# CHAPTER 5. ONNUT-1 TELEPHONE EXCHANGE

CONTENTS

## CHAPTER 5. ONNUT-1 TELEPHONE EXCHANGE

### 5.1 Service Area

The Onnut-1 Telephone Exchange is located in the eastern part of the city of Bangkok. In part of the service area the Muangton Mobile Exchange is in operation.

On the west, this service area adjoins that of the Phrakanong Exchange across the Klong Kled. On the north, it borders with the service area of the Klongchan Exchange across the Thai National Railways' East Line. On the east spreads the swampy land which abuts on the service area of the Onnut-2 Exchange.

The whole service area embraces approximately 3,000 hectares.

Onnut-1 Exchange Building Site (recommended)

### 5.2 Demand Potential and Locational Features

In the Onnut-1 Telephone Exchange service area which is the swampy land for the most part, the residential land development is actively in progress since recently.

At present, almost the whole service area is the housing area. The housing construction is making rapid progress along the Onnut Road and Phathanakarn Road.

In this service area the subscribers that belong to the special demand category do not exist. The growth of telephone demand depends upon the progress of residential land development in the future.

Based on these locational features, the demand distribution map was formulated and the demand forecast was conducted. The demand estimates are as follows:

General Demand Forecast

| | 1981 | 1986 | 1991 |
|---|---|---|---|
| Demand | 4,810 | 7,090 | 9,370 |
| Growth rate | 100.0 | 147.4 | 194.8 |

## 5.3 Primary Cable Network Design

### 5.3.1 Selection of Exchange Building Site

After the demand survey, the cable distribution areas were established and the network center was selected. In the case of the Onnut-1 Exchange, the network center was located at the intersection of the Onnut Road and Phathanakarn Road. Accordingly, TOT was advised to expropriate the land in this neighborhood as the exchange building site.

### 5.3.2 Design of MDF

a. MDF is to be the same type as in other existing exchanges, i.e., the combined distribution frame type.

b. On the line side, the 258R terminal blocks with 600 pairs per vertical are to be installed.

c. At MDF, the junction cables are to be terminated firstly and then the local cables are to be terminated.

### 5.3.3 Number of Pairs of Entrance Cable

To be newly installed:

| | | |
|---|---|---|
| 2400-.4 AP-FSF | | 3 lines |
| 1800-.4 AP-FSF | | 1 line |
| Total | 9,000 pairs | 4 lines |

5.3.4 New Cable Installation in Various Directions

(1) Phathanakarn Road Direction

a. The existing feeder cable to CAB No. 007 is installed through the private land. Therefore, the new cable line is to be installed on the entrance road to the Tee Chid San Ue Chue Liang housing complex and the existing cable is to be withdraw.

b. CAB No. 016 is scheduled to be established in 1979 (according to the Buddhist calendar, 2525) in the Huamak Exchange opening work.
In this design, which is based on the Huamak Exchange design drawing, 300 pairs are assigned to that cabinet area.

c. For the Onnut Road, the road expansion work is expected in the future so that, for the cross-connecting cabinets, the locations which will not obstruct the road expansion work have been selected.
As regards the MEA poles, their present locations are used in this design because the road expansion work period could not be known definitely.
The related cross-connecting cabinets are CAB Nos. 018, 019, 021, 024, 027 and 028.

d. For the end part of each underground cable line, the aerial line is designed for less than 600 pairs.

5.3.5 Line Loss and DC Resistance

For all subscribers in the Onnut-1 Exchange service area the line loss and the d.c. resistance are within the limit values prescribed in the transmission sheet.

## 5.4 Subscriber Cut-over Design

(1) This work is to cut-over the subscribers of the Phathanakarn Mobile Exchange now in operation and a part of subscribers of the Phrakanong Exchange and Huamak Exchange.

(2) For the Phrakanong Exchange and Huamak Exchange subscribers who are located distantly from their respective exchanges, the multiple cut-over is to be carried out.

(3) Cut-over of Phathanakarn Mobile Exchange Subscribers

a. The 01 cable cut-over is by multi-connection to the newly laid 03 cable in the pulling box No. 003. And, on the pole No. 8 in front of the mobile exchange, the existing cable (59-01:1-100, 201-400) and the newly laid cable (42-03:1-300) are to be multi-connected.
In CAB No. 007, 200 pairs (59-01:1-200) are terminated. The subscribers accommodated in the cable (59-01:101-200) are to be jumpered to the vacant circuits of (59-01:1-100) in the cabinet or at the MDF and thereby returned to the new exchange.

b. The 02 cable cut-over is by multi-connection of all circuits to the newly laid cable (42-03:601-1800) in the pulling boxes Nos. 001 and 005.

(4) Cut-over of Phrakanong Exchange Subscribers

a. On the MEA pole No. 1 in front of CAB No. 029 the Phrakanong Exchange cable (30-01:1-600) and the newly laid cable (42-01:1-600) are to be multi-connected for the purpose of multiple cut-over of subscribers in the direction of Mittapub Road.

b. CAB No. 027 secondary cable (027-01:1-300) is to be installed in advance and is to be multi-connected to the existing secondary cable (008-02:1-100) on the MEA pole No. 183.

c. By the abovementioned arrangements the multiple cut-over of the Phrakanong Exchange subscribers in the Onnut-1 Exchange area can be realized.

(5) Cut-over of Huamak Exchange Subscribers

In the Huamak Exchange CAB No. 040, the Huamak Exchange cable (28-01:2101-2300) and the newly laid cable (42-04:301-500) are to be multi-connected.
However, the number of working circuits could not be ascertained because of being before the completion of the Huamak Exchange work.

(6) After the Onnut-1 Exchange service-in, the mobile exchange is to be withdrawn.

## 5.5 Design of Underground Conduit

(1) The entrance manhole and entrance duct design will be made by TOT.
On the assumption that the new exchange building site would be the Phathanakarn Road and Onnut Road intersection, it has been tentatively decided that the entrance manhole be the V-3 type, and on this assumption the related design has been prepared.

(2) The conduit location on the Phathanakarn Road has been decided to be on the western side of the road at the request of the road administrator.
For this reason, the road-occupying location of the conduit changes in the section where the conduit is to be connected to the TOT-planned underground conduit that uses the highway.

(3) The pulling box No. 013 (JUF-11) cannot but be the deformed pulling box becuase of the narrowness of the sidewalk and the existence of the drain pipe under the road.

(4) The locations and types of bridges are as follows:

i) Between manholes Nos. 002 and 003

Number of pipe lines: 9 (3 lines in 3 steps)
Bridge length: 117.0m with 13 spans

ii) Between manholes Nos. 011 and 012
Number of pipe lines: 9 (3 lines in 3 steps)
Bridge length: 21.2m with 3 spans

iii) Between manholes Nos. 016 and 017
Number of pipe lines: 9 (3 lines in 3 steps)
Bridge length: 22.0m with 3 spans

iv) Between manholes Nos. 001 and 022
Number of pipe lines: 12 (3 lines in 4 steps)
Bridge length: 27.8m with 3 spans

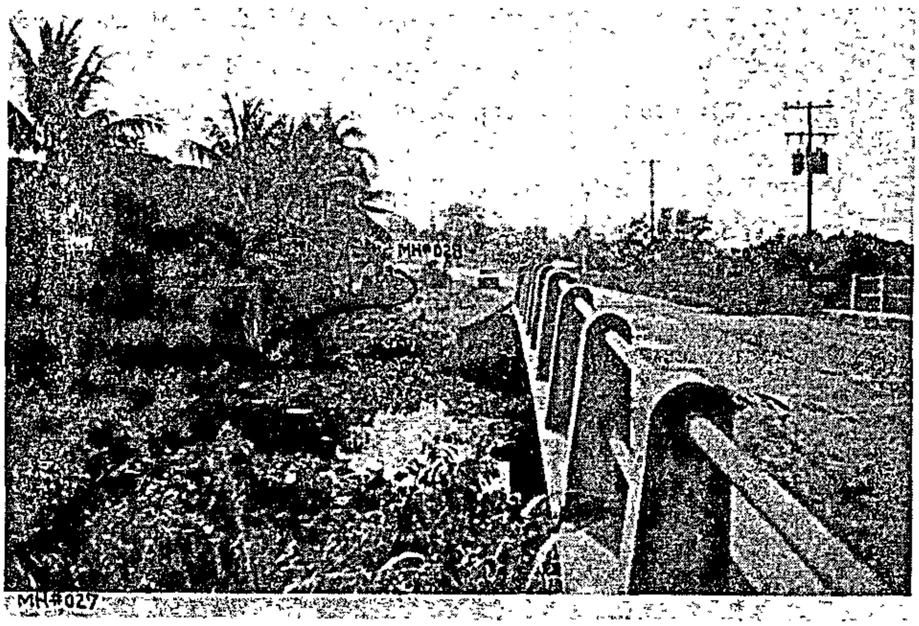
v) Between manholes Nos. 027 and 028
Number of pipe lines: 12 (3 lines in 4 steps)
Bridge length: 36.0m with 5 spans

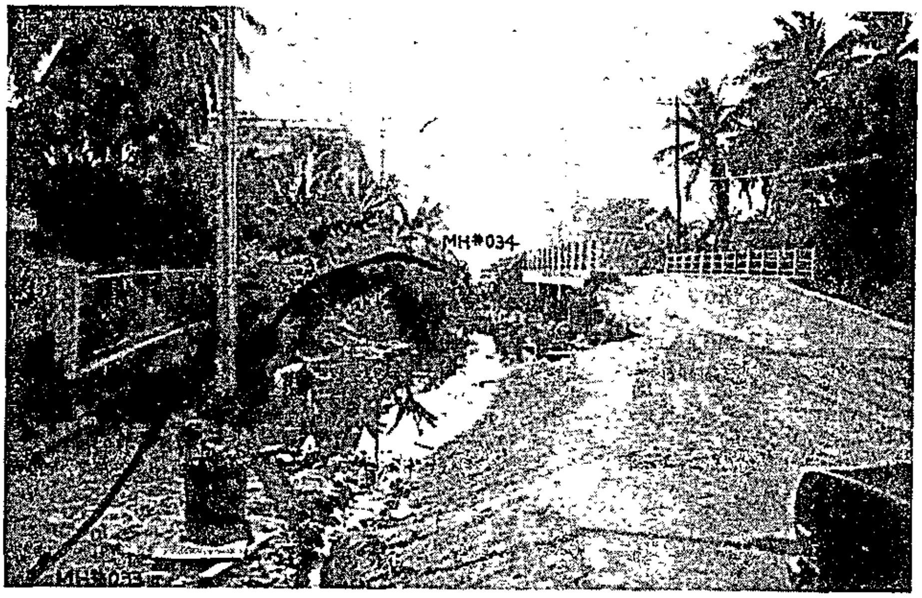
vi) Between manholes Nos. 030 and 031
Number of pipe lines: 12 (3 lines in 4 steps)
Bridge length: 36.2m with 5 spans

vii) Between manholes Nos. 033 and 034
Number of pipe lines: 12 (3 lines in 4 steps)
Bridge length: 32.1m with 5 spans

viii) Between manholes Nos. 035 and 036
Number of pipe lines: 12 (3 lines in 4 steps)
Bridge length: 32.0m with 5 spans

(5) The bridge strength calculations are refer to appendixes.






TOT bridge will be installed in the canal

## 5.6 Secondary Cable Network Design

(1) CAB No. 010 (Siam Phathanakarn Center)

The pole erection in the Center is planned by MEA. However, the work design has not yet been completed.
In this cable work design, 400 pairs are to be installed on the Phathanakarn Road in order to reserve the cable conductors to be assigned to the Center.

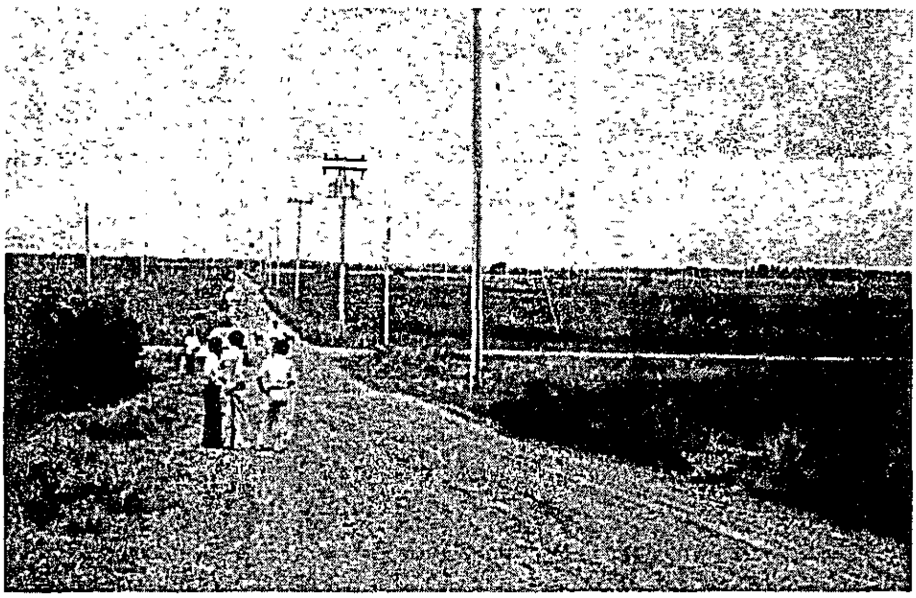
(2) In the undermentioned areas, the housing construction has not yet been completed. Therefore, the cable distribution plan for those areas is a desk plan based on MEA's pole erection plan drawing. For the areas where MEA's pole erection plan is still pending, the cable work is withheld this time so as to avoid the duplication of MEA's and TOT's pole erections.

CAB No. 011 area
CAB No. 012 area
CAB No. 015 area
CAB No. 022 area
CAB No. 023 area

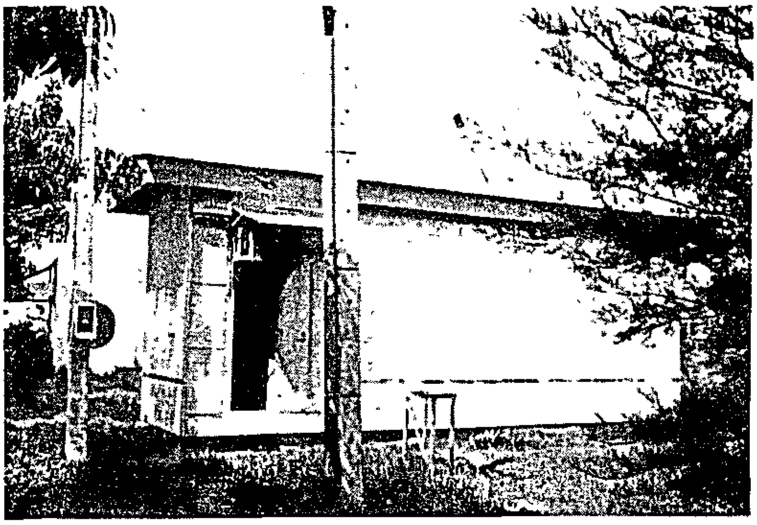
(3) The Huamak Exchange CAB No. 040 primary cable is to be used as CAB Nos. 014 and 017 secondary cable after the Onnut-1 Exchange service-in.

(4) For the areas where the land development has been completed but the housing construction is being delayed, the cable work design is withheld. Those areas are as follows:

    CAB No. 020: Phathanachumchon Tua Yang Village
    CAB No. 025: Chadsankrahuang Kalkung Village
    CAB No. 026: Chadsanveera Village



Reservation Area for the Residential Land Development Project



Phathanakarn Mobile System (in service)

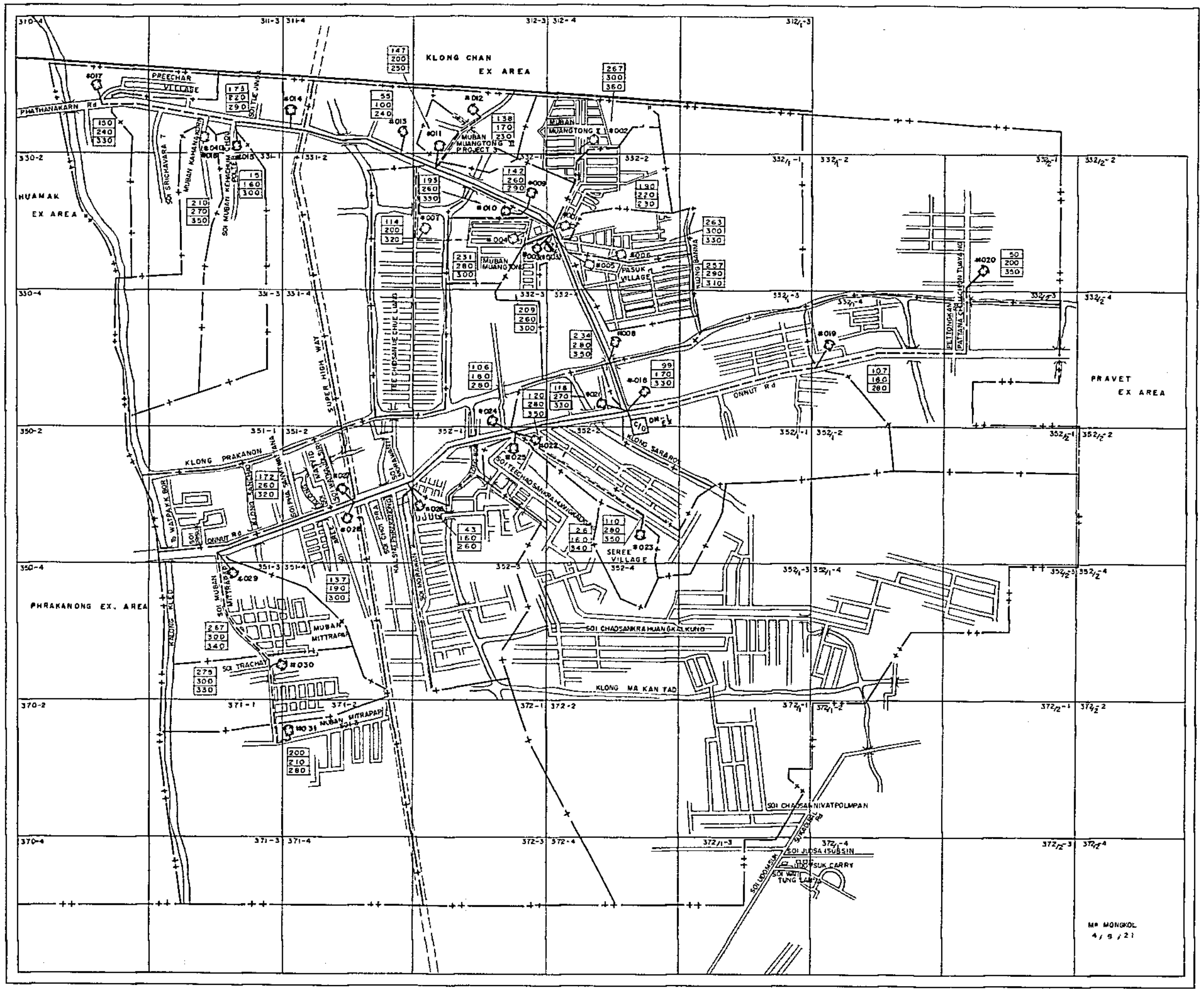
## 5.7 Amount of Construction Work

AMOUNT OF CONSTRUCTION WORK

| SECTION | DESIGNATION | UNIT | QUANTITY | | | REMARKS |
|---|---|---|---|---|---|---|
| | | | PRIMARY | SECONDARY | TOTAL | |
| A | A - 8 | ea | 13 | 2 | 15 | |
| B | BIBS | " | 17 | 197 | 214 | |
| | BICS | " | 17 | | 17 | |
| | BIFS | " | | 3 | 3 | |
| | B2B | " | | 10 | 10 | |
| | Section "B" Total | " | 34 | 210 | 244 | |
| C | C5A2B | " | 34 | 200 | 234 | |
| E | E 10 . 4 A2 | 100m | | 22.3 | 22.3 | |
| | E 25 . 4 A2 | " | | 67.8 | 67.8 | |
| | E 50 . 4 A2 | " | | 67.5 | 67.5 | |
| | E 100 . 4 A2 | " | 9.1 | 73.6 | 82.7 | |
| | E 200 . 4 A2 | " | | 30.8 | 30.8 | |
| | E 300 . 4 A2 | " | 33.9 | 61.3 | 95.2 | |
| | E 400 . 4 A2 | " | 2.2 | 25.0 | 27.2 | |
| | E 600 . 4 A2 | " | 12.9 | | 12.9 | |
| | E 10 . 5 A2 | " | | 0.4 | 0.4 | |
| | E 25 . 5 A2 | " | | 6.9 | 6.9 | |
| | E 50 . 5 A2 | " | | 16.5 | 16.5 | |
| | E 100 . 5 A2 | " | | 4.4 | 4.4 | |
| | E 200 . 5 A2 | " | | 1.0 | 1.0 | |
| | E 300 . 5 A2 | " | 11.8 | 5.0 | 16.8 | |
| | E 400 . 5 A2 | " | | 9.0 | 9.0 | |
| | E 600 . 5 A2 | " | 11.3 | | 11.3 | |
| | Section "E" Total | " | 81.2 | 391.5 | 472.7 | |
| G | G . . 4 | 100 m | 7.3 | | 7.3 | AP-FSF cable |
| | G 600 . 4 | " | 9.8 | | 9 8 | " |
| | G 900 . 4 | " | 0.8 | | 0.8 | " |

| SECTION | DESIGNATION | UNIT | QUANTITY | | | REMARKS |
|---|---|---|---|---|---|---|
| | | | PRIMARY | SECONDARY | TOTAL | |
| G | G1200 . 4 | 100 m | 1.3 | | 1.3 | AP-FSF cable |
| | G1800 . 4 | " | 29.0 | | 29.0 | " |
| | G2400 . 4 | " | 27.6 | | 27.6 | " |
| | G 600 . 5 | " | 0.4 | | 0.4 | " |
| | G 900 . 5 | " | 13.1 | | 13.1 | " |
| | G1200 . 5 | " | 10.8 | | 10.8 | " |
| | G 100 . 4 A2 | " | 0.2 | 0.1 | 0.3 | |
| | G 300 . 4 A2 | " | 1.0 | 0.5 | 1.5 | |
| | G 400 . 4 A2 | " | | 1.1 | 1.1 | |
| | G 200 . 5 A2 | " | | 1.6 | 1.6 | |
| | G 300 . 5 A2 | " | 0.1 | | 0.1 | |
| | Section "G" Total | " | 101.4 | 3.3 | 104.7 | |
| J | J 300 . 5 P3 | 10 m | 15.5 | | 15.5 | |
| K | KA11G2 | ea | | 5 | 5 | |
| | KB12 | " | | 452 | 452 | |
| | Section "K" Total | " | | 457 | 457 | |
| L | L 900 | ea | 24 | | 24 | |
| | L 50 B2 | " | | 1 | 1 | |
| | L 100 B2 | " | 5 | 2 | 7 | |
| | L 50 B2 | " | 1 | | 1 | For new type cabinet |
| | L 100 B2 | " | 59 | 85 | 144 | " |
| | Section "L" Total | " | 89 | 88 | 177 | |
| M | MIAP | ea | 2 | 58 | 60 | |
| | MIBP | " | 23 | 72 | 95 | |
| | MICP | " | 6 | | 6 | |
| | M3AP | " | 3 | 4 | 7 | |
| | M3BP | " | 40 | 24 | 64 | |
| | M3CP | " | 48 | | 48 | |
| | Section "M" Total | " | 122 | 158 | 280 | |

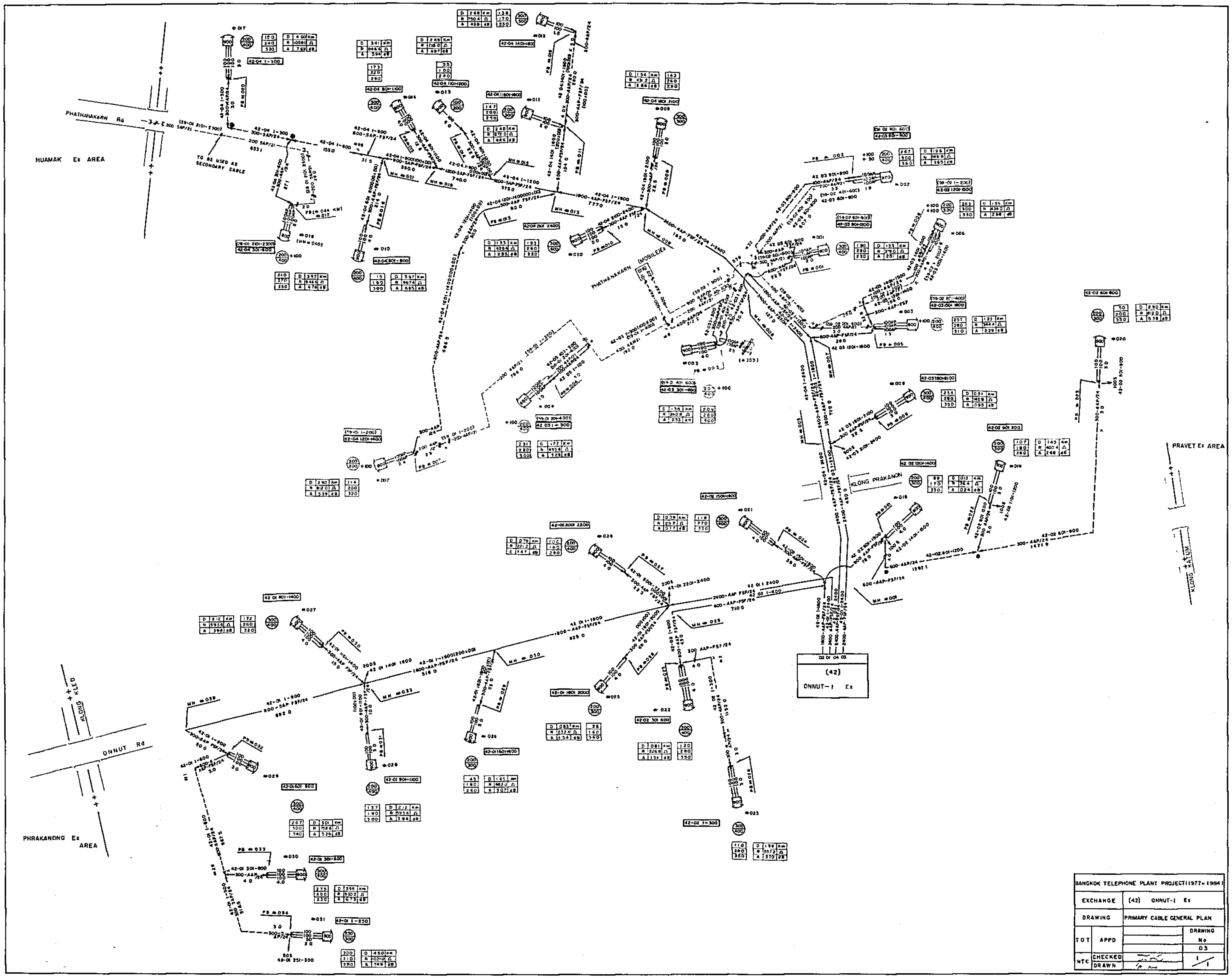
| SECTION | DESIGNATION | UNIT | QUANTITY | | | REMARKS |
|---|---|---|---|---|---|---|
| | | | PRIMARY | SECONDARY | TOTAL | |
| P | PP9B | 100 m | 2.0 | | 2.0 | |
| | PP12B | " | 2.1 | | 2.1 | |
| | PV9B | " | 9.4 | | 9.4 | |
| | PV12B | " | 1.8 | | 1.8 | |
| | Px9B | " | 21.8 | | 21.8 | x: TOT determined |
| | Px12B | " | 24.4 | | 24.4 | " |
| | PP2A | " | 0.4 | | 0.4 | Riser to pole |
| | PV2A | " | 5.7 | | 5.7 | " |
| | PV4A | " | 0.9 | | 0.9 | " |
| | PP4A | " | 1.7 | | 1.7 | Riser to cabinet |
| | PV4A | " | 7.6 | | 7.6 | " |
| | Section "P" Total | " | 77.8 | | 77.8 | |
| Q | QA-2 | ea | 33 | | 33 | |
| | QJUF-11 | " | 25 | | 25 | |
| | Section "Q" Total | " | 57 | | 57 | |



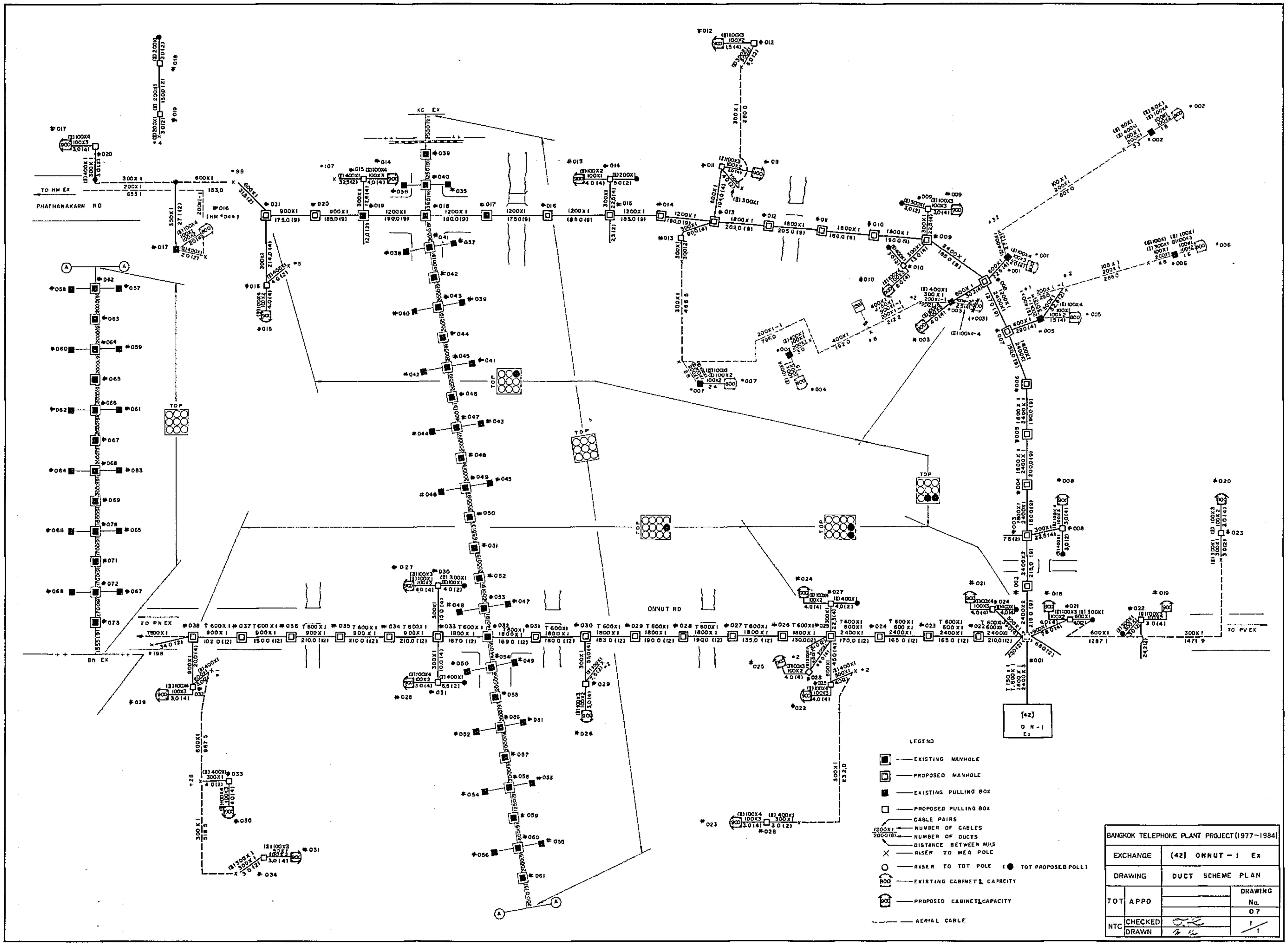

LEGEND

- —++— BOUNDARY OF EXCHANGE AREA
- —+— BOUNDARY OF CABINET AREA
- ——— FEEDER CABLE ROUTE (UNDER GROUND)
- - - - FEEDER CABLE ROUTE(AERIAL)
- CABINET
- 210 NUMBER OF PRESENT DEMAND
- 270 NUMBER OF 5 YEARS DEMAND
- 350 NUMBER OF 10 YEARS DEMAND

| | | | |
|---|---|---|---|
| BANGKOK TELEPHONE PLANT PROJECT (1977~1984) | | | |
| EXCHANGE | | (42) ONNUT-I Ex. | |
| DRAWING | | KEY PLAN | |
| TOT | APPD | | DRAWING No |
| | | | 01 |
| NTC | CHECKED | | 1/1 |
| | DRAWN | | |



HUAMAK Ex AREA
PHATHANAKARN Rd
PHATHANAKARN (MOBILE)Ex
KLONG PRAKANON
PRAVET Ex AREA
KLONG KATUM
KLONG KLED
ONNUT Rd
PHRAKANONG Ex AREA
(42)
ONNUT-1 Ex
BANGKOK TELEPHONE PLANT PROJECT(1977-1984)
EXCHANGE
(42) ONNUT-1 Ex
DRAWING
PRIMARY CABLE GENERAL PLAN
DRAWING No
03
TOT
APPD
NTC
CHECKED
DRAWN
1/1



KC EX
TO HM EX
PHATHANAKARN RD
TO PN EX
BN EX
ONNUT RD
TO PV EX
TOP
LEGEND
EXISTING MANHOLE
PROPOSED MANHOLE
EXISTING PULLING BOX
PROPOSED PULLING BOX
CABLE PAIRS
NUMBER OF CABLES
NUMBER OF DUCTS
DISTANCE BETWEEN MHS
RISER TO MEA POLE
RISER TO TOT POLE
TOT PROPOSED POLE
EXISTING CABINET & CAPACITY
PROPOSED CABINET & CAPACITY
AERIAL CABLE
BANGKOK TELEPHONE PLANT PROJECT (1977~1984)
EXCHANGE
(42) ONNUT - 1 Ex
DRAWING
DUCT SCHEME PLAN
TOT
APPO
NTC
CHECKED
DRAWN
DRAWING No.
07
1/1

APPENDIX

# CALCULATIONS OF BRIDGE STRENGTH

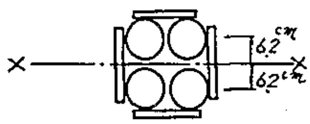
Design Calculations for TOT Bridge between MH #061 - MH #063 in CW Ex. area

A. Calculations as Combination Type GIP Bridge (ø12 mm iron bars welded around the pipe)

a. Design Conditions

| | | |
|---|---|---|
| i. | Conduit lines | 4 lines |
| ii. | Interval between piers | 16.0 m |
| iii. | Weight per m | |
| | Pipe (GIP ø4") | 12.4 x 4 = 49.6 kg/m |
| | Cable (0.4-2400P) | 8.7 x 3 = 26.1 kg/m |
| | | Total = 75.7 kg/m = 0.757 kg/cm |

b. Calculation of Moment of Inertia




Ix : Total moment of inertia in case of combined pipes

Io : Moment of inertia of one pipe 232.6 $cm^4$

A : Sectional area of one pipe

$$(D^2-d^2)\times\frac{\pi}{4} = (11.41^2-10.51^2)\times\frac{\pi}{4} = 15.49\ cm^2$$

$$Ix = 4I_o + 4Ay^2 = 4\times 232.6 + 4\times 15.49\times 6.2^2 = 3312.12 \fallingdotseq 3312\ cm^4$$

c. Calculation of Deflection

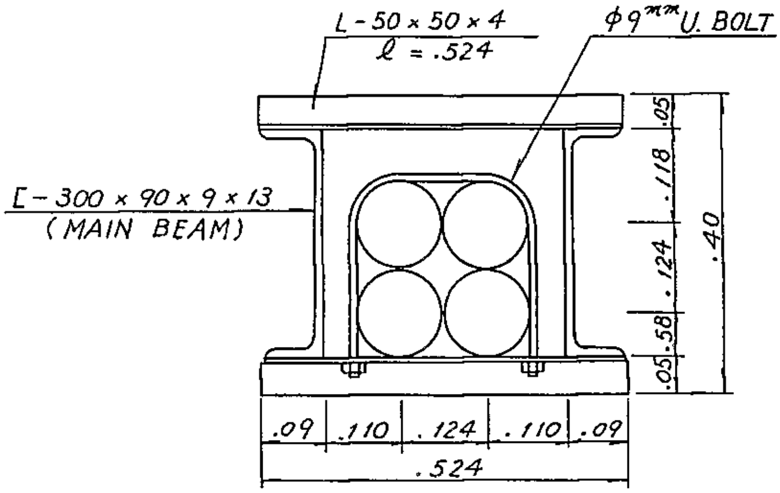
$$y = \frac{5\omega l^4}{384EI} = \frac{5\times 0.76\times 1600^4}{384\times 2.1\times 10^6\times 3312\times 0.7}$$

$$= 13.3\ cm > \frac{l}{300} = \frac{1600}{300} = 5.33\ cm$$

In the case of the combination Type GIP bridge specified above, the deflection becomes too large. Therefore, the bridge is to be composed of the specially established main beam with the pipes attached thereto. On the basis of this type of bridge, the following calculations are made.

B. Calculations for Bridge Using Channel Steel as Main Beam

(Refer to the illustration below.)




a. Design Conditions

| | | | |
|---|---|---|---|
| i. | Length of bridge | 16.0 m | |
| ii. | Main beam | [ - 300 x 90 x 9 x 13 | Unit weight: 38.1 kg/m |
| iii. | Cross beam | L - 50 x 50 x 4 | Unit weight: 3.06 kg/m |
| iv. | Lateral bracing | L - 50 x 50 x 4 | Unit weight: 3.06 kg/m |
| v. | Allowable stress | $\sigma ca = 1400 - 24(\ell/b - 4.5)$ | |

: Bending compressive stress

where

ℓ : Length between flange fixing points

b : Breadth of flange

$\therefore\ \sigma ca = 1400 - 24(190/9 - 4.5) = 1001.3\ kg/cm^2$

$\sigma ta = 1400\ kg/cm^2$ : Bending tensile stress

$\tau a = 800\ kg$ : Shearing stress

b. Load

Pipe $12.4 \times 4 = \underline{49.6\ kg/m}$

Cable $8.7 \times 3 = \underline{26.1\ kg/m}$

U-bolt $3.42 \div 16.0 = \underline{0.21\ kg/m}$

(Weight per piece ($\ell$ = 75 cm):

$0.499 \times 0.75 = 0.38$ kg

Since nine pieces are used,

$0.38 \times 9 = 3.42$ kg)

Cross beam (L- 50 x 50 x 4)

$22.4 \div 16.0 = \underline{1.4\ kg/m}$

(Weight per piece ($\ell$ = 52.4 cm):

$3.06 \times 0.524 = 1.6$ kg

Since five pieces are used in the upper part and nine pieces in the lower part,

$1.6 \times 14 = 22.4$ kg)

Lateral bracing (L- 50 x 50 x 4)

$45.12 \div 16.0 = \underline{2.82\ kg/m}$

(Weight per piece ($\ell$ = 92.3 cm):

$3.06 \times 0.923 = 2.82$ kg

Since 16 pieces are used,

$2.82 \times 16 = 45.12$ kg)

Total weight = $\underline{80.13\ kg/m}$

Dead load of main beam ( [- 300 x 90 x 9 x 13)

38.1 kg/m

Uniform load $q_1$ on one main beam

$$q_1 = \frac{80.13 + (38.1 \times 2)}{2} = 78.165 \fallingdotseq 79\ kg/m$$

c. Calculation of Maximum Bending Moment

$$M_{max} = \frac{g_1 \cdot l^2}{8} \times 100 = \frac{79 \times 160^2}{8} \times 100 = 252{,}800 \text{ kg·cm}$$

d. Calculation of Shearing Force

$$S_A = S_B = \frac{gl}{2} = \frac{79 \times 16.0}{2} = 632 \text{ kg}$$

e. Calculation of Each Stress

$$\sigma_c = \sigma_x = \frac{M_{max}}{Z} = \frac{252.800}{429} = 589.3 \text{ kg/cm}^2$$

$$\sigma_c < \sigma_{ca} = 1001.3 \text{ kg/cm}^2 \text{ or } \sigma_{ta} = 1400 \text{ kg/cm}^2$$

∴ Z = Section modulus of [- 300 x 90 x 9 x 13 = 429

$$\tau = \frac{S_A}{A_W} = \frac{632}{24.66} = 25.6 \text{ kg/cm}^2 < \tau_a = 800 \text{ kg/cm}^2$$

(AW = Sectional area of web = 0.9 x (30-2.6) = 24.66 $cm^2$)

f. Calculation of Deflection

$$y = \frac{5gl^4}{384EI} = \frac{5 \times 0.79 \times 1600^4}{384 \times 2.1 \times 10^6 \times 6440} = 4.98 \text{ cm}$$

$$= 5.0 \text{ cm} < \frac{l}{300} = \frac{1600}{300} = 5.33 \text{ cm}$$

Therefore, the bridge using [- 300 x 90 x 9 x 13 as main beam will be adopted.

C. Calculation of Pile Length

a. Load

i. Load from upper structure

$$\frac{(46.9+26.1+38.1 \times 2) \times 16.0+3.42+22.4+45.12}{2} = 1{,}229.1 \text{ kg}$$

ii. Abutment

$$(0.6 \times 0.5 + \frac{0.6 + 1.3}{2} \times 1.3 \times 0.5) \times 1.0 = 1.14 \text{ m}^3$$

$$(1.3 + 0.2) \times (1.0 + 0.2) \times 0.10 = 0.18 \text{ m}^3$$

$$(1.14 + 0.18) \times 2{,}300 = 3{,}036.0 \text{ kg}$$

Load on one pipe, P

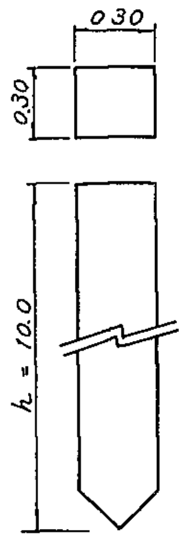
$P = 1{,}299.1 + 3{,}036.0 = 4{,}265.1 \doteqdot 4.3$ ton

b. Calculation of Pile Length

i. Kind of pipe 0.30 solid square PC pile

ii. Pipe root depth 10.0 m (provisional)

iii. Soil Soft soil




P = 4,300 kg

A : Circumferential length of pipe

h : Pipe root depth

F : Frection of pile (based on TOT data shown below)

Frection of Pipe (F)

| Ground Level | F (kg/m²) | Kind of Soil |
|---|---|---|
| 0 - 6 m | 1,470 | Soft soil |
| 0 - 6 m | 2,000 | Hard soil |
| 6 - 14 m | 1,800 | Soft soil |
| 6 - 14 m | 2,000 | Hard soil |
| 14 - 15 m | 4,000 | " |
| 15 - 21 m | 6,000 | " |

$A = 0.3 \times 4 = 1.2$ m

$= 10.0$ m

$F = 1{,}800$ kg/m$^2$

$$R = \frac{1.2 \times 10.0 \times 1{,}800}{2}$$

$= 10{,}800$ kg $> 4{,}300$ kg

Therefore, the pipe root depth provisionally set at 10.0 m ensures full safety.

Bridge Strength Calculations

(Ramindra Exchange)

Contents

1. Design Calculations for Combination Type GIP Bridge (ø12 mm iron bars welded around the pipe)

For the six-line bridge in which case the interval between piers is 6.6 m, the iron bars welded method is not adopted.

## 1.1 Nine-Line Bridge

a. Design Conditions

i. Interval between piers 10.0 m

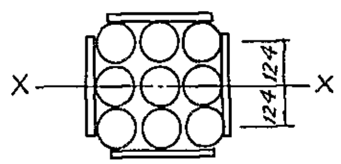
ii. Weight per m

| | | |
|---|---|---|
| Pipe | 9 x 12.4 = | 111.6 kg/m |
| Cable | 8 x 8.7 = | 69.6 kg/m |
| | Total W = | 181.2 kg/m |

iii. Allowable Stress

$\sigma_{ta}$ = 1,400 kg/cm$^2$ (bending tensile stress)

b. Calculation of Moment of Inertia




Ix : Total moment of inertia in case of combined pipes

Io : Moment of inertia of one pipe 232.6 cm$^4$

A : Sectional area of one pipe

$A = (D^2 - d^2) \times \frac{\pi}{4} = (11.41^2 - 10.51^2) = 15.49$ cm$^2$

| Io + | A x | $Y^2$ | = Ix (cm$^4$) |
|---|---|---|---|
| 232.6 x 6 = 1,395.6 | 15.49 x 6 = 92.94 | 12.4 = 153.76 | 15,686.1 |
| 232.6 x 2 = 465.2 | 15.49 x 2 = 30.98 | 0 | 465.2 |
| Total | | | 16,151.3 |

c. Calculation of Deflection

$$y = \frac{5Wl^4}{384EI_x} = \frac{5 \times 1812 \times 1000^4}{384 \times 2.1 \times 10^6 \times 16{,}151.3 \times 0.7} \fallingdotseq 1.0\,\text{Cm}$$

$$1.0\,\text{Cm} < \frac{l}{300} = \frac{1000}{300} = 3.33\,\text{Cm}$$

Therefore, safe.

d. Maximum Bending Moment

$$M_{max} = \frac{Wl^2}{8} = \frac{1.812 \times 1000^2}{8} = 226{,}500\ \text{Kg}\cdot\text{Cm}$$

e. Calculation of Stress

Z : Modulus of section

$$Z = \frac{I_x}{y} = \frac{16151.3 \times 0.7}{12.4} = 1302.5\,\text{Cm}^3$$

$$\sigma = \frac{M}{Z} = \frac{226{,}500}{1302.5} = 173.9\,\text{Kg/Cm}^2 < \sigma_{ta} = 1400\,\text{Kg/Cm}^2$$

Therefore, safe.

1.2 24-Line Bridge

1. In case of 6 lines x 4 steps = 24 lines, the interval between piers is 7.0 m, so that the pipe bridge is adopted.

2. In case of 4 lines x 6 steps = 24 lines

a. Design Conditions

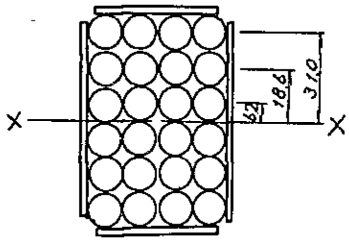
i. Interval between piers 10.0 m

ii. Weight per m

| | | |
|---|---|---|
| Pipe : | 24 x 12.4 = | 297.6 kg/m |
| Cable : | 23 x 8.7 = | 200.1 kg/m |
| | Total | 497.7 kg/m |

b. Calculation of Moment of Inertia

| Io | + A | x | = Ix ($cm^4$) |
|---|---|---|---|
| 232.6 x 8 = 1,860.8 | 15.49 x 8 = 123.92 | $31.0^2$ = 961.0 | 120,947.92 |
| 232.6 x 4 = 930.4 | 15.49 x 4 = 61.96 | $18.6^2$ = 345.96 | 22,366.08 |
| 232.6 x 4 = 930.4 | 15.49 x 4 = 61.96 | $6.2^2$ = 38.44 | 3,312.14 |
| Total | | | 146,621.1 |




c. Calculation of Deflection

$$Y = \frac{5 W l^4}{384 E I_x} = \frac{5 \times 4.98 \times 1000^4}{384 \times 2.1 \times 10^6 \times 146626.1 \times 0.7} = 0.3 \text{ cm}$$

$$Y = 0.3 \text{ cm} < \frac{l}{300} = \frac{1000}{300} = 3.33 \text{ cm}$$

Therefore, safe.

d. Calculation of Maximum Bending Moment

$$M_{max} = \frac{W l^2}{8} = \frac{4.98 \times 1000^2}{8} = 622500 \text{ kg·cm}$$

e. Calculation of Stress

$$Z = \frac{I_x}{\Sigma Y} = \frac{146626.1 \times 0.7}{31.0 + 18.6 + 6.2} = 2627.7 \text{ cm}^3$$

$$\sigma = \frac{M}{Z} = \frac{622500}{2627.7} = 236.9 \text{ kg/cm}^2 < \sigma_{ta} = 1400 \text{ kg/cm}^2$$

Therefore, safe.

## 2. Calculation of Bridge Pier

### 2.1 Type of Upper Part of Bridge Pier

The six-line bridge is of the type shown in Figure (a).

The nine-line and 24-line bridges are of the type shown in Figure (b).

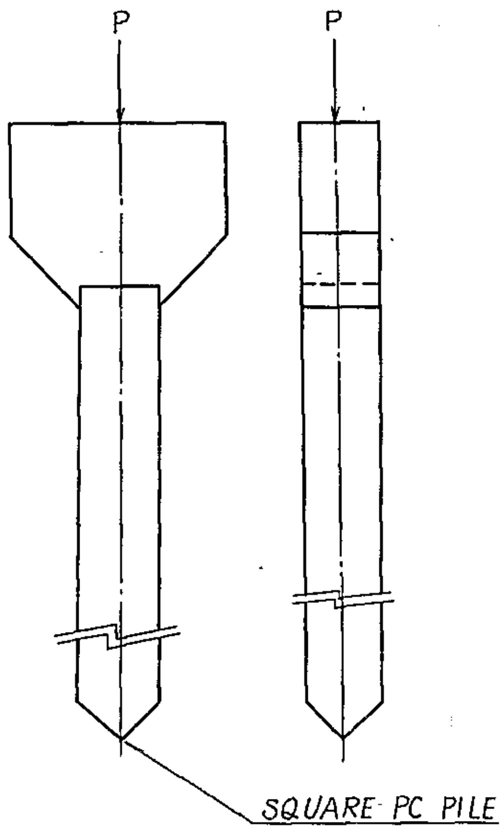



Figure (a)

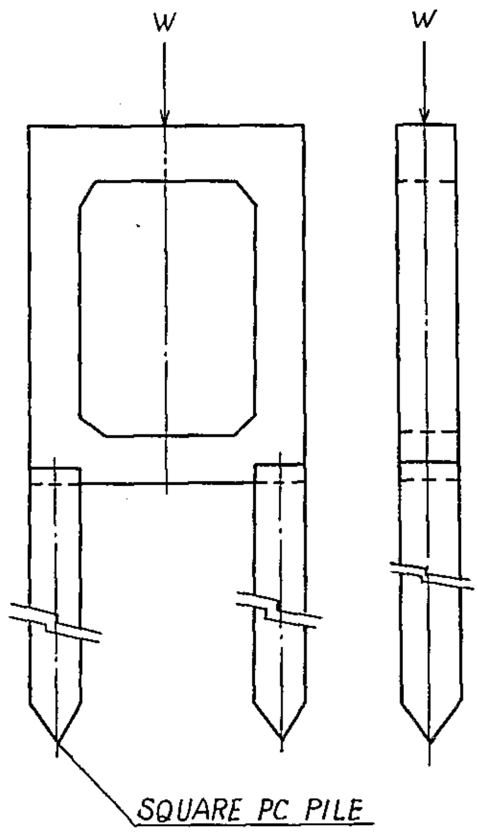
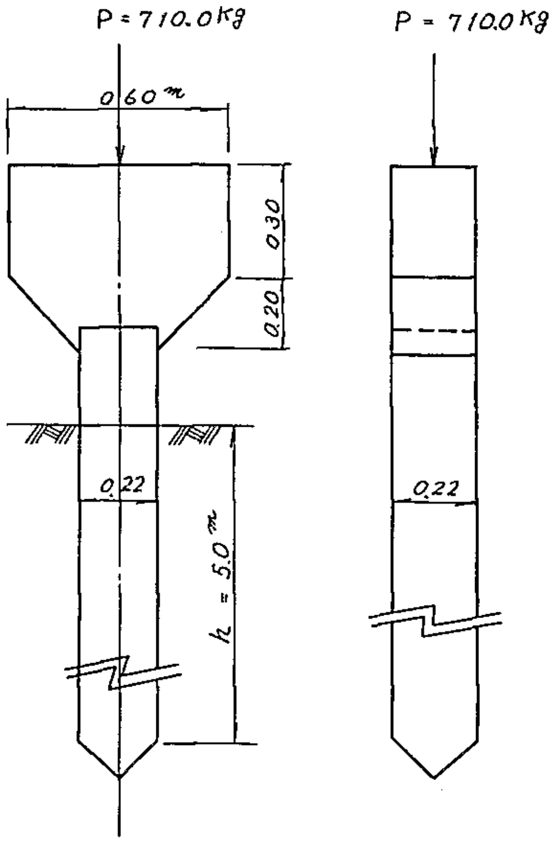



Figure (b) Rigid Frame

2.2 Six-Line Bridge Pier

Calculation of Pile Root Length




a. Load

i. Load from upper structure

Pipe : 12.4 x 6 x 6.0 = 446.4 kg

Cable : 8.7 x 5 x 6.0 = 261.0 kg

Total P = 707.4 kg = 710 kg

ii. Weight upper part of bridge pier

$$(0.6 \times 0.3 + \frac{0.6 + 0.22}{2} \times 0.20) \times 0.22 \doteqdot 0.06 \text{ m}^3$$

$W = 0.06 \times 2{,}400 = 144 \text{ kg} \doteqdot 150 \text{ kg}$

Load on one pile

$P + W = 710 + 150 = 860 \text{ kg}$

b. Pipe Root Length

i. Type of Pile: 0.22 solid square PC pile

ii. Pile root length: 5.0 m (tentative)

iii. Soil: Soft soil

$P + W = 860$ kg

$$R = \frac{A h \overline{F}}{2}$$

$A = 0.22 \times 4 = 0.88$ m

$h = 5.0$ m

$F = 1,470$ kg/m$^2$

A : Circumferential length of pile

h : Pile root length

F : Frection of pile

Based on pile length calculation terms in the case of Chaengwatana Exchange.

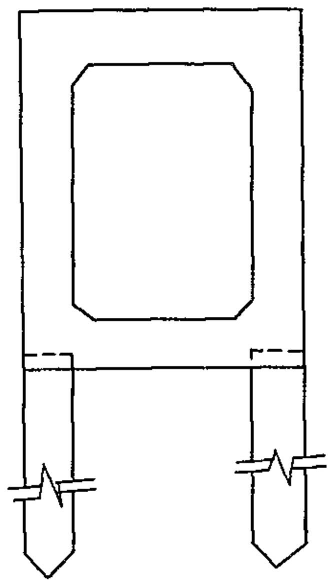
$$R = \frac{0.88 \times 5.0 \times 1,470}{2} = 3,234 \text{ kg}$$

$R = 3,234 \text{ kg} > P + W = 860$ kg

Therefore, the tentatively set pile root length of 5.0 m is sufficient.

## 2.3 Calculation of Pier of Rigid Frame Bridge

Design Conditions



i. For the upper part of the bridge pier, the rigid frame as shown in the illustration is to be adopted.

ii. Concrete should be the type that satisfies the following requirements:

Unit strength

$\sigma'_{ck}$ = 210 kg/cm$^2$

Maximum size of coarse aggregate

25 cm

iii. Unit weight by volume of reinforced concrete

Wc = 2,400 kg/cm$^3$

iv. Allowable stress

Reinforcement

$\sigma'_{sa}$ = 1,400 kg/cm$^2$

Concrete

Bending compressive stress

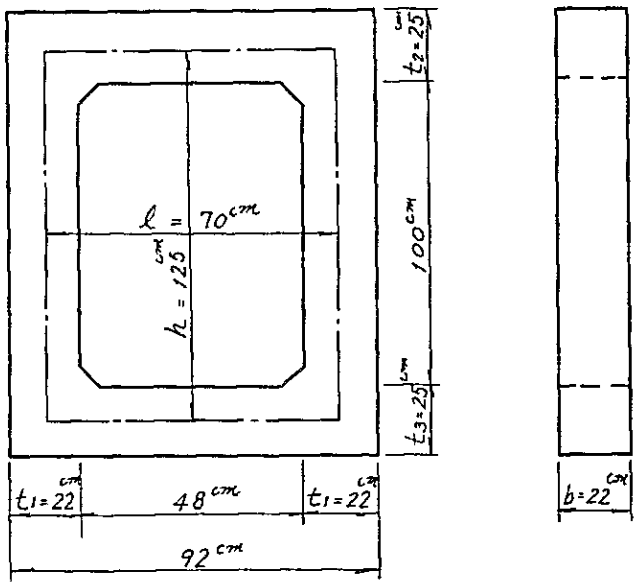
$\sigma_{ca}$ = 70 kg/cm2

Shearing stress

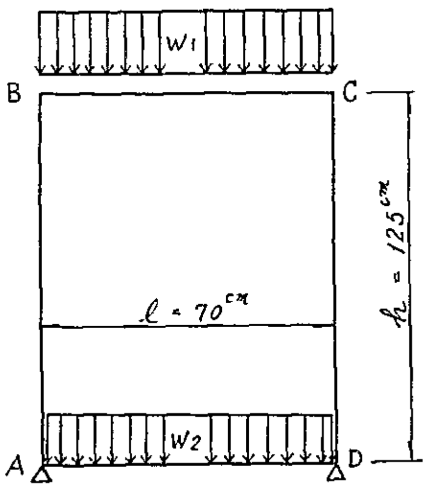
$\tau_a$ = 8 kg/cm$^2$

2.4 Pier Calculation for Nine-Line Bridge

(1) Section




(2) Load




The rigid frame is applied on condition that the load is as shown in the illustration at left.

where

$W_1$ : Load to act on upper member (kg/m)

$W_2$ : Load to act on lower member (kg/m)

$t_1$ : Thickness of vertical member (m)

$t_2$ : Thickness of upper member (m)

$t_3$ : Thickness of lower member (m)

1) Upper Load

| | | |
|---|---|---|
| Pipe | 12.4 x 9 = | 111.6 kg/m |
| Cable | 8.7 x 8 = | 69.6 kg/m |
| Other | | 1.8 kg/m |
| | Total | 183.0 kg/m |

$P = 183.0 \times 10.0 = 1{,}830.0$ kg

$W_P = 1{,}830.0 \div 0.70 = 2{,}615.0$ kg

$W_1 = W_P + 2{,}400 \times t \times b$

$= 2{,}615.0 + 2{,}400 \times 0.25 \times 0.22 = 2{,}747.0$ kg/m

$W_2 = 2{,}400 \times t \times b$

$= 2{,}400 \times 0.25 \times 0.22 = 132$ kg/m

(3) Stress

1) Relative stiffness

$$\alpha = \beta = \left(\frac{t_2}{t_1}\right) \times \frac{h}{\ell} = \left(\frac{0.25}{0.22}\right)^3 \times \frac{1.25}{0.70} = 2.62$$

$n_1 = N_2 = 2 + \alpha = 4.62$

2) Load term

$$C_{BC} = \frac{W_1 \ell^2}{12} = \frac{2747.0 \times 0.7^2}{12} = 112.2 \text{ Kg·m}$$

$C_{BA} = C_{AB} = 0$

$$C_{AD} = -\frac{W_2 \ell^2}{12} = \frac{132.0 \times 0.7^2}{12} = -5.39 \text{ Kg·m}$$

3) Angle of rotation of joint

$$\theta_A = \frac{N_1(C_{AB}-C_{AD})-(C_{BC}-C_{BA})}{N_1 N_2 - 1}$$

$$= \frac{4.62(0+5.39)-(112.2-0)}{4.62^2-1} = -4.29\ \mathrm{Kg\cdot m}$$

$$\theta_B = \frac{N_2(C_{BC}-C_{BA})-(C_{AB}-C_{AD})}{N_1 N_2 - 1}$$

$$= \frac{4.62(112.2-0)-(0+5.39)}{4.62^2-1} = 25.21\ \mathrm{Kg\cdot m}$$

4) End moment

$$M_{AB} = 2\theta_A + \theta_B - C_{AB}$$

$$= 2 \times (-4.29) + 25.21 - 0 = 16.63\ \mathrm{kg.m}$$

$$M_{AD} = \beta\theta_A + C_{AD}$$

$$= 2.62 \times (-4.29) + (-5.39) = -16.63\ \mathrm{kg.m}$$

$$M_{BA} = 2\theta_B + \theta_A + C_{BA}$$

$$= 2 \times 25.21 + (-4.29) + 0 = 46.13\ \mathrm{kg.}$$

$$M_{BC} = \alpha\theta_B - C_{BC}$$

$$= 2.62 \times 25.21 - 112.2 = -46.13\ \mathrm{kg.m}$$

5) Shearing force and axial force

$$N_{AB} = \frac{W_1 \ell}{2} + 2{,}400 \times b \cdot t_1 \cdot x$$

$$= \frac{2747.0 \times 0.7}{2} + 2{,}400 \times 0.22 \times 0.22x$$

$$= 961.45 + 116.16\,x$$

$$N_{BC} = -S_{AB} = 0$$

$$N_{AD} = S_{AB} = 0$$

$$S_{BC} = \frac{W_1 \ell}{2} = \frac{2747.0 \times 0.7}{2} = 961.45\ \mathrm{kg}$$

$$S_{AD} = \frac{W_2 \ell}{2} = -\frac{132.0 \times 0.7}{2} = -46.2\ \mathrm{kg}$$

6) Maximum bending moment

$$M_{E2} = \frac{W_1 \ell^2}{8} - M_{BC}$$

$$= \frac{2747.0 \times 0.7^2}{8} - (-46.13) = 214.4 \text{ kg·m}$$

$$M_{E3} = \frac{-W_2 \ell^2}{8} - M_{AD}$$

$$= \frac{-132.0 \times 0.7^2}{8} - (-16.63) = 8.55 \text{ kg·m}$$

7) Inflection point

Member AD

No inflection point

Member AB

$$X_1 = \frac{h}{\dfrac{M_{BA} + M_{AB}}{M_{AB}}} = \frac{1.25}{\dfrac{46.13+16.63}{16.63}} = 0.33\,m$$

Member BC

$$X_2 = \frac{S_{BC} \pm \sqrt{S_{BC}^2 + 2W_1 + M_{BC}}}{W_1}$$

$$= \frac{961.45 \pm \sqrt{961.45^2 + 2 \times 2747.0 \times (-46.13)}}{}$$

$$= 0.65\,cm \text{ or } 0.05\,cm$$

8) Bending moment diagram and shearing force diagram

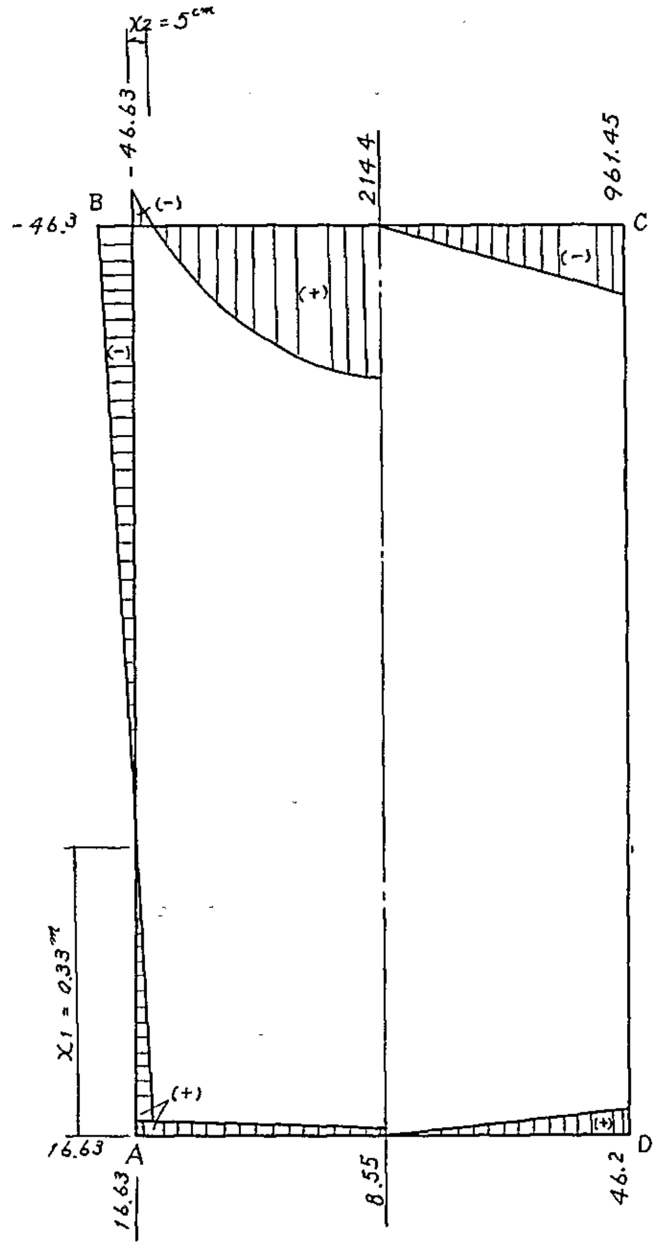


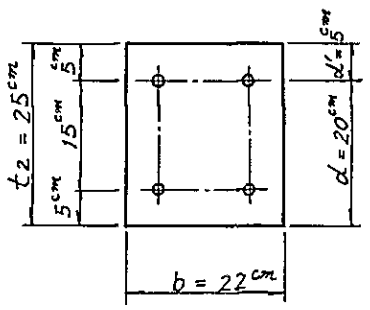

Figure-M (kg/m)

Figure-S (kg)

(4) Calculation of Section

The calculation of section is made by the bending moment in the central part of Member BC. The value obtained applies to the total cross section. The shearing force is calculated at the edge of Member BC.

1) Assumption of section




2) Calculation of volume of reinforcement

$$A_S = \frac{M}{\sigma_{sa} \cdot j \cdot d} \qquad \text{provided } j = 7/8$$

$$= \frac{21440}{1400 \times 7/8 \times 20} = 0.9\ cm^2$$

When two pieces of ø9 mm reinforcement are used, the tension side volume of reinforcement is

$$A_S = 0.6362 \times 2 = 1.2724\ cm^2$$

On the compression side also the reinforcement is used so that the compression side volume of re-inforcement is

$$A_S' = 1.2724\ cm^2$$

The study is made on the basis of such double reinforcement beam.

3) Calculation of neutral axis

$$x = -\frac{n(As+A's)}{b}+\sqrt{\left\{\frac{n(As+A's)}{b}\right\}^2+\frac{2n}{b}(As\,d+A's\cdot d')}$$

$$= -\frac{15\times1.2724\times2}{22}+\sqrt{\left\{\frac{15\times1.2724\times2}{22}\right\}^2+\frac{2\times15}{22}\times1.2724(20+5)}$$

= 8.546 cm

4) Study of stress

a) Compressive stress

$$\sigma_c = \frac{M}{\frac{bx}{2}(d-\frac{x}{3})+nA's\frac{(x-d')}{x}(d-d')}$$

$$= \frac{M}{\frac{22\times8.546}{2}(20-\frac{8.546}{3})+15\times1.2724\times(\frac{8.546-5}{8.546})(20-5)}$$

$$= 11.81\ kg/cm^2 < \sigma_{ca} = 70\ kg/cm^2 \quad \therefore OK$$

b) Tensile stress

$$\sigma_s = \frac{n\,\sigma_c(d-x)}{x}$$

$$= \frac{15\times11.81(20-8.546)}{8.546} = 237.4\ kg/cm^2 < \sigma_{sa} = 1400\ kg/cm^2$$

$\therefore$ OK

c) Shearing stress

$$\tau = \frac{961.45}{22(20-\frac{8.546}{3})} = 2.55\ kg/cm^2 < \tau_a = 8\ kg/cm^2$$

$\therefore$ OK

Therefore, the tentatively set ø9 mm reinforcement provides the required safety.

(5) Calculation of Pile Root Length

a) Load

i) Load from upper structure

$P_1$ = 1,830 kg

ii) Weight of upper part of bridge pier

From Figure 1:

$$W = (0.92 \times 1.50 - 0.48 \times 1.0) \times 0.22 \times 2,400$$
$$= 475.2 \text{ kg}$$

Load on one pile

$$1,830 + 475.2 \doteqdot 2,310 \text{ kg}$$

$$\therefore \; P = 2,310 \div 2 = 1,155 \text{ kg}$$

b) Pile root length

i. Type of pile: 0.22 solid square PC pile

ii. Pile root length: 6.0 m (tentative)

iii. Soil: Soft soil

$$P = 1,155 \text{ kg}$$

$$R = \frac{A\ h\ F}{2}$$

$$= \frac{0.88 \times 6.0 \times 1,470}{2}$$

$$= 3,881 \text{ kg} > 1,155 \text{ kg}$$

$$A = 0.22 \times 4 = 0.88 \text{ m}$$

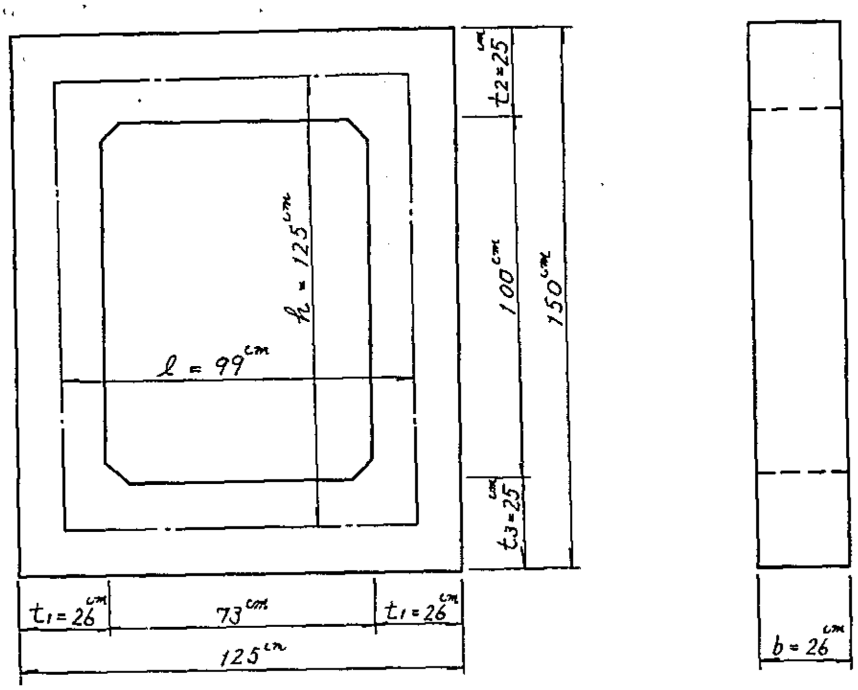
$$h = 6.0 \text{ m}$$

$$F = 1,470 \text{ kg/m}^2$$

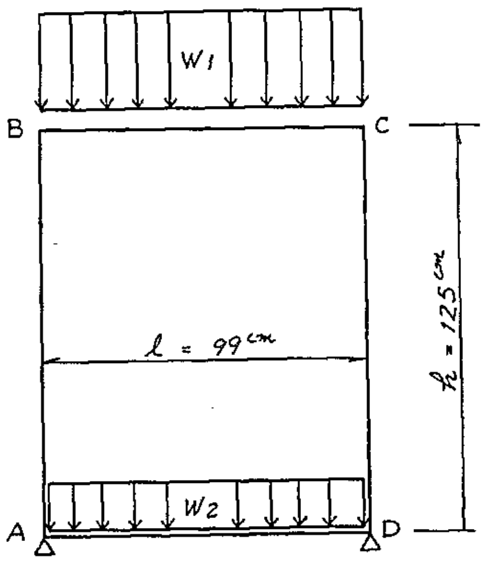
Therefore, the tentatively set 6.0 m pile root length is sufficient.

2.5 Pier Calculation for 24-Line (6 lines x 4 steps) Bridge

(1) Section




(2) Load




The rigid frame is applied on condition that the load is as shown in the illustration at left.

where

$W_1$ : Load to act on upper member (kg/m)

$W_2$ : Load to act on lower member (kg/m)

$t_1$ : Thickness of vertical member (m)

$t_2$ : Thickness of upper member (m)

$t_3$ : Thickness of lower member (m)

1) Upper Load

| | | |
|---|---|---|
| Pipe | 12.4 x 24 = | 297.6 kg/m |
| Cable | 0.7 x 23 = | 200.1 kg/m |
| Other | | 2.3 kg/m |
| | Total | 500.0 kg/m |

$$P = 500.0 \text{ kg/m} \times 10.0 \text{ m} = 5{,}000.0 \text{ kg}$$

$$W_P = 5{,}000.0 \text{ kg}/0.99 \text{ m} = 5{,}050.5 \text{ kg/m}$$

$$W_1 = W_P + 2{,}400 \text{ kg/m}^3 \times t_2 \times b$$

$$= 5{,}050.5 + 2{,}400 \times 0.25 \times 0.26 = 5{,}206.5 \text{ kg/m}$$

$$W_2 = 2{,}400 \text{ kg/m}^3 \times t_3 \times b$$

$$= 2{,}400 \times 0.25 \times 0.26 = 156.0 \text{ kg/m}$$

(3) Stress

1) Relative stiffness

$$\alpha = \beta = \left(\frac{t_2}{t_1}\right)^3 \times \frac{h}{l}$$

$$= \left(\frac{0.25}{0.26}\right)^3 \times \frac{1.25}{0.99} = 1.122$$

$$N_1 = N_2 = 2 + \alpha = 3.122$$

2) Load term

$$C_{BC} = \frac{W_1 l^2}{12} = \frac{5206.5 \times 0.99^2}{12} = 425.2 \text{ kg·m}$$

$$C_{BA} = C_{AB} = 0$$

$$C_{AD} = \frac{-W_2 l^2}{12} = \frac{-156.0 \times 0.99^2}{12} = -12.7 \text{ kg·m}$$

3) Angle of rotation of joint

$$\theta_A = \frac{N_1(C_{AB} - C_{AD}) - (C_{BC} - C_{BA})}{N_1 \cdot N_2 - 1}$$

$$= \frac{3.122(0+12.7) - (425.2-0)}{3.122^2 - 1}$$

$$= -44.1 \text{ kg·m}$$

$$\theta_B = \frac{N_2(C_{BC} - C_{BA}) - (C_{AB} - C_{AD})}{}$$

$$= \frac{3.122(425.2-0) - (0+12.7)}{3.122^2 - 1}$$

$$= 150.3 \text{ kg·m}$$

4) End moment

$$M_{AB} = 2\theta_A + \theta_B - C_{AB}$$

$$= 2 \times (-44.1) + 150.3 - 0$$

$$= 62 \text{ kg·m}$$

$$M_{AD} = \beta \cdot \theta_A + C_{AD}$$

$$= 1.122 \times (-44.1) + (-12.7)$$

$$= -62 \text{ kg·m}$$

$$M_{BA} = 2\theta_B + \theta_A + C_{BA}$$

$$= 2 \times 150.3 - 44.1 + 0$$

$$= 257 \text{ kg·m}$$

$$M_{BC} = \alpha \cdot \theta_B - C_{BC}$$

$$= 1.122 \times 150.3 - 425.2$$

$$= -257 \text{ kg·m}$$

5) Shearing force and axial force

$$N_{AB} = \frac{W_1 \cdot \ell}{2} + 2400 \times b \cdot t_1 \cdot x$$

$$= \frac{5206.5 \times 0.99}{2} + 2400 \times 0.26 \times 0.26x$$

$$= 2577.2 + 162.2x$$

$N_{BC} = -S_{BA} = 0$

$N_{AD} = S_{AB} = 0$

$$S_{BC} = \frac{W_1 \cdot \ell}{2} = \frac{5206.5 \times 0.99}{2}$$

$$= 2577.2\ \text{Kg}$$

$$S_{AD} = \frac{-W_2 \cdot \ell}{2} = \frac{-156.0 \times 0.99}{2}$$

$$= -77.2\ \text{Kg}$$

6) Maximum bending moment

$$M_{E2} = \frac{W_1 \cdot \ell^2}{8} - M_{BC} = \frac{5206.5 \times 0.99^2}{8} + 257$$

$$= 895\ \text{Kg}\cdot\text{m}$$

$$M_{E3} = \frac{-W_2 \cdot \ell^2}{8} - M_{AD} = \frac{-156.0 \times 0.99^2}{8} + 62$$

$$= 43\ \text{Kg}\cdot\text{m}$$

7) Inflection point

Member AD

No inflection point

Member AB

$$x_1 = \frac{h}{\frac{M_{BA} + M_{AB}}{M_{AB}}} = \frac{1.25}{\frac{257+62}{62}} = 0.24\text{m}$$

Member BC

$$x_2 = \frac{S_{BC} \pm \sqrt{S_{BC}^2 + 2W_1 \cdot M_{BC}}}{W_1}$$

$$= \frac{2577.2 \pm \sqrt{2577.2^2 + 2 \times 5206.5 \times (-257)}}{5206.5}$$

$$= 0.11\ \text{m} \quad \text{or} \quad 0.88\ \text{m}$$

8) Bending moment diagram and shearing force diagram

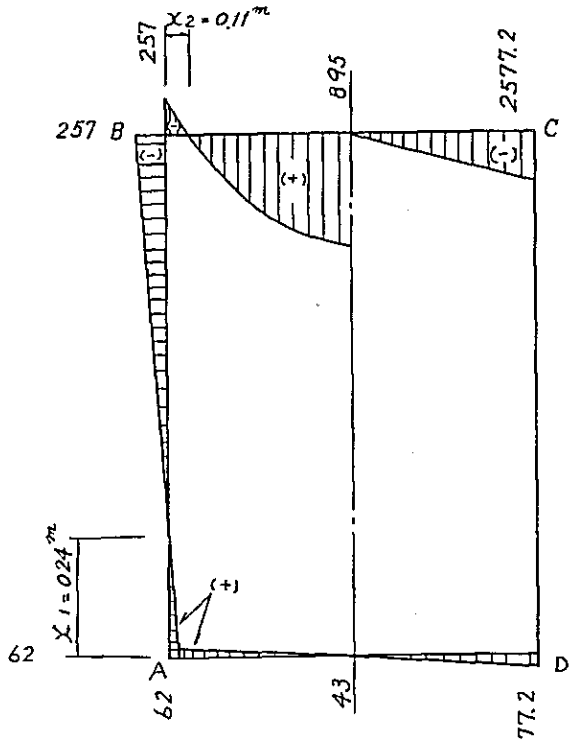
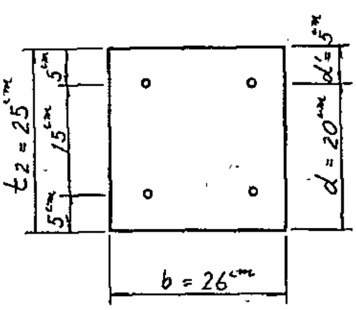



Figure-M (Kg/m) Figure-S (Kg)

(4) Calculation of Section

The calculation of section is made by the bending moment in the central part of Member BC. The value obtained applies to the total cross section. The shearing force is calculated at the edge of Member BC.

1) Assumption of section



2) Calculation of volume of reinforcement

$$A_S = \frac{M}{\sigma_{sa} \cdot j \cdot d} \qquad \text{provided } j = 7/8$$

$$= \frac{89500}{1400 \times (7/8) \times 20}$$

$$= 3.65 \text{ cm}^2$$

When two pieces of ø16 mm reinforcement are used, the tension side volume of reinforcement is

$$A_S = 2.011 \times 2 = 4.002 \text{ cm}^2$$

On the compression side also the reinforcement is used so that the compression side volume of reinforcement is

$$A_S' = 4.002 \text{ cm}^2$$

The study is made on the basis of such double reinforcement beam.

3) Calculation of neutral axis

$$x = -\frac{n(A_s + A_s')}{b} + \sqrt{\left\{\frac{n(A_s+A_s')}{b}\right\}^2 + \frac{2n}{b}(A_s d + A_s' \cdot d')}$$

$$= -\frac{15 \times 2 \times 4.022}{26} + \sqrt{\left\{\frac{15 \times 2 \times 4.022}{26}\right\}^2 + \frac{2 \times 15 \times 4.022}{26}(20+5)}$$

$$= 7.088 \text{ cm}$$

4) Study of stress

a) Compressive stress

$$\sigma_c = \frac{M}{\frac{bx}{2}(d - \frac{x}{3}) + nA_s' \frac{(x-d')}{x}(d-d')}$$

$$= \frac{89500}{\frac{26 \times 7.088}{2}(20 - \frac{7.088}{3}) + 15 \times 4.022 \frac{(7.088-5)}{7.088}(20-5)}$$

$$= 47.3 \text{ Kg/cm}^2 < \sigma_{ca} = 70 \text{ Kg/cm}^2$$

$\therefore$ OK

b) Tensile stress

$$\sigma_s = \frac{n\,\sigma_c (d - x)}{x}$$

$$= \frac{15 \times 47.3 (20 - 7.088)}{7.088}$$

$$= 1292.5\ \mathrm{kg/cm^2} < \sigma_{sa} = 1400\ \mathrm{kg/cm^2} \quad \therefore \mathrm{OK}$$

c) Shearing stress

$$\tau = \frac{S}{b\left(d - \frac{x}{3}\right)}$$

$$= \frac{2577.2}{26\left(20 - \frac{7.088}{3}\right)}$$

$$= 5.6\ \mathrm{kg/cm^2} < \tau_a = 8\ \mathrm{kg/cm^2} \quad \therefore \mathrm{OK}$$

Therefore, the tentatively set ø16 mm reinforcement provides the required safety.

(5) Calculation of Pile Root Length

a) Load

i) Load from upper structure

$P_1 = 5{,}000$ kg

ii) Weight of upper part of bridge pier

From Figure 1:

$W = (1.25 \times 1.50 - 0.73 \times 1.0) \times 0.26 \times 2{,}400 = 715$ kg

$= 715$ kg

Load on one pile

$5{,}000 + 715 = 5{,}715$ kg

$P = 5{,}715 \div 2 \fallingdotseq 2{,}860$ kg

b) Pile root length

i) Type of pile: 0.26 solid square PC pile

ii) Pile root length: 10.0 m (tentative)

iii) Soil: Soft soil

$$P = 2,860 \text{ kg}$$

$$R = \frac{A\ h\ F}{2}$$

$$= \frac{1.04 \times 10.0 \times 1,800}{2}$$

$$= 9,360 \text{ kg} > 2,860 \text{ kg}$$

$$A = 0.26 \times 4 = 1.04 \text{ m}$$

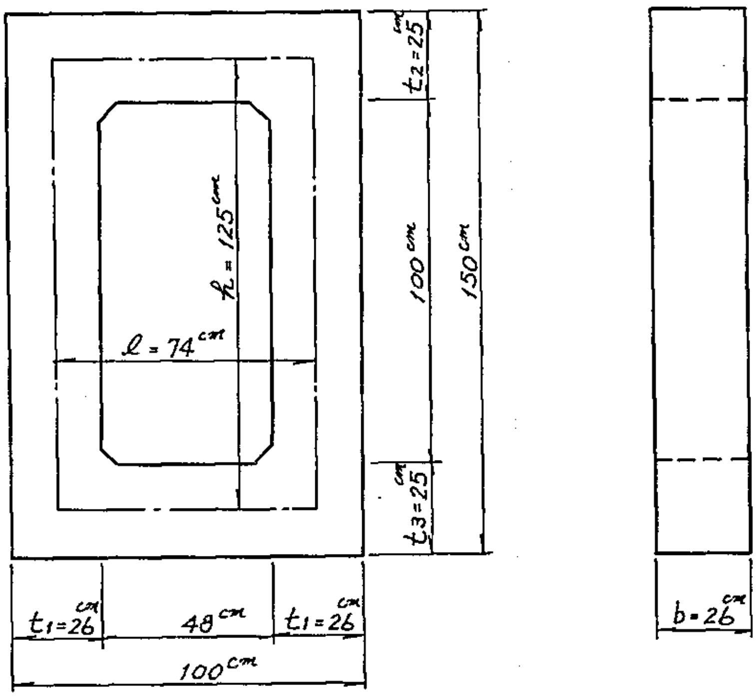
$$h = 10.0 \text{ m}$$

$$F = 1,800 \text{ kg/m}^2$$

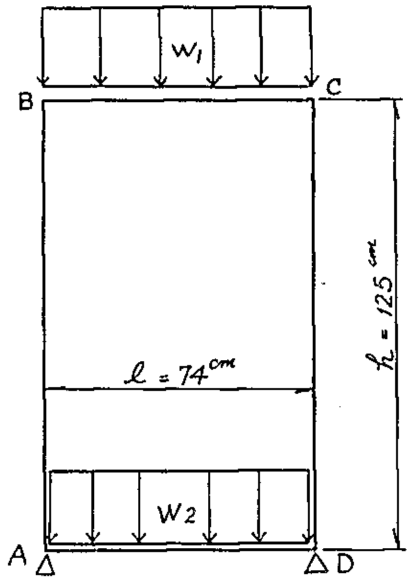
Therefore, the tentatively set 10.0 m pile root length is sufficient.

## 2.6 Pier Calculation for 24-Line (4 lines x 6 steps) Bridge

(1) Section



(2) Load




The rigid frame is applied on condition that the load is as shown in the illustration at left.

where

$W_1$ : Load to act on upper member (kg/m)

$W_2$ : Load to act on lower member (kg/m)

$t_1$ : Thickness of vertical member (m)

$t_2$ : Thickness of upper member (m)

$t_3$ : Thickness of lower member (m)

1) Upper Load

| | | |
|---|---|---|
| Pipe | 12.4 x 24 = | 297.6 kg/m |
| Cable | 8.4 x 23 = | 200.1 kg/m |
| Other | | 2.3 kg/m |
| | Total | 500.0 kg/m |

$P = 500.0$ kg/m x 10.0 m = 5,000 kg

$W_P = 5{,}000$ kg/0.74 m = 6,756.8 kg/m

$W_1 = W_P + 2{,}400 \times t_2 \times 0.26$

$= 6{,}756.8 + 2{,}400 \times 0.25 \times 0.26 = 6{,}912.8$ kg/m

$W_2 = 2{,}400 \times t_3 \times 0.26$

$= 2{,}400 \times 0.25 \times 0.26 = 156.0$ kg/m

(3) Stress

1) Relative stiffness

$$\alpha = \beta = \left(\frac{t_1}{t_2}\right)^3 \times \frac{h}{\ell}$$

$$= \left(\frac{0.25}{0.26}\right)^3 \times \frac{1.25}{0.74} = 1.502$$

$N_1 = N_2 = 2 + \alpha = 3.502$

2) Load term

$$C_{BC} = \frac{W_1 \ell^2}{12} = \frac{6912.8 \times 0.74^2}{12} = 315.5 \text{ kg}\cdot\text{m}$$

$$C_{BA} = C_{AB} = 0$$

$$C_{AD} = \frac{-W_2 \ell^2}{12} = \frac{-156.0 \times 0.74^2}{12} = -7.12 \text{ kg}\cdot\text{m}$$

3) Angle of rotation of joint

$$\theta_A = \frac{N_1 (C_{AB} - C_{AD}) - (C_{BC} - C_{BA})}{N_1 \cdot N_2 - 1}$$

$$= \frac{3.502(0 - 7.12) - (315.5 - 0)}{3.502^2 - 1}$$

$$= -25.8 \text{ kg.m}$$

$$\theta_B = \frac{N_2 (C_{BC} - C_{BA}) - (C_{AB} - C_{AD})}{N_1 \cdot N_2 - 1}$$

$$= \frac{3.502(315.5 - 0) - (0 + 7.12)}{3.502^2 - 1}$$

$$= 97.46 \text{ kg.m}$$

4) End moment

$$M_{AB} = 2\theta_A + \theta_B - C_{AB}$$

$$= 2 \times (-25.8) + 97.46 - 0$$

$$= 45.86 \text{ kg.m}$$

$$M_{AD} = \beta\theta_A + C_{AD}$$

$$= 1.502 \times (-25.8) - 7.12$$

$$= -45.86 \text{ kg.m}$$

$$M_{BA} = 2\theta_B + \theta_A + C_{BA}$$

$$= 2 \times 97.46 + (-25.8) + 0$$

$$= 169.12 \text{ kg.m}$$

$$M_{BC} = \alpha \cdot \theta_B - C_{BC}$$

$$= 1.502 \times 97.46 - 315.5$$

$$= -169.12 \text{ kg.m}$$

5) Shearing force and axial force

$$N_{AB} = \frac{W_1 \cdot \ell}{2} + 2400 \times 0.26 \times t_1 \times x$$

$$= \frac{6912.8 \times 0.74}{2} + 2400 \times 0.26 \times 0.26\, x$$

$$= 2557.7 + 162.2\, x$$

$$N_{BC} = -S_{BA} = 0$$

$$N_{AD} = S_{AB} = 0$$

$$S_{BC} = \frac{W_1 \cdot \ell}{2}$$

$$= \frac{6912.8 \times 0.74}{2}$$

$$= 2{,}557.7 \text{ kg}$$

$$S_{AD} = \frac{W_2 \cdot \ell}{2}$$

$$= \frac{156.0 \times 0.74}{2}$$

$$= 57.7 \text{ kg}$$

6) Maximum bending moment

$$M_{E2} = \frac{W_1 \cdot \ell^2}{8} - M_{BC}$$

$$= \frac{6912.8 \times 0.74^2}{8} + 169.1$$

$$= 642.3 \text{ kg.m}$$

$$M_{E3} = \frac{-W_2 \cdot \ell^2}{8} - M_{AD}$$

$$= \frac{-156 \times 0.74^2}{8} + 45.9$$

= 35.2 kg.m

7) Inflection point

Member AD

No inflection point

Member AB

$$x_1 = \frac{h}{\frac{M_{BA}+M_{AB}}{M_{AB}}} = \frac{1.25}{\frac{169.1+45.9}{45.9}} = 0.27\ m$$

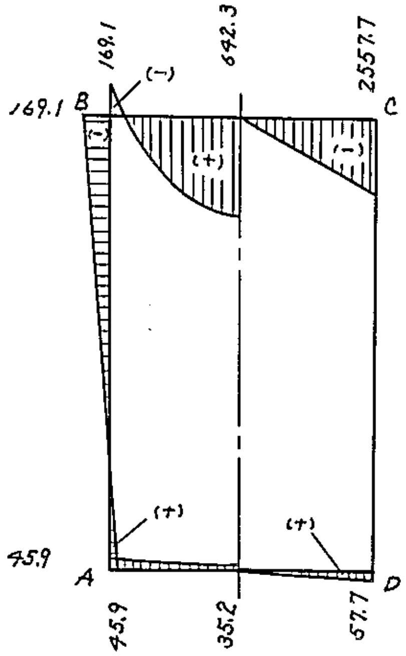
Member BC

$$x_2 = \frac{S_{BC} \pm \sqrt{S_{BC}^2 + 2W_1 \cdot M_{BC}}}{W_1}$$

$$= \frac{2557.7 \pm \sqrt{2557.7^2 + 2 \times 6912.8 \times (-169.1)}}{6912.8}$$

= 0.667 m or 0.073 m

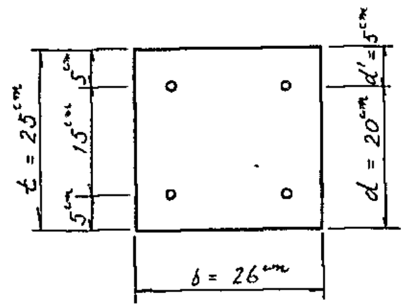
8) Bending moment diagram and shearing force diagram



(4) Calculation of Section

The calculation of section is made by the bending moment in the central part of Member BC. The value obtained applies to the total cross section. The sharing force is calculated at the edge of Member BC.

1) Assumption of section




b = 26 cm

d = 20 cm

d' = 5 cm

2) Calculation of volume of reinforcement

$$A_S = \frac{M}{\sigma_{SA} \cdot j \cdot d} = \frac{64230}{1400 \times 7/8 \times 20} = 2.62\ cm^2$$

Sectional area per piece of ϕ16 mm reinforcement

2.011 $cm^2$

Tension side volume of reinforcement

2 pcs x 2.011 = 4.022 $cm^2$

Compression side volume of reinforcement

2 pcs x 2.011 = 4.022 $cm^2$

3) Calculation of neutral axis

$$x = -\frac{n(As+As')}{b} + \sqrt{\left\{\frac{n(As+As')}{b}\right\}^2 + \frac{2n}{b}(As \cdot d + As' \cdot d')}$$

$$= -\frac{15 \times 2 \times 4.022}{26} + \sqrt{\left\{\frac{15 \times 2 \times 4.022}{26}\right\}^2 + \frac{2 \times 15}{26} \times 4.022(20+5)}$$

$$= 7.088\ cm$$

4) Study of stress

a) Compressive stress

$$\sigma_c = \frac{M}{\frac{bx}{2}(d-\frac{x}{3}) + n \cdot As' \frac{(x-d')}{x} \cdot (d-d')}$$

$$= \frac{M}{\frac{26 \times 7.088}{2}(20-\frac{7.088}{3}) + \frac{15 \times 4.022(7.088-5)(20-5)}{7.088}}$$

$$= 33.95 \, kg/cm^2 < \sigma_{ca} = 70 \, kg/cm^2 \qquad \therefore OK$$

b) Tensile stress

$$\sigma_s = \frac{n \sigma_c (d-x)}{x} = \frac{15 \times 33.95(20-7.088)}{7.088}$$

$$= 927.7 \, kg/cm^2 < \sigma_{sa} = 1400 \, kg/cm^2 \qquad \therefore OK$$

c) Shearing stress

$$\tau = \frac{S}{b(d-\frac{x}{3})} = \frac{2557.7}{26(20-\frac{7.088}{3})}$$

$$= 5.57 \, kg/cm^2 < \tau_a = 8 \, kg/cm^2 \qquad \therefore OK$$

Therefore, the tentatively set ϕ16 mm reinforcement provides the required safety.

(5) Calculation of Pile Root Length

a) Load

i) Load from upper structure

$P_1$ = 5,000 kg

ii) Weight of abutment

From Figure 1:

W = (1.0 x 1.5 - 0.48 x 1.0) x 0.26 x 2,400 = 637 kg

Load on one pile

P = (5,000 + 637) ÷ 2 = 2,820 kg

Therefore, as in the case of 6 lines x 4 steps pier, 0.26 solid square PC pile is adopted and the pile root length is set at 10.0 m.

Bridge Strength Calculations

(Onnut-1 Exchange)

In the Onnut-1 Exchange underground conduit section, two kinds of bridges are used: the bridge with 9 pipe lines and the bridge with 12 pipe lines.

In the following pages, the strength calculations are made for the bridge with 12 pipe lines.

For the strength calculations of the bridge with 9 pipe lines, refer to the corresponding statement in the case of the Ramindra Exchange.

1. Design Calculations for 12-Line Combination Type GIP Bridge

($\phi$12 mm iron bars welded around the pipe).

a. Design Conditions

i. Interval between piers 13.8 m

ii. Weight per m

Pipe (GIP $\phi$4") 12.4 x 12 = 148.8 kg/m

Cable (0.4 - 2400 P) 8.7 x 11 = 95.7 kg/m

Total 244.5 kg/m

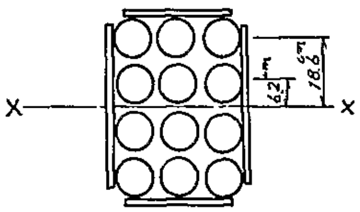
b. Moment of Inertia

Ix : Total moment of inertia in case of combined pipes

Io : Moment of inertia of one pipe 232.6 $cm^4$

A : Sectional area of one pipe

$$(D^4 - d^4) \times \frac{\pi}{4} = (11.41^2 - 10.51^2) \times \frac{\pi}{4} = 15.49\ cm^2$$




| Io + | A x | $Y^2$ = | Ix $cm^4$ |
|---|---|---|---|
| 232.6 x 6 = 1,395.6 | 15.49 x 6 = 92.94 | $18.6^2$ = 345.96 | 33,549.1 |
| 232.6 x 4 = 930.4 | 15.49 x 4 = 61.96 | $6.2^2$ = 38.44 | 3,312.14 |
| Total | | | 36,861.2 |

c. Calculation of Deflection

$$Y = \frac{5 W \ell^4}{384 EI} = \frac{5 \times 2.445 \times 1380^4}{384 \times 2.1 \times 10^6 \times 36861.2 \times 0.7} = 2.13 \text{ cm}$$

$$Y = 2.13 \text{ cm} < \frac{\ell}{300} = \frac{1380}{300} = 4.6 \text{ cm}$$

Therefore, safe.

d. Maximum bending moment

$$M_{max} = \frac{W \ell^2}{8} = \frac{2.445 \times 1380^2}{8} = 582032.3 \text{ kg-cm}$$

e. Stress

$$Z = \frac{I_x}{\Sigma y} = \frac{36861.2 \times 0.7}{18.6 + 6.2} = 1040.4 \text{ cm}^3$$

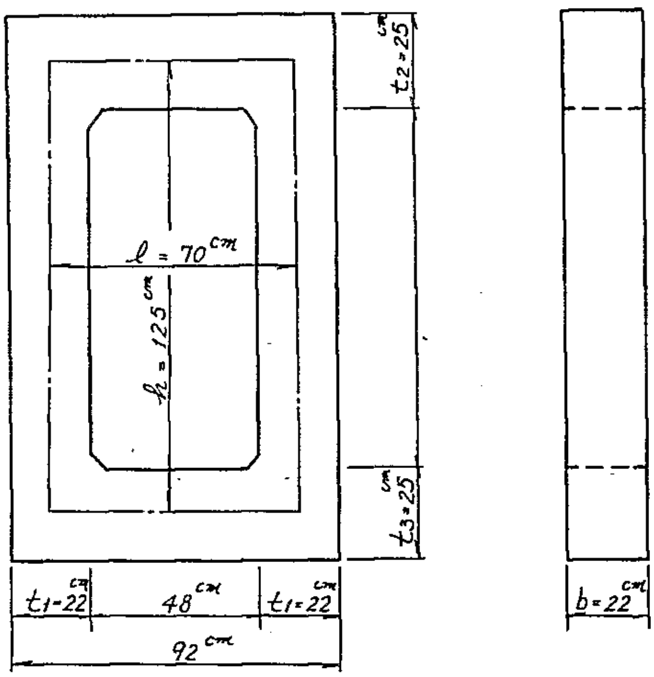
$$\sigma = \frac{M}{Z} = \frac{582032.3}{1040.4} = 559.4 \text{ kg/cm}^2 < \sigma_{ta} = 1400 \text{ kg/cm}^2$$

Therefore, safe.

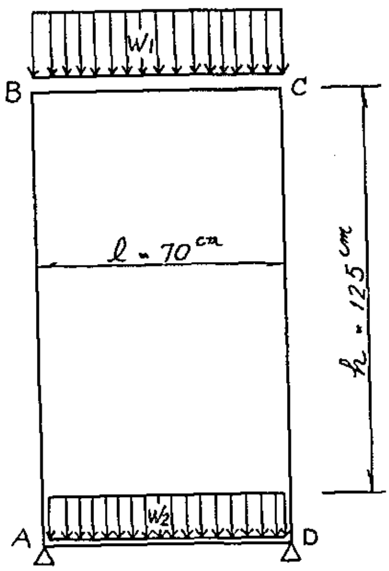
2. Design Calculations for 12-Line Bridge Pier

Design Conditions are the same as those of calculations for the rigid frame bridge pier in the case of Ramindra Exchange.

(1) Section



(2) Load




The rigid frame is applied on condition that the load is as shown in the illustration at left.

where

$W_1$ : Load to act on upper member (kg/m)

$W_2$ : Load to act on lower member (kg/m)

$t_1$ : Thickness of vertical member (m)

$t_2$ : Thickness of upper member (m)

$t_3$ : Thickness of lower member (m)

1) Upper load

| | | |
|---|---|---|
| Pipe | 12.4 x 12 = | 148.8 kg/m |
| Cable | 8.7 x 11 = | 95.7 kg/m |
| Other | | 2.5 kg/m |
| | Total | 247.0 kg/m (span: $\frac{13.8 + 7.0}{2} \doteqdot 10.4$) |

$P = 247 \times 10.4 \doteqdot 2{,}569.0$ kg

$W_p = 2{,}569.0 / 0.70 \doteqdot 3{,}670.0$ kg/m

$W_1 = 3{,}670.0 + 2{,}400 \times 0.25 \times 0.22 = 3{,}802$ kg/m

$W_2 = 2{,}400 \times t_3 \times b$

$= 2{,}400 \times 0.25 \times 0.22 = 132$ kg/m

(3) Stress

1) Relative stiffness

$$\alpha = \beta = \left(\frac{t_2}{t_1}\right)^3 \times \frac{h}{\ell}$$

$$= \left(\frac{0.25}{0.22}\right)^3 \times \frac{1.25}{0.70} = 2.62$$

$N_1 = N_2 = 2 + \alpha = 4.62$

2) Load term

$$C_{BC} = \frac{W_1 \ell^2}{12} = \frac{3802 \times 0.7^2}{12} = 155.25 \text{ kg·m}$$

$C_{BA} = C_{AB} = 0$

$$C_{AD} = -\frac{W_2 \ell^2}{12} = \frac{-132.0 \times 0.7^2}{12} = -5.39 \text{ kg·m}$$

3) Angle of rotation of joint

$$\theta_A = \frac{N_1(C_{AB} - C_{AD}) - (C_{BC} - C_{BA})}{N_1 N_2 - 1}$$

$$= \frac{4.62(0+5.39) - (155.25-0)}{4.62^2 - 1}$$

$= -6.41$ kg.m

$$\theta_B = \frac{N_2(C_{BC} - C_{BA}) - (C_{AB} - C_{AD})}{N_1 N_2 - 1}$$

$$= \frac{4.62(155.25-0) - (0+5.39)}{4.62^2 - 1}$$

$= 34.99$ kg.m

4) End moment

$M_{AB} = 2\theta_A + \theta_B - C_{AB}$

$= -2 \times 6.41 + 34.99 = 22.2$ kg.m

$M_{AD} = \beta\theta_A + C_{AD}$

$= 2.62 \times (-6.41) + (-5.39) \doteqdot -22.2$ kg.m

$M_{BA} = 2\theta_B + \theta_A + C_{BA}$

$= 2 \times 34.99 + (-6.41) + 0 = 63.6$ kg.m

$$M_{BC} = \alpha\,\theta_B - C_{BC}$$

$$= 2.62 \times 34.99 - 155.25 = -63.6 \text{ kg.m}$$

5) Shearing force and axial force

$$N_{AB} = \frac{W_1 \ell}{2} + 2400 \times b \cdot t_1 \cdot x$$

$$= \frac{3{,}802 \times 0.7}{2} + 2{,}400 \times 0.22 \times 0.22 \cdot x$$

$$= 1{,}330.7 + 116.16 \cdot x$$

$$N_{BC} = -S_{BA} = 0$$

$$N_{AD} = S_{AB} = 0$$

$$S_{BC} = \frac{W_1 \ell}{2} = \frac{3802 \times 0.7}{2} = 1330.7 \text{ kg}$$

$$S_{AD} = \frac{W_2 \ell}{2} = -\frac{132.0 \times 0.7}{2} = -46.2 \text{ kg}$$

6) Maximum bending moment

$$M_{E2} = \frac{W_1 \ell^2}{8} - M_{BC} = \frac{3802 \times 0.7^2}{8} + 63.6 = 296.5 \text{ kg}\cdot\text{m}$$

$$M_{E3} = -\frac{W_2 \ell^2}{8} - M_{AD} = \frac{-132 \times 0.7^2}{8} + 22.2 = 14.1 \text{ kg}\cdot\text{m}$$

7) Inflection point

Member AD

No inflection point

Member AB

$$x_1 = \frac{h}{\frac{M_{BA} + M_{AB}}{M_{AB}}} = \frac{1.25}{\frac{63.6 + 22.2}{22.2}} = 0.32 \text{ cm}$$

Member BC

$$x_2 = \frac{S_{BC} \pm \sqrt{S_{BC}^2 + 2W_1 \cdot M_{BC}}}{W}$$

$$= \frac{1330.7 \pm \sqrt{1330.7^2 + 2 \times 3802 \times (-63.6)}}{3802} = 0.65^m \text{ or } 0.05m$$

8) Bending moment diagram and shearing force diagram

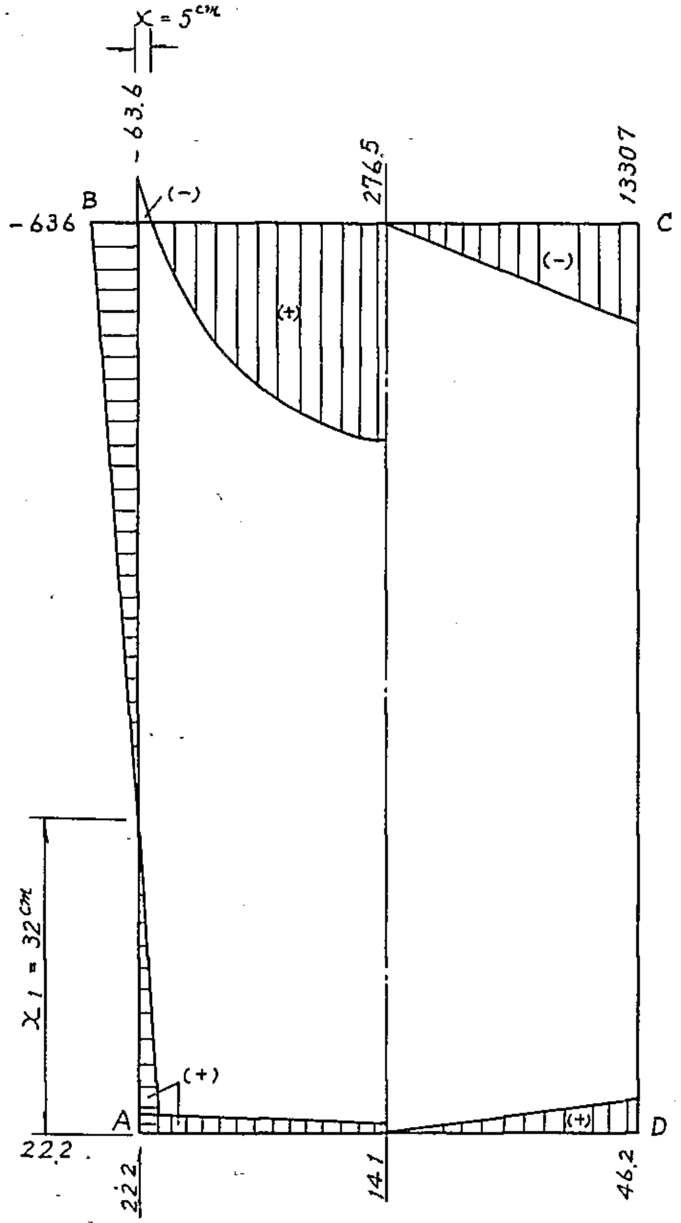
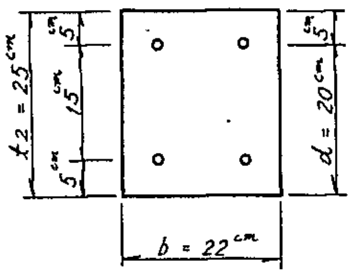



Figure-M (Kg/m) Figure-S (Kg)

(4) Calculation of Section

The calculation of section is made by the bending moment in the central part of Member BC. The value obtained applies to the total cross section. The shearing force is calculated at the edge of Member BC.

1) Assumption of section




b = 22 cm

d = 20 cm

d' = 5 cm

2) Calculation of volume of reinforcement

$$A_S = \frac{M}{\sigma_{SA} \cdot J \cdot d}$$

$$= \frac{29650}{1400 \times 7/8 \times 20} = 1.21\ \mathrm{cm}^2 \quad \text{provided } j = 7/8$$

When two pieces of $\phi$9 mm reinforcement are used, the tension side volume of reinforcement is

$A_S = 0.6362 \times 2 = 1.2724\ \mathrm{cm}^2$

On the compression side also the reinforcement is used so that the compression side volume of reinforcement is

$A_S' = 0.6362 \times 2 = 1.2724\ \mathrm{cm}^2$

The study is made on the basis of such double reinforcement beam.

3) Calculation of neutral axis

$$x = -\frac{n(As+As')}{b} + \sqrt{\left\{\frac{n(As+As')}{b}\right\}^2 + \frac{2n}{b}(Asd+As'd')}$$

$$= \frac{15 \times 1.2724 \times 2}{22} + \sqrt{\left\{\frac{15 \times 1.2724 \times 2}{22}\right\}^2 + \frac{2 \times 15}{22} 1.2724(20+5)}$$

$= 8.546$ m

4) Study of stress

a) Compressive stress

$$\sigma_c = \frac{M}{\frac{b \cdot x}{2}(d-\frac{x}{3}) + nA_s' \frac{(x-d')}{x}(d-d')}$$

$$= \frac{29650}{\frac{22 \times 8.546}{2}(20-\frac{8.546}{3}) + 15 \times 1.2724 \times \frac{(8546-5)}{8546}(20-5)}$$

$$= 19.85 \ kg/cm^2 < \sigma_{ca} = 70 \ kg/cm^2 \qquad \therefore OK$$

b) Tensile stress

$$\sigma_s = \frac{n\sigma_c(d-x)}{x}$$

$$= \frac{15 \times 19.85(20-8.546)}{8.546} = 399.1 \ kg/cm^2 < \sigma_{sA} = 1400 \ kg/cm^2$$

$$\therefore OK$$

c) Shearing stress

$$\tau = \frac{S}{b(d-\frac{x}{3})} = \frac{1330.7}{22(20-\frac{8.546}{3})}$$

$$= 4.56 \ kg/cm^2 < \tau_a = 8 \ kg/cm^2$$

Therefore, the tentatively set ∅9 mm reinforcement provides the required safety.

(5) Calculation of Pile Root Length

a) Load

i) Load from upper structure

$P_1 = 2{,}569$ kg

ii) Weight of upper part of bridge pier

From Figure 1:

$W = (0.92 \times 1.50 - 0.48 \times 1.0) \times 0.22 \times 2{,}400 = 475.2$ kg

Load on one pile

$2{,}569 + 475.2 \doteqdot 3{,}045$ kg

$P = 3{,}045 \div 2 = 1{,}523.0$ kg

b) Pile root length

i. Type of pile: 0.22 solid square PC pile

ii. Pile root length: 7.0 m (tentative)

iii. Soil: Soft soil

$$R = \frac{0.88 \times 7.0 \times 1,800}{2}$$

$$= 5,544 \text{ kg} > 1,523 \text{ kg}$$

$A = 0.22 \times 4 = 0.88$ m

$h = 7.0$ m

$F = 1,800$ kg/m$^2$

(F is based on the pile length calculation table in the case of Chaengwatana Exchange.)

Therefore, the tentatively set 7.0 m pile root length is sufficient.

JICA