	Average	Minimum	BS15 Requirement
Yield Stress (N/mm^2)	300	242	247 min.
Ultimate Stress (N/mm^2)	444	384	432 min.
Elongation (%)	32	25	22 min.

As far as yield stress is concerned, the original structure fairly complies with the BS15 requirement. However, ultimate stress does not satisfy the minimum requirement.

130 Kafue Road Bridge was constructed approximately thirty (30) years after construction of Kafue Rail Bridge. It is reasonable to assume that steel materials had been improved during these 30- years. However, it would be safer for the existing steelwork to presume lower stresses. It is proposed to use 210 N/mm^2 (21 kg/mm^2) which is equivalent to approximately 85% of BS15-247 N/mm^2, as an assumed permissible stress, and 1,100 kg/cm^2 as an allowable stress.

5-C. SUBSTRUCTURE AND FOUNDATION

(1) Substructure Analysis

131 The existing piers and abutments have been analyzed on the assumption that the original (1952) structure elements and the subsequent (1968) raising resulted in a monolithic mass concrete structure. The analysis has been carried out in order to determine whether the bridge can carry the loading required as a Class 1B road.

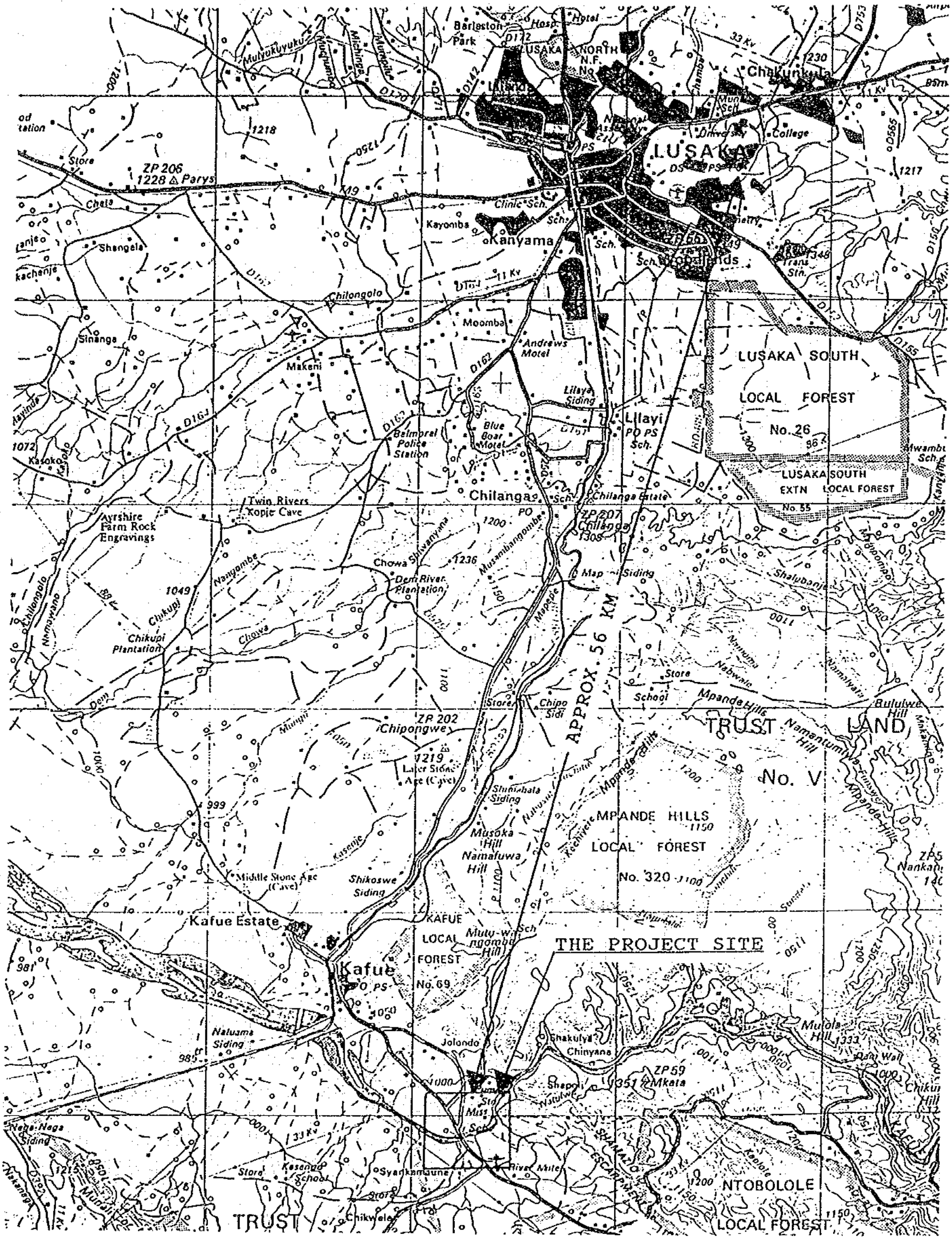
(2) Piers and Abutments

132 The piers can be analyzed as monolithic structure, provided the original concrete and the added concrete during the raising are fairly in good condition and surfaces between the two maintain adequate bonding.

According to drawings as raised, the surfaces were exposed, some concrete was removed and the reinforcing bars were exposed, cleaned and primed with UNIBOND.

133 According to drawings prepared by Freeman Fox and Partners in 1989, concrete proportioned in 1:2:4 was used for the top 2'-6" of pier shaft, while concrete proportioned in 1:3:6 was used for the rest.

Fig. 5-1 LOCATION OF THE KAFUE ROAD BRIDGE



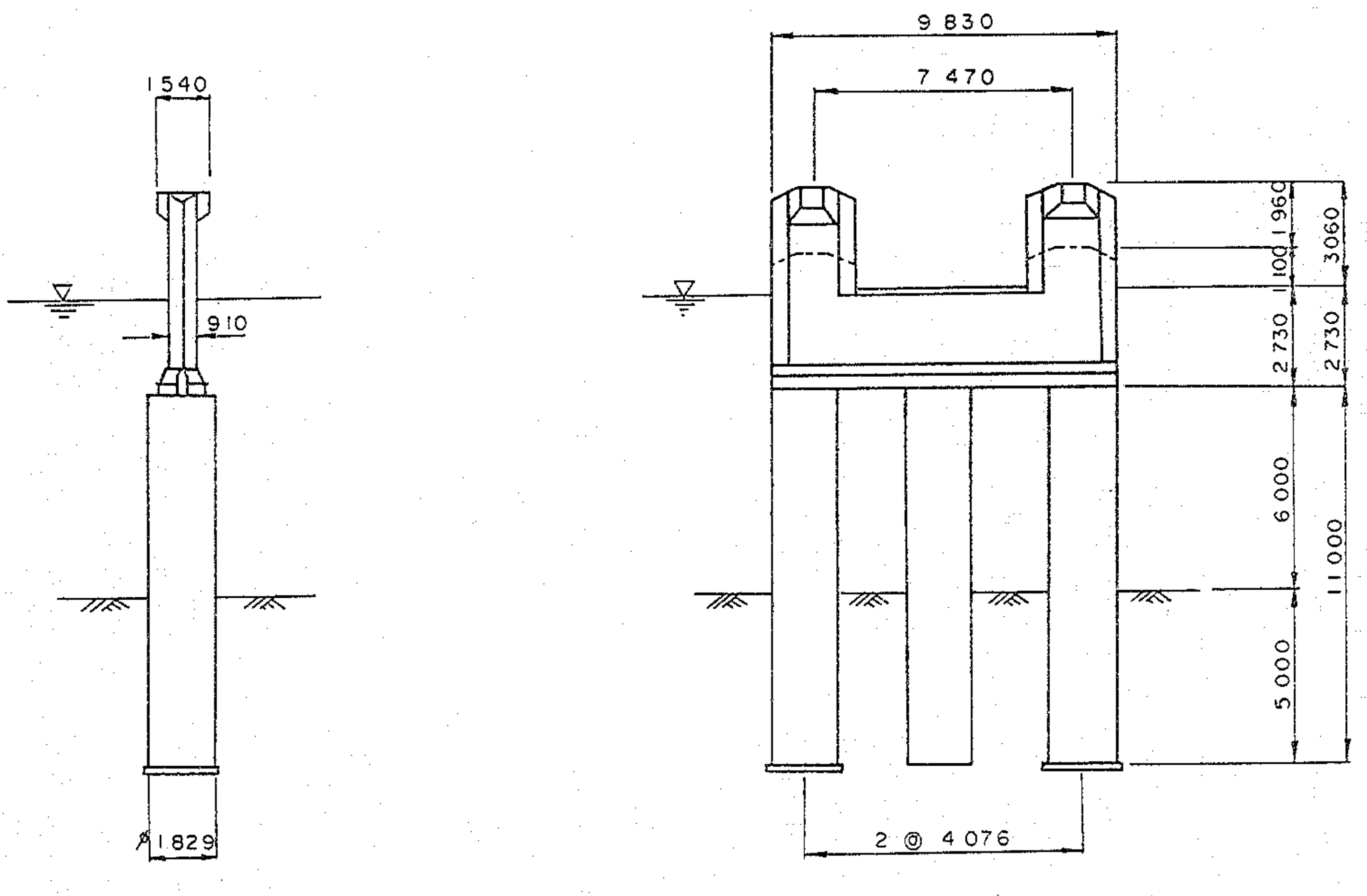


Fig. 5-2 Pier of Existing Bridge
(by Original Drawing 1949)   All demensions are in mm

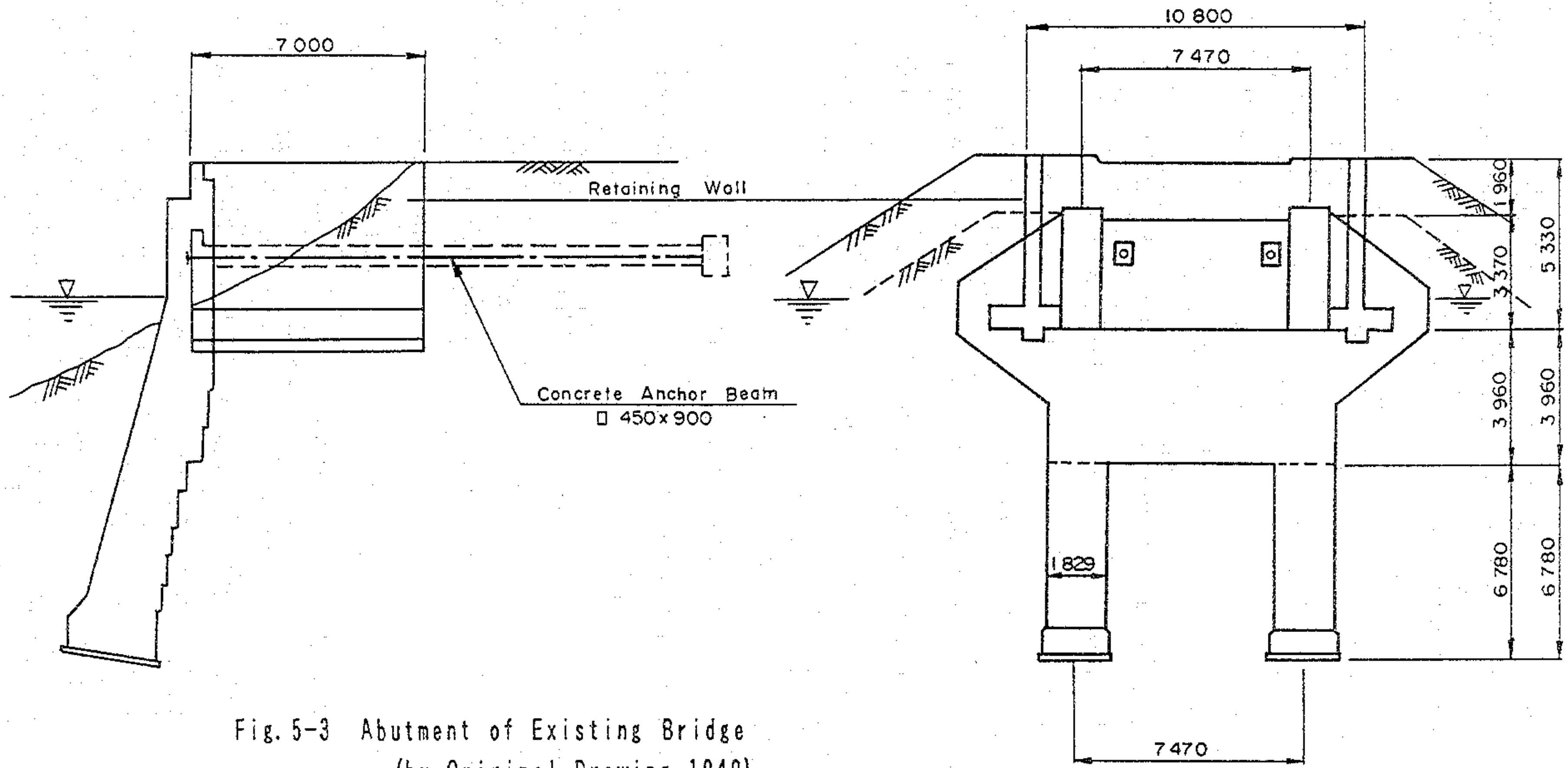


Fig. 5-3 Abutment of Existing Bridge
(by Original Drawing 1949)
All demensions are in mm.

134 As a result of the visual inspection, the bonding surface was inspected to ensure sufficient bonding. Although surfaces of both piers and abutments show some weathering and with lime seeping due to rain water, no progressive crack can be observed.

135 In compliance with a concrete strength test made using a mechanical hammer, the strengths of concrete at the time the original and raising were estimated to be more than 210 kg/cm^2. As the bulk of the original structure had been in the water, the concrete strengths were estimated by checking visible portions only.

Because of the above reason, it was impossible to verify accurately the concrete quality as well as bonding conditions. However, structural elements of piers and abutments were judged reasonably sound.

136 A retaining wall-parallel wing- was constructed when structural elements of abutments were raised. A gap has developed between this wing and structural elements.

The original structure was provided with lateral wing wall. The lateral wing of the downstream Lusaka side seemed unable to withstand the lateral pressure from the surcharge occasioned by the backfill upon being raised and was deformed, as a result.

137 Leaving the present condition alone might induce flow-out of back filling and subsequently result in the collapse of the road surfaces and the collapse of embankment.

138 As expressed in Paragraph No. 169, three-span trusses get reciprocal contact and form a linked condition. As a result, the expansion joints begin to contact each other. This condition has been brought by either deformation of at least one abutment due to soil pressure of the back fill material or deformation of parapet wall.

139 According to visual inspection, crack in parapet wall base cannot be observed and it seems the deformations either moved or rotated the abutments. This means abutment foundations are at risk of collapse if water way gets scoured in the future.

140 It is thought that, after the raising, the abutments were pushed forward due to soil pressure on the back face of abutments.

(3) Foundations

141 According to "as constructed" drawing of 1953, foundations at the abutments have a structure of so-called retaining wall with no-reinforced cement concrete. Footings are founded over red clay or grey clay stratum. Front surfaces of the abutments are not scoured at all. Based on visual inspection, movement of the abutments appeared minimal, although it was not possible to determine an absolute value of the movement since the dimensions upon completion were uncertain.

142 When raised, an anchorage with 2-tie bars (Macalloy High Tensile Bar with 1 3/8" diameter) is provided to resist earth pressure due to extra filling and vehicle loads. The tie bars are protected by wrapping and "DENSO" paste. No adverse effect due to the anchorage is seen on the road surfaces and accordingly it is judged that no deformation exists.

143 Back surfaces of foundations at the abutments were bench cut. Each layer of soils is clearly shown on drawing No. A-45605 "as constructed" (5-1-53) and the surfaces proved to be in good condition.

144 Foundations at both north and south piers have the same structure and are composed of three (3) 6'-0" dia shafts. Each shaft is consisted of 3/8" thick steel cylinder, which contains 6 ft high base segment and 3'-0", 2'-0" and 1'-0" high adjustable segment, filled with reinforced concrete. Structurally it can be said that the foundations are classified to be reinforced concrete structure.

145 In accordance with drawings reproduced in June, 1989, each shaft shall have 25' ± penetration to be varied depending upon ground conditions. A projection from the river bed shall be as short as possible and assumed to be approximately 8'-0".
The drawings designate enlarging the bases due to undercut at the bottom of cylinder can be implemented if the Engineer's directive is made.

146 It must be borne in mind that each shaft has been penetrated to white clay layers, viewing from log of boreholes shown in Drawing No. A42900 16-2-48 and possible directive by the Engineer as delineated above.

147 Referring comprehensively to "Cross Section Kafue River at Road Bridge, Water Development & Irrigation Dept., Dec. 1952" and each construction drawing of Kafue Road Bridge, an approximately 1.2 m of scouring has occurred in the river bed between 1938 and 1952.

Though neither report nor drawing concerning river bed elevation at a time of the raising were found, a soil investigation during the Study can prove that the river bed had been considerably scoured.

148 The borings show rocks designated as white clay exist beneath sediment being located 1.5-3.0 m below river bed. The strength of white clay is great and this layer is presumed to have resisted scour. Additionally, the present river bed at the Bridge site become 0.5-1.0 m higher than height of river bed in 1952 as seen by river cross section survey. (See Drawing River Cross Section survey.) It is considered that the river bed deeply scoured at once through 20 years from 1952 to 1972, and then sediment layer of sand/soil has settled on the scoured river bed after completion of Kafue Gorge Dam.

149 From results of the soil investigation there emerge several essential facts. First of all, white clay layer has appeared shallower than the depth indicated on the drawing of "The Best Trust Road Bridge over the Kafue River, General Arrangement 16-2-48". Observing core and test results of standard penetration, it is extremely unlikely that the shaft of pier foundations would penetrate into the white clay layer.

150 In a bid to know exact depth of footing, it would be most reliable to bore through piers and cylinders. However, in this Study, this was not carried out, since many of "as constructed" drawings were maintained.

151 Nevertheless, it seems adequate that the shaft penetrates no more than 0.5 m into the white clay stratum, though neither any reports nor materials to prove exact bedding depths were found. This judgement is made based upon an assumption that the Engineers would generally not direct to penetrate deep into white clay layer at the time of the construction of foundations.

5-D. SUPERSTRUCTURE

(1) Features

152 Sections of main structures are comprised of steel plates and shapes that are connected by rivets. Diagonals, which are usually adopted to be tension members in pratt-truss structures, also are composed of channels. Thus, the bridge appears to have relatively high stiffness for an archaic truss.

153 Bolts are applied to connect members, and their diameter is shown as 15/16" (23.8 m) for main structural elements according to available drawings. However, it was not possible by only visual inspections to distinguish whether these are ordinary bolts or high strength bolts. Hence, it shall be assumed to be ordinary bolts, since the drawings do not designate high strength bolts and the bridge was originally constructed some fifty (50) years ago.

155 Both stringers and floorbeams are comprised of I-shaped beams.
A particular facet of this bridge is on geometric correlation of top surfaces between stringers and floorbeams, that is, top surfaces of stringers are located beneath those of floorbeams. This is one of typical railway bridge features and as a result the bridge can be said to have peculiar geometry for road bridges.

156 Portals and top lateral bracings are comprised of miscellaneous angles and seem to be common structures for these old type trusses.

(2) Damages to Members and Road Surfaces

(a) General :

157 A visual inspection was carried out to identify damages against structural members and road surface.
According to the inspection, several severe damages presumably due to vehicles collisions have been occasioned on such structural members as portals, top lateral bracings and a vertical.

158 Various damaged portions are shown in Photo 5-1 to 5-8.

Photo 5-1. Buckled Vertical

Photo 5-2. Buckled Vertical

Photo 5-3. Partly Missing Bracings

Photo 5-4. Several Bent Bracings

Photo 5-5. Pot Holes on Carriageway

Photo 5-6. Pot Holes on Carriageway

Photo 5-7. Intermittent Loss of Pavement

Photo 5-8. Intermittent Loss of Pavement

(b) Top Lateral Bracings :

159 Although clearances between top of road surfaces and bottom of upper structures are properly furnished as stipulated by "Highway Design Standard" of Roads Department of Ministry of Works and Supply in the Republic of Zambia, portals and top lateral bracings were yet severely damaged due to collisions during passage of heavy trucks and/or trailers. It appears that the insufficient provision of the clearances, and inadequate restriction on sizes of vehicles as well as loads have contributed to these severe damages.

160 Particularly, two (2) top lateral bracings are partly lost apparently due to repetitive collisions by oversize consignment and/or vehicles. Refer to the Appendix 5-4 for this location. (See Photo 5-3 and 5-4.)

(c) Verticals :

161 Similar incidents to the aforementioned have befallen the guardrails. As relatively smaller carriageway (in fact, carriageway widths are 6.10 meters) which at present hinders mid and large-sized vehicles from passing each other on the bridge, Guardrails have been damaged here and there due to contacts by vehicles. Worse yet, one of the verticals on the downstream side is heavily buckled due to a presumed crash of a vehicle. This curvature has extended to the member's entire length and most likely caused substantial decrease in compressive capacity of the member. (See Photo 5-1 and 5-2.)

(d) Surface of Pavement :

162 At close visual inspection of the road surfaces, three (3) large pot holes and intermittent loss of pavement at every expansion joints were observed. At close look at these pot holes, it was indicated that an existing thickness of asphalt pavement is approximately 10 mm. (See Photos. 5-5 and 5-6.)

(3) Corrosion

163 Concurrent with the inspection of the damages, visual inspection throughout the entire bridge was implemented to examine conditions of painted surfaces.

164 Generally speaking, it seems that the environmental circumstance around the bridge site are such that less maintenance requirement for corrosion resistance are needed. Repainting that was done recently shows that painted surfaces are in fairly good condition. Consequently, no considerable corrosion was noticed on painted surfaces.

165 However, such damaged portions as part of portals, top lateral bracings and verticals are left unpainted, exposed to the weather, and accordingly have heavily rusted. Refer to Appendix 5-4 for locations of the rusty areas.

Several photographs taken at every visible rusty areas are shown in the Photo 5-1 to 5-4, and are self-explanatory.

(4) Movement of Truss Girders

166 Bearing shoes in three-spans truss girders have been arranged to enable each span to move longitudinally, as shown in a schematic drawing of Fig. 5-10.

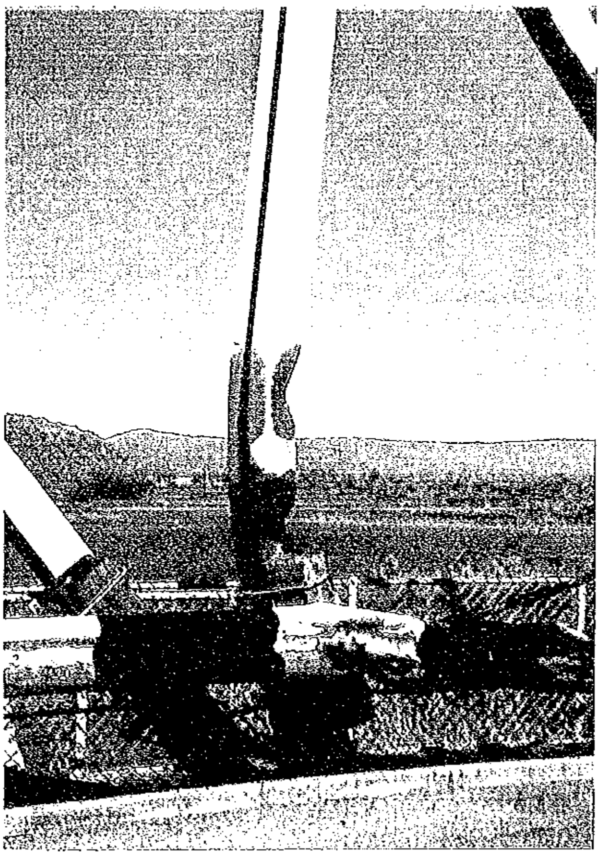


Photo. 5-1　Curved Vertical

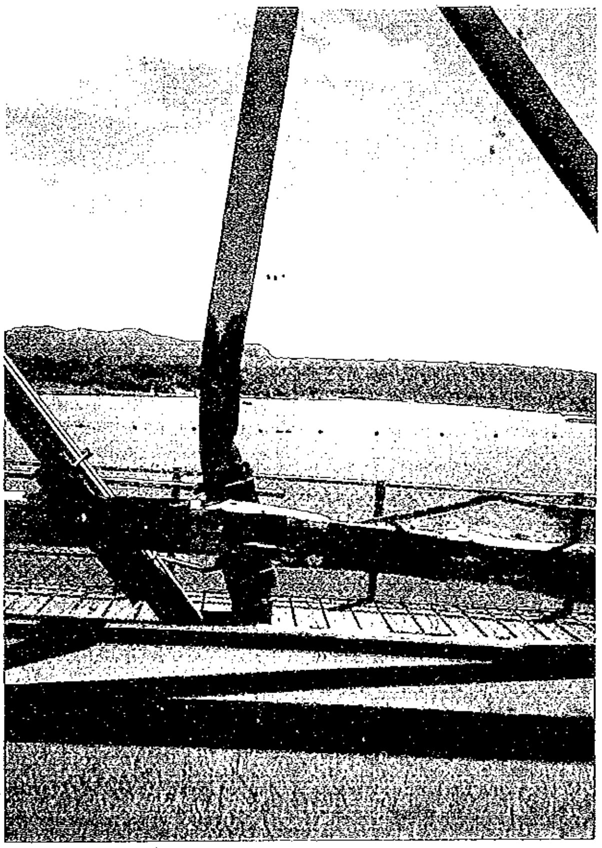


Photo. 5-2　Curved Vertical

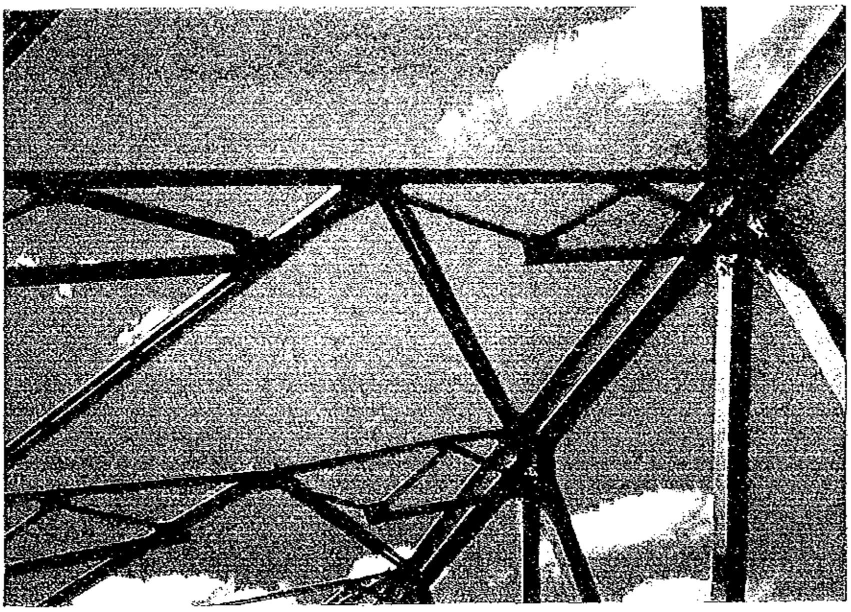


Photo. 5-3. Partly Missing Bracings

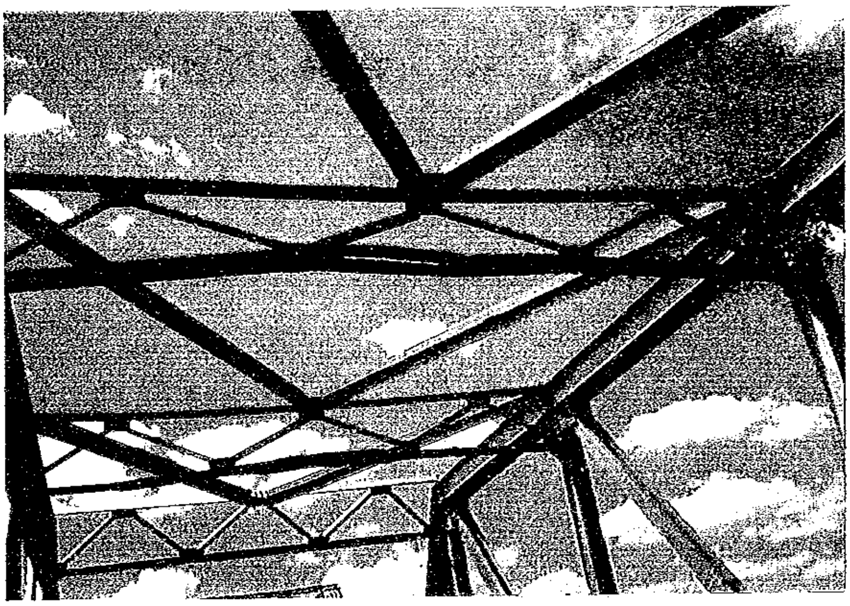


Photo. 5-4. Severely Bent Bracings

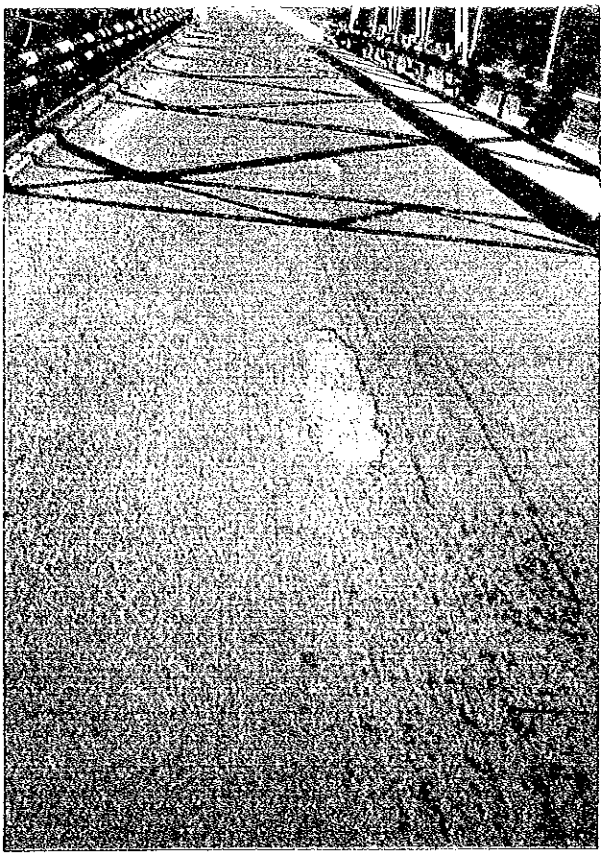
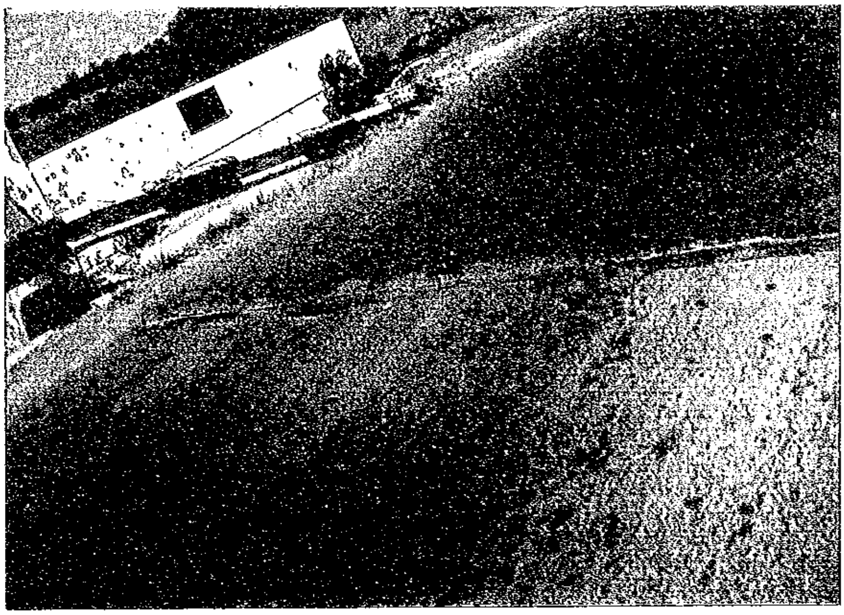


Photo 5-5. Pot Holes
on Carriageway

Photo 5-6. Pot Holes
on Carriageway



Phto 5-7. Intermittent Loss of Pavement

Fig. 5-8. Intermittent Loss of Pavement

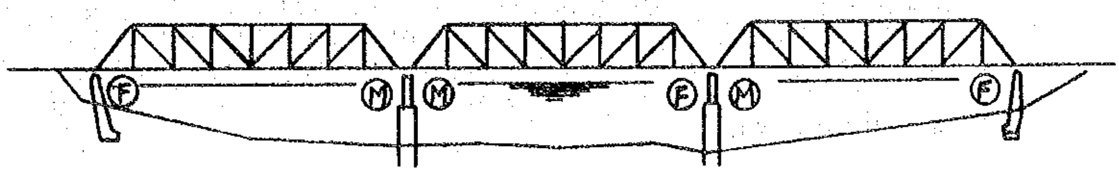


Fig. 5-4 Arrangement of Bearing Shoes

167 With effects of temperature variation and live load, each span will experience expansion and contraction horizontally as well as vertical deformation and rotation at girder ends.

168 Assuming atmospheric temperature variation ranging from -2°C to + 31°C and temperature variation in structural elements to be ± 20°C, each span would move approx. ± 1.0 cm respectively. Considering construction tolerances and structural allowances at least 4.0 - 5.0 cm clearance at the north pier and 3.0 - 4.0 cm clearance at the south pier are generally required.

169 According to original drawings at the time of raising, each expansion joint has been designed to provide for sufficient movement.

Expansion joints come into contact each other and longitudinal forces from abutments always act against floor slab. Meanwhile, in main trusses, web plates of lower chords mutually contact at the north pier, movable shoes at the south pier slide fully toward north, thus making the entire structure similar to a three-spans pinned connection girders.

5-E. DURABILITY OF BRIDGE AND RECOMMENDATION

(1) Superstructure

170 Structural elements are maintained considerably sound because an environment around Kafue region that has no air pollution gives an advantage to mitigate steel corrosion. Good maintenance has also been kept. It has been some 40 years since the bridge was transferred to

Zambia after its original construction as the emergency bridge on the Thames River. Heavy vehiclular traffic has been carried on the Bridge for many years, therefore possible fatigue may be assumed to have affected part of floor systems.

171 A vertical has been heavily deformed due to vehicle's crash. As generally known, compressive stress affects the vertical members in pratt -truss and the deformation significantly effects the compressive strength of the vertical. Since only one heavy vehicle is carried on the bridge at any one time, no serious accident has been occurred. However, for the sake of safety, the vertical must be immediately replaced. The Bridge would be relatively sound and its reliability would quickly be restored provided the vertical is replaced.

172 Sway bracings are also damaged by collision of vehicles' cargoes. These damges do not adversely affect normal traffic, as the bracings are not main structural elements to sustain vehicle loads. But, these are important members to such transverse loads as wind loads, to improve stiffness of bridge, and to secure safety of bridge. Consequently, the sway bracings need replacement as well.

173 The Bridge was originally designed and constructed for response to air raid, and therefore, was designed to enable cantilever erection and to have equal spans and equal sections individually in top and bottom chords, diagonals and verticals through the span. As long as nominal design loads are concerned, each main structural member satisfies permissible stresses and has sufficient durability. However, as shown in Paragraph 172, a vertical and a diagonal in south span have been damaged due to collision, and as a result, weight restriction for vehicles shall be required, consequently.

174 As stated in Paragraph 169, each span has been linked longitudinally. This would produce a secondary stress equal to 1,500 kg/cm^2 in upper chords and verticals, and a secondary bending moment in lower chord ends, thus each member presumably has an excessive stress.

175 The width of the existing bridge is 6.11 metres between curbs and holds good for ordinary 2-lanes road. However, in this bridge, a clearance between carriageway and truss member is insufficient forcing large-size trucks and trailers to give way and wait for opposite traffic.

Eventually, the Bridge can function practically as a 1-lane bridge.

176 Roads connecting Lusaka with Chirundu/Livingstone have the open field of vision and only Kafue road Bridge itself visually furnishes a closed space. Because of this, the width is seen narrower by drivers. From a standpoint of safety driving, it can be stated that the width is too narrow.

(2) Substructure

177 Subsequent to the visual inspection, concrete bonding of the original and raised structural elements of piers seems relatively sound. Stress review in compliance with drawings shows the structural elements have sufficient durability.

178 As previously mentioned, stability of pier foundations has been remarkably deteriorated since the river bed is extremely scoured. On the other hand, soil/sand sedimentation has taken place and soft rock stratum, so called white clay became exposed in these 20 years after Kafue Gorge Dam construction. Therefore, it is assumed stability will no longer progressively became worst.

179 In general, possibility of bridge collapse is hypothetically expressed as follows:

(a) In Zambia, no substantial earthquake has been experienced up to date, thus no crash of building/structures has been occasioned. But, since maximum magnitude of 4.9 was recorded in Zambia and the height of piers reaches to 16 m, the pier has possibility to fall due to excess in the ultimate strength of the bearing rock if an earthquake equivalent to seismic coefficient of more than 0.03 would occur.

(b) Once Kafue Rail Bridge is replaced and its span is expanded, the influence by floating islands onto Kafue Road Bridge becomes more serious. In this assumption with an another assumption of extreme flood, piers would receive eccentric loads and be heavily damaged since the present piers have less torsional resistance.

180 Regarding the abutments, the parapet walls make completely contact with the end of floor slab. On the other hand, it was observed by visual inspection that the retaining walls are get slide and there are some gaps between the abutments and the retaining walls. If the gap is in progress, the stability of abutment systems would be subsequently more deteriorated.

(3) Recommendation

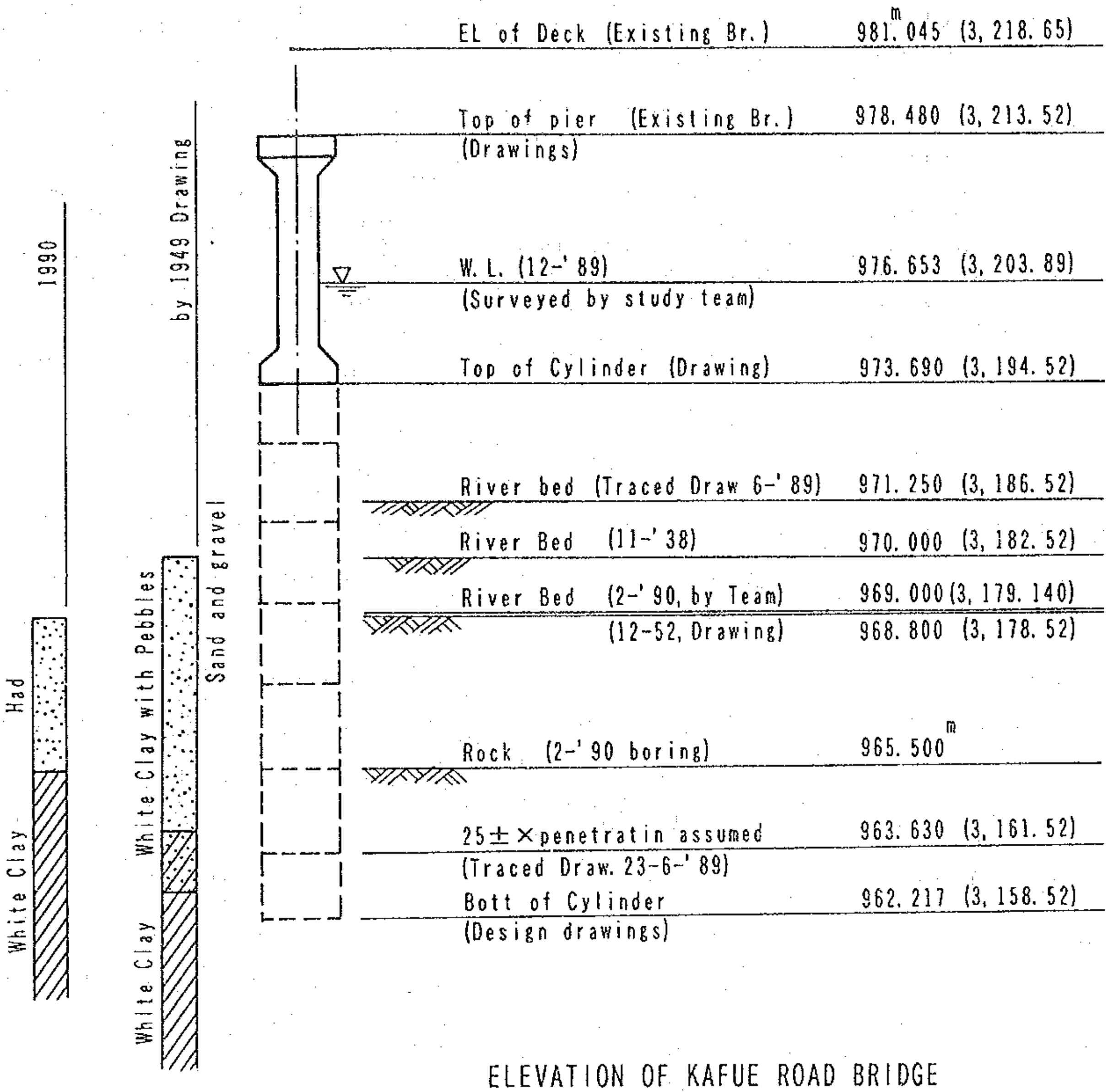
181 As far as superstructure is concerned, the structural elements are fairly sound and could last longer if part of damaged members is rehabilitated.

182 However, as a bridge in the main trunk road linking Lusaka-Livingstone and Chirundu, durability seems insufficient. To upgrade the durability, reinforcement must be given to the entire structural systems, and accordingly, strengthening with maintaining daily traffic seems impractical. Further, the present width can allow merely a single lane to large-sized vehicles. This means the Bridge could be a bottle-neck in the main trunk road if, in future, traffic volume is increased. Consequently, it is recommended that the Bridge should be replaced, taking into consideration the Bridge's essential role in the traffic.

183 The Bridge is sufficiently usable if it is relocated to the roads of class 2 or below. In such a case, sway bracings, shoes, handrails and other damaged structural members shall be replaced.

184 Pier foundations that were expected to have 25' (7.6 m) depth when designed have been scoured and are at present standing in the water as independent columns. Worse yet, the bridge was unexpectedly raised by approximately 2 m and foundations were made more unstable. Should the Bridge be used hereafter, rehabilitation of foundations seems imperative, although, for such an effect, high-grade construction technology and excessive expenditure would be needed.

185 In summary, the Kafue Road Bridge is on delicate balancing condition and might suffer possible destruction by becoming unstable due to unidentified factors. To predict collapse is beyond the binds of structural analysis and most likely impossible.



ELEVATION OF KAFUE ROAD BRIDGE

Guage Hight;Port Elizabeth Datum

Fig. 5-5

186 It is required in above context to urgently replace the Bridge, considering importance of the Bridge and social consequence by its possible destruction.

188 If, in the future, the Bridge is demolished and transferred to another site for further use necessitating repair or reinforcement, it is required to conduct metallurgical tests to determine material strengths and sulphur contents. Particularly, if welding is applied to repairs or replacement, care shall be taken to ascertain that no excessive sulphur is contained.

CHAPTER 6

DESIGN CONDITIONS

CHAPTER 6 DESIGN CONDITIONS

6-A. WIDTH OF CARRIAGEWAY AND FOOTPATH

(1) Road System

189 According to the Road and Road Traffic Ordinance, CAP 173 of the Law, road system and classification in the Republic of Zambia are classified in accordance with "Highway Design Standards" enacted in 1965 and revised in 1971 by Roads Department of Ministry of Works and Supply. (MOWS)

190 Although no specific definition is given for various categories of road, the Highway Design Standards specify that roads are international main roads which form the primary road network, connecting the Zambian highway system with that of neighbouring countries, linking Provincial seats of Government with the main centers of population.

In compliance with the Highway Design Standards, the road passing through the existing Kafue Road Bridge is categorized as a T road and called T2, accordingly.

(2) Road Classification

(a) Access Roads to the Bridge :

191 The Kafue - Chirundu Road, southern road of the present Kafue road Bridge, is a two (2) lanes road having a width (including shoulders) of 10.8 m. Rehabilitation of this road, funded by USAID, was completed in 1988. The road is classified as Class 1B in accordance with the Highwy Design Standards.

192 The Kafue - Lusaka Road, northern access road of the bridge, is also a two (2) lanes road, but almost with no surfacing on the shoulders. This road is programmed to be incorporated into the USAID rehabilitation project scheduled for 1990. Classification of this road is also defined as Class 1B for the road section of Kafue - Kafue Bridge.

(b) Road Classification for the New Bridge & Approaches :

193 According to the Highway Design Standards, various classes of road are provided depending on traffic ranges. The classification as stipulated by the Highway Design Standards is tabulated in the appendix 6-1 "Road Classification in the Republic of Zambia".

194 Since the access road to the bridge is classified as Class 1B, it is hereby recommended the road passing through the new bridge shall be also classified as Class 1B.

195 The standards for Class 1B relating to designing bridges are as summarized below;

a. Average Daily Traffic Estimated for 10th Year after Construction (ADT) .... 500 to 1,500

b. Width of Surfaced Carriageway .... 6.70 m

c. Shoulder Widths .... 2.00 - 3.00 m

d. Design Speed in Flat Topography 100

e. Limiting Grade in Flat Topography ...... 5 %

(3) Carriageway Widths

196 In as much as the new bridge is longer than 10 m between abutments, and on Class 1B roads with 6.70 m carriageway, it is stipulated in Section 6.1.1.b) of the Highway Design Standards that the width of the bridge shall be 7.30 m between kerbs. A composition of the width is as follows;

a. Widths of Traffic Lane ..... 3.35 m × 2 = 6.70 m

b. Widths of Marginal Strip ... 0.30 m × 2 = 0.60 m

(4) Footpath

197 Section 6.1.2. of the Highway Design Standards specifies that on all Class 1 roads, footpath shall be provided on

both sides of the carriageway irrespective of the length of the bridge. Consequently, the bridge shall have footpath on both sides of carriageway.

198 The same part of the Highway Design Standards also states that footpath shall have a normal width of 1 m but where greater widths are required they shall be made in increments of 0.50 m.

Considering the observations during the Study, that pedestrians are not too frequent nor forced to wade through the footpath, a greater width than 1 m does not seem to be necessary.

6-B. GEOMETRIC AND STRUCTURAL DESIGN CONDITIONS

(1) General

199 Since it was determined in the previous section of the Report that the road to pass through the new bridge ought to be Class 1B, the basic geometry and structure concerning the new bridge and approaches is given by the Highway Design Standards.

200 Geometry and/or structure consideration not clearly specified by the said Standards will necessitate, modification of the Standards, "Ordinance of Road Structure" enacted by Ministry of Construction of Japan is proposed for use in an effort to attain more secure bridge traffic.

(2) Geometric Design Conditions

201 Geometric design standards for the new Kafue Road Bridge are tabulated in the Appendix 5-2 "Geometric Design Conditions".
These values are in compliance with Section 6.1 of the Highway Design Standards.

Other principal conditions not listed above, as well as modified conditions are delineated hereinafter.

(a) Design Vehicle Dimensions :

202 Design vehicle dimensions are tabulated in Appendix 5-3 "Design Vehicle Dimensions", in compliance with the Highway Design Standards.

All dimensions are quoted from ASSHTO Standards, considering that they fairly represent types of vehicles using the roads in this country.

203 Section 6.1.6 of the Highway Design Standards requires that "underpasses shall have a minimum overhead clearance of 5 meters over the full carriageway width".

When compared with the design vehicle heights, this requirement of the overhead clearance seems reasonable.

(b) Moundup of Footpath :

204 Pertaining to Section 6.1.2. of the Highway Design Standards, the footpath shall be protected from traffic by a 150 mm high barrier kerb. Since this requirement seems to offer less secure bridge traffic, it is hereby recommended that a 250 mm high barrier kerb in lieu of the 150 mm shall be provided in accordance with the Ordinance of Road Structure of Japan.

(c) Verge Widths :

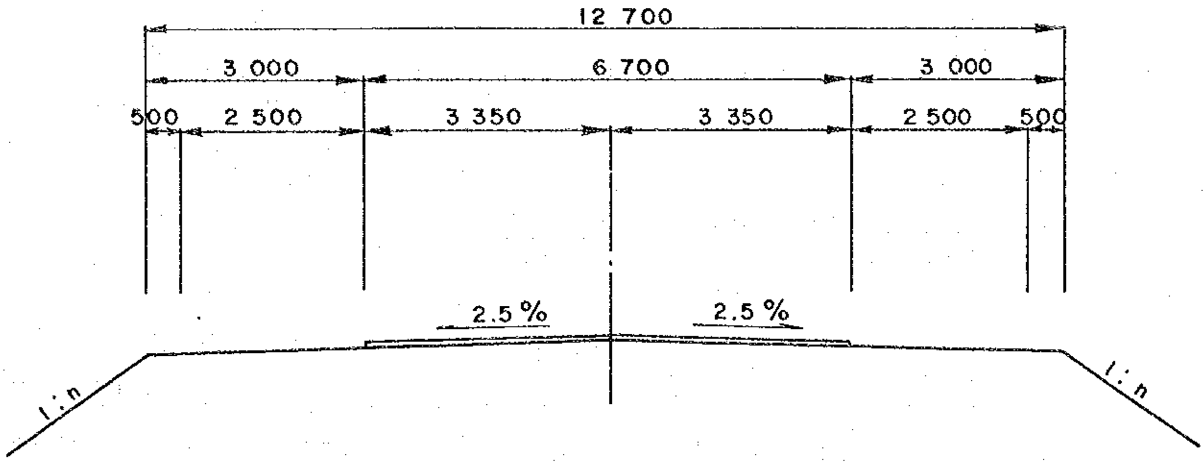
205 There is no specific requirement in the Highway Design Standards regarding widths of verges.

Therefore, it is proposed a 500 mm width of verge on each side of the bridge shall be provided.

(d) Crossfall of Footpath :

206 Requirement on crossfall of footpath is not found in the Highway Desing Standards.

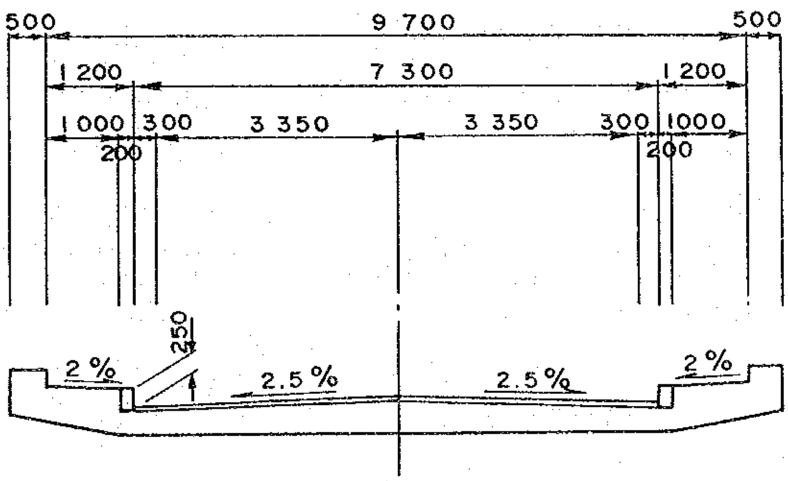
Accordingly 2% crossfall subject to the requirement of the Ordinance of road Structure of Japan is adopted.



n=2, Fills over 3 m in height

n=4, Below 3 m in height

Approach Embankment



Bridge

Fig. 6-1 TYPICAL CROSS SECTION

6-C. UTILITIES

207 There are power and telephone lines on downstream side of the present bridge.

A distance between the telephone line and the bridge is approximately 15 m, while that of the power line is 37 m.

208 Although it is commonly known that relocation of the existing utilities ought to be avoided in a planned alignment, it is often times inevitable to transfer, during construction, such utilities like the telephone line when they are located too close to a bridge.

209 The location of power line meets with requirement of the minimum distance by the Highway Design Standards and the distance shall be maintained as control points when planning the new alignment.

210 To provide for utilities, the followoing loads are assumed to be considered with the bridge loading:

(1) Water Main (400 mm) including attachments
------- 300 kg/m
(2) Power Line Facilities including attachments
------- 50 kg/m
(3) Telephone Line Facilities including attachments
------- 30 kg/m

210A Traffic safety device such as guardrails, deflector, lighting and posts shall be provided in order to cope with upgrading of the bridge and potential smoother traffic.

6-D. DESIGN SPECIFICATIONS FOR BRIDGE AND APPROACHES

(1) General

211 In the Republic of Zambia, the "Highway Design Standards" were enacted in August, 1965 and revised in April 1971. The "Standard Specification for Roads and Bridge" current edition is dated December, 1973. Both documents are published by Roads Department of Ministry of Works and Supply.

212 In compliance with Section 7 of the Highway Design Standards, bridges shall be designed to meet the requirements of "British Standard 153 Part 3A - Girder Bridges". It also specifies that unless a bridge is to be subjected to abnormally heavy live load, TYPE HA equivalent lane loading will be adopted.

213 Notwithstanding a full knowledge of the above stipulation in this country, it is proposed that the bridge design, except as set forth hereinafter, shall be in accordance with "Standard Specification for Highway Bridge" enacted in 1980 by Japan Road Association, while being mindful of compatibility with the local design standards.

(2) Affects due to Temperature change

214 During the Survey, three (3) sources of information were found to be available regarding temperature in Zambia. They are ;

(1) Records of July 1938 - July 1958 Meteorological Station

Lusaka Airport

(2) Records of April 1987 - March 1989 Meteorological Station, Kafue Polar

(3) An Outline of the Geography and Geology of Zambia

215 The maximum and minimum temperature collected from the above sources are summarized below :

(1) max. 32.4°C min. 8.2°C

(2) max. 39.5°C min. 2.0°C

(3) max. 37.0°C min. 2.0°C

Consequently, it is proposed that temperature variation to be considered for bridge design shall be ranged between 0°C and 45°C.

(3) Seismic Intensity

(a) Return period and Maximum Acceleration :

216 In general, correlation between sizes of earthquakes that broke out on the circumference of construction sites of any structure and their frequency is defined in accordance with the following equation by Gutenberg-Richter.

$$\log_{10} N_{(m)} = a - bm \qquad \cdots\cdots\cdots ①$$

where,

$N_{(m)}$: frequency of earthquakes of which magnitude "M" is not less than "m"

a , b : constants

217 When return period "T_R" of earthquakes with having not less than magnitude "m" broken out in past T-years is to be $T/N_{(m)}$, then the equation ① is converted as follows;

$$\log_{10} T_R = \log_{10} T/N_{(m)}$$

$$= \log_{10} T - a + bm \qquad \cdots\cdots\cdots ②$$

$$= bm + C$$

where,

C : a constant

218 And if the following experienced equation is applicable to express correlation of magnitude "M" with max. acceleration "A_{max}",

$$A_{max} = A \times 10^{BM} \times (R+R_9)^c \qquad \cdots\cdots\cdots ③$$

the return period "T_R" and max. acceleration "A_{max}" will be linear on both logarithms, as follows;

$$\log_{10} T_R = \frac{b}{B} \log_{10} A_{max} + C - \frac{b}{B} \log_{10} A - \frac{b}{B} C\log_{10}(R+R_0)$$

$$= \alpha \log_{10} A_{max} + \beta \qquad \cdots\cdots\cdots ④$$

α, β : constants

therefore, assuming that probability of earthquakes in future is equivalent to that of previously occurred earthquakes, expected value of reappearance of max. acceleration is determined by the above equation.

(b) Seismic Coefficient :

219 Table 6-1 shows earthquakes occurred around Zambia during 1983 through 1988. The table implies no major size of earthquakes had taken place in Zambia and its environment.

220 On the table, max. acceleration at the project location of Kafue Road Bridge assumed by the following equation has been also shown

$$A_{max} = 987.4 \times 10^{0.216M} \times (R+30)^{-1.218}$$

221 This is an equation stipulated in "Standard Specification for Highway Bridge" by Japan Road Association, subject to Type-1 ground.

222 Correlation between max. acceleration and retern period at Kafue Road Bridge due to these earthquakes is as appeared in Fig. 6-2 and the expected value of the max. acceleration against 100-year of return period is assumed to be approx. 40-60 gal, in accordance with Gutenberg-Richter's equation. Seismic intensity is defined dividing acceleration of earthquakes dynamity by acceleration of gravity. Considering a short analyzed term, max. acceleration of 100-years return period is required to be thought safer and 100 gal accordingly. In this condition, seismic coefficient at Kafue Road Bridge shall be

$$K_h = 100\,\text{gal} / 980\,\text{gal} = 0.10$$

223 The coefficient of 0.10 is not thought too small since seismic coefficients as listed below are applied at locations with frequent earthquakes.

Japan	0.20
New Zealand	0.10 - 0.15
USA	0.12
Yugoslavia	0.10

(4) Design Wind Speed

(a) A Method to Analyze

224 Distribution function F(V) for annual max. wind speed is, in general, shown by Fishier-Tippett's I-Type ultimate value distribution and is as follows:

$$F(V) = \exp\{-\exp(-y)\}$$

$$y = (V - \mu)/\sigma \qquad \text{.......... ①}$$

where

- y ; a standardizing variable
- μ ; parameter to designate dispersion of wind speed Vi
- σ ; parameter to designate dispersion of wind speed Vi

225 A standardizing variable yi against wind speed Vi can be expressed as follows;

$$yi = -\ell_n\{-\ell_n F(Vi)\} \qquad \text{.......... ②}$$

and

$$F(Vi) = 1 - p(Vi) \qquad \text{.......... ③}$$

is a relation between distribution function F(Vi) and excess probability P(Vi) for wind speed.

226 Also, the following experienced equation by Gringorten is given to the excess probability (probability that wind speed vi would exceed a certain wind speed value of Vn).

$$P(Vn) = (n-a)/(T+1-2a) \quad a = 0.44 \qquad \text{.......... ④}$$

T ; years to be evaluated

227 Recurrence line shall be given by the following equation, when the line is determined by min. square method of N-sets of wind speed Vi and standardizing variable yi plotted on probability paper.

$$(y-\mu_y)/Sy = (V-V)/S^y \qquad \text{.......... ⑤}$$

where, V and S_v are mean value and standard deviation of wind speed date V_1, V_2, ···, V_n, respectively, while μ_y and S_y are mean value and standard deviation of standardizing variables. Accordingly, expected wind speed value V_T against reappearance term T_R shall be

$$V_T = V + S_v\,(y_T - \mu_y)/S_y$$

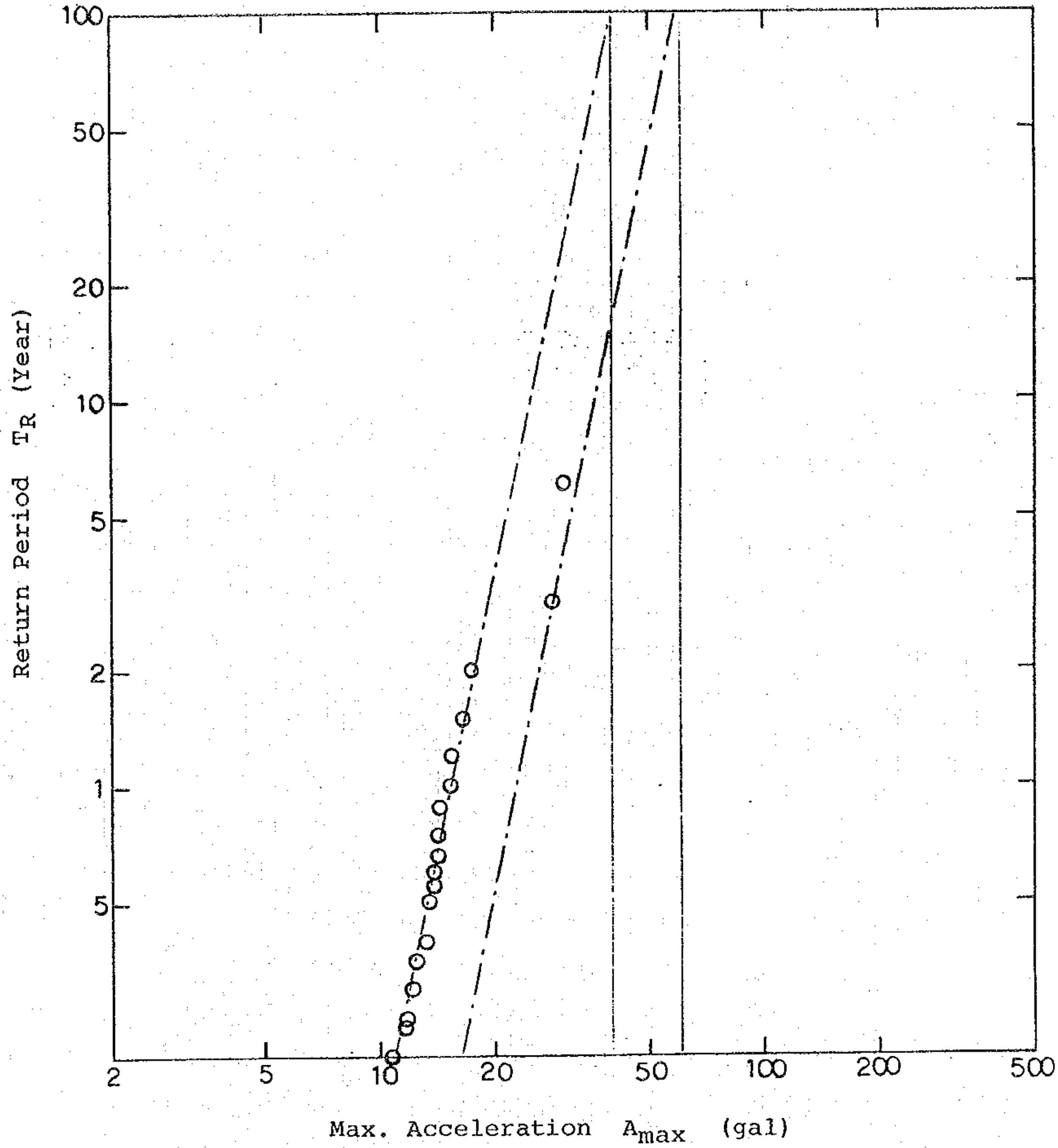


Fig. 6-2 Correlation between Max. Acceleration and Return Period

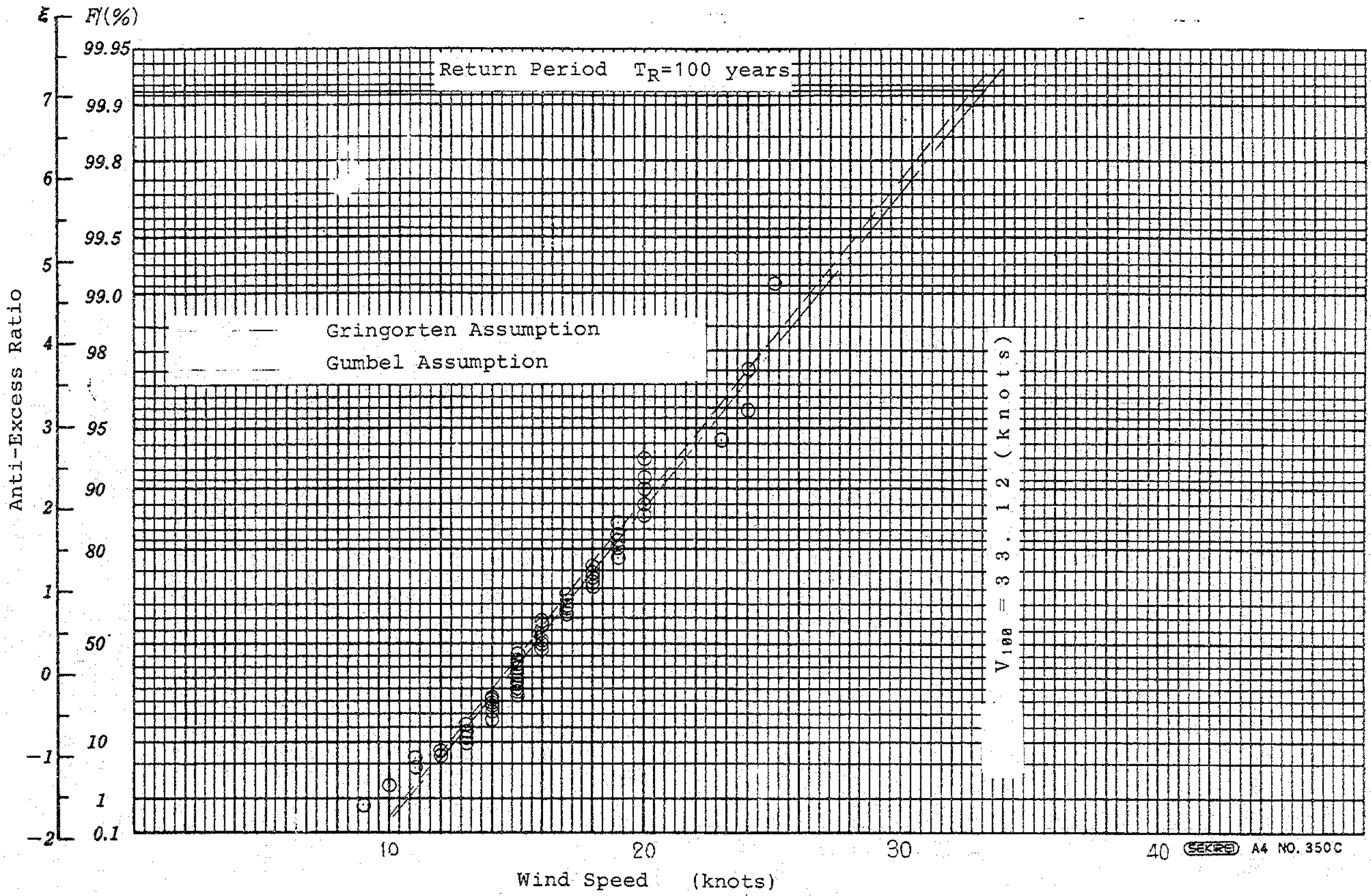


Fig. 6-3 Correlation of Wind Speed with Return Period

Table 6-1 Earthquake Records around Zambia (1983.1-1988.5)

Year, Month, Day	Region	Seismic center lat.	long.	Distance from Epicenter R (km)	Magnitude M	Acceleration A (gal)
1983. 1.23	C/PROV.ZAMBIA	14.5	29.4	177.2	3.1	6.96
2. 2	N/PROV.ZAMBIA	8.1	29.8	846.9	3.7	1.62
2.11	N/PROV.ZAMBIA	10.7	32.3	700.3	3.1	1.50
2.13	L.KARIBA	16.4	28.6	98.3	3.0	11.88
2.22	N/PROV.ZAMBIA	8.7	29.6	777.6	3.5	1.62
2.26	L.KARIBA	17.4	27.5	212.6	2.9	5.20
4. 7	S/PROV.ZAMBIA	17.6	25.5	363.0	3.4	3.71
4. 8	C/PROV.ZAMBIA	14.9	28.2	77.4	2.8	13.36
4.17	W/PROV.ZAMBIA	14.0	26.4	262.2	4.0	7.17
4.19	C/PROV.ZAMBIA	15.4	29.6	151.7	3.2	8.59
4.21	C/PROV.ZAMBIA	14.5	27.5	143.0	3.2	9.11
4.28	E/PROV.ZAMBIA	14.7	30.6	276.3	3.7	5.83
5.12	C/PROV.ZAMBIA	15.3	29.2	112.3	3.0	10.47
5.28	W/PROV.ZAMBIA	15.3	22.1	654.9	3.1	1.62
8.18	C/PROV.ZAMBIA	14.9	27.1	141.1	3.5	10.72
10.13	N/PROV.ZAMBIA	8.7	29.7	779.8	3.4	1.54
11. 6	W/PROV.ZAMBIA	11.2	24.8	609.8	3.2	1.85
1984. 5. 7	NW/PROV.ZAMBIA	13.8	26.2	293.1	3.4	4.70
7. 8	C/BELT.ZAMBIA	13.6	27.4	237.3	2.9	4.62
7.30	NW/PROV.ZAMBIA	11.8	26.6	454.2	3.6	3.17
9.18	NW/PROV.ZAMBIA	13.9	27.5	202.4	2.8	5.21
10. 2	NW/PROV.ZAMBIA	12.4	26.3	408.9	3.0	2.66
10. 6	C/PROV.ZAMBIA	14.4	27.2	170.7	3.5	8.83
10.20	C/PROV.ZAMBIA	14.3	29.2	179.5	3.2	7.22
11. 4	C/PROV.ZAMBIA	13.7	28.9	223.1	3.2	5.73
11. 4	L.KARIBA	16.3	28.9	107.7	3.4	13.29
11. 4	L.KARIBA	16.3	28.7	94.1	3.0	12.37
11. 5	L.KARIBA	16.3	28.7	94.1	2.9	11.77
11. 8	L.KARIBA	16.3	28.9	107.7	3.7	15.43
11.10	L.KARIBA	16.2	28.9	100.1	3.4	14.25
11.11	L.KARIBA	16.2	28.9	100.1	3.0	11.68
11.13	L.KARIBA	16.2	28.9	100.1	3.4	14.25
11.13	L.KARIBA	16.2	28.9	100.1	3.1	12.28
11.14	L.KARIBA	16.2	28.9	100.1	2.9	11.11
11.21	L.KARIBA	16.2	28.9	100.1	3.1	12.28
11.24	L.KARIBA	16.2	28.7	85.2	3.0	13.54
12. 5	L.KARIBA	16.7	29.0	148.6	3.8	11.81
12. 6	L.KARIBA	16.4	28.7	103.4	2.9	10.78
12. 7	L.KARIBA	16.4	28.7	103.4	3.8	16.86
12.12	L.KARIBA	16.6	28.7	122.8	2.9	9.13
12.26	L.KARIBA	17.0	27.9	158.1	2.8	6.75
1985. 1. 6	L.KARIBA	16.2	28.9	100.1	3.0	11.68
1.14	L.KARIBA	16.3	28.8	100.5	3.1	12.22
1.28	L.KARIBA	16.2	28.8	92.3	2.9	11.98

Table 6-1 Earthquake Records around Zambia (1983.1-1988.5)

Year,Month, Day	Region	Seismic center lat.	long.	Distance from Epicenter R (km)	Magnitude M	Acceleration A (gal)
1985. 2.22	L.KARIBA	16.6	28.8	127.8	3.1	9.70
2.23	L.KARIBA	16.3	28.9	107.7	3.2	12.04
2.27	L.KARIBA	16.2	28.9	100.1	2.8	10.57
3. 7	C/PROV.ZAMBIA	8.7	29.4	773.6	3.4	1.55
3.13	L.KARIBA	16.6	28.6	118.5	2.8	9.00
8.15	NW/PROV.ZAMBIA	12.6	25.4	448.6	4.0	3.93
11.16	NW/PROV.ZAMBIA	12.4	26.5	398.5	3.6	3.68
11.18	ZAMBIA	12.8	27.2	327.8	2.9	3.24
11.19	NW/PROV.ZAMBIA	13.9	25.5	345.9	3.3	3.72
11.21	NW/PROV.ZAMBIA	13.6	25.4	373.8	3.4	3.59
1986. 1.20	W/PROV.ZAMBIA	14.4	24.9	378.6	4.2	5.26
1.22	W/PROV.ZAMBIA	14.2	24.9	387.0	4.0	4.65
4. 4	E/PROV.ZAMBIA	13.6	26.9	261.7	2.9	4.16
4.10	C/BELT.ZAMBIA	12.3	27.9	366.2	2.6	2.47
4.10	C/BELT.ZAMBIA	12.6	28.1	331.8	2.5	2.62
4.10	C/BELT.ZAMBIA	12.6	28.1	331.8	2.4	2.49
4.12	C/BELT.ZAMBIA	12.6	28.0	332.3	2.4	2.49
4.28	L.MWERU	9.0	29.8	749.8	4.1	2.28
5. 4	KARIBA AREA	17.6	27.6	230.2	3.5	6.44
5. 7	KARIBA AREA	17.9	27.5	265.0	3.4	5.26
6. 1	S/PROV.ZAMBIA	16.9	27.1	185.6	3.3	7.33
6. 3	NE/ZAMBIA	10.9	32.4	690.3	2.8	1.31
6. 8	L.MWERU	9.0	28.9	738.9	2.1	0.86
6.13	L.MWERU	8.8	29.8	771.4	3.0	1.28
7. 3	N/PROV.ZAMBIA	9.2	29.9	731.0	3.1	1.43
7. 6	KARIBA AREA	17.8	27.8	247.0	3.3	5.40
7. 9	L.MWERUWANTIPA	8.8	29.7	769.0	3.2	1.41
7. 9	ISOKA AREA	10.1	32.3	753.0	3.4	1.60
7.15	C/BELT.ZAMBIA	13.5	27.8	236.1	3.1	5.13
7.18	L.KARIBA	16.3	28.5	83.8	4.5	28.99
7.20	L.KARIBA	16.4	28.6	98.3	3.1	12.49
7.30	BANGWEULU AREA	10.9	30.0	554.8	3.0	1.87
7.31	SIAVONGA AREA	16.3	28.3	78.1	3.1	15.37
8. 8	C/PROV.ZAMBIA	15.1	28.7	77.0	3.3	17.20
8. 9	W/PROV.ZAMBIA	14.8	23.5	512.2	3.4	2.50
8.10	L.KARIBA	16.5	28.4	101.8	3.3	13.34
8.11	L.KARIBA	16.4	28.4	91.0	2.8	11.54
8.14	S/PROV.ZAMBIA	17.4	25.5	350.1	2.6	2.59
8.14	C/BELT.ZAMBIA	12.9	28.3	298.7	2.6	3.10
8.16	KARIBA AREA	17.7	27.7	238.2	3.6	6.52
8.16	N/PROV.ZAMBIA	12.4	31.5	501.9	2.8	1.90
8.18	ZAMBIA	13.7	31.6	421.9	3.0	2.56
8.31	KARIBA	16.3	28.3	78.1	2.9	13.91
9.24	L.KARIBA	16.4	28.5	94.1	3.3	14.36

Table 6-1 Earthquake Records around Zambia (1983.1-1988.5)

Year, Month, Day	Region	Seismic center lat.	Seismic center long.	Distance from Epicenter R (km)	Magnitude M	Acceleration A (gal)
1986.10. 1	C/BELT.ZAMBIA	14.6	26.7	195.4	2.8	5.41
10. 5	C/BELT.ZAMBIA	14.4	27.1	177.7	3.3	7.67
10. 8	W/PROV.ZAMBIA	17.5	24.4	456.4	3.3	2.72
10. 9	W/PROV.ZAMBIA	17.4	24.4	451.6	3.2	2.62
10.16	KARIBA	15.8	28.1	24.6	2.8	30.44
11.10	W/PROV.ZAMBIA	15.6	22.4	621.5	3.0	1.64
1987. 1. 5	SERENJE AREA	13.5	30.1	309.4	3.1	3.82
1.25	CHINSALI AREA	10.6	32.1	695.6	3.1	1.51
2. 6	N.ZAMBIA	11.3	31.7	607.6	3.1	1.77
2.22	ZAMBIA	12.2	31.6	525.3	3.1	2.10
4.21	KARIBA	16.5	28.4	101.8	3.4	14.02
5. 4	ZAMBIA	12.5	28.0	343.3	3.0	3.23
6.24	KARIBA AREA	14.0	26.0	295.4	4.0	6.29
6.25	N.ZAMBIA	9.0	30.0	755.2	3.2	1.44
7. 7	N.ZAMBIA	9.0	30.0	755.2	3.9	2.05
7.12	LUAPULA	11.0	29.0	515.8	3.5	2.61
7.13	NW.ZAMBIA	12.0	25.0	527.1	3.7	2.81
8.22	N/PROV.ZAMBIA	11.0	32.0	653.9	3.2	1.71
8.23	C/PROV.ZAMBIA	15.0	27.0	144.8	3.2	9.00
9.18	S/PROV.ZAMBIA	17.0	25.0	375.1	3.7	4.15
9.19	NW.ZAMBIA	11.0	24.0	682.0	3.4	1.80
10.24	E/PROV.ZAMBIA	11.0	33.0	726.9	3.5	1.75
10.29	N/PROV.ZAMBIA	11.0	31.0	591.9	3.3	2.02
12. 4	LUAPULA	12.0	30.0	442.9	3.5	3.11
12.15	N/PROV.ZAMBIA	10.0	32.0	743.5	3.6	1.79
1988 1. 3	NW.ZAMBIA	14.1	25.9	297.8	4.2	6.88
1.11	LUAPULA	10.4	29.3	587.0	3.2	1.94
1.21	CHIRUNDU	16.3	29.0	115.4	2.7	8.79
3.15	C/BELT ZAMBIA	12.3	27.2	380.4	3.3	3.34
3.30	N/PROV.ZAMBIA	10.8	30.3	577.2	4.1	3.09
3.31	E/PROV.ZAMBIA	12.1	31.1	497.7	4.1	3.67
5.26	ZAMBIA	13.7	31.0	367.3	2.5	2.34
5.30	N/PROV.ZAMBIA	9.8	29.9	665.2	2.9	1.44

TABLE 6-2 MONTHLY MAX. WIND SPEED (knots)

	JAN	FEB	MAR	APR	MAY	JUN	JLY	AUG	SEP	OCT	NOV	DEC
1984	16.0	15.0	13.0	20.0	17.0	19.0	19.0	24.0	25.0	18.0	23.0	19.0
1985	13.0	13.0	17.0	14.0	16.0	15.0	20.0	24.0	17.0	18.0	20.0	19.0
1986	17.0	16.0	16.0	16.0	20.0	16.0	14.0	18.0	16.0	9.0	16.0	15.0
1987	15.0	14.0	15.0	15.0	15.0	15.0	14.0	19.0	18.0	20.0	16.0	16.0
1988	11.0	15.0	10.0	14.0	14.0	16.0	15.0	17.0	15.0	17.0	19.0	17.0
1989	12.0	12.0	11.0	–	–	–	–	–	–	–	–	–

$$y_T = -l_n\{-l_n(1-1/T_R)\} \qquad \cdots\cdots\cdots ⑥$$

(b) Design Wind Speed :

228 Table 6-2 shows monthly max. wind speed observed at Kafue during 1984 through 1989. Expected wind speed value was obtained from these data in accordance with the analysis previously mentioned. Although it is common to utilize annual max. Wind velocity, monthly max. Wind speed was used because of shortage of data. In this connection, return period used in the analysis was 1,200 months (= 100 years × 12-months).
Correlation of wind speed with return period is as shown in Fig. 6-3 and expected wind speed for return period of 100-years is assumed 33.1 knots (17.0 m/sec).
Considering the data used are relatively short, it is recommended design wind velocity shall be

$$V_{100} = 39 \text{ knots} (20\text{m/sec})$$

(5) <u>Foating Island</u>

229 Almost without exception, earlier reports on the Bridge have made reference to the problem of blocking of the spans by floating islands of vegetation. This has been an annual occurence spread over 4 to 6 months, commencing with the first of the local summer floods in November and ending with the main Kafue flood peak.

230

Despite the long history of the floating island problem at the Kafue Rail Bridge, quantitative information on the number, size and frequency of occurrence of the island is sparsely recorded.

231 Accordingly, theory of stream pressure shall be applied to floating islands as though they would be pushed continuously by river water. According to Standard Specifications for Highway Bridge, the stream pressure is thought as follows;

$$P_{HC} = KV^2A$$

where

K : resistance coefficient
$0.07 \times 2 = 0.14$
V : water velocity
ordinary time : 1.0 m/sec
extraordinary time : 2.5 m/sec
A : Normal projected area of floating island
$43.0\ m \times 0.6\ m = 25.8\ m^2$

232 Since a width of floating island to be shared by a single pier is 43.0 m (=span length), horizontal force to work against the pier is

$$\text{(ordinary)}\ P_{HC} = 0.14 \times 1.0^2 \times 25.8$$

$$= 3.612\ \text{t/pier}$$

$$\text{(extraordinary)}\ P_{HC} = 0.14 \times 2.5^2 \times 25.8$$

$$= 22.575\ \text{t/pier}$$

CHAPTER 7

ALTERNATIVE PLANS

CHAPTER 7 ALTERNATIVE PLANS

7-A. GENERAL

233 As previously mentioned, the Kafue road Bridge is located along the international trunk roak linking Lusaka, the capital city, and the southern region.

234 The Roads Department has planned a rehabilitation project of the road between Lusaka and Kafue, and the project will be implemented starting in 1991. Furhter, the T2 road (Kafue-Chirundu) had been rehabilitated up to the south abutment of the Bridge.

235 Therefore, it is desirable that the new alignment regarding the Bridge reconstruction be coincide with these on-going or completed project.

7-B. SELECTION OF HORIZONTAL ALIGNMENT

(1) Scope of Horizontal Alignment

236 The key factors/control-points on selection of horizontal alignment are summarized below:

(a) The telephone lines are located parallel to, and 15 m away from the Bridge on the downstream side.

(b) The power lines are likewise located parallel to, and 37 m away from the Bridge on the downstream side.

(c) The Nansenga River is located approx. 200 m upstream of the Bridge.

(d) The customs inspection area, toll gate, exists at the south side of the Bridge.

(e) The weighbridge is located at the south side of the Bridge and approx. 750 m toward Lusaka from the intersection of Route T1 (Kafue-Livingstone) and Route T2.

(f) Along both sides of the existing road, and beginning from the customs area to the intersection of Route T1 and Route T2, there exists private property.

(g) The basic concepts on geometrical design of the bridge and approach road are as follows,

i) In as much as long straight tangents and large radius curves are used on the adjacent roads, employing a steep curvature to shorten approaches shall not be considered,

ii) Since a reverse curve has been applied to the south side of the Bridge, it is not wise to introduce another reverse curve on the north side,

iii) To insert a short tangent between curves in the same direction shall be avoided, and

iv) It is advantageous to have the Bridge in a straight alignment.

(h) The sufficient distance between existing bridge and the Bridge on the new alignment shall be ensured not to adversely affect on the existing structures due to constructing the new bridge structure, as well as to provide the required space for construction works.

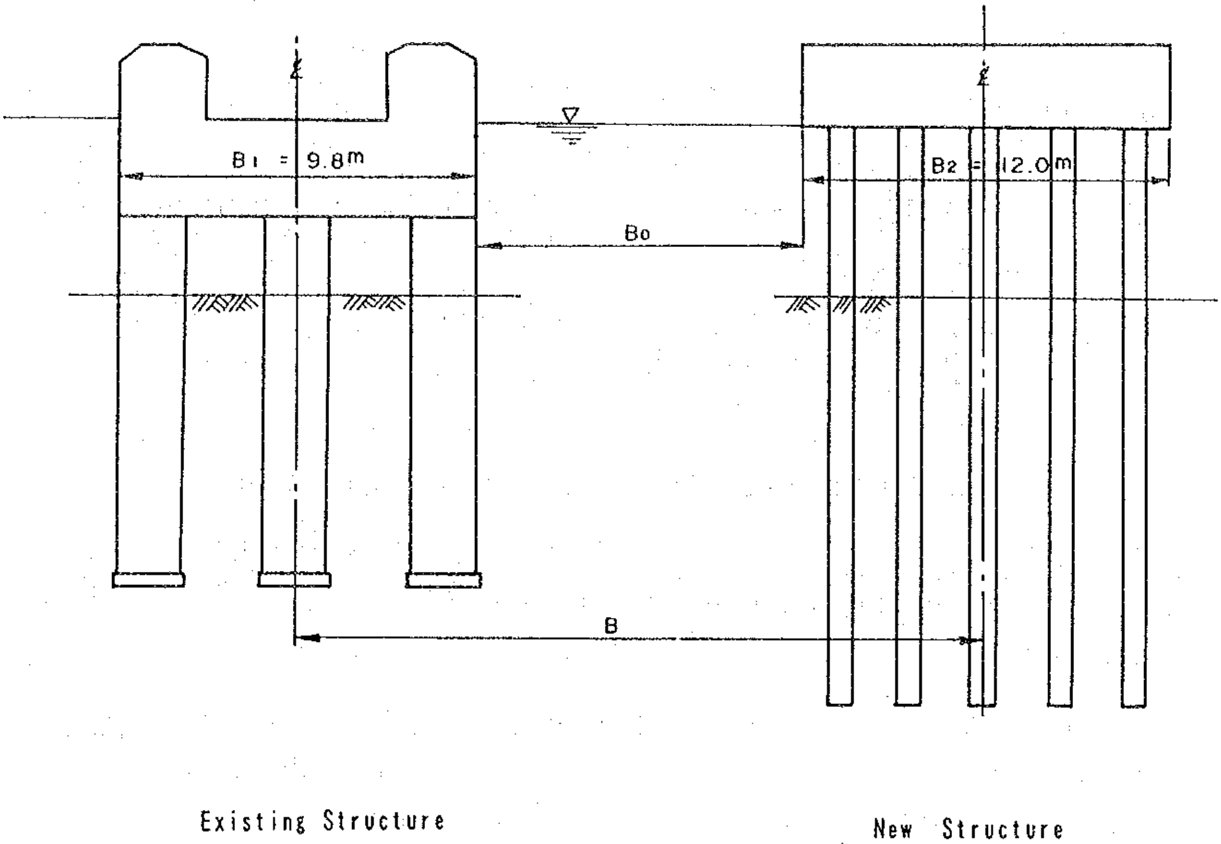
The distance shall not be less than 15 m, center to center of adjacent structure, provided driving pile foundation type is applied for new foundations. The estimates are based on the relationship:

$B0 > B2$ when $3 < N < 5$

$B0 > \frac{1}{2}B2 = 6.0m$ when $N > 10$

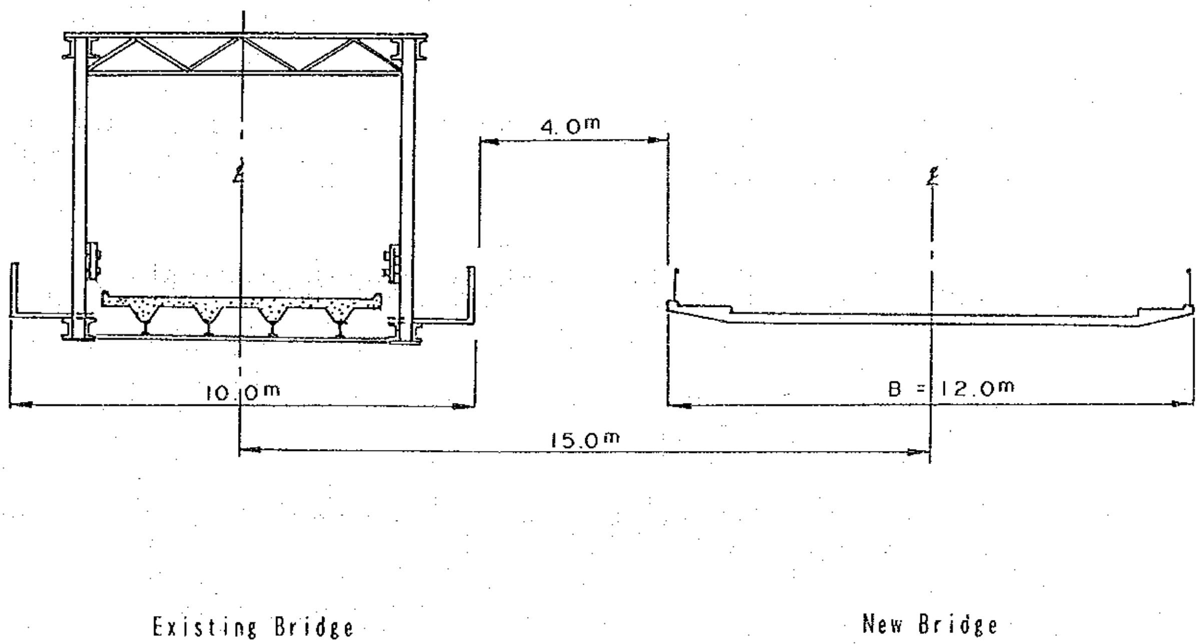
$B^B = 9.8/2 + 12/2 = 16.9m$

where B1 = 9.8 m and b2 = 12 m are assumed.



B1 = 9.8m
B2 = 12.0m
Bo
B
Existing Structure
New Structure

Fig. 7-1



4.0m
10.0m
B = 12.0m
15.0m
Existing Bridge
New Bridge

Fig. 7-2

Definition of B, B0, B1 and B2 are shown in Fig. 7-1.

The smaller B could be applicable in a case of the new bridge construction, since the existing bridge would be demolished upon completion of the new bridge.

The required minimum net opening for construction would be 4.0 m. Therefore, the distance between new and old structure turns out to be 15 m.

(2) Allowable Scope of Horizontal Alignment

237 Regarding the location of horizontal alignment, allowable scope of alignment shall be as indicated in Fig. 7-3.

(a) Line 1
Line 1 is on the extension of the T2's straight alignment.

(b) Line 2
Line 2 is selected so as to avoid any curves on the Bridge while maintaining required distance.

(c) Line 3
Line 3 is located on the upstream side based on same concern of Line 2.

(d) Line 4
Line 4 is to be attached to Line 1 at T.P. Line 4 is considered so as not to eliminate a straight alingment of the existing road.

238 The study of horizontal alignment is determined to be within a space between Lines 2 and 3.

(3) Alternative Route

239 Three routes will be considered as the alternative routes based on above-mentioned concept.

(a) A-Route

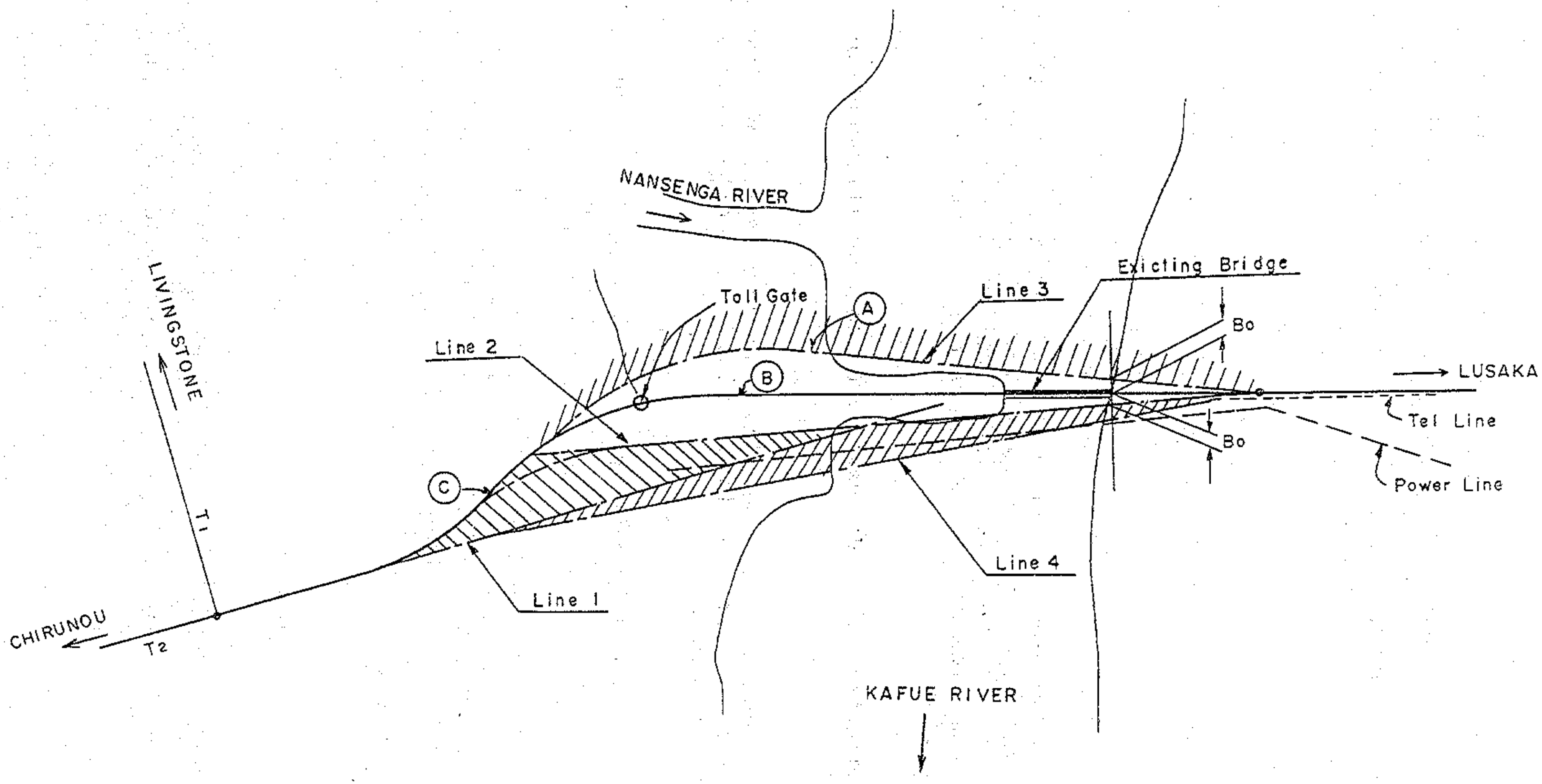


Fig. 7-3 Allowable Scope of Horizontal Alignment

The A-route is provided to clear restriction of adjacent construction requirement for the abutments (which is to provide at least 15 m between ends of old and new abutments) and be on upstream. The route passes west of the custom area, and is attached to the existing route before the weigh bridge.

(b) B-Route

The B-route utilizes the alignment of the existing road.

(c) C-Route

The C-route clears restriction of adjacent construction requirement for the aubtments and allows at least 16 m clearance for the power line. The route shall be on the downstream side and connects to the existing road at the weigh bridge.

240 The characteristics of each alternative alignment are as follows.

(a) The A-route will be close to the mouth of Nansenga River and enters into private property around the customs area.

(b) The B-route needs no geometric improvement and provides satisfactory width of approach roads.

(c) The C-route shall be provided as close to the existing bridge as possible to reduce construction costs and to avoid relocation of the power line. However, the telephone line should be relocated. As in the case of the A-route, this route encroaches upon private property.

(4) <u>Recommendation</u>

241 In any of the above alternatives, pier locations are influenced by the existing bridge unless existing piers are removed. It is hoped that the route to be selected shall not encroach on the private proprty.

7-C. SELECTION OF VERTICAL ALIGNMENT

242 The vertical alignment should be designed based on following conditions:

(a) Soffit level of the new bridge shall be at the same elevation as that of the existing bridge.

(b) To minimize construction cost, crest shall be incorporated into the new alignment of the Bridge.

(c) Approaches shall be minimal and have the lowest banking.

(d) The scheme to utilizing the existing road shall have an alignment which can be topped with overlay.

(e) Geometrically, elements for alignment more than standard values shall be used.

(f) For the Bridge span, it is planned to elevate the vertical alignment by 1.5 m from existing elevation.

(g) In main bridge span, the crest of vertical curve should be set in the center of the bridge so as to make symmetric longitudinal profile for the whole bridge spans.

7-D. ALTERNATIVE SCHEMES

(1) Fundamental Suggestion on New Bridge

243 As previously mentioned, the Kafue Road Bridge was transferred from the Thames River and constructed in 1949. The original bridge was built as five (5) single spans. Three (3) of these were transferred to Kafue River. Thus, the bridge length was inevitably restrictive.

244 In 1969, it was necessary to raise the Kafue Road Bridge by 6'-5" (approximately 1.96 m) due to a rise of water level caused by impounding Kafue gorge Dam which was constructed downstream of the bridge. Since the bridge length remained unchanged at that time, the embankment projected into the stream channel at right angle.

245 On the basis of these particulars, reconstruction of the Kafue Road Bridge can not be confined to the same bridge size.

246 The improvement of the traffic function and the wide view of the landscope which the topographic offers through the new bridge will heighten the aesthetics of this large scale bridge on main trunk road of South African countries making it a monumental structures.

(2) River Condition and Its Effect on Construction

247 Water Level at the project site is influenced by discharge of both Kafue-Gorge and Iteshi-Teshi Dams. Considering requirements of hydroelectric power and water for irrigation and living, the discharge of both dams is artificially controlled to enable 650 m^3/sec of normal water flow and to adjust water level at Kasaka Gauge Station to below 976.6 m.

248 Further, as gradient of Kafue River is extremely small, water level at the project site is deeply influenced by that of the dams. As a matter of fact, water depth at the project site is always deeper than 8 m, while annual variation of water level is assumed to approximately 1 m.

249 Therefore, construction of the bridge will be free from seasonal conditions, though it is obvious that construction during dry season is advantageous.

(3) Current and Scouring

250 The largest histrical water flow at the project site is recorded as to be 1,200 m^2/sec. The maximum current at the present bridge location including its adjacent area is assumed to be about 1.5 m/sec. In fact, the current can be close to 0.4-0.5 m/sec, if the water flow at the

project site is presumed to be coincide with discharge of Kafue-gorge Dam.

251 Under this current, there is little possibility to cause scouring, instead, sedimentation of earth and sand is more likely. Oppositely, the current velocity more than 2.5 m/sec, as discussed previously in Paragraph 122, is expected, when the bridge spans are to be designed in conformity with planned water flow for 100-years return period of Kafue Gorge Dam. Under this velocity, it is necessary to consider the river bed scouring or to widen the river width.

(4) Clearance of Span

(a) Minimum Span Length

(a)-1 Minimum Span Length between Center to Center of Piers

252 The span length is decided 43.0 m in the following circumstances mentioned below.

253 The following formula shall be applied to clear space (L_{min}) between piers according to Japanese government regulations.

$$L_{min} = 20.0 + 0.005 \times Q \text{ (m)}$$

where Q: flow quantity per second (m^3/sec)

254 Since, in Kafue River, Q of 100-years return period is estimated to be 2,120 m^3/sec, L_{min} is calculated 30.6 m.

Accordingly, not less than 35 m, that is a rounded number of 30.6 m, of clear space is desirable.

255 Floating islands would temporarily stick on existing piers. These floating islands, however, would be swept down by river flow and would not cause any damage of piers, viewing the fact that the soundness of existing piers have been kept up to this time.

256 There is no positive reasons to stretch the clearance between piers for navigation of vessels and so forth.

(a)-2 Span Length between Abutment and Pier

257 The span length is determined to be 38.0 m by the following reasons.

258 Construction cost and workability at the site in the case of the 38.0 m span length is most economic and suitable, as reviewed by the paragraph No. 263.

259 To ensure a span length not less than 35.0 m is required, as delineated in the paragraph No. 254.

(b) Review on Side Span Length

260 In deciding the side span length, economical aspect and easy execution of site work should be taken into consideration as well as the affect on river nature.

261 There would be three (3) ideas considered as tabulated below. Among these three variations, we adopt the 2nd one as final proposal from the following assessment table.

262 The evaluation of each idea is done only for A1 abutment, as the depth of bearing layer does not vary so much around A2 abutment.

263 Remarkable features of side span length for each idea is

1st idea

Side span length is 33.6 m which is the critical limitation that makes site-work execution feasible on land.
In this case allowance for floating island passing by does not seem sufficient.

2nd idea

Side span length is 38.0 m, which corresponds to minimum span length between abutment and pier.

3rd idea

Side span length is 43.0 m which produces sufficient space for pass of floating islands. This gives four (4) continuous equal span bridge. The more longer span is getting expensive.

	Side span length	Bridge span ratio	Facilitation of site work	Relative cost (exclusive of piers)	Affect on river nature	Total evaluation
1sr Idea	33.6 m	33.2+43+43+33.2 (1:1.3:1.3:1) ○	○	100 ○	△	△
2nd Idea	38.0 m	37.6+43+43+37.6 (1:1.14:1.14:1) ○	○	101 ○	○	○
3rd Idea	43.0 m	42.6+43+43+42.6 (1 : 1 : 1 : 1) △	○	106 ×	◎	△

(5) Types of Structure

(a) Deck-or Through-Bridge :

264 Although, at the existing bridge, overhead clearances between top road surfaces and bottom of sway bracings are properly furnished as stipulated by Highway Design Standards, most of top lateral are damaged and/or destroyed.

Notwithstanding restrictions by Roads Department that consignments over 3.8 m high are required to obtain special permissions, facts are as described above and causes of accidents are unidentified.

265 In Zambia, roads that require limit of overhead clearances are fairly few and Kafue Road Bridge is one of the few on main trunk roads.

Due to these circumstances, a deck bridge is highly recommended as long as it is feasible. In adopting a larger span, a through bridge is of course necessitated. However, a provision of special sencers at both portals of the bridge are advisable when choosing a through bridge.

(b) A Grand View :

266 It is desirable not to provide any structure hampering wide vision, since the site offers a wide field of vision.

(c) Continuous Bridge or simple Bridge :

267 A continuous bridge is recommendable from s standpoint of improving structural safety and smooth traffics.

(6) Construction Equipment/Materials

268 Major construction equipment and a greater part of construction materials have to be imported from other

countries. As Zambia is a land-locked country, a distance of inland transportation amounts to nearly 2,000 km.

Consequently, transportation of equipment/materials is one of the major factors to determine economics of bridge construction.

(7) Approaches

269 Present approaches are 170 m embankment placed in swampland. The embankment is fairly high and reaches to approximately 13 m. As is previously mentioned, this uncommon structure has appeared as result of the rise of approximately 2 m in the water level caused by the Kafue-Gorge Dam construction.

270 Counter anchorages with tension bars have been provided to protect existing abutments from destruction due to the rise. From an engineering standpoint, the length of the embankment should have been reestimated at the time when water level was rised.

271 In view of these circumstances, the elevation of road surfaces on the new bridge will necessarily be higher than those on the existing bridge, and an appropriate length of new embankment shall be determined upon reviewing the embankment height as well as assessment that either viaduct or embankment should be considered.

(8) Essential Factors of Bridge Alternative

272 Summarizing all discussions as developed above, a bridge length and type of bridge shall be proposed subject to folloiwng essential factors :

1) Total Construction Cost

2) Construction Simplicity

3) Duration of Construction

4) Correlation with Existing Piers

(9) Alternative Schemes

273 As a result of the comprehensive review on all options discussed in the foregoing sections, thirteen (13) alternative schemes are have been identified. Fig. 7-4

274 Schemes 1 through 12 shows the types of main span crossing over the river, while Scheme 13 covers the approaches on the right bank of the river. These schemes are summarized in Table 7-1.

Among these, Schemes 2 and (steel) correspond to brand new alignment, while other schemes utilize the present and new alignments.

275 For bridge alternatives including approaches combination of schemes 1 through 12 with scheme 13 have been proposed as for as the new alignment is concerned.

On the other hand, for the existing alignment, the proposed scheme includes the main bridge and takes advantage of the existing embankment.

In cases of other schemes on the existing alignment, it is imperative that construction cost include removal of the existing super-and sub-structure.

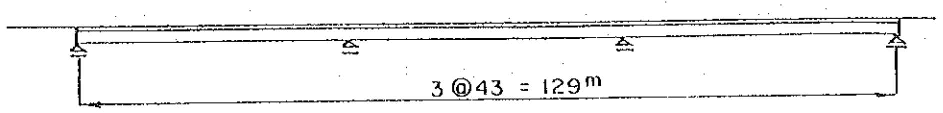
(a) Schemes for the New Alignment :

276 In schemes corresponding to new alignment, the existing substructure is assumed not to be removd over the long term, and locations of new piers are nearly the same as those of existing piers since the new and old alignment are adjacent each other. Schemes 1 and 12 will also be adopted to the new alignment.

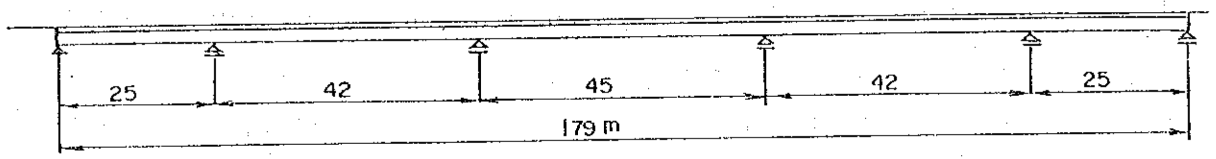
Scheme 1

3-spans continuous steel plate girder.
Approx. same span configuration as the existing bridge.
Since abutments are to be constructed in deep water, the need for dewatering would add up the cost of construction.

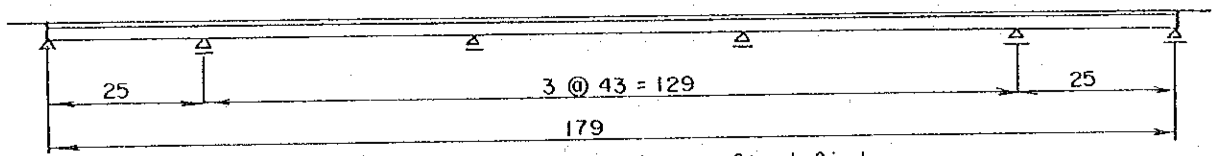
Scheme 2



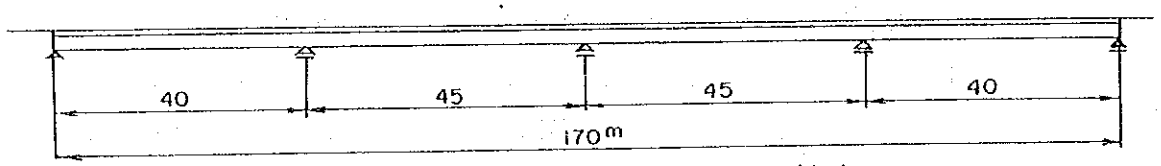
Scheme 1 3-Spans Continuous Steel Girder



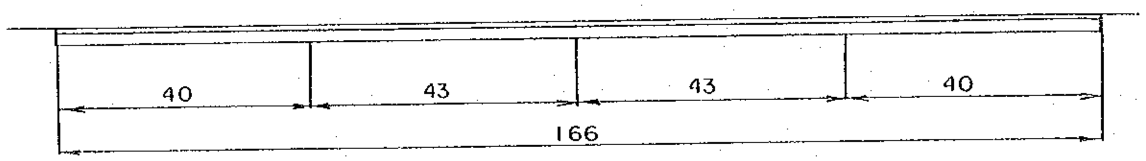
Scheme 2 5-Spans Continuous Steel Girder



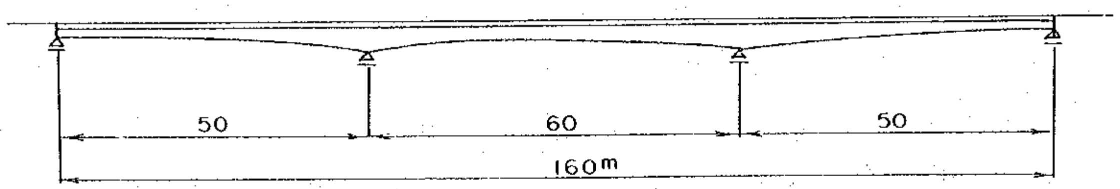
Scheme 3 5-Spans Continuous Steel Girder



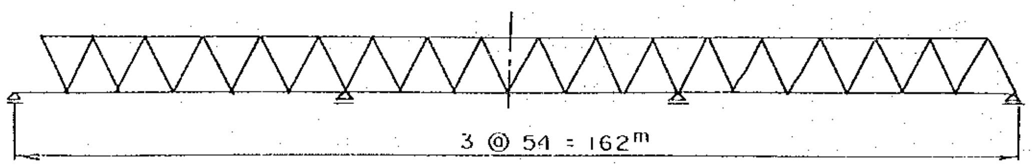
Scheme 4 4-Spans Continuous Steel Girder



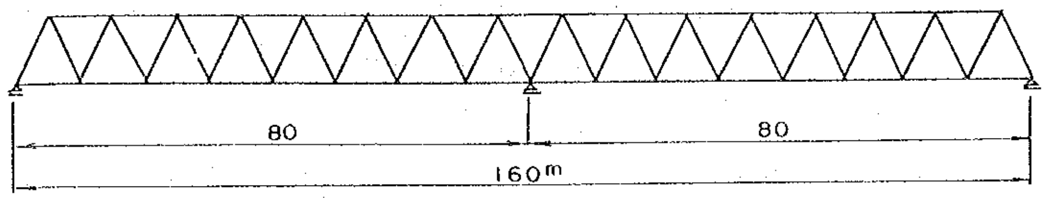
Scheme 5 4-Spans Continuous Steel Girder



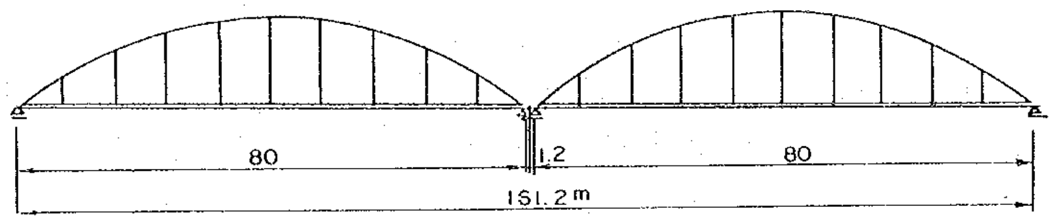
Scheme 6 3-Spans Continuous Steel Girder



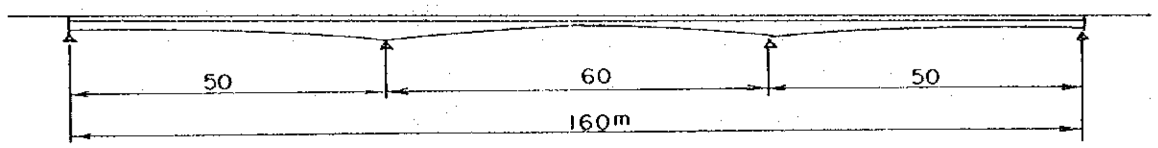
Scheme 7 3-Spans Continuous Truss



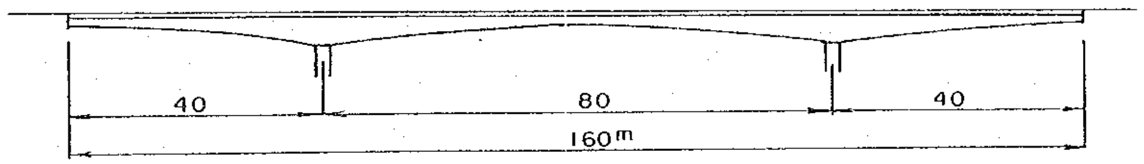
Scheme 8 2-Spans Continuous Truss



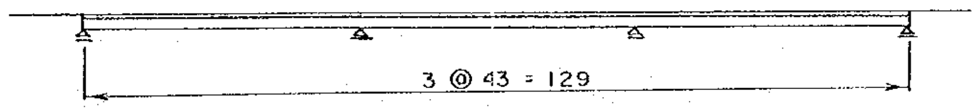
Scheme 9 2-Spans Steel Arch



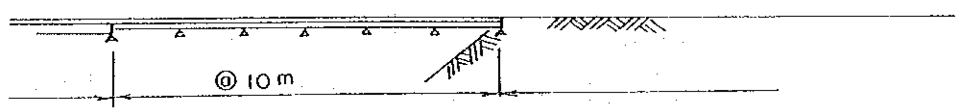
Scheme 10 3-Spans P.C. Girder



Scheme 11 3-Spans P.C. Girder



Scheme 12 3-Spans P.C. Girder



Scheme 13 Viaduct and Aproach Embankment

Fig. 7-4 ALTERNATIVE SCHEMES

Table 7-1 SUMMARY FOR SCREENING OF ALTERNATIVE SCHEMES

Good ○ Fair □ Poor △

Scheme	Type	Length of Bridge	Max. Span Length	Alignment Corres-ponded	Height of Abutment (m)	Number of Pier in Water	Eco-nomy	Dura-bility	Safety on Const.	Mainte-nance	Others	Evalu-ation
1	3-Spans Steel Girder	129m	43m	new/exist.	13m	2	○	○	△	Enbankment to be settled for new alignemnt		○
2	5-Spans Steel Girder	179m	45m	new	6m	3	□	○	○	To be care settlement of embankment	Restriction of Pier Location	
3	5-Spans Steel Girder	179m	43m	new	6m	3	○	○	○	do	do	○
4	4-Spans Steel Girder	170m	45m	(new)/exist.	7m	3	○	○	○			
5	4-Spans Steel Girder	166m	43m	(new)/exist.	8m	3	○	○	○			○
6	3-Spans Steel Girder	160m	60m	(new)/exist.	8m	2	△	○	○			○
7	3-Spans Steel Truss	162m	54m	(new)/exist.	8m	2	□	□	□			
8	2-Spans Steel Truss	160m	80m	(new)/exist.	8m	2	△	△	△			
9	2-Spans Steel Arch	161.2m	80m	(new)/exist.	8m	1	△	△	△			
10	3-Spans P.C. Girder	160m	60m	(new)/exist.	8m	2	△	△	△	Free		
11	3-Spans P.C. Girder	160m	80m	(new)/exist.	8m	2	△	△	△	Free		
12	3-Spans P.C. Girder	129m	43m	new/exist.	13m	2	○	○	△	Free		

Note : (new) designates that Schemes 4 through 12 can also be applied to new alignment, provided existing piers are demolished.

5-spans continuos steel plate girder. 3-central spans have same spans as the existing bridge. Abutments are to be constructed at land.

Scheme 3

Variation of Scheme 2.

Sheme 12

3-spans prestressed concrete girder. Same concept as Scheme 1.

(b) Schemes for the Existing Alignment :

277 Pier locations in these schemes are not influenced by the existing pier locations. These schemes are proposed with the understanding that the existing substructures shall be demolished.

278 Further, Schemes 4 through 9 involve steel superstructure and Schemes 10 through 12 involve prestressed concrete girder bridge by means of post-tension method, both with main span lengths ranging from 43 to 80 m. All these schemes are applicable also to the new alignment, on condition that the existing piers are demolished, although these piers, generally, would be remained even after dismantling the superstructure.

Scheme 4

4-spans continuous steel plate girder with a total bridge length of 170 m. Side span lengths of 40 m, especially for economical reason, are to be considered.

Scheme 5

Variation of scheme 4, center span is 43 m.

Scheme 6

3-spans continuous steel plate girder. Piers constructed in deep water shall be reduced.

Scheme 7

3-spans continuous trusses with each span lengths of 54 m. This scheme has the following features:

a. The elevation of road surfaces can be maintained relatively low, since the scheme offers a through bridge. This enables less longitudinal slope and lower elevations of abutments.

b. Compact sections of member will give advantages to transportation as well as erection, since less quantity of cargo and erection lifts are expected.

c. Because of a through-bridge, permissible let-style shall be strictly controlled.

Scheme 8

2-spans continuous trusses with span lengths of 80 m. This scheme will be similar to the Kafue Rail Bridge which is being replaced with 80 m spans. As long as the same span lengths are kept, the flow of floating islands is not disturbed.

Scheme 9

Arch bridge (Langer Girder) with a span length of 80 m. This arch bridge will be an imposing landmark in Kafue.

Scheme 10

3-spans continuous prestressed concrete box girder with a center span of 60 m. Staging erection is assumed.

Scheme 11

3-spans continuous prestressed concrete box girder with a center span of 80 m and the same bridge length as Scheme 10. Cantilever erection is assumed.

Scheme 13

Viaduct having 10 m span lengths. This viaduct will be provided only on the right bank of the river and only for cases using new alignment. The water depth of the right bank is approx. 2-2.5 m.

7-E. SCREENING OF ALTERNATIVE SCHEMES

(1) Steel Versus Concrete Structure

279 It is highly recommended the new bridge construction shall be implemented coincidently with the Kafue-Lusaka Road Rehabilitation Project financed by USAID (Kafue-Lusaka Road Project). Construction schedule of Kafue-Lusaka Road is expected to commence in March, 1991 and to be completed in November, 1993.

280 Reasons to synchronize the construction schedule with the above project are :

(a) The Kafue Road Bridge shall form a part of Kafue-Lusaka Road Rehabilitation Program and is not ideal to be scheduled to open to traffic after the road is completed.

(b) There is no asphalt plant in operation around Kafue-Lusaka area, and it is absolutely impossible to be supplied hotmixed bitumen asphalt concrete all the time. Meantime, quantity of asphalt concrete to be required for this project is merely 700 m^3 and to provide an exclusive asphalt plant at its own expense is uneconomical and practically impossible. As the Road Rehabilitation would set up the asphalt plant, applying the plant to the bridge construction is desirable provided construction schedule could be synchronized.

281 To ensure pavement strength and warrant its quality, construction equipment such as finishers, rollers, and asphalt plant shall be prepared. From a standpoint of economics, asphalt paving shall be carried out by using the plant mobilized for the Kafue-Lusaka road Project or even entrusting the entire work to that project.

282 Construction period of the concrete schemes 10 through 12 is anticipated to take 4-5 months longer than equivalent steel schemes. Consequently, more construction equipment will be required in order to complete concrete structures

within a desired period. This will necessarily increase the construction costs.

Due to these circumstances, steel structure is expected to reduce construction period and costs.

283 Generally, maintenance cost for concrete structures is less than corresponding steel structures. The difference is almost equivalent to cost to be required for protection system of painting that shall be inevitable in a case of steel structures.

284 Because of Zambia being a land locked country, there is no concern about salt contamination. Further, there is no acid smog nor water pollution which accelerate progress of rust. Thus, painting in every ten (10) years seems sufficient to protect steel.
Maintenance costs throughout the lifetime of the bridge can be estimated by converting future costs to present day value in compliance with specified rates.

285 Assuming painting area to be 5,300 m^2 and costs of coating will paint to be K 90/m^2 in Scheme 4, one time placing would amont to approx. K 477,000. If the bridge is durable for 50-years, this amount is converted to K 684,000 of present day value based upon 10% compound interest. If project evaluation is assumed 20 years, this corresponds to K 298,000.

Thus, in comparing the capital cost of steel and concrete schemes, at most K 0.3 million of capitalized cost of maintaining the paint work is added.

286 Prestressed concrete structure would require highly skilled labour and more attention to quality control and supervision throughout site works as compared to steel structures. On the other hand, steel structure are prefabricated at the shop and can be erected by a few skilled technician and local labour under supervision of expatriate engineers.

287 Furthermore, a long span prestressed concrete bridge would require concrete with high quality and high strength of more than 350 kg/cm^2. To fulfill these requirements, good quality sand and gravel and good cement quality would have to be ensured. It is nearly impractical as well as

expensive to obtain good quality sand around Kafue district.
It also requires staging with sufficient strengths, a large quantity of water-proof plywood or metal forms to ensure quality of concrete structure and satisfy esthetic requirements.

288 Concrete structure would have more dead load than equivalent steel spans. Therefore, substantial substructure would be necessitated.

289 On the other hand, the height between girder's soffit level and road surfaces in steel structure would be 0.2 m higher than that in concrete structure. This will result in higher approach embankment and raise cost of steel structure. However, this additional cost is to be considered when comparing the costs of bridge structure.

290 Should the superstructure be re-used upon dismantling and relocation to another place, further economic advantage would be expected.

291 Viewing in economic terms, concrete structure seems advantageous to Zambian Government, because of less imported materials. But, costs of local materials at present shall be confirmed, considering prestressing falsework, launching girders and current cement productivity.

292 In this connection, concrete structure shall be eliminated considering financial and economic terms as well as site conditions, although in the future, advantages of concrete structures may be established.

(2) Selection of Bridge Types

Several alternatives among prescribed schemes will be chosen to determine final bridge types including span lengths and number of spans.

The selection shall be in accordance with.
a) Economy,
b) Workability (including construction period),
c) Safety

d) Correlation with other projects, and
e) Care of maintenance.

293 Conditions to select alternatives for an objective of design comparison are as summarized below:

(a) River width at least not less than the existing bridge length shall be maintained.

(b) Minimum span length in the main water channel shall be the present 43 m.

(c) In the case of the new alignment, new piers shall be on the stream lines of the existing piers unless the existing piers are removed for a while. However, existing piers have to be demolished when the existing superstructure is dismantled, since they would be expected to adversely influence the new bridge if left in place. In case the existing bridge is removed, pier locations therefore, could be freely taken.

(d) When the new alignment is planned on the existing route, present piers and abutments will be demolished.

(e) A cutting device to protect from floating islands shall be provided at the piers.
A load to consider the effect of floating islands shall be included in design conditions.

(f) In a bid to compare construction costs in equivalent term, the cost of approaches should be included as may be directed by the respective schemes. Consequently, the length of approach spans vary in accordance with main bridge length and scheme.

(g) The rehabilitation and use of existing substructure has been ruled out.

(3) Proposed Alternatives

294 Considering various conditions as stated in Sections (1) & (2), main bridge alternatives to be selected for the further study shall be as follows:

Alternative	Spans	Bridge types	Alignment
A	40.0+43.0+43.0+40.0 =166.0m	4-spans continuous steel plate Girder	existing
B/C	3×43.0=129.	3-spans continuous steel plate Girder	existing /new
D	25.0+3×43.0+25.0 =179.0m	5-spans continuous steel plate Girder	new
E	50.0+60.0+50.0 =160.0m	3-spans continuous steel plate Girder	new

(4) Structural Type

(a) Superstructure

295 Reasons to have selected continuous girders are as follows:

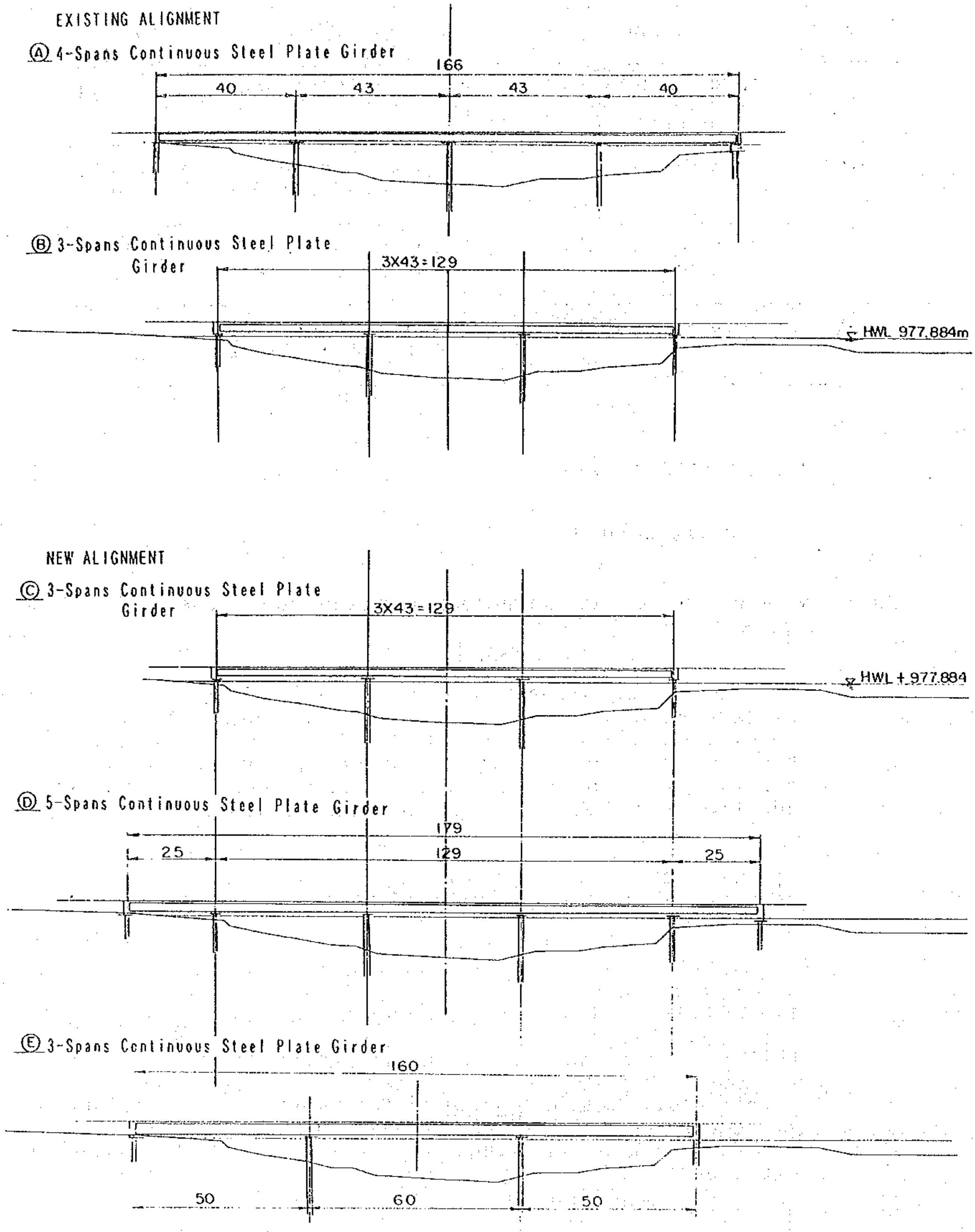
1) Continuous girders are safer than simply supported girders under ultimate fracture conditions. When a soil conditions are good such as in this bridge, continuous type would be more suitable since no loss of structural safety represented by unequal settlement is expected. Additionally this scheme is more economical.

2) The number of expansion joints will be less and this results in smoother driving surface.

3) The water depth of the bridge site is approx. 8 m and a variation in the water depth is only about ±1 m. Since 1.0-2.5 m of sediment layer was discovered beneath the river bed and the bearing stratum was subsequent to that as determined by the soil investigation, rigid body foundation would be more expensive and take longer construction period. It is advantageous from standpoint of both economics and construction to make the superstructure continuous and the piers flexible structure.

ALTERNATIVES

FIG. 7-5



296 Selected reasons of individual types are as summarized below:

Alternative B/C:

297 A shorter bridge length is economical in general provided a minimum required river width is satisfied. Alternative B/C has the same spans as the existing bridge and keeps present river conditions as they are.

Alternative A:

298 In alternative B/C, cofferdam will be required since water depth when constructing abutments is deep. Thus this alternative would be expensive and require longer construction period. The alternative improves the above disadvantages by backing the north abutment toward Lusaka. Though cost of the superstructure would be pushed up, the total cost would be expected to be reduced. In the alternative A, river width will necessarily broaden and river condition shall improve. This alternative can be applied to both existing and new alignment. Removal of the existing bridge is, however, absolutely necessary for the new alingment.

Alternative D;

299 Assuming the existing bridge would not be removed for a substantial period, alternative C would attempt to reduce construction cost and obtain wider river while backing abutments toward Lusaka. The length of end span is determined to be approx. 25 m considering no possibility of uplift at girder ends. Although several variations could provide shoes to handle uplift, by placing a counterweight or to make end spans simple support, 5-span continuous girder would sufficiently represent the concept of this alternative in the phase of alternative design. This scheme is only applicable to the new alignment.

Alternative E;

300 Similar to alternative A, Alternative E has similar bridge length to alternative A. By expanding span lengths, it is expected to reduce quantity of foundations in water.

(b) Substructure :

(b)-1 Pier

301 We propose the pile-bent type as pier foundation with cutting device of floating islands, which has great advantages compared to caisson type foundation in terms of cost and facilitation of site-work is proposed.

302 The pile should be steel pipe filled with concrete inside pile throughout its entire length to give much resisting capability against bending moment.

303 The diameter and wall thickness of steel pipe is to be ϕ1000 mm × 22 mm.

304 The above characteristics of pile are determined by examining the cost, facilitation at site work and appearance as well as strength and durability of pile itself. (Refer to next page)

Dimension of pile & number	Relative cost	Facilitation of site work	Appearance	Rigidity	Total evaluation
① ϕ800×22 2×5 =10 piles Weight:74.7t	130 ×	Requires relatively long term ×	○	△	×
② ϕ1000×22 1×5 piles Weight:47.0t	100 ○	○	△	△	○
③ ϕ1200×19 1×5 piles Weight:48.9t	120 △	△	×	○	△

(b)-2 Abutment

(i) A1 abutment (Lusaka side)

305 The bearing layer locates approximately 11 m below the land surface. The height of footing foundation, then, becomes 13.5 m.

306 As the result of discussion on the following two realistic types of footing from the view point of the cost, easy construction work at site and its rigidity in longitudinal direction of the bridge, we choose 1st type as A1 abutment.

1st idea --- inverted T type footing foundation
(simple structure and easy site work)

2nd idea --- inverted T type pile foundation
(less amount of temporary site work such as excavation and preventive measurement against land slide/collaps, however requires much expense for pile driven cost)

Comparison Table (A1 Abutment)

	Cost	Facilitation of site work	Rigidity in longi. direction of bridge	Total evaluation
1st idea Inverted T type pile foundation	100 ○	○	○	○
2nd idea Inverted T type footing foundation	155 △	Pile driven is required △	○	△

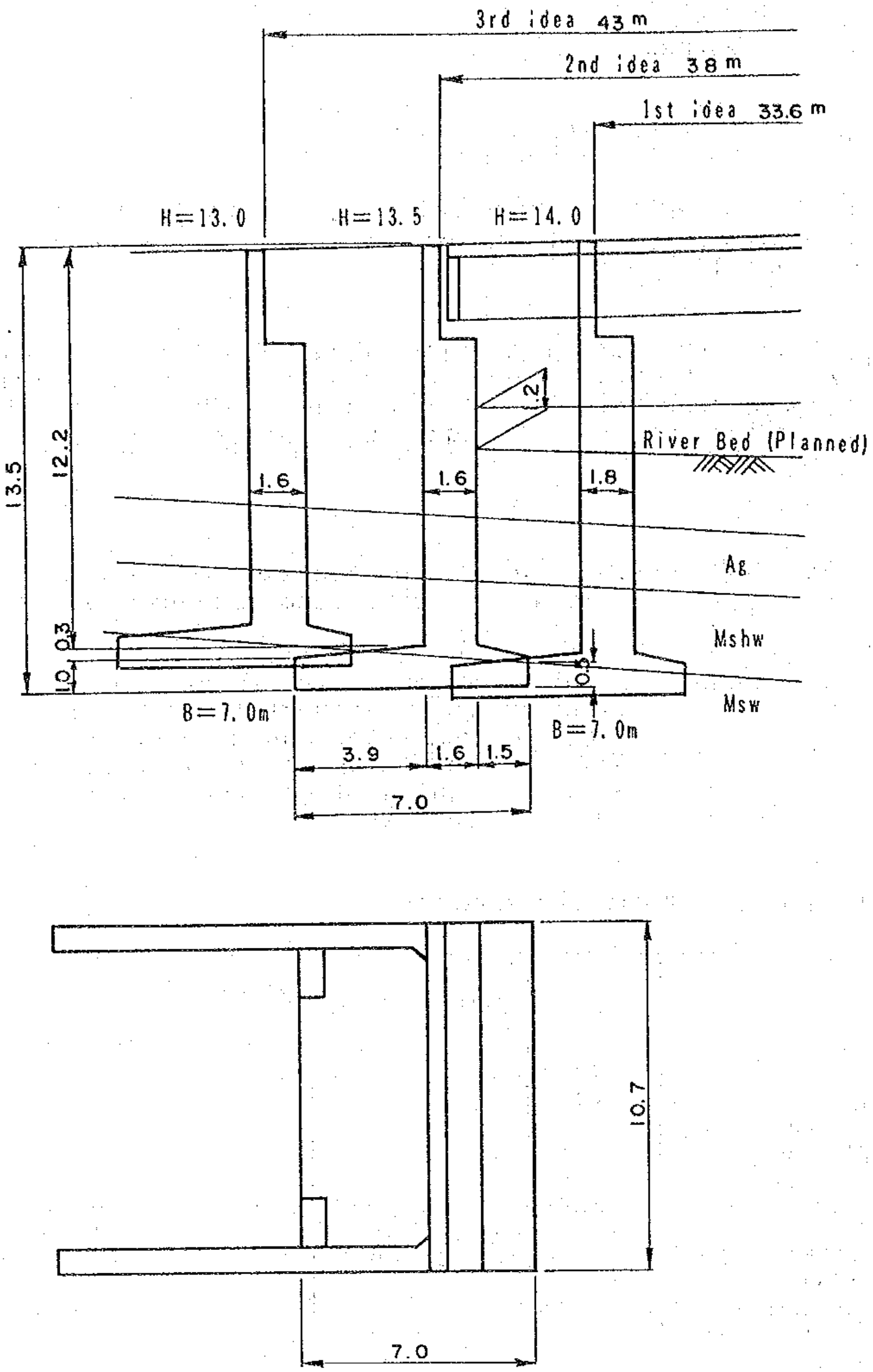


Fig. 7-6 Abutment 1 - Proposed Ideas

A₁ Abutment

1st Idea

Cantilever Type

Footing Foundation

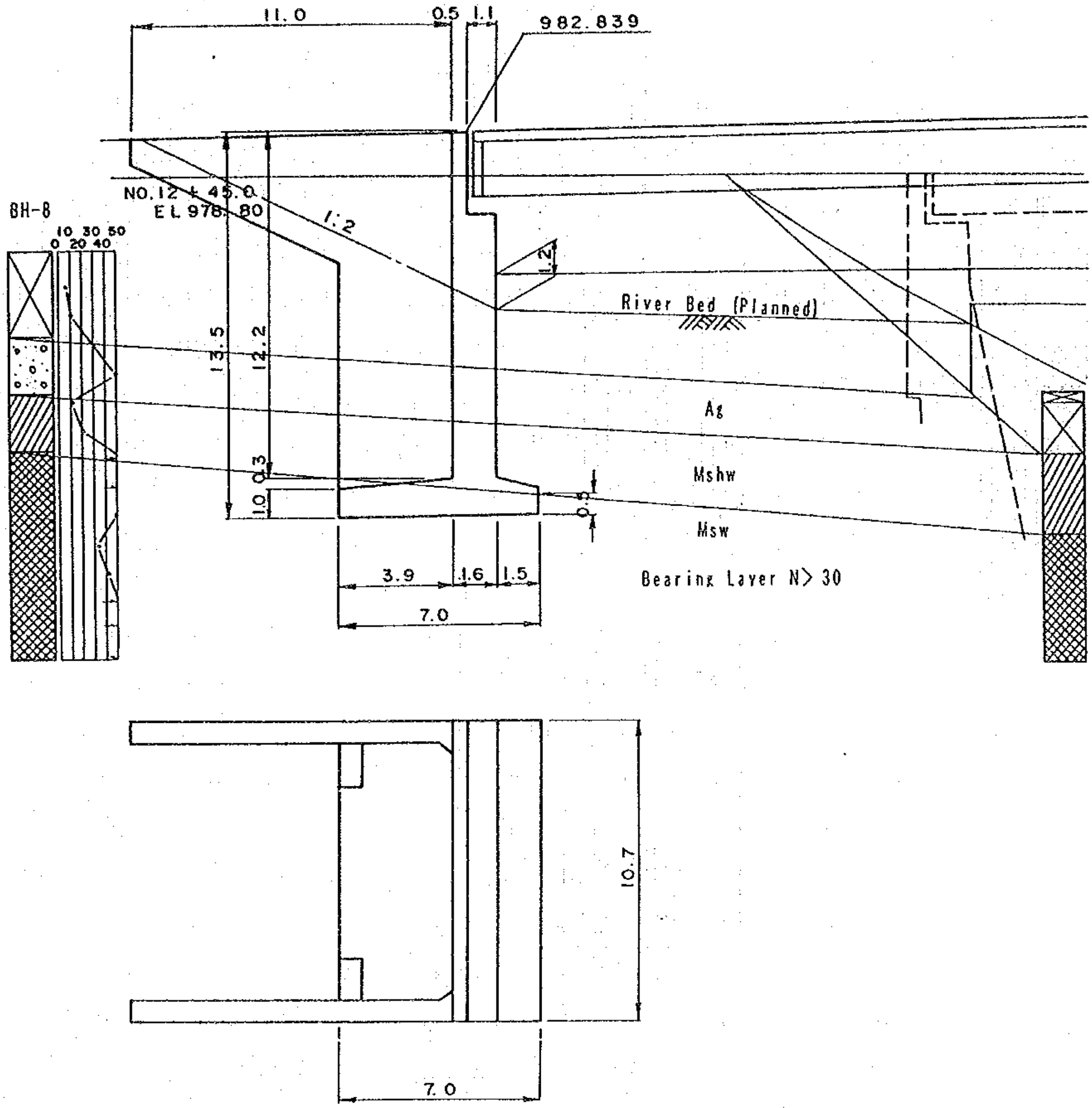


Fig. 7-7 Abutment 1 - 1st Idea

A₁ Abutment

2nd Idea

Cantilever Type

Pile Foundation

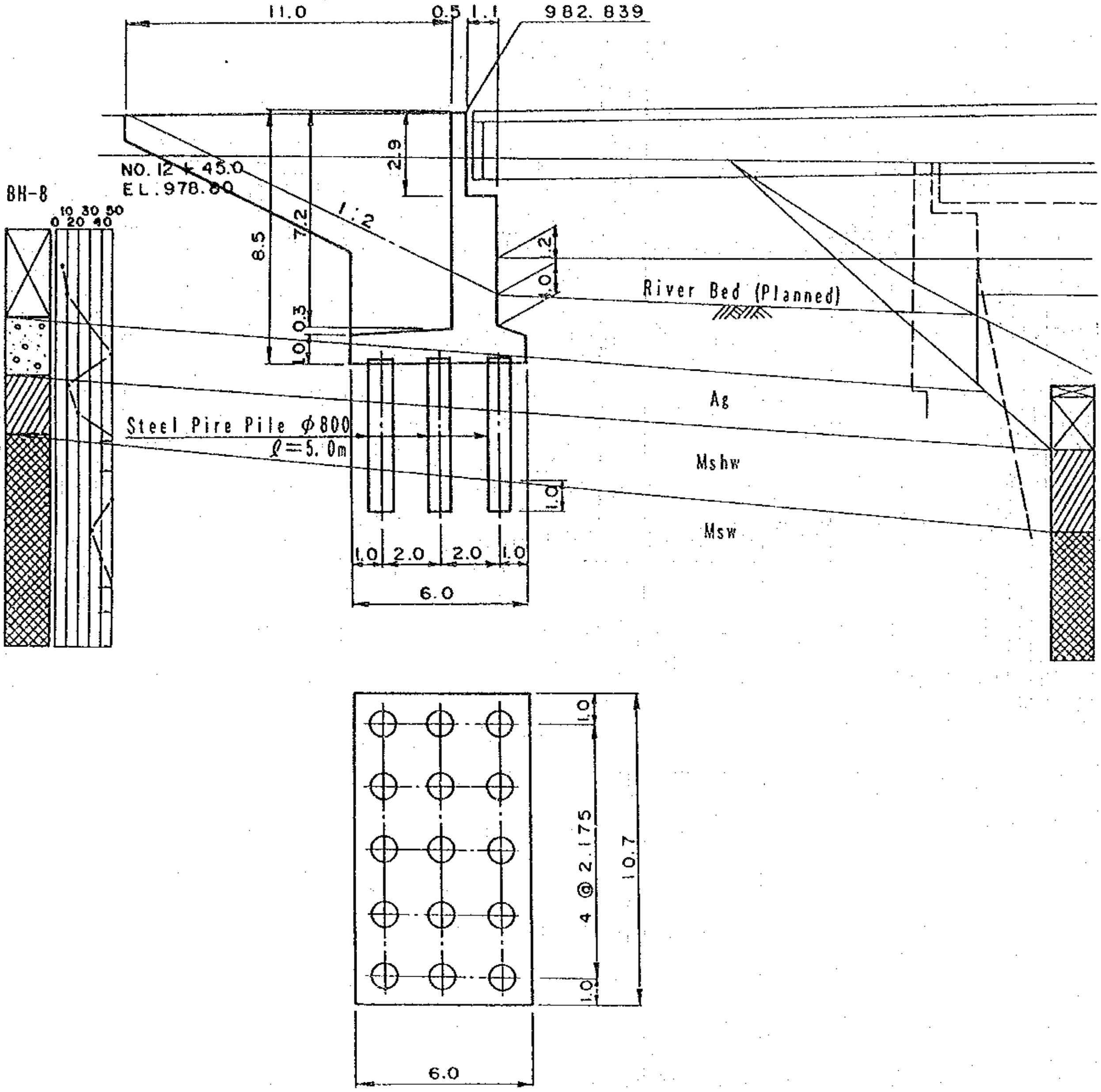


Fig. 7-8 Abutment 1 - 2nd Idea

(ii) A2 abutment (Livingstone side)

307 The bearing layer locates approximately 12 m below the land surface. The height of footing foundation, then, becomes 14.5 m.

308 As the result of examining the following three realistic types of footing foundation from the view point of cost, facilitation at site and rigidity in longitudinal direction of the bridge, we select 1st type as A2 abutment, though this type is inferior to other two ideas in terms of site work implementation.

1st idea --- inverted T type footing foundation
(less wing wall length)

2nd idea --- inverted T type footing foundation
(requires retaining wall instead of wing)

3rd idea --- inverted T type footing foundation
(enlarging bridge length and less wing wall)

Comparison Table (A2 Abutment)

	Relative Cost	Facilitation of site work	Rigidity in longi. direction of bridge	Total evaluation
1st idea footing foundation	100 ○	△	○	○
2nd idea inverted T type abutment with retaining wall	156 ×	Water shield is required ×	△	×
3rd idea inverted T type abutment (longer span)	111 △	○	△	△

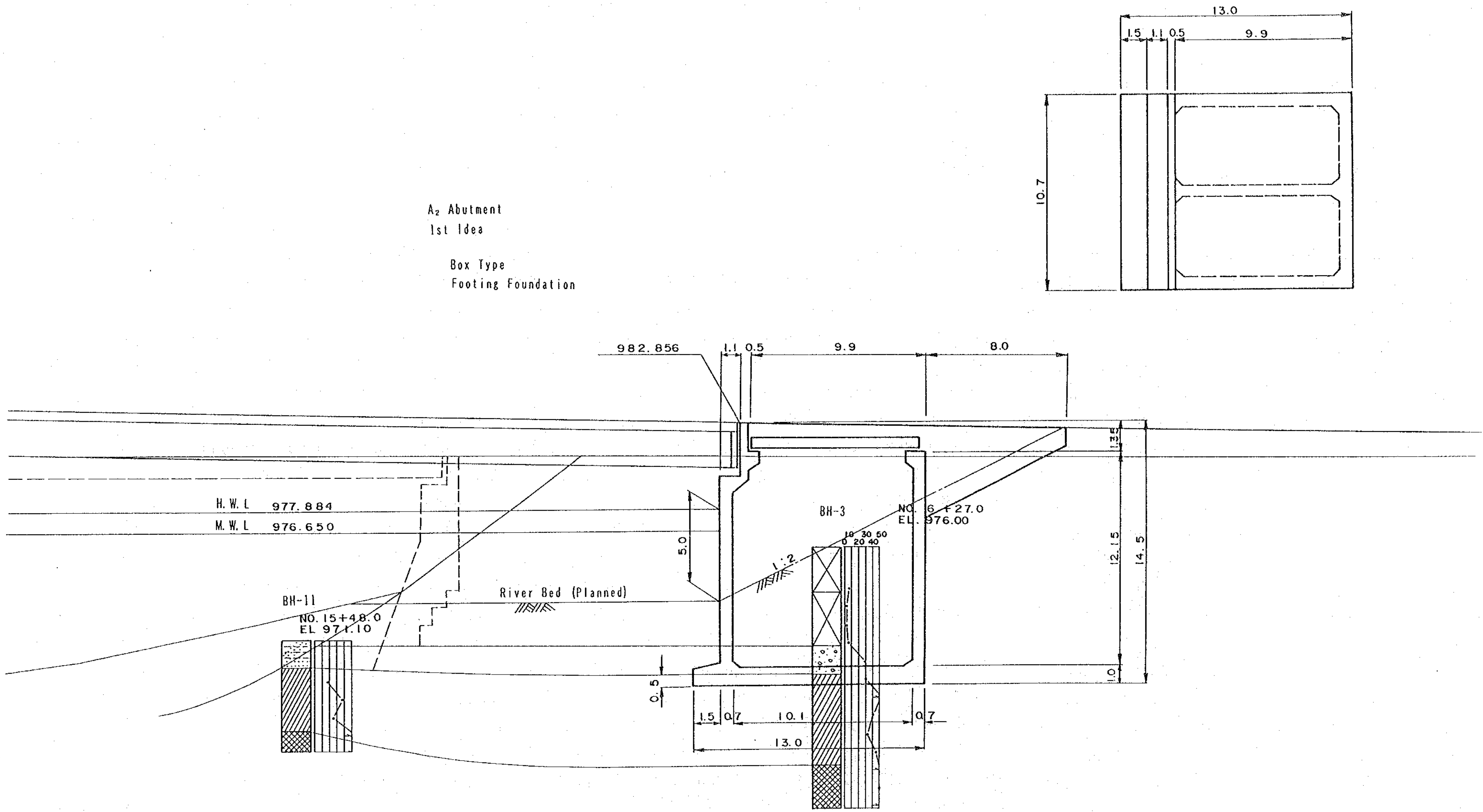


Fig. 7-9 Abutment 2 - 1st Idea

A₂ Abutment
2nd Idea

Cantilever Type Abutment with Cantilever Type Retaining Wall
Footing Foundation

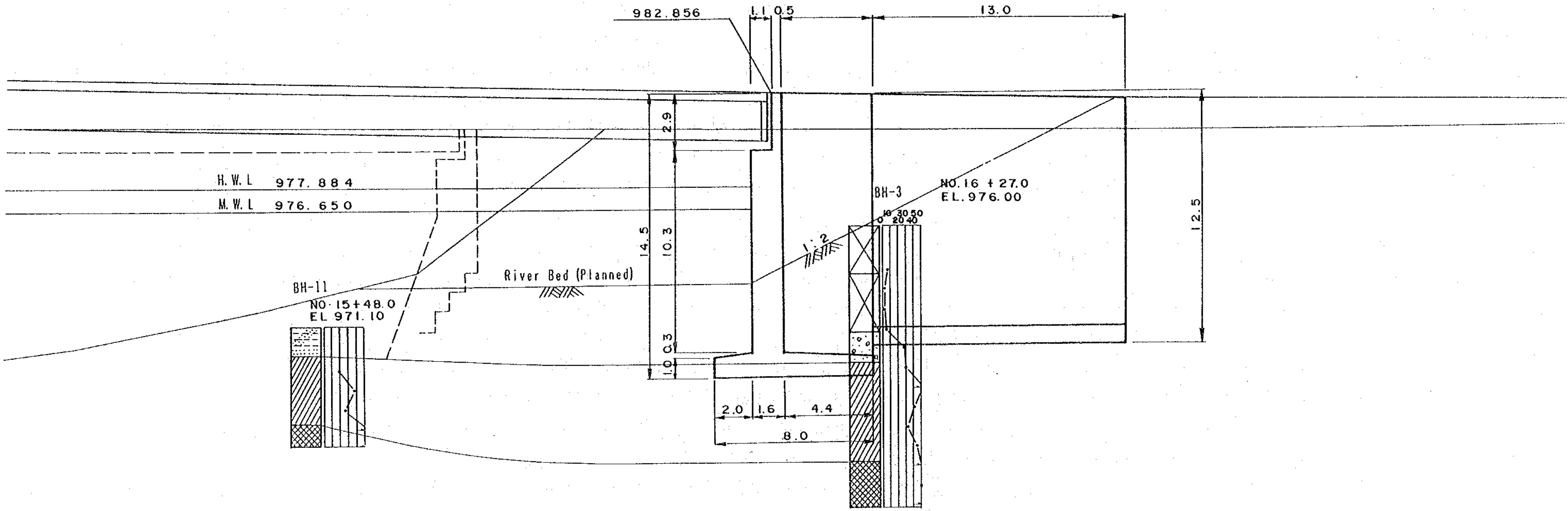


Fig. 7-10 Abutment 2 - 2nd Idea

A₂ Abutment
3rd Idea

Cantilever Type
Footing Foundation

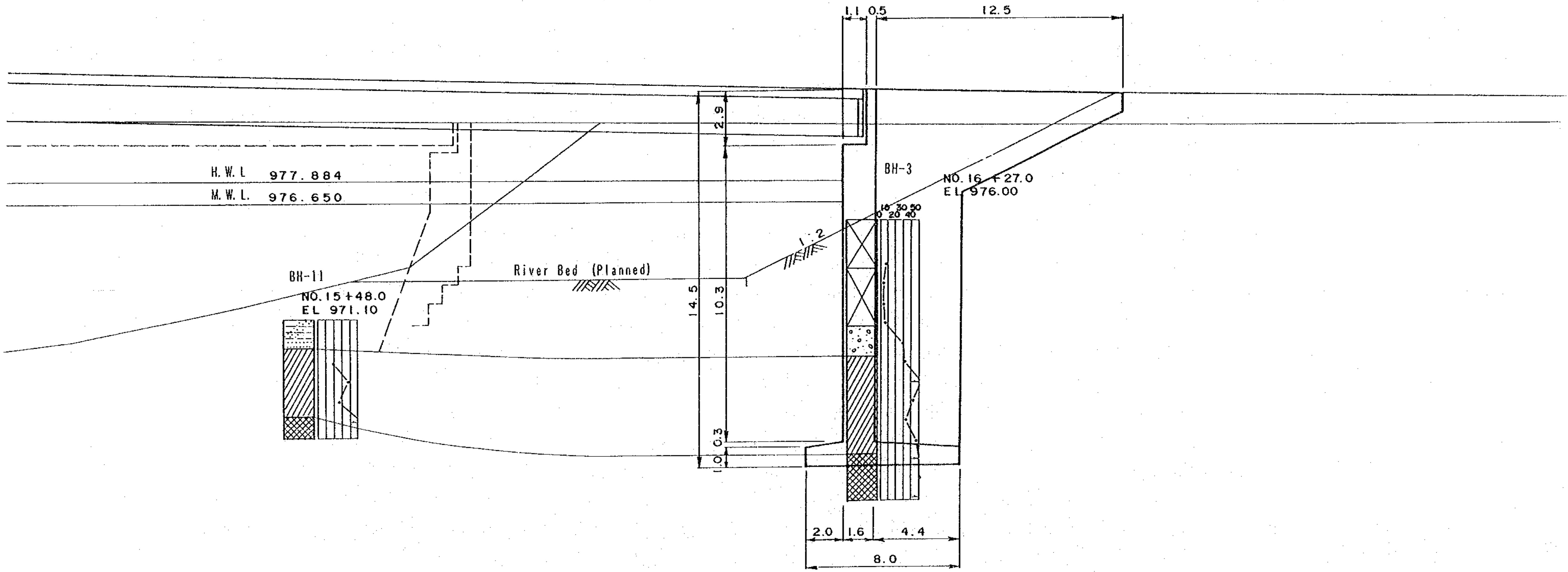


Fig. 7-11 Abutment 2 - 3rd Idea

7-F. APPROACH EMBANKMENTS

309 Laterite, that is broadly available in these regions, is applied to embankment. Laterite contains relatively low water ratio and makes control of construction possible. It also furnishes fairly good compactions.

Material used on the existing road is also laterite. Pertaining to visual inspection, the material seems reasonably sound.

CHAPTER 8

TECHNICAL APPRAISAL OF ALTERNATIVES

8-A PRELIMINARY COST ESTIMATION
8-B TECHNICAL OPTIMIZATION
8-C DEMOLITION OF EXISTING BRIDGE

CHAPTER 8 TECHNICAL APPRAISAL OF ALTERNATIVES

8-A. PRELIMINARY COST ESTIMATION

310 Preliminary cost estimates have been prepared for each of the five (5) Alternatives described in Paragraph 294 of Section 7-E.

311 Estimating information has been collected through discussions with several major local contractors who have experience in roads and bridge construction in south African countries and Japanese constructors and transport firms who possess work experiences under similar circumstances. Discussion was made regarding supplies, delivery, assembly and erection at site for the new bridge, including dismantling the existing bridge, supply condition of cement, gravel, sand, timber, steel bars and concrete forms.

312 Transportation routes for imported construction materials and equipment are Dal Es Saram - Kafue route and Durban - Zimbabwe - Kafue route. There is no significant difference in the distance of the inland transportation.

313 Quantities of every alternative has been computed in accordance with preliminary outline design and cost obtained as a results of multiplying the quantities required by the unit prices. Taxes and duties for such construction equipment and steel materials as to be imported are considered exempt.

314 Net cost excludes preliminary and general charge, allowance for unmeasured items, engineering and contingencies. These allowance are assumed approx. 60 % of net measured cost.

315 Supply conditions on major items of construction materials and equipment have been settled as follows:

a) Aggregates (crushed stone, sand)

Crushed stone is obtained from a factory near the site and other several locations in Zambia. River sand is gathered at various places near the site. These are usable should