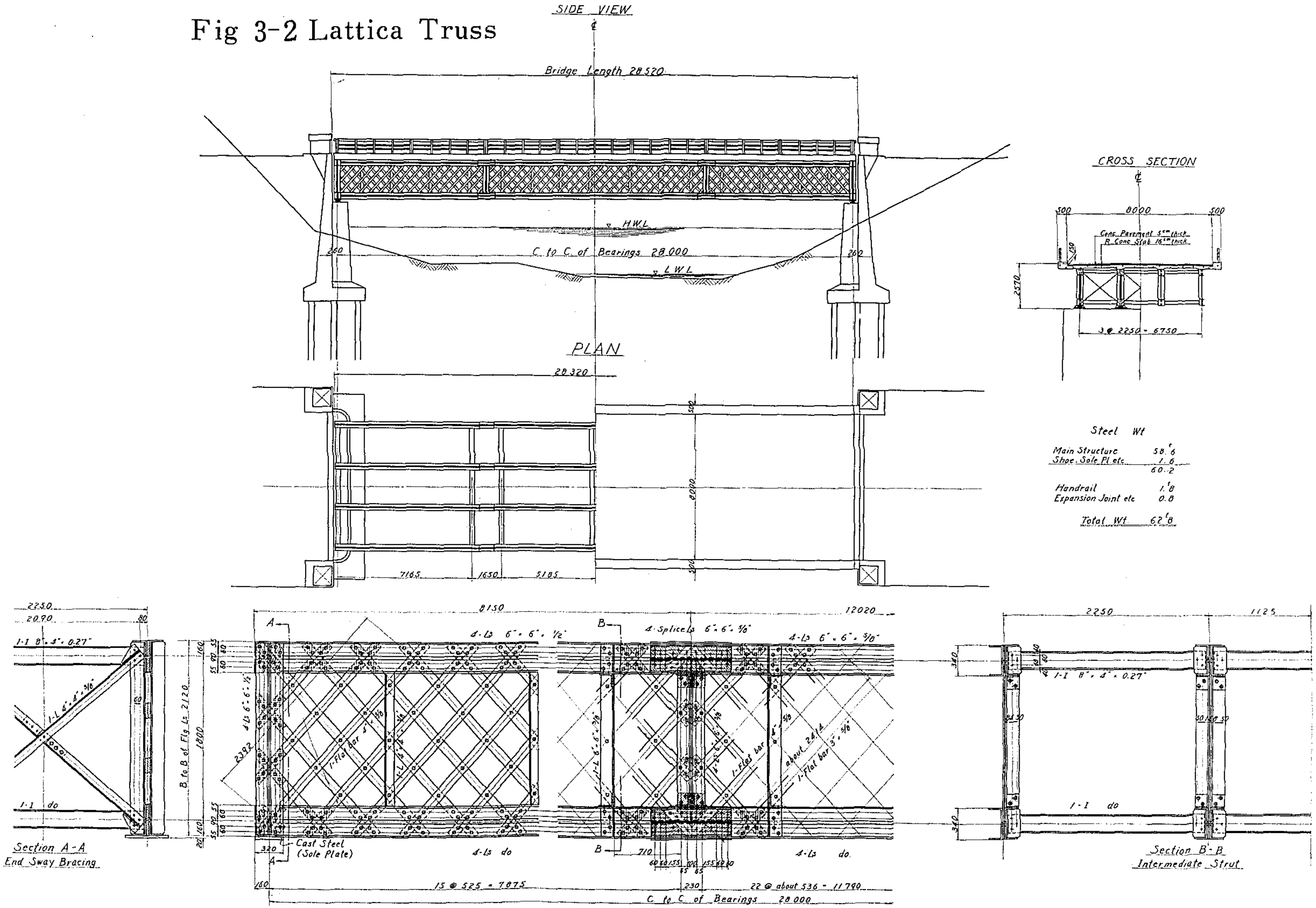
If large-sized plate of about 9-12 mm is available, various types of bridge may be built.

Table 3-I. Acerias (Paz del Rio)

PALANQUILLAS Y BARRAS CUADRADAS	100mm, 80mm, 70mm, 60mm, 50mm, 1¼", 1⅛", 1", ⅞", ¾", ⅝", ½", ⅜"
REDONDOS	8", 6", 5", 4", 3", 2½", 2", 1½", 1¼", 1⅛", 1", ⅞", ¾", ⅝", ½", ⅜", ¼"
ANGULOS	6", 5", 4", 3", 2½", 2", 1½", 1"
PERFILES EN "T"	4", 3", 2½", 2", 1½", 1"
PERFILES EN "U"	2.26", 1.92", 1.58", 1.41", 1", 1½"
VIGAS EN "I"	4", 3.33", 2.66", 2.33"
RIELES Y ECLISAS	75 lb, 60 lb, 40 lb, 75 lb, 60 lb, 40 lb
FLEJES O PLATINAS	5", 4½", 4", 3½", 3", 2½", 2", 1¾", 1½", 1¼", 1", ¾", ⅝"
ALAMBRES	4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14 Y 12½"

HOSIO NO. 20
PROMOCION DE VENTAS
BOGOTA APRIL 1958

Fig 3-2 Lattica Truss



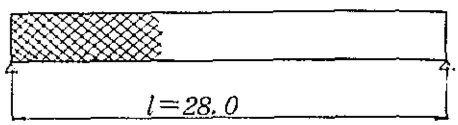
b. Example of design.

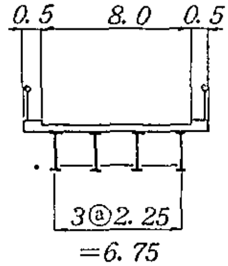
An example of design is shown in Fig. 3-1, and the computation processes stated below.

i General Data

Span Length	28.0 m
Effective Width	8.0 m
Type	Deck Lattice Truss

Fig. 3-1-a

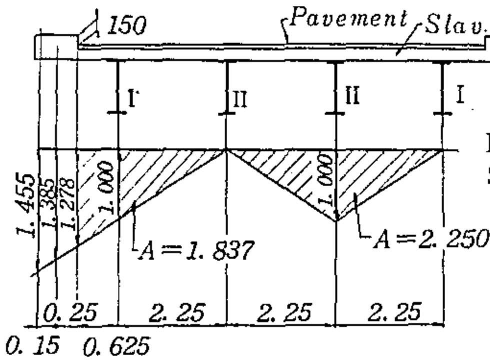




ii Bending Moment &. Shear

ii-1 Dead Load

Fig. 3-1-b



Pavement: Concrete, 5 cm thick
Slab: R. Concrete, 16 cm thick

Girder I

Pavement	2.35 × 0.05 × 1.837	= 0.216 t/m
Slab	2.50 × 0.16 × 1.837	= 0.735 〃
Coping	2.50 × 0.5 × 0.36 × 1.389	= 0.625 〃
Handrail	0.03 × 1.45	= 0.044 〃
Steel Wt.	0.25 t/m^2 × 1.837	= 0.460 〃
Others		20 〃
		2.100 〃

Girder II

Pavement	2.35 × 0.05 × 2.25	= 0.265 t/m
Slab	2.50 × 0.16 × 2.25	= 0.900 〃
Steet Wt.	0.25 × 2.25	= 0.563 〃
Others		= 22 〃
		1.750 〃

ii−2 Live Load (With Impact) $i = 0.257$

Girder I

$$P = 5.0 \times 1.837 \times 1.257 = 11.546\ t$$
$$p = 0.35 \times 1.837 \times 1.257 = 0.808\ t/m$$

Girder II

$$P = 5.0 \times 2.25 \times 1.257 = 14.141\ t$$
$$p = 0.35 \times 2.25 \times 1.257 = 0.991\ t/m$$

ii−3 Max. Bending Moment

Girder I

$$M_d = 0.125 \times 28.0^2 \times 2.100 = 205.8\ t/m$$
$$M_{eP} = 0.25 \times 28.0 \times 11.546 = 80.9\ 〃$$
$$M_{lp} = 0.125 \times 28.0^2 \times 0.808 = 79.3\ 〃$$
$$366.0\ 〃$$

Girder II

$$M_d = 0.125 \times 28.0^2 \times 1.750 = 171.6\ t/m$$
$$M_{lP} = 0.25 \times 28.0 \times 14.141 = 99.0\ 〃$$
$$M_{lp} = 0.125 \times 28.0^2 \times 0.991 = 97.2\ 〃$$
$$367.8\ 〃$$

ii−4 Max. Shearing Force

Girder I

$$S_d = 0.5 \times 28.0 \times 2.1 = 29.4$$
$$S_l = 11.546 + 0.5 \times 28.0 \times 0.808 = 22.8$$
$$52.2\ t$$

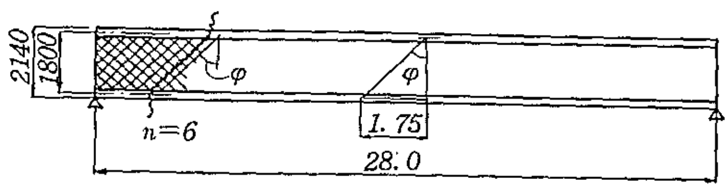
Girder II

$$S_d = 0.5 \times 28.0 \times 1.75 = 24.5$$
$$S_l = 14.141 + 0.5 \times 28.0 \times 0.991 = 28.0$$
$$52.5\ t$$

iii Stress &. Section

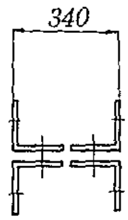
$$\sec\varphi = \frac{\sqrt{1.8^2 + 1.75^2}}{1.8} = \frac{2.51}{1.8} = 1.39$$

Fig. 3-1-c



iii −1

Fig. 3-1-d



Stress

Chord Member $\quad P = \frac{M}{1.8} = \frac{367.8}{1.8} = 204.2\ t$

Web Member $\quad P = \frac{S}{n} \sec\varphi$

$$= \frac{52.5}{6} \times 1.39 = 12.15\,t$$

iii −2

Section

Chord Member $\quad (1^{\square''} \fallingdotseq 6.45\ \text{cm}^2)$

① $4L_s \quad 6'' \times 6'' \times 1/2'' \quad A_g = 4 \times 5.75^{\square''}$

$$= 148.5\ \text{cm}^2$$

$A_n = 148.5 - 8 \times 2.5 \times 1.27 = 123.1\ \text{cm}^2$

② $4L_s \quad 6'' \times 6'' \times 5/8'' \quad A_g = 4 \times 7.11^{\square''}$

$$= 183.5\ \text{cm}^2$$

$A_n = 183.5 - 8 \times 2.5 \times 1.59 = 151.7\ \text{cm}^2$

$max\sigma_c = \frac{204.2}{183.5} = 1112\ \text{kg/cm}^2 < 1300\ \text{kg/cm}^2$

$max\sigma_t = \frac{204.2}{151.7} = 1348\ \text{kg/cm}^2 < 1400 \quad$ 〃

Web Member

Using $\qquad$ Flat Bar

Thickness $= t \quad r_x = 0.2887\,t, \quad l = 41\ \text{cm}, \quad \sigma_{ca} = 1300 - 0.06(l/r_x)^2$

	Section		A_g (cm²)	A_n (cm²)	r_x (cm)	l/r_x	σ_{ca} (kg/cm²)
1-$F.B.$	4″ (10.16cm)	5/8″ (1.59cm)	16.13	12.15	0.459	89.3	822
1-$F.B.$	3″ (7.62cm)	5/8″ (1.59cm)	12.10	8.60	0.459	89.3	822

7/8″ϕ(22ϕ) Rivet Hole $\quad 2.5 \times 1.59 = 3.98\ \text{cm}^2$

3/4″ϕ(19ϕ) $\quad$ 〃 $\quad 2.2 \times 1.59 = 3.5\ \text{cm}^2$

Chord Member Full Strength

① $4L_s \quad 6'' \times 6'' \times 1/2''$

$F_c = 1300 \times 148.5 = 193\,t \qquad {}_rM_c = 347\ tm$

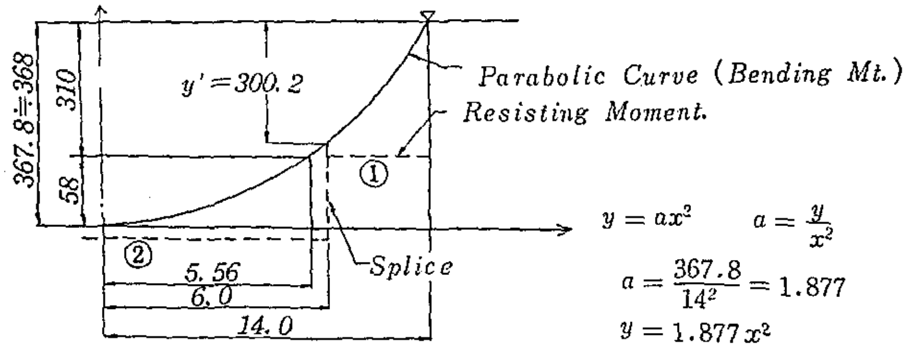
$F_t = 1400 \times 123.1 = 172\,t \qquad {}_rM_t = 310\ tm$

② $4L_s \quad 6'' \times 6'' \times 5/8''$

$F_c = 1300 \times 183.5 = 239\,t \qquad {}_rM_c = 430\ tm$

$F_t = 1400 \times 151.7 = 212\,t \qquad {}_rM_t = 382\ tm$

Fig. 3-1-e



$$y = ax^2 \qquad a = \frac{y}{x^2}$$

$$a = \frac{367.8}{14^2} = 1.877$$

$$y = 1.877\,x^2$$

$$x = \sqrt{\frac{y}{1.877}}$$

$$= \sqrt{\frac{58}{1.877}} = 5.56\,m$$

$x = 6.0\,m$

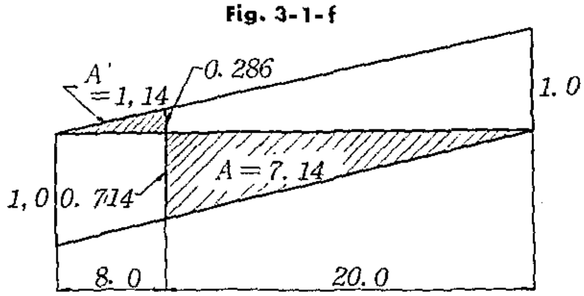
$$y = 1.877 \times 6.0^2 = 67.6$$

$$y' = 367.8 - 67.6$$

$$= 300.2$$

iii Shearing Force at Splicept.

Fig. 3-1-f



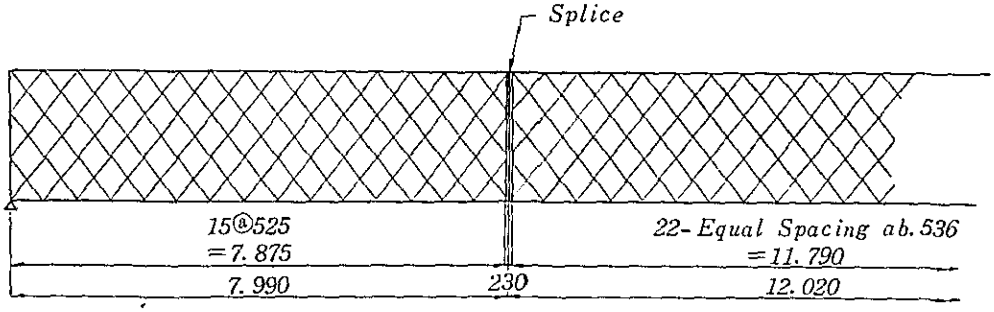
$$\frac{8}{28} = 0.286$$

$$S = 1.75 \times (7.14 - 1.14) + 14.141 \times 0.714 + 0.991 \times 7.14$$

$$= 10.5 + 10.1 + 7.1 = 27.7\,t$$

$$P = \frac{27.7}{6} \times 1.39 = 6.42\,t$$

Fig. 3-1-g



Splice ($M = 300.2\,tm$, $S = 27.7\,t$)

Chord Member

4-L_s 6" × 6" × 1/2" $\quad A_g = 148.5\ \text{cm}^2$

$A_n = 123.1\ \text{cm}^2$

$${}_sM_c = \frac{347+300}{2} = 323\,tm \quad {}_sP_c = \frac{323}{1.8} = 179.4\,t$$

$${}_sM_t = \frac{310+300}{2} = 305\,tm \quad {}_sP_t = \frac{305}{1.8} = 169.4\,t$$

Req'd Number of Rivet

$$n_c = \frac{179.4}{4 \times 3.763} = 11.9 \cdots\cdots\cdots\cdots\cdots 12$$

$$n_t = \frac{169.4}{4 \times 3.763} = 11.3 \cdots\cdots\cdots\cdots\cdots 12$$

① Web. Member 1-FB. 4" × 5/8" $\quad A_g = 16.13\ \text{cm}^2 \quad A_n = 12.15\ \text{cm}^2$

Full Strength $F_c = 822 \times 16.13 = 13.28\,t$

$F_t = 1400 \times 12.15 = 17.02\,t$

② Web Member 1-FB. 3" × 5/8" $\quad A_g = 12.10\ \text{cm}^2 \quad A_n = 8.6\ \text{cm}^2$

Full Strength $F_c = 822 \times 12.1 = 9.94\,t$

$F_t = 1400 \times 8.6 = 12.03\,t$

Spliceing & Connecting Rivet

$$n = \frac{12.15 + 13.28}{2} \times \frac{1}{3.763} = 3.38 \cdots\cdots\cdots\cdots\cdots 4 \quad \text{req' number}$$

3-2. Augumentation of Traffic Facility by a Provisional Bridge.

A. *General description.*

While it is most desirable that a traffic network is steadily developed in good proportion to the demand with appropriate high-ways and bridges which conform with refined standard specifications, a tremendous investment is needed to meet a rapidly rising demand or to make appreciable contributions to industrial promotion in the underdeveloped areas.

Therefore, a very difficult problem arises.

In such places where the volume of traffic is high and a key network of trunk high-way is present, an appropriate measure is to provide bridges of permanent structure not only through public investment but also, as already referred to, through employment of toll-bridge system. On the other hand, in the underdeveloped areas or for high-ways of secondary importance, it is highly advisable to extend the range of vehicular transportation by building even bridges and high-ways of provisional structure in parallel to the orthodox long-term policy.

The description to follow deals with such techniques as would be conceivable in the light of the above proposition.

B. *Wooden earth bridge.*

It is an wooden bridge that can be erected with ease by using handily available material. Of various techniques of wooden bridge, some make use of thick planks for the floor system, but it is the technique using soil for the floor system that is distinguished in the properties of material and mechanical principles. According to this technique, even deformed material is still usable and the mechanical composition ensures an excellent distribution of live load. This technique shall be reviewed below.

a. Floor system. The objective of a floor system is to provide a smooth surface for cruising vehicles, and at the same time to transmit such live load as uniformly as possible through floor beams (arrange-log) to main beams. Accordingly, the surface must have a suitable degree of rigidity to ensure a smooth vehicular traffic, but the members to be used in the lower portion of the system must have increasing flexibility so that they may be fitted in against some unevenness of floor beams or deformity of material.

The basic technique consists in placing several layers of impermeable material, such as banana leaves, on the floor beam, then spreading highly cohesive clay uniformly on them, paving lightly compacted sand-pebble mixture and finally spreading gravels on the finishing surface.

It is also effective to use a mixture with calcareous powder or cement at places where such seems feasible.

The transversal gradient of the surface may well be 3~5% for drainage.

b. Main bracings. The member for the main girder is a scaled timber which is whittled flatly only on the upper face with both ends scraped on the downward face to fit in tthe supporting devices. For the floor beam is used a timber of about 9-12 cm in diameter which is whittled on the upper and downward faces in such a way that no appreciable unevenness may remain on the finished faces to impair their parallelism. Such processing work may be performed either manually or with aid or portable timber machines.

c. Substructure. Figs. 3-3 (B) and (C) show a design example of the structure of an abutment and pier.

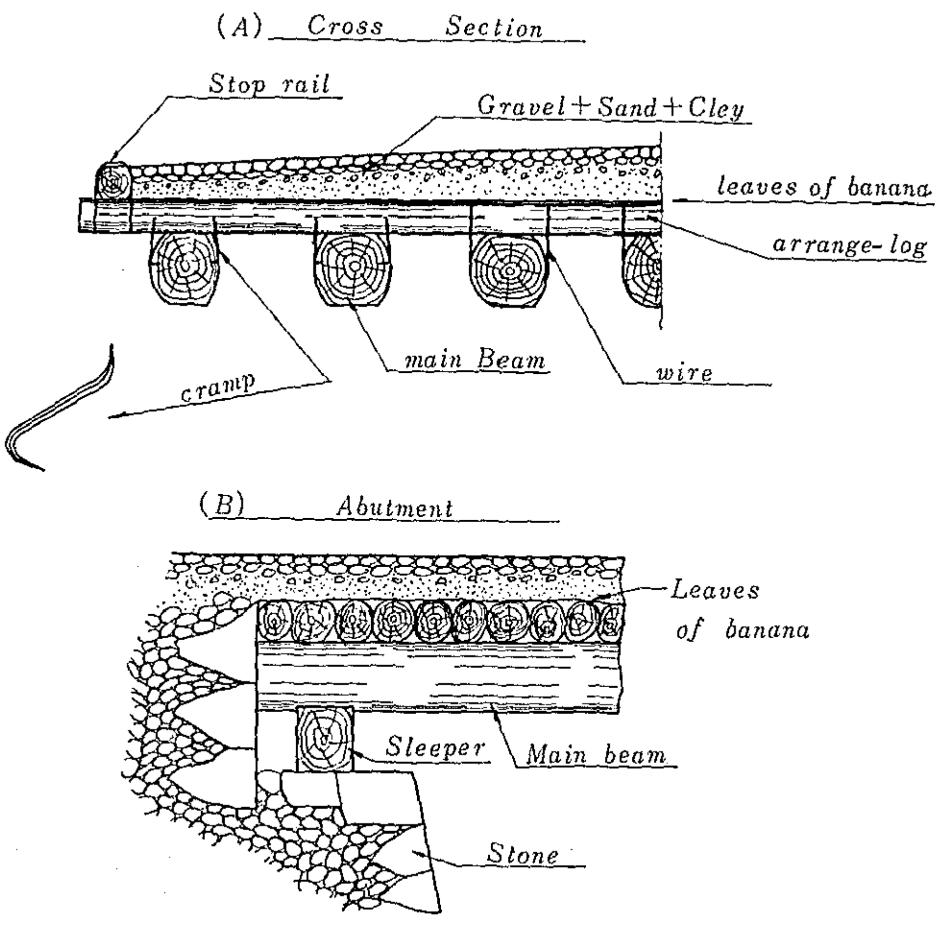
d. Antiseptic, etc.

The timber must be a well-dried material. The most advisable process is to inject the creosote solution (in exchange with sap), but since this method requires some bulky equipment, coating of creosote solution will do also.

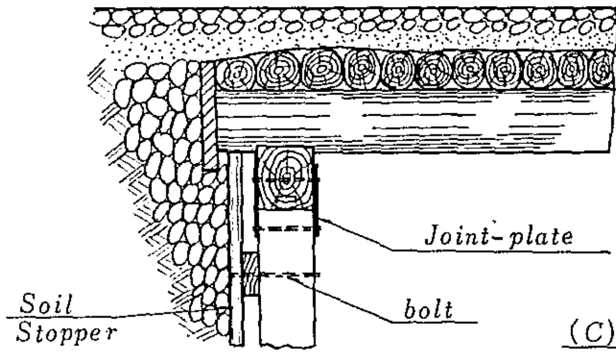
Bolt connection must be tightened again in about a half or one year after inauguration of the bridge, and it must not be overlooked to use hard wood for the packing piece of a bolt.

Attention must be paid to ensure efficient drainage. It is necessary to apply regularly the antiseptic coating because the timber is liable to corruption at the cut end.

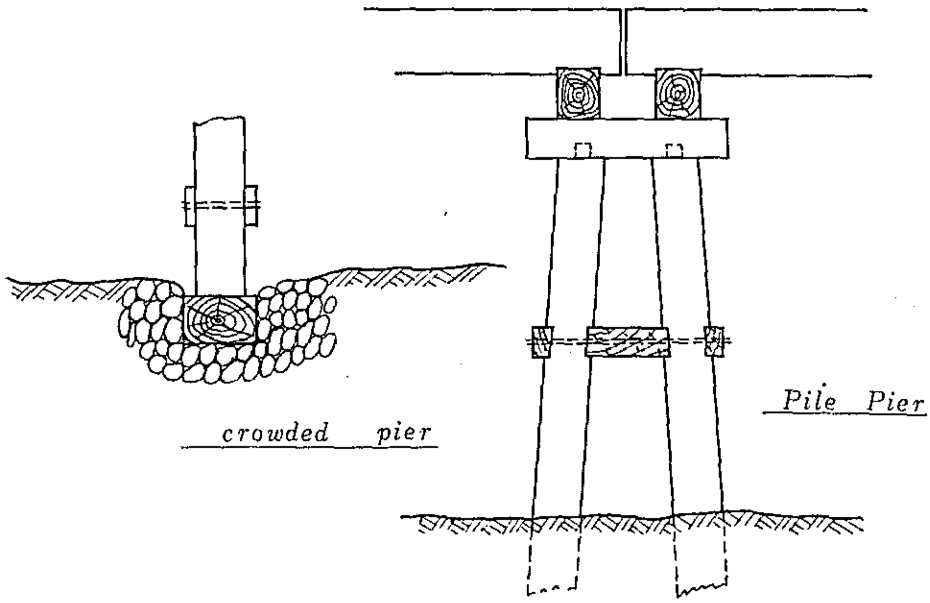
Fig. 3-3 Wooden Earth bridge



(B) Abutment



(C) Pier



C. Wood-concrete composite girder.

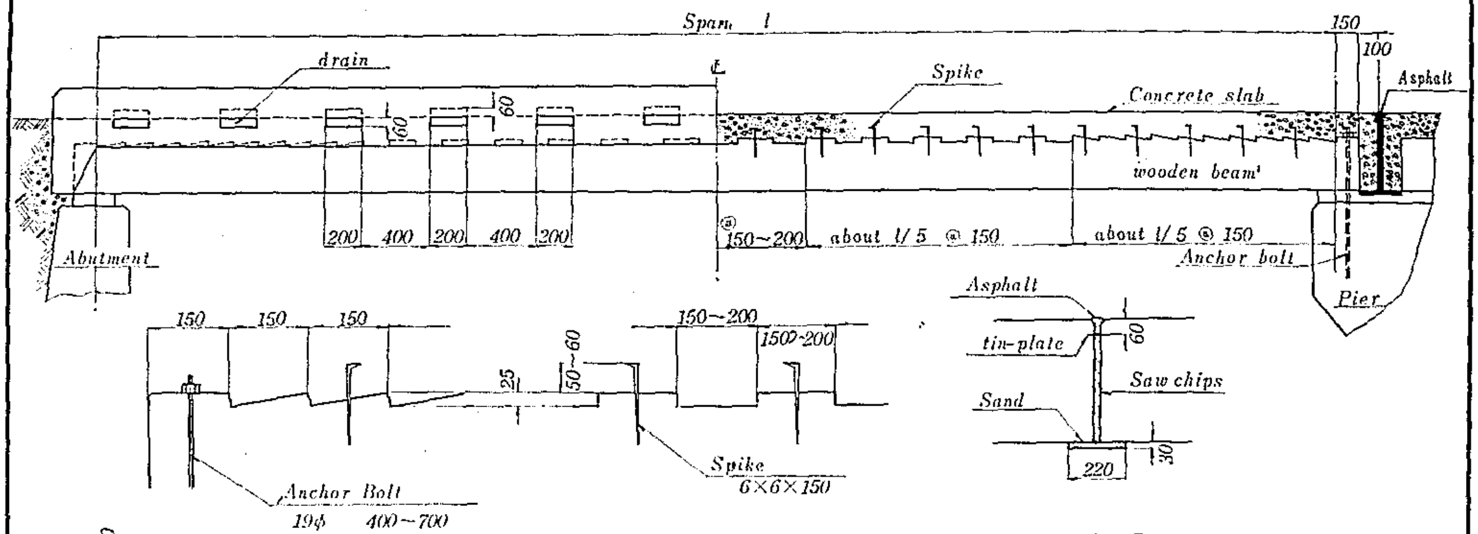
Of the half-permanent structures which are stronger than an wooden earth bridge and durable for about 15 to 25 years, there is a wood-concrete composite bridge. In Japan, this structure has been in use since 30 years ago, and is very handily built at places where timbers are easily available. The dead load of the slab concrete acts upon the wooden girder, but the live load due to automobile is reacted by the composite girder. Named "live-load composite girder," this type of bridge can afford to have a smaller cross-section for the main girder than other types of woden bridges. Also, being highly water-tight, this is less liable to corruption.

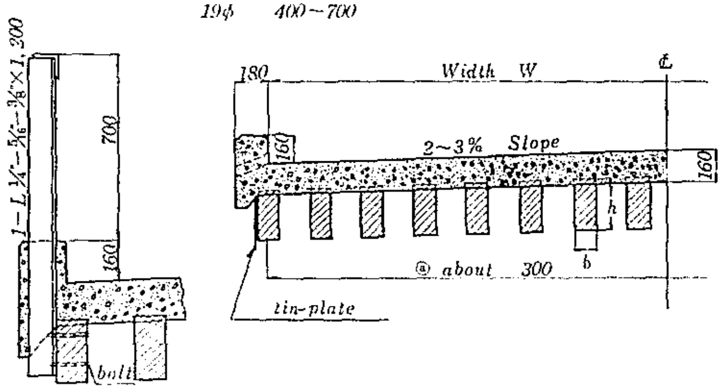
A design example is shown in Fig. 3-4.

The sliding stress which occurs between the upper face of the main girder and the concrete slab is resisted by dented teeth, whil the scaling action by spikes. For hand rails, L-type member manufactured by Paz del Rio has been adopted.

Fig. 3 - 4 Composite-beam (Wooden beam-Concrete slab)

(dimension ; ; mm)





Main Wooden Beam

Spovn l (m)	Width b (cm)	Hight h (cm)	Length L (cm)	Remarks
3.0	10.5	15	330	
4.0	10.5	15	430	
5.0	10.5	18	530	
6.0	12	21	530	
6.5	12	24	680	
7.0	12	27	730	
7.5	12	30	790	
8.0	13.5	30	840	

The process of computation for cross-section is outlined below.

a. Bending stress

According the distance between main beams is usually narrow, we may adopt this distance as the effective width of concrete slab.

If we assumed that the section-form does not transform and the strain is directly proportional to the stress, we find that the bending stress is directly proportional to the distance from the neutral axis as follows:

$$\frac{\sigma_w}{\sigma_c} = n\frac{h-x}{x}, \quad \sigma_w = n\,\sigma_c\frac{h-x}{x}$$

where

σ_c = Compressive fibre stress

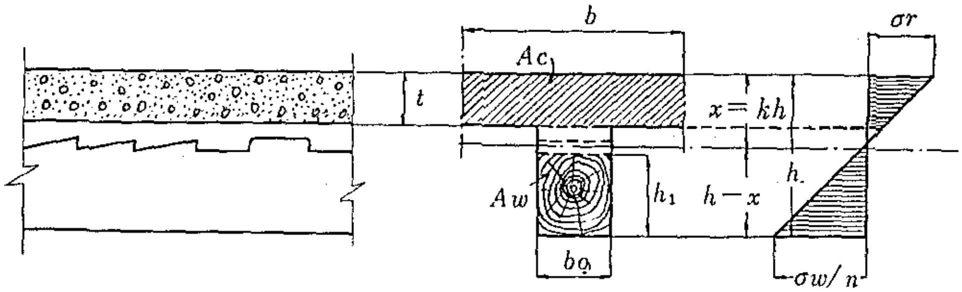
σ_t = Tensile fibre stress

E_w = Young's modulus of wood, 70,000~100,000 kg/cm^2

E_c = Young's modulus of concrete, 140,000~210,000 kg/cm^2

E_c/E_w = Poison's ratio, nearly 0.5

Fig. 3-5-a



Neglecting the concrete section of web, a foundamental equation is obtained as follows, from the principle of moment balance.

$$A_c = (x-t/2) - nA_w(h-x-h_o/2) = 0 \quad \cdots\cdots① $$

where

$A_c = bt, \quad A_w = b_o h_o$

Therefore, the equation ① may be rewritten

$$x = \frac{A_c\frac{t}{2} + nA_w(h-\frac{h_o}{2})}{A_c + nA_w} \quad \cdots\cdots② $$

Now, let

$$K = \frac{x}{h}, \quad i = \frac{t}{h}, \quad \alpha = \frac{h_o}{h}, \quad \beta = \frac{b_o}{b}, \quad \alpha\beta = \frac{A_w}{A_c}$$

The equation ② may be follows:

$$K = \frac{\frac{i^2}{2} + n\alpha\beta(1-\frac{\alpha}{2})}{i + n\alpha\beta} \quad \cdots\cdots③ $$

Consequently, the moment of inertia of the effective section area, about the neutral axis, is given as follows:

$$I = \frac{b}{3}\left[x^3-(x-t)^3\right] + \frac{nb_o}{3}\left[(h-x)^3-(h-x-h_o)^3\right]$$
$$= \frac{bh^3}{3}\left\{\left[K^3-(K-i)^3\right] + n\beta\left[(1-K)^3-(1-K-\alpha)^3\right]\right\} \cdots\cdots\cdots\cdots ④$$

So, the stress is represented as follows:

$$\sigma_c = \frac{M}{I}x = \frac{M}{L_c\, bh^2}, \quad \sigma_w = n\sigma_c\frac{h-x}{x} = n\sigma_c\frac{1-K}{K} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots ⑤$$

where

$$L_c = \frac{I}{xbh^2} = \frac{1}{3K}\left\{\left[K^3-(K-i)^3\right] + n\beta\left[(1-K)^3-(1-K-\alpha)^3\right]\right\}$$

b. Shearing stress

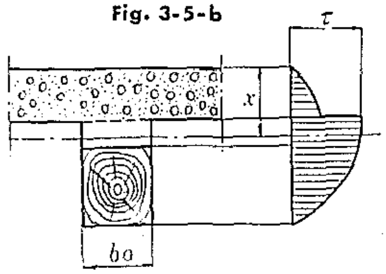
Generally, the shearing stress is given as follows:

$$\tau = \frac{S\,G}{I\,b}$$

where

S = Shearing force
G = Section modulus of effective area
I = Moment of inertia of effective area

Fig. 3-5-b



If we show the maximum τ as τ_o,

$$\tau_o = \frac{SG}{Ib_o}, \quad G = nb_o h_o\left(h-x-\frac{h_o}{2}\right)$$

Usually, we know that I/G_o is nearly 2/3, therefore τ_o is obtained as follows:

$$\tau_o = \frac{3}{2}\frac{S}{b_o h}$$

c. Connecting

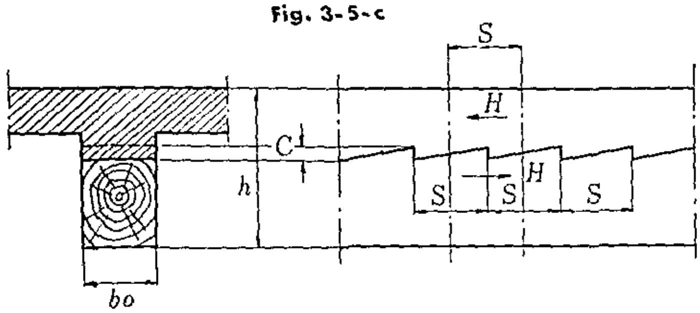
If we consider that the horizontal shearing force H acting on the distance s, in the connected face, is uniform,

$$H = \frac{SG_s}{I} \doteqdot \frac{3}{2}\frac{S_s}{h}$$

Assuming that the bearing stress acting on the face between wood and concrete is P, and the horizontal shearing force acting on the wooden tooth is τ, in Fig. 3−5−C.

$$P = \frac{H}{b_o C}, \quad \tau = \frac{H}{b_o s} = \frac{3}{2}\frac{S}{b_o h}$$

Fig. 3-5-c



In the design of connection, it is necessary that the bearing stress P is smaller than the allowable stress P_a and the horizontal shearing stress τ is smaller than the allowable stress τ_a.

Usually, τ_a of concrete is 5 kg/cm^2 and τ_a of wood is 8 kg/cm^2, therefore we must adopt 5 kg/cm^2 as the τ_a.

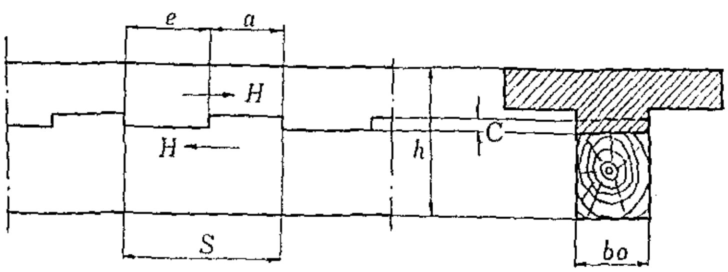
But, when the wooden teeth are buried under concrete, we may adopt 8 kg/cm^2 as the τ_a, according to the distribution of concrete shearing stress.

Decided the value of τ_a, we may rewritten the equation $\tau \leqq \tau_a$ as follows:

$$b_o \geqq \frac{3}{2}\frac{S}{\tau_a h}$$

Usually, Pa of concrete is 35~45 kg/cm^2 and Pa of wood is 80kg/cm^2, therefore we must adopt 35~45 kg/cm^2 as Pa.

Fig. 3-5-d



Similarly we get next relations, in Fig. 3−5−d

$$P = \frac{H}{b_o C}, \quad \tau_c = \frac{H}{b_o e}, \quad \tau_w = \frac{H}{b_o a}$$

where

τ_c = shearing stress of concrete

τ_w = shearing stress of wood

Therefore,

$$C \geqq \frac{H}{P_{ca}\, b_o}, \quad e \geqq \frac{H}{\tau_{ca}\, b_o}$$

where

P_{ca} = allowable bearing stress of concrete

τ_{ca} = allowable shearing stress of concrete

d. Holding power of nail

The holding power of nail depends on the friction between nail and wood, therefore the resistance for pulling is nearly proportional to the connected area.

$$N_P = CA$$

where

N_P = holding power

C = holding power for unit area

A = connected area

When we drived nails at right angle for wood fibre, the value of C are given as follows, by Dr. Mori.

Table 3-2 Value of C

Sorts of wood	Specific Gravity	Limit Value of C(kg/cm²)
Cedar	0.40	10.4
Fir	0.42	14.0
White pine	0.52	18.0
Oak	0.69	27.0
Beech	0.72	27.2
Zelkova	0.64	32.8

In deciding the allowable value of C, we must consider the safety factor 5~6 and 2~3 for the temporary project.

D. Other techniques.

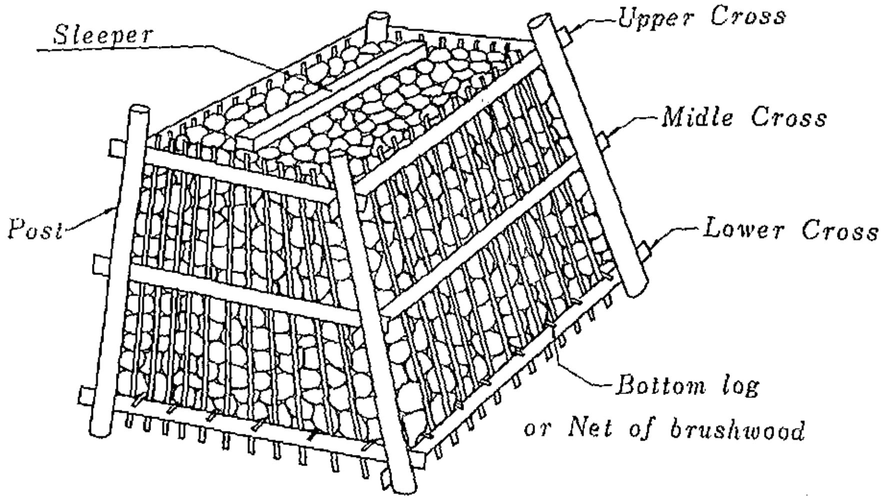
a. Frame filled with stone. As a technique to retain earth pressure or for groin, a frame filed with stone is used. This comprises a timber frame equipped with bottom net of twigs and filled with stone in it, as shown in Fig. 3-6. This is linked both longitudinally and laterally with each other until a required size is attained.

This technique presents a good stability aginst running water, and is hence a highly useful method adaptible for both rigid and flexible functions for abutments and piers of a provisional bridge in cases where the river bed of gravel layers or other hard formation defies excavation or penetration of a foundation pile. Durability can be improved by using antiseptic such as creosote solution plus absorbent.

b. Protection with brushwood. Use of brushwood is effective as a protective procedure for revetment and slopes at the front and wings of a bridge abutment. Such an example is shown in Fig. 3-7. This is built with timber piles, driven into the ground and firmly entwined by a net of woven brushwood, filled with stones which are so arranged as to form a smooth face on the finished surface.

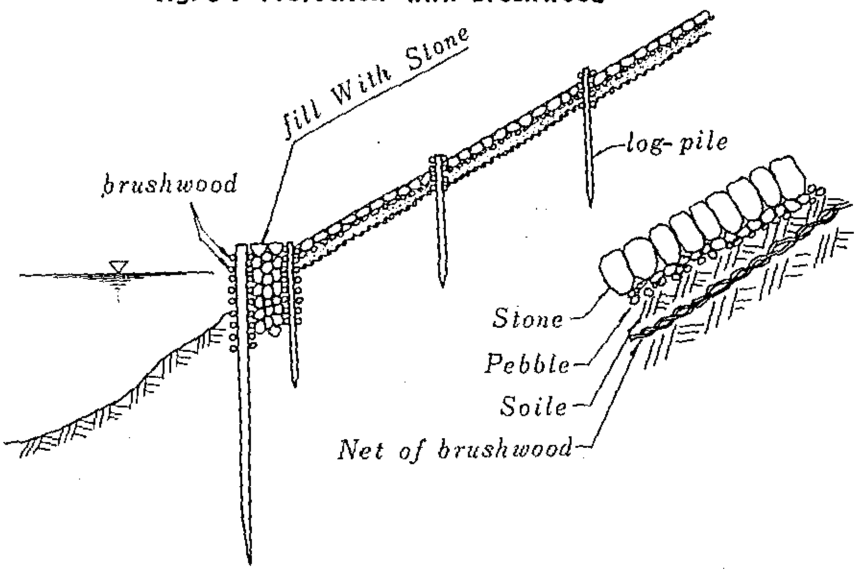
c. Use of bamboo. The above technique may be used by using bamboo grass instead of brushwood in places where bamboo is plenty. There is some other use of a long bamboo, according to which the bamboo joints are cleared to fill the mixture of sand and pebble and then consolidate it with cement intrusion. Thus a strong material is manufactured, which is protected on the exterior face by powerful and

Fig. 3-6 Frame filled with stome



burable fibre of bamboo and reinforced in the interior core by hardened concrete. Such a product may be used for the floor system of a wooden bridge or a stone frame.

Fig. 3-7 Protection with brushwood



Chapter 4. Welding and Hight Strength Steel

4-1. On the Use of Welding

Most bridges now existing in Colombia are almost riveted ones. We expect, however, that for the future welding will be used widely in composing girders or members and in their connection, especially in shop connection. In welded construction, we need not bother about the size of available angles or shapes and various other restrictions that usually trouble us in designing riveted structures. In welded structures, the section and connection of girders or members can be designed freely and rationally and, moreover, there is no need of the reduction of section by rivet holes. The saving of steel weight of welded bridges as compared to riveted bridges generally amounts at least to 20 percent and in some instances to about 30 percent.

To make use of welding, however, it is necessary and hoped that steel of excellent weldability, especially thick steel plates of 8 to 25 mm in thickness, shall be produced and the technique of welding itself shall be studied and promoted.

4-2. On the Use of High Strength Steels.

It goes without saying and is a common sence in bridge engineering that, in designing large bridges, we must endeavor to reduce as much as possible the dead weight of the bridge and to this effect we use welding and high strength steels in the right place. We hope and expect, therefore, that high strength steel of excellent weldability will be produced in Colombia.

It is, however, no truly economical design that we, merely to reduce the steel weight, design too complicated details or make needless use of high strength steels. For a member that will be acted upon by a great tensile force or when the reduction of dead weight will result in a exceedingly favorable effect, high strength steel deserves to be employed. The strength of compression members, however, does not depend upon the ultimate strength or the yield point of the steel used but is determined by the resistance to buckling. The buckling of long compression members depends upon the Young's modulus of the material and the slenderness ratio (ratio of the length of the member to the radius of gyration of the cross-section of the member). While the Young's modulus of steels remains almost invariable notwithstanding the variation of their strength, if we use high strength steel, the required cross-section will as a matter of course become smaller and the slenderness ratio of the member will increase with a result that the resistance to buckling will become lower. Therefore, it is not improbable that the use of high strength steel for large compression members may in some cases become disadvantageous.

By the use of high strength steel the required stel weight of a bridge will ap a matter of course decrease, but it must be mentioned here that the rate of the steel weight saving isby no means proportional to the rate of increase of the allowable stress or the ultimate strength of steels. This is due not only to the resistance to buckling but also to that there are various restrictions regardings designing, such as regarding the minimum thickness, rivet pitches, maximum deflection, etc., that hinder us from using high strength steel freely.

The saving of steel by the use of high strength steel in right places, however, usually overcomes the increase of steel cost. In the appendix is shown a deck plate birder bridge of composite structure in which high strength steel SM50 (ultimate strength greater than 50 kg/mm^2) was used in the central part where large bending moments occur. This SM50 is of very good weldability and has become in present-day Japan to be used so commonly as the ordinary structural steel SS41 or SM41 that, if we say "high strength steel", it means the steel having an ultimate strength greater than 70 or 80 kg/mm^2.

The example of composite-girder using SM50 is shown in Fig. 4-1.

Fig.

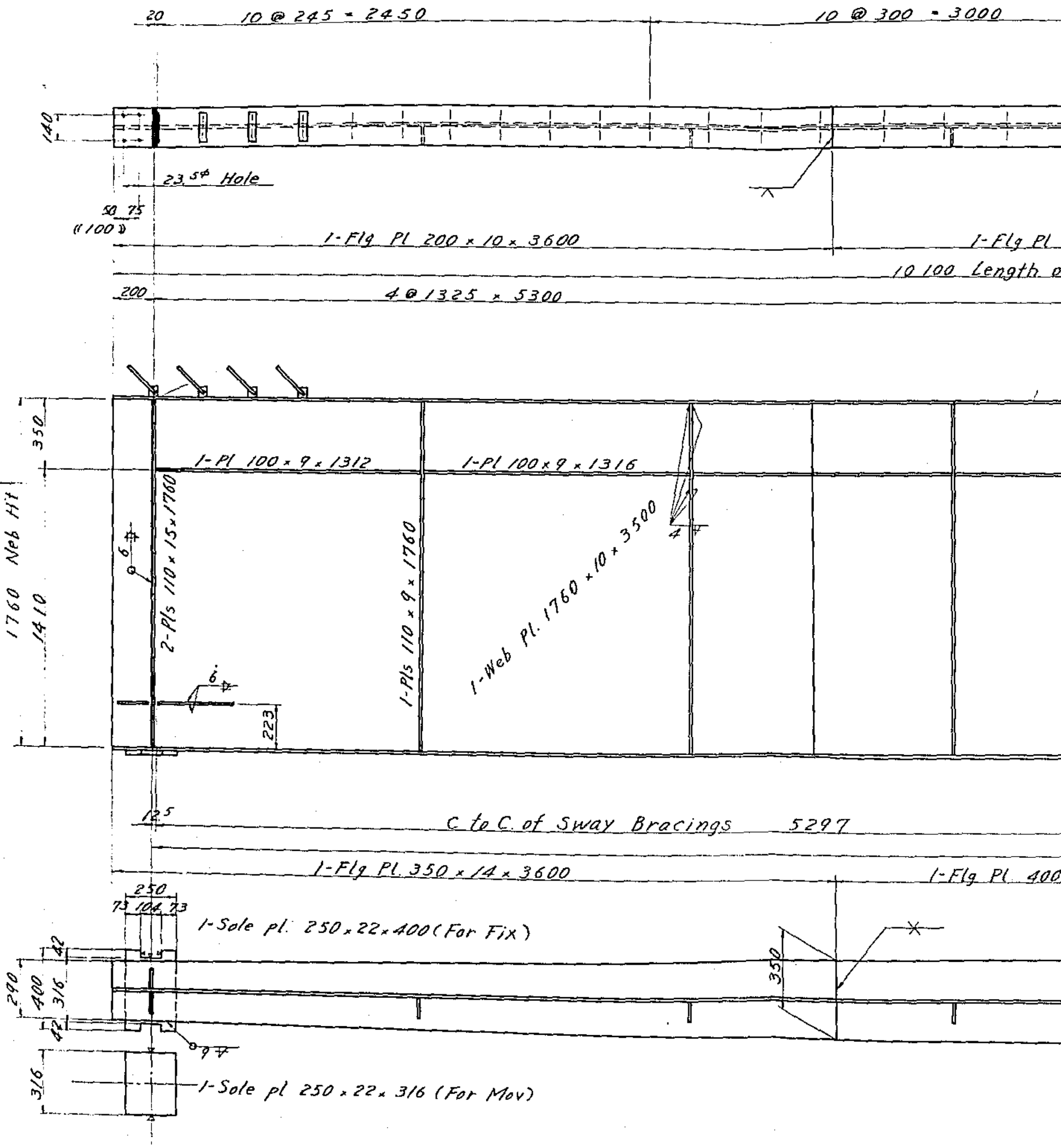

20
10 @ 245 = 2450
10 @ 300 = 3000
140
23.5φ Hole
50 75
((100))
1-Flg Pl 200 × 10 × 3600
1-Flg Pl
10 100 Length o
200
4 @ 1325 = 5300
350
1-Pl 100 × 9 × 1312
1-Pl 100 × 9 × 1316
1760 Web Ht
1410
2-Pls 110 × 15 × 1760
1-Pls 110 × 9 × 1760
1-Web Pl. 1760 × 10 × 3500
223
125
C to C of Sway Bracings 5297
1-Flg Pl. 350 × 14 × 3600
1-Flg Pl. 400
250
73 104 73
1-Sole pl. 250 × 22 × 400 (For Fix)
42
290
400
316
350
1-Sole pl 250 × 22 × 316 (For Mov)

Fig. 4-1 Details of Composite-Girder

(Using high tensil steel SM 50 A)

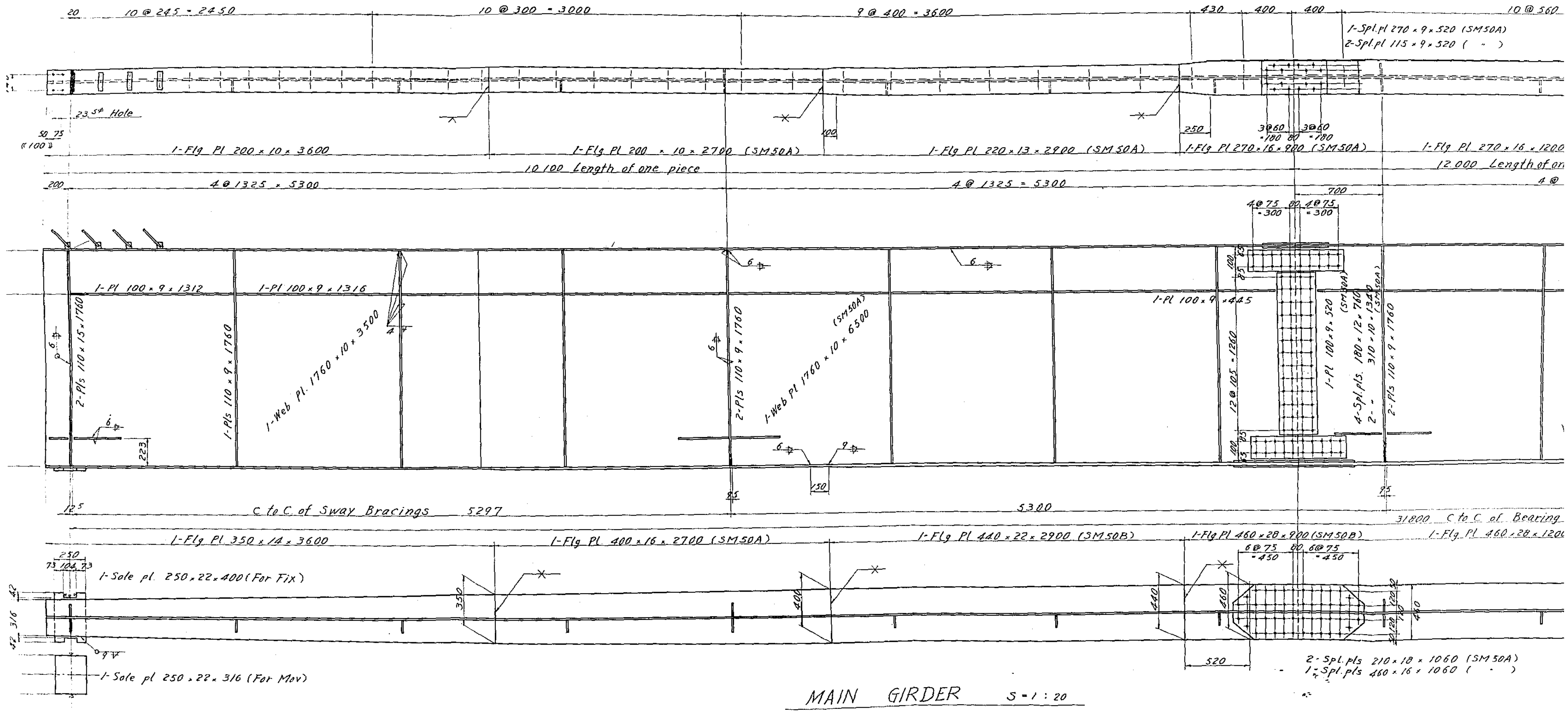


Fig. 4-1 Details of Composite-Girder
(Using high tensil steel SM 50A)