The shape of the pier varies considerably from scheme to scheme. The basic reason for choosing these shapes is to harmonize the design with the superstructure and the environment.

Foundation

Two alternates are considered. The first one is a big-diameter pile which has been used in this area. Another is open caisson construction which has been used in the Island for many years and is familiar to most contractors. Further study is required to define the selection for the foundation system, pending information from the new borings which are being conducted and should be available very soon.

MATERIAL

Cement

Local produced Type 3 Cement according to the National Standard Specifications of the Republic of China.

Sand

Natural sand from which any clay element was washed away.

Coarse Aggregate

Crushed gravel, maximum size 5 cms. for the piers and foundation work; 2.5 cms. for the beam and the slab construction of the super-structure. The strength of the concrete to be as follows:

Deck Slab	*4000 psi*
Prestressed Concrete Beams	*4000 psi*
Piers and abutments	*4000 psi*
Foundation caissons	*3000 psi*

Steel

All reinforcing steel, the naminal diameter less than 16 mm (included 16 mm) should be conformed with National Standard of Republic of China with ultimate strength not less than 40 ksi. All reinforcing steel the naminal diameter larger than 16 mm should be conformed to the Standard with ultimate strength not less than 60 ksi. All prestressing steel to be 1/2" ± round stress-relieved 7 wire strands with minimum ultimate strength of 270 ksi.

Anchorage for post-tensioning strands should develop the full strength of the strands with maximum slippage of 1/8" during anchoring. The anchorage system should be one which has been successfully used for similar projects.

Conduit for post-tensioned tendons should be made of galvanized flexible sheet metal. Connection between the conduit should be made in such a way that no leakage of cement mortar enters the conduit during concreting. Thickness of the sheet metal for the conduit should not be less than 22 ga. Rigid conduits may be required when necessary to reduce friction.

Other materials such as bearings, expansion joints and lighting system to be selected during the working drawings stage.

WORKMANSHIP

Formwork for concrete to be preferably steel or fiberglass. Plywood may be permitted if shown to be satisfactory. The construction of the forms should be approved by the TAFCB.

Foreman for post-tensioning work should be a qualified person with sufficient experience subject to the approval of the TAFCB. All work shall be accomplished per detail specifications which will be presented with the construction documents

COST ESTIMATE - SCHEME 1

SPAN: 10 @ 70 M ± TOTAL LENGTH = 615 M

SUPERSTRUCTURE: COMBINED TEE & BOX SECTION

SUBSTRUCTURE: HOLLOW CIRCULAR BENT ON PILES

I T E M	*Quantity*	*Unit*	*Unit Cost*	*Total*
SUPERSTRUCTURE				
Railing	615	M	1,500	922,500
Barrier	615	M	600	369,000
Wearing Surface (A.C.)	2,500	T	400	1,000,000
4,000 psi Concrete	13,840	M^3	2,100	29,064,000
Rebars	1,030	T	8,000	8,240,000
Tendon	580	T	32,000	18,560,000
Expansion Joint	140	M	6,000	840,000
Bearing (Glass Rein. Teflon)	60	M^2	22,000	1,320,000
Surface Drainage				100,000
Light Fixtures	32	Set	15,000	480,000
Shoring		-		15,000,000
5,000 psi Precast Concrete				---
Lifting & Transportation				---
Pipe				---
SUB-TOTAL				(75,895,500)
SUBSTRUCTRE				
Dewater & Coffer Dam				8,000,000
4,000 psi Concrete	7,120	M^3	1,670	11,890,400
3,000 psi Concrete	24,800	M^3	850	21,080,000
Rebars	3,150	T	8,000	25,200,000
Piles	13,460	M	3,000	40,380,000
Caisson Excavation				---
Caisson Sinking				---
SUB-TOTAL				(106,550,400)
TOTAL OF SUPERSTRUCTURE & SUBSTRUCTURE				(182,445,900)
MOBILIZATION & GENERAL CONDITION:	2±	%		3,700,000
OTHERS:	2±	%		3,700,000
TOTAL				(189,845,900)
OVERHEAD, PROFIT & TAX FOR GENERAL CONTRACTOR:	10±	%		19,154,100
GRAND TOTAL				NT$ 209,000,000

$$\text{NOTE: AVERAGE UNIT COST} = \frac{209,000,000}{615} = \text{NT\$ } 339,000/M$$

COST ESTIMATE - SCHEME 2

SPAN: # @ 150M, 2 @ 135M AND 1 @ 86M - TOTAL LENGTH = 671M

SUPERSTRUCTURE: CANTILEVER BOX GIRDER

SUBSTRUCTURE: HOLLOW CIRCULAR BENT ON CAISSONS

I T E M	Quantity	Unit	Unit Cost	Total
SUPERSTRUCTURE				
Railing	671	M	1,500	1,006,500
Barrier	671	M	600	402,600
Wearing Surface (A.C.)	2,670	T	400	1,068,000
4,000 psi Concrete	21,800	M^3	1,750	38,150,000
Rebars	2,090	T	8,000	16,720,000
Tendon	1,370	T	32,000	43,840,000
Expansion Joint	272	M	6,000	1,632,000
Bearing (Glass Rein. Teflon)				---
Surface Drainage				100,000
Light Fixtures	32	Set	15,000	480,000
Shoring				---
5,000 psi Precast Concrete				---
Lifting & Transportation				---
Pipe				---
SUB-TOTAL				(103,399,100)
SUBSTRUCTURE				
Dewater & Coffer Dam				4,000,000
4,000 psi Concrete	6,670	M^3	1,670	11,138,900
3,000 psi Concrete	31,000	M^3	1,300	40,300,000
Rebars	4,030	T	8,000	32,240,000
Piles				---
Caisson Excavation	71,000	M^3	100	7,100,000
Caisson Sinking	13,500	M^2	700	9,450,000
Tendon	121	T	32,000	3,872,000
SUB-TOTAL				(108,100,900)
TOTAL OF SUPERSTRUCTURE & SUBSTRUCTURE				(211,500,000)
MOBILIZATION & GENERAL CONDITION:	2±	%		4,000,000
OTHERS:	2±	%		4,000,000
TOTAL				(219,500,000)
OVERHEAD, PROFIT & TAX FOR GENERAL CONTRACTOR:	10±	%		22,000,000
GRAND TOTAL				NT$ 241,500,000

NOTES: 1. $\text{AVERAGE UNIT COST} = \frac{241{,}500{,}000}{671} = \text{NT\$359,000/M}$

2. The actual.cost of construction to compare with the same length (615M) as Scheme 1 = 241,500,000 - 56M x 34.6 M x 6000 = 229,874,400

TABLE II

COST ESTIMATE - SCHEME 3

SPAN: 1 @ 170M, 2 @ 100M, 4 @ 70M, TOTAL LENGTH = 650M

SUPERSTRUCTURE: CABLE STAYED DECK

SUBSTRUCTURE: HOLLOW CIRCULAR BENT ON PILES AND CAISSON

I T E M	*Quantity*	*Unit*	*Unit Cost*	*Total*
SUPERSTRUCTURE				
Railing	650	M	1500	975,000
Barrier	650	M	600	390,000
Wearing Surface (A.C.)	2,586	T	400	1,034,400
4,000 psi Concrete	11,850	M^3	1500	17,775,000
Rebars	1,075	T	8000	8,600,000
Tendon	1,065	T	32000	34,080,000
Expansion Joint	210	M	6000	1,260,000
Bearing (Glass Rein. Teflon)	28	M^2	22000	616,000
Surface Drainage				100,000
Light Fixtures	32	Set	15000	480,000
Shroing				5,000,000
5000 psi Precast Concrete	4,800	M^3	1850	8,880,000
Lifting & Transportation	180	Set	35000	6,300,000
Pipe	350	T	16000	5,600,000
				(91,090,400)
SUBSTRUCTURE				
Dewater & Coffer Dam				4,000,000
4,000 psi Concrete	6,900	M^3	1850	12,765,000
3,000 psi Concrete	27,200	M^3	1260	34,272,000
Rebars	3,220	T	8000	25,760,000
Piles	9,040	M	3000	27,120,000
Caisson Excavation	44,350	M^3	100	4,435,000
Caisson Sinking	6,100	M^2	700	4,270,000
SUB-TOTAL				(112,622,000)
TOTAL OF SUPERSTRUCTURE AND SUBSTRUCTURE				(203,712,400)
MOBILIZATION & GENERAL CONDITION	2±	%		4,000,000
OTHERS	2±	%		4,000,000
TOTAL				(211,712,400)
OVERHEAD, PROFIT & TAX FOR GENERAL CONTRACTOR	10±	%		21,287,600
GRAND TOTAL				NT$233,000,000

NOTE: 1. *Average Unit Cost* $\frac{233,000,000}{650}$ *= NT$358,000.*

2. *The actual cost of construction to compare with the same length (615M) as Scheme 1*
= 233,000,000 - 35 x 34.6 x 6000 = 225,734,000

COST ESTIMATE - SCHEME 4

SPAN: MAX. 120 METERS, TOTAL LENGTH = 637.5M

SUPERSTRUCTURE: TEE-SECTION ON Y SHAPE PIERS

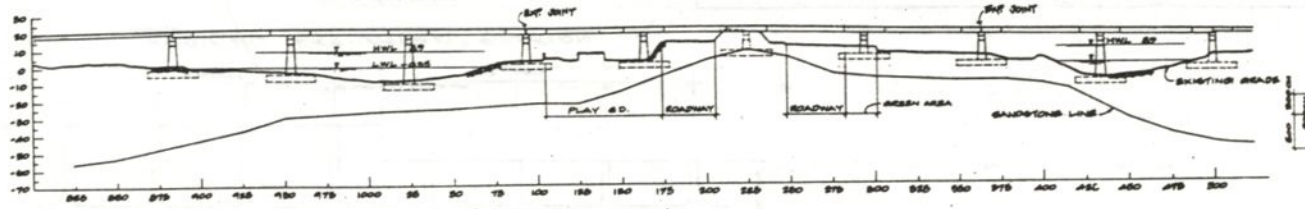
SUBSTRUCTURE: HOLLOW CIRCULAR BENT ON PILES AND CAISSONS

I T E M	*Quantity*	*Unit*	*Unit Cost*	*Total*
SUPERSTRUCTURE				
Railing	637.5	M	1,500	956,300
Barrier	637.5	M	600	382,500
Wearing Surface (A.C.)	2,540	T	400	1,016,000
4,000 psi Concrete	14,270	M^3	2,100	29,967,000
Rebars	1,490	T	8,000	11,920,000
Tendon	1,180	T	32,000	37,760,000
Expansion Joint	242	M	6,000	1,452,000
Bearing (Glass Rein. Teflon)	24	M^2	22,000	528,000
Surface Drainage				100,000
Light Fixtures	32	Set	15,000	480,000
Shoring				15,000,000
5,000 psi Precast Concrete				---
Lifting & Transportation				---
Pipe				---
SUB-TOTAL				(99,567,800)
SUBSTRUCTURE				
Dewater & Coffer Dam				4,000,000
4,000 psi Concrete	11,700	M^3	1,800	21,060,000
3,000 psi Concrete	21,200	M^3	1,300	27,560,000
Rebars	3,990	T	8,000	31,920,000
Piles	7,959	M	3,000	23,877,000
Caisson Excavation	44,500	M^3	100	4,450,000
Caisson Sinking	9,100	M^2	700	6,370,000
SUB-TOTAL				(119,237,000)
TOTAL OF SUPERSTRUCTURE & SUBSTRUCTURE				(218,804,800)
MOBILIZATION & GENERAL CONDITION:	2±	%		4,000,000
OTHERS:	2±	%		4,000,000
TOTAL				(226,804,800)
OVERHEAD, PROFIT & TAX FOR GENERAL CONTRACTOR:	10±	%		22,695,200
GRAND TOTAL				NT$ 249,500,000

NOTES: 1. *AVERAGE UNIT COST* $= \frac{249,500,000}{637.5} =$ *NT$ 391,000/M*

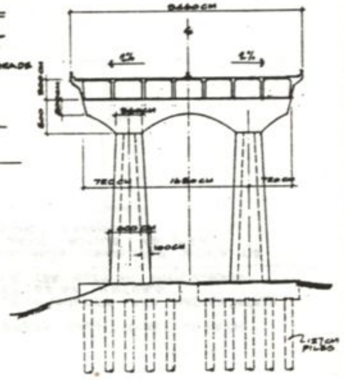
2. *The actual cost of construction to compare with the same length (615M) as Scheme 1* $= 249,500,000 - 22.5 \times 34.6 \times 6,000 = 244,829,000$

TABLE IV



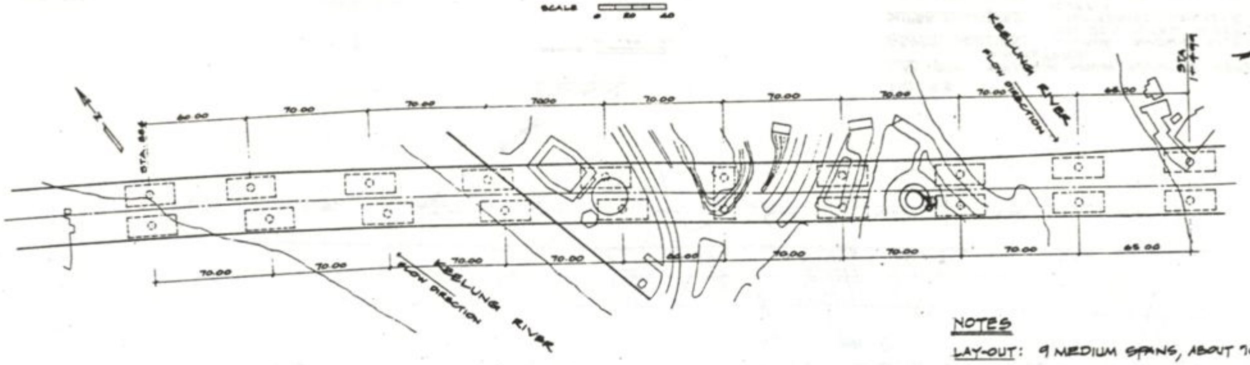
PROFILE

SCALE 0 20 40



PIER SECTION

SCALE 0 5 10 15



PLAN

SCALE 0 20 40

NOTES

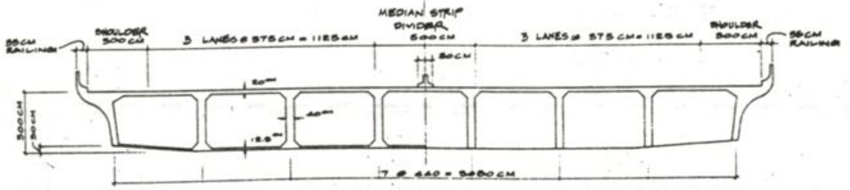
LAY-OUT: 9 MEDIUM SPANS, ABOUT 70 METERS EACH, TOTAL LENGTH 615 METERS

CROSS-SECTION: 6 LANE ROADWAY WITH SHOULDER AND MEDIAN STRIP DIVIDER
TOTAL WIDTH 34.60 METERS

SUPER STRUCTURE: PRE-STRESSED CONCRETE BOX GIRDER AT SUPPORT,
TEE SECTION AT CENTRAL SPAN

SUB STRUCTURE: TWO (2) CIRCULAR COLUMNS CONNECTED WITH A TIE
BEAM TO FORM A BENT ACTING AS PIER.

FOUNDATION: LARGE DIAMETER PILES OR CAISSONS.



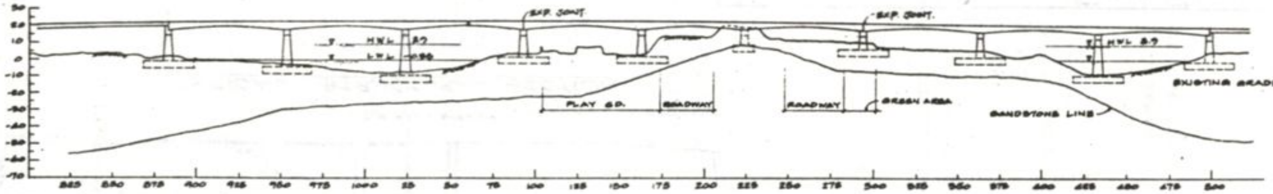
TYPICAL STRUCTURAL SECTION

SCALE 0 1 2 3 4

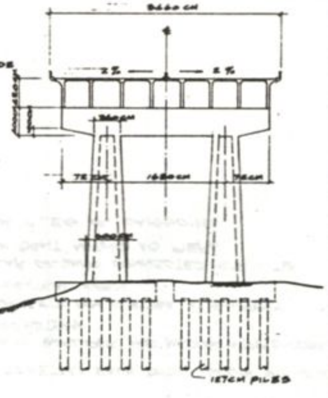
T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-72003	SCHEME 1A- PLAN, PROFILE & SECTIONS
NOV. 1. 73	YUAN SHAN BRIDGE
Y. C	TAIWAN AREA FREEWAY CONST. BUREAU
Y. L. YANG	

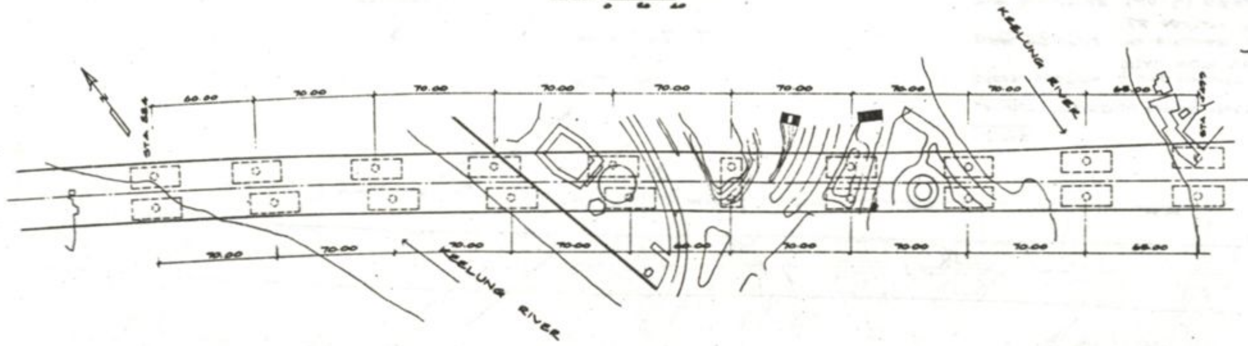
P-1



PROFILE



PIER SECTION



PLAN

NOTES

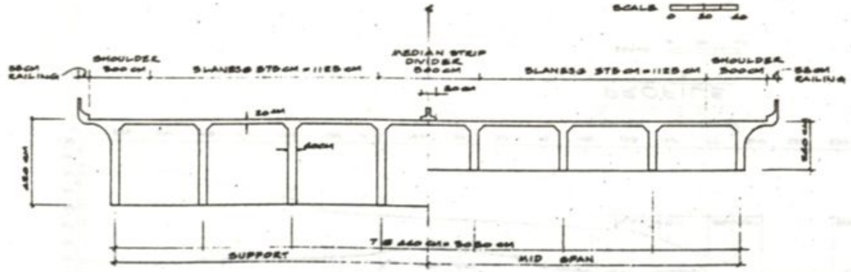
LAY-OUT: 9 MEDIUM SPANS, ABOUT 70 METERS EACH, TOTAL LENGTH 615 METERS

CROSS-SECTION: 6 LANE ROADWAY WITH SHOULDER AND MEDIAN STRIP DIVIDER TOTAL WIDTH 34.60 METERS.

SUPER STRUCTURE: PRESTRESSED CONCRETE TEE SECTION WITH CURVED SOFFIT

SUB STRUCTURE: TWO (2) CIRCULAR COLUMNS CONNECTED WITH A TIE BEAM TO FORM A BENT ACTING AS PIER

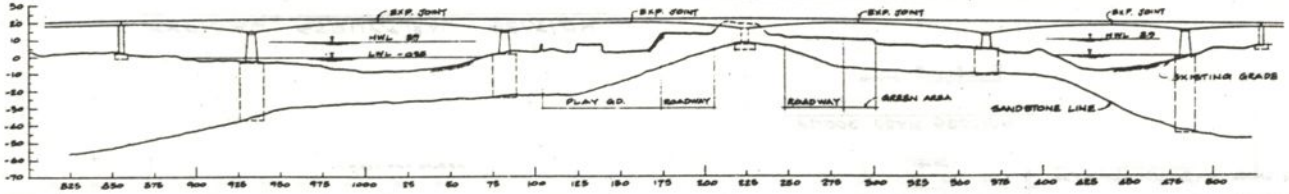
FOUNDATION : LARGE DIAMETER PIERS OR CAISSONS



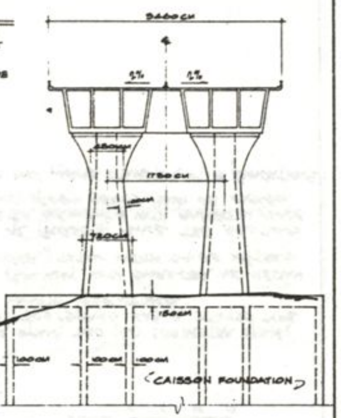
TYPICAL STRUCTURAL SECTION

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

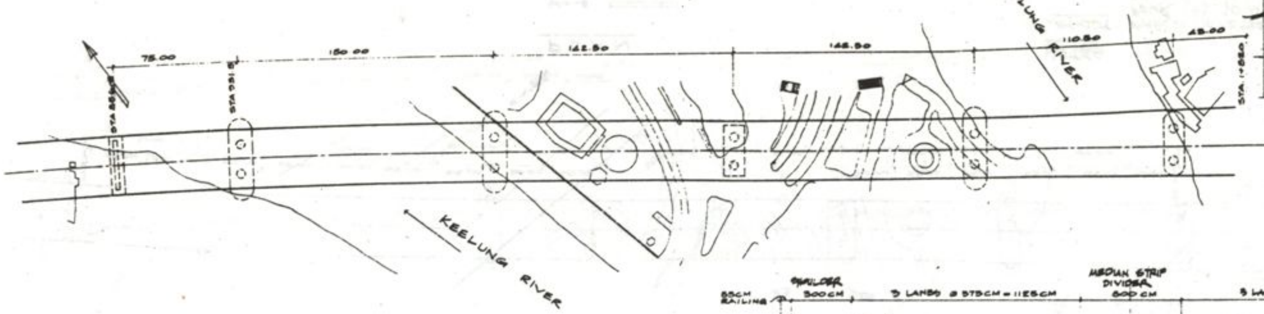
T-72003	SCHEME 1B- PLAN, PROFILE & SECTIONS	P-2
NOV. 7, 72	YUAN SHAN BRIDGE	
Y. C	TAIWAN AREA FREEWAY CONST. BUREAU	
T. L. YANG		



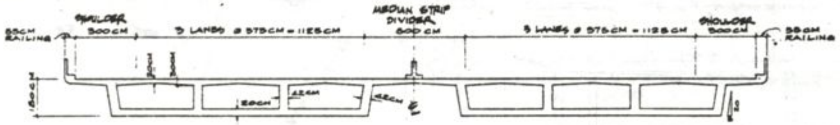
PROFILE



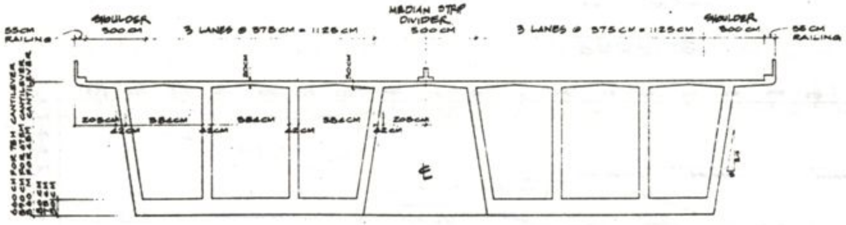
PIER SECTION



PLAN



SECTION AT MID SPAN



SECTION AT CENTER OF PIER

NOTES

LAY-OUT : 4 LONG SPANS ABOUT 150 METERS, AND TWO (2) MEDIUM SPAN TOTAL LENGTH 671 METERS

CROSS-SECTION : 6 LANE ROADWAY WITH SHOULDER AND MEDIAN STRIP DIVIDER TOTAL WIDTH 34.60 METERS

SUPER STRUCTURE : PRESTRESSED CANTILEVERED CONCRETE BOX GIRDER WITH THREE (3) CELLS FOR EACH ROADWAY

SUB STRUCTURE : TWO (2) CIRCULAR HOLLOW CONCRETE COLUMNS CONNECTED WITH BENT DIAFHRAM TO FORM A BENT ACTING AS PIER

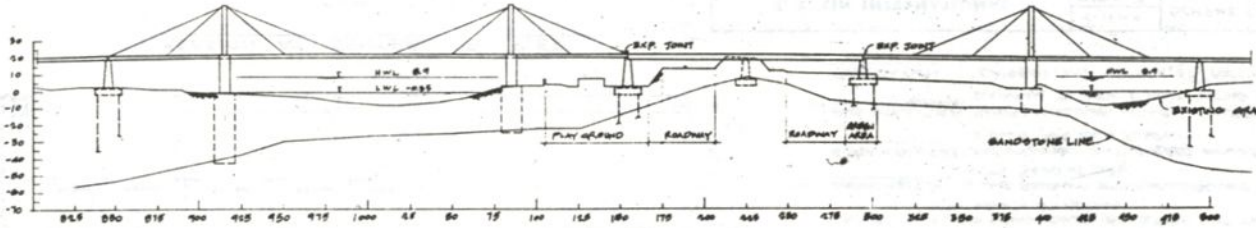
FOUNDATION : CAISSON AND SEPERATED FOOTING FOUNDATIONS

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-72003
NOV 11. 72
Y. C
T. L. YANG

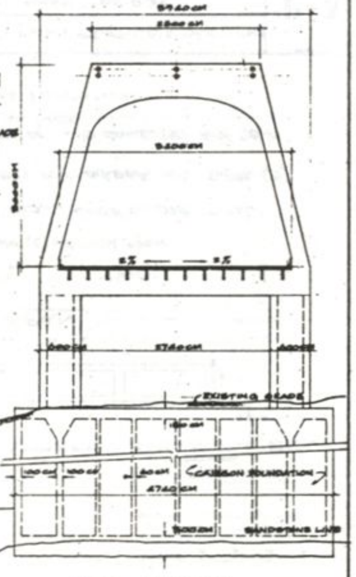
SCHEME 2- PLAN, PROFILE & SECTIONS
YUAN SHAN BRIDGE
TAIWAN AREA FREEWAY CONST. BUREAU

P-3



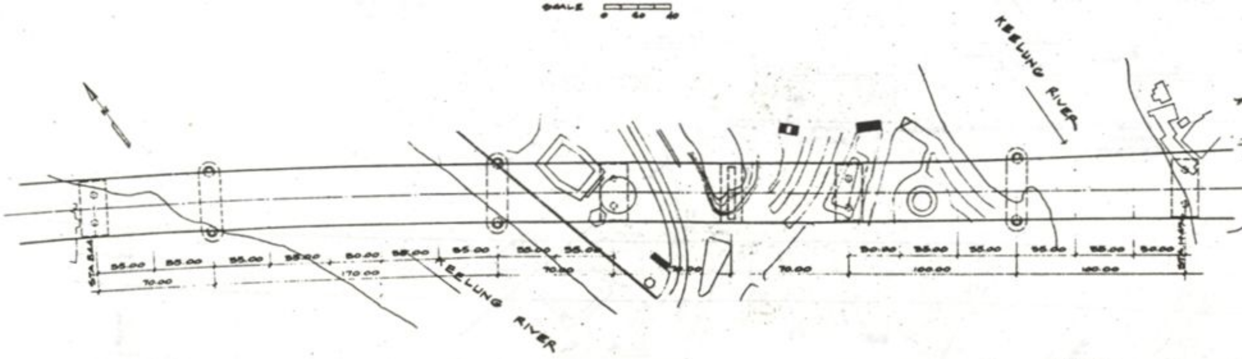
PROFILE

SCALE 0 20 40



PIER SECTION

SCALE 0 5 10 15



PLAN

SCALE 0 20 40

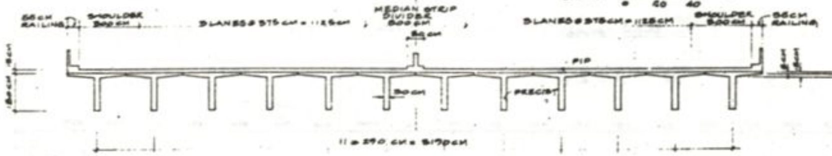
NOTES

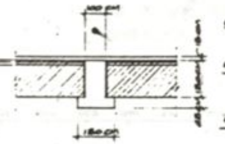
LAY-OUT: ONE(1) 170METER SPAN; TWO (2) 100 METER SPAN; FOUR (4) 70 METERS SPANS CABLE STAYED TYPE BRIDGE. TOTAL LENGTH 650 METERS.

CROSS-SECTION: 6-LANE ROADWAY WITH SHOULDER AND MEDIAN STRIP DIVIDER; TOTAL WIDTH 34.00 METERS.

SUPER STRUCTURE: PRECAST OR POURED IN PLACE TEE SECTIONS OF 35 METER SEGMENTS WITH POURED IN PLACE DECK AND CROSS BEAMS HUNG ON CABLES.

SUB-STRUCTURE: CAISSONS AND LARGE DIAMETER PILE FOUNDATIONS.





FLOOR BEAM SECTION

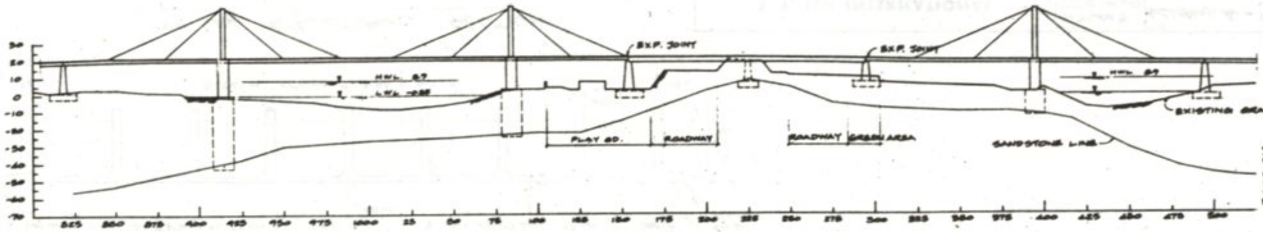
SCALE 0 2 4

TYPICAL STRUCTURAL SECTION

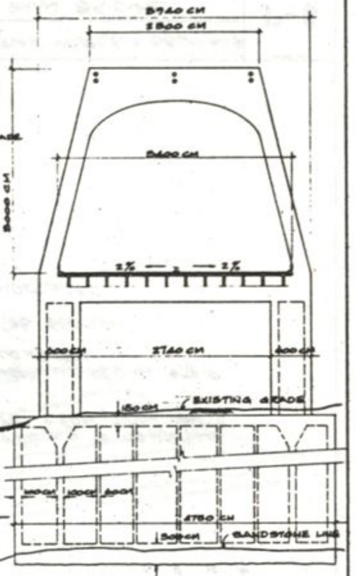
SCALE 0 2 4

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

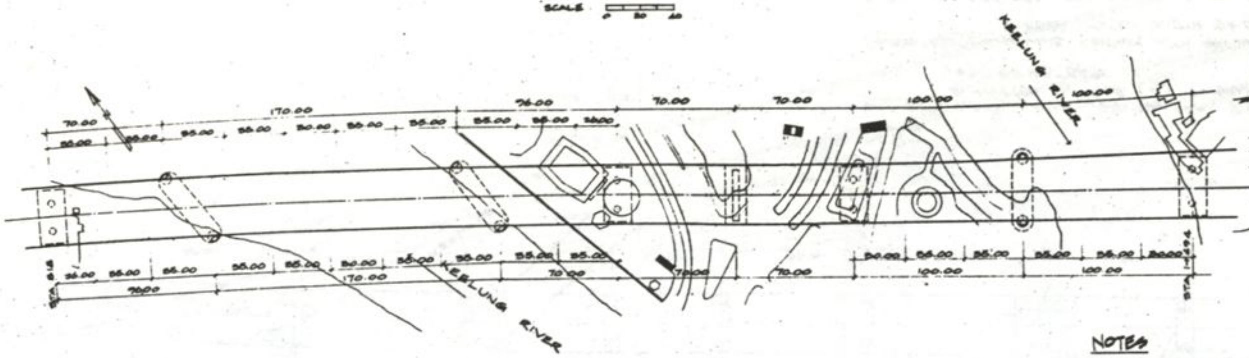
T-72003 NOV. 24, 72 T.Y. Y.L.YANG	SCHEME 5A-PLAN, PROFILE & SECTIONS YUAN SHAN BRIDGE TAIWAN AREA FREEWAY CONST. BUREAU	P-4



PROFILE



PIER SECTION



PLAN

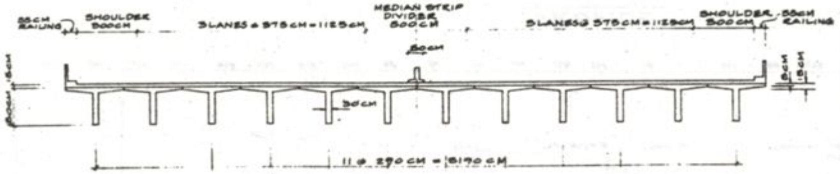
NOTES

LAY-OUT: ONE (1) 170 METER SPAN; TWO (2) 100 METER SPANS; FOUR (4) 70 METER SPANS CABLE STAYED TYPE BRIDGE. TOTAL LENGTH 650 METERS.

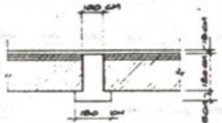
CROSS-SECTION: 6-LANE ROADWAY WITH SHOULDER AND MEDIAN STRIP DIVIDER; TOTAL WIDTH 34.60 METERS.

SUPER STRUCTURE: PRECAST OR POURED IN PLACE TEE-SECTIONS OF 35 METERS SEGMENTS WITH POURED IN PLACE DECK AND CROSS BEAMS HUNG ON CABLES.

SUB-STRUCTURE: TWO (2) CAISSONS FOUNDATION IN SKEWED POSITION PARALLEL TO RIVER CURRENT FLOW AND LARGE DIAMETER PILE FOUNDATIONS.



TYPICAL STRUCTURAL SECTION



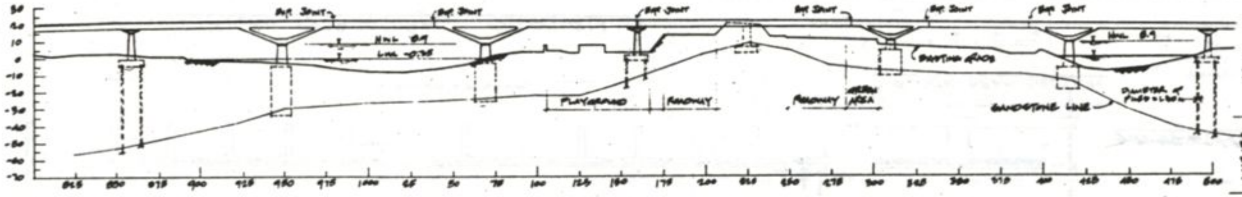
FLOOR BEAM SECTION

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

JOB NO.	T-72003
DATE	NOV. 28, 72
DESIGNED BY	T.Y.
DRAWN BY	Y.L. YANG
CHECKED BY	
ISSUE DATE	

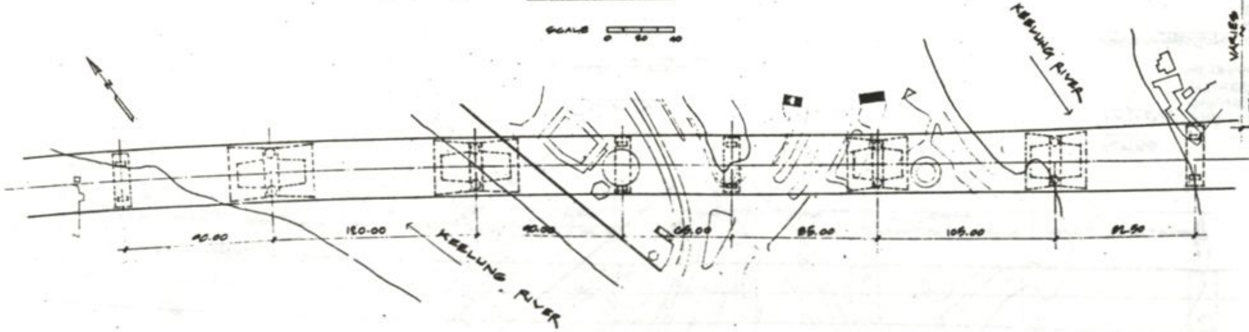
TITLE: SCHEME 3B-PLAN, PROFILE & SECTIONS
PROJECT: YUAN SHAN BRIDGE
FOR: TAIWAN AREA FREEWAY CONST. BUREAU

DWG. NO. P-5



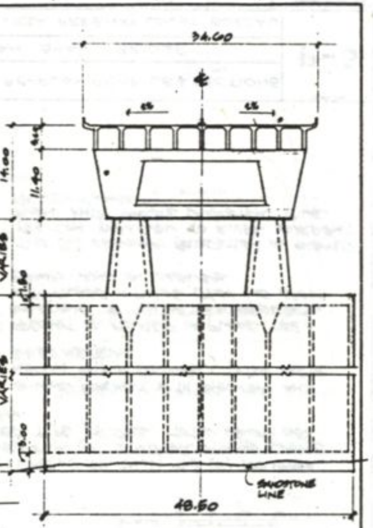
PROFILE

SCALE 0 20 40



PLAN

SCALE 0 20 40



PIER SECTION

SCALE 0 5 10 15

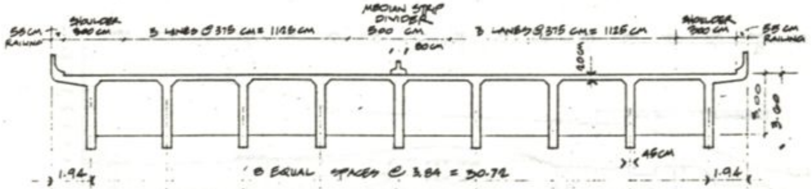
NOTES

LAY-OUT: VARYING SPAN LENGTHS FROM 120 METERS TO 80 METERS SUPPORTED ON FOUR (4) Y-SHAPED PIERS, TOTAL LENGTH 637.50 METERS.

CROSS-SECTION: 6-LANE ROADWAY WITH SHOULDER AND MEDIAN STRIP DIVIDER; TOTAL WIDTH 34.60 METERS.

SUPER STRUCTURE: PRESTRESSED CONCRETE TEE SECTION.

SUB STRUCTURE: CAISSONS AND PILE FOUNDATIONS.



TYPICAL STRUCTURAL SECTION

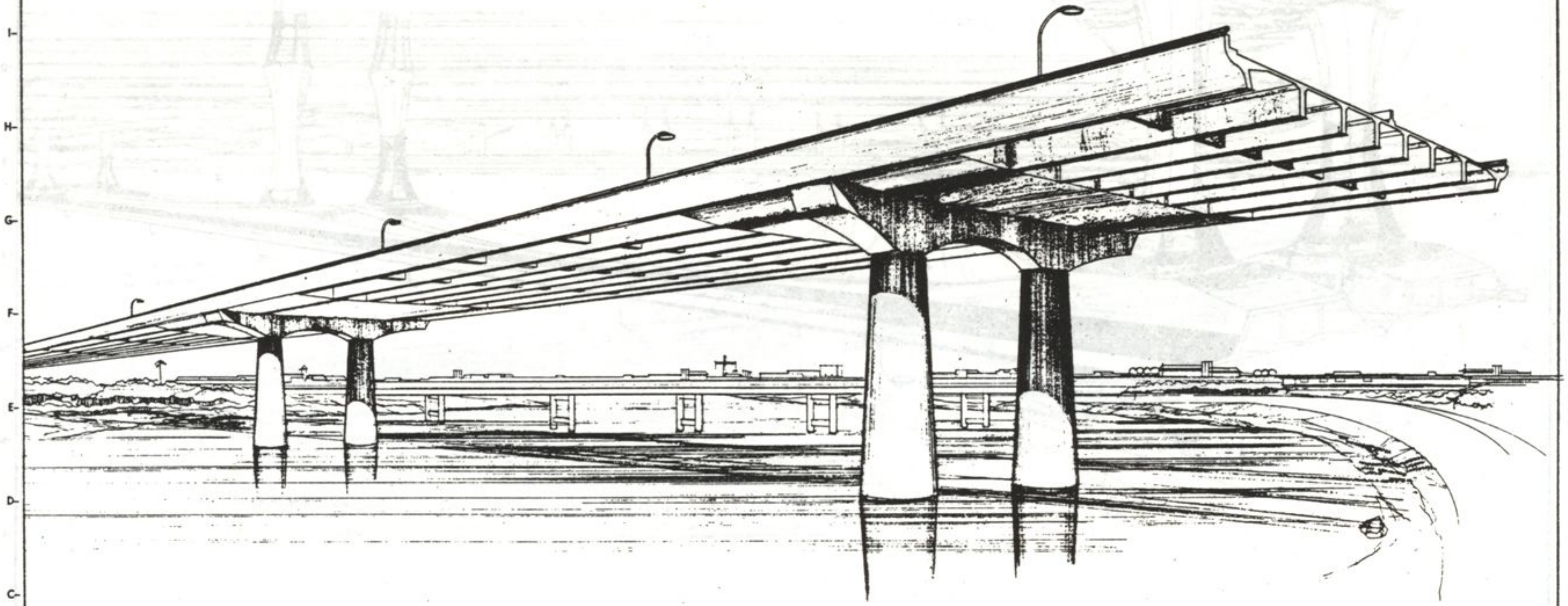
SCALE 0 2 4

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-72003
NOV 1st 1972
Y. G. YANG

SCHEME 4 - PLAN PROFILE & SECTIONS
YUAN SHAN BRIDGE
TAIWAN AREA FREEWAY CONST. BUREAU

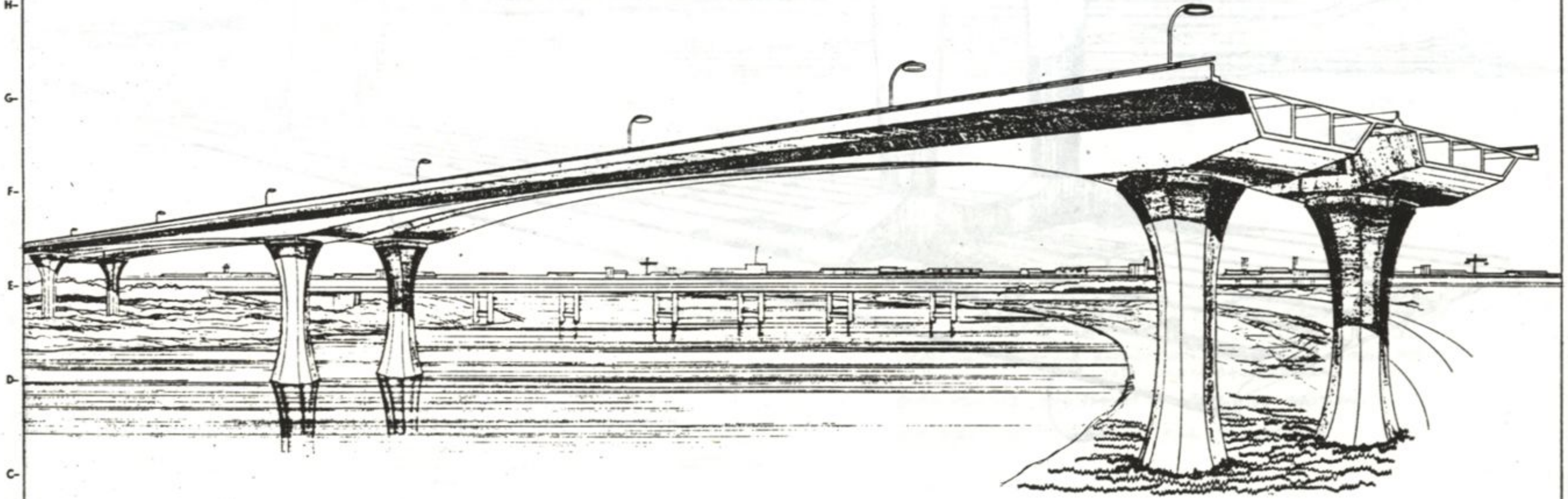
P-6



T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T- 72003	PERSPECTIVE SCHEME 1
NOV. 7 1972	YUAN SHAN BRIDGE
Y.C. YANG	TAIWAN AREA FREEWAY CONST. BUREAU
BERMUDEZ	

P-7



T. Y. LIN INTERNATIONAL

STRUCTURAL ENGINEERING

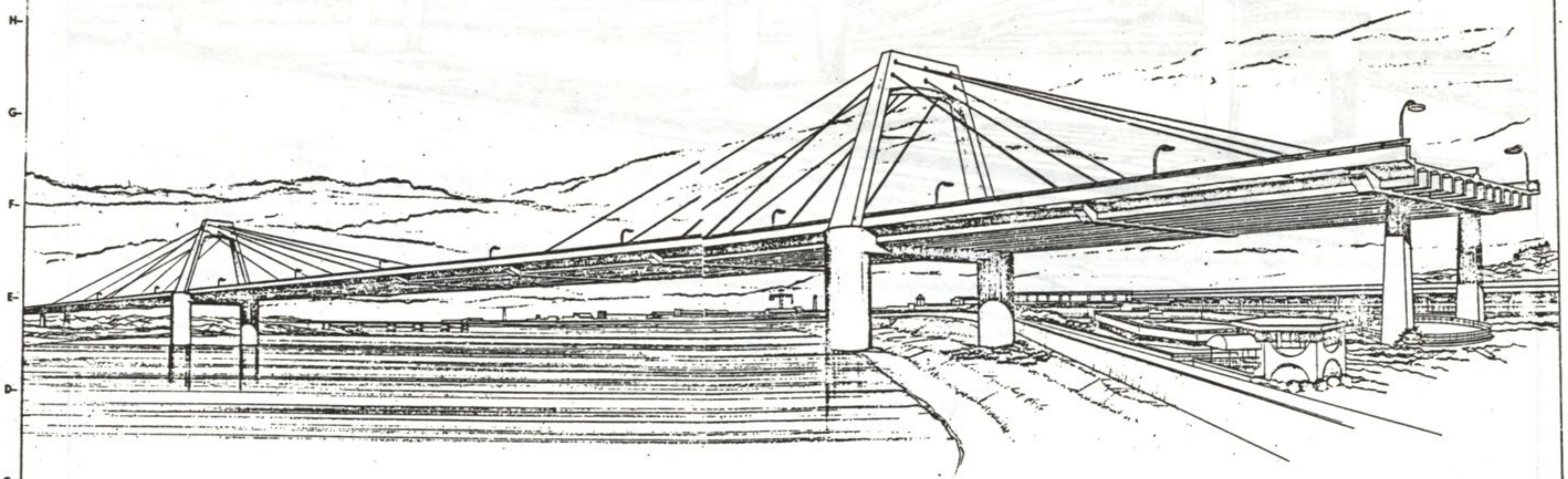
T-72008
6 NOV 1972
Y.C. YANG

PERSPECTIVE SCHEME 2

YUAN SHAN BRIDGE

TAIWAN AREA FREEWAY CONST. BUREAU

P-8

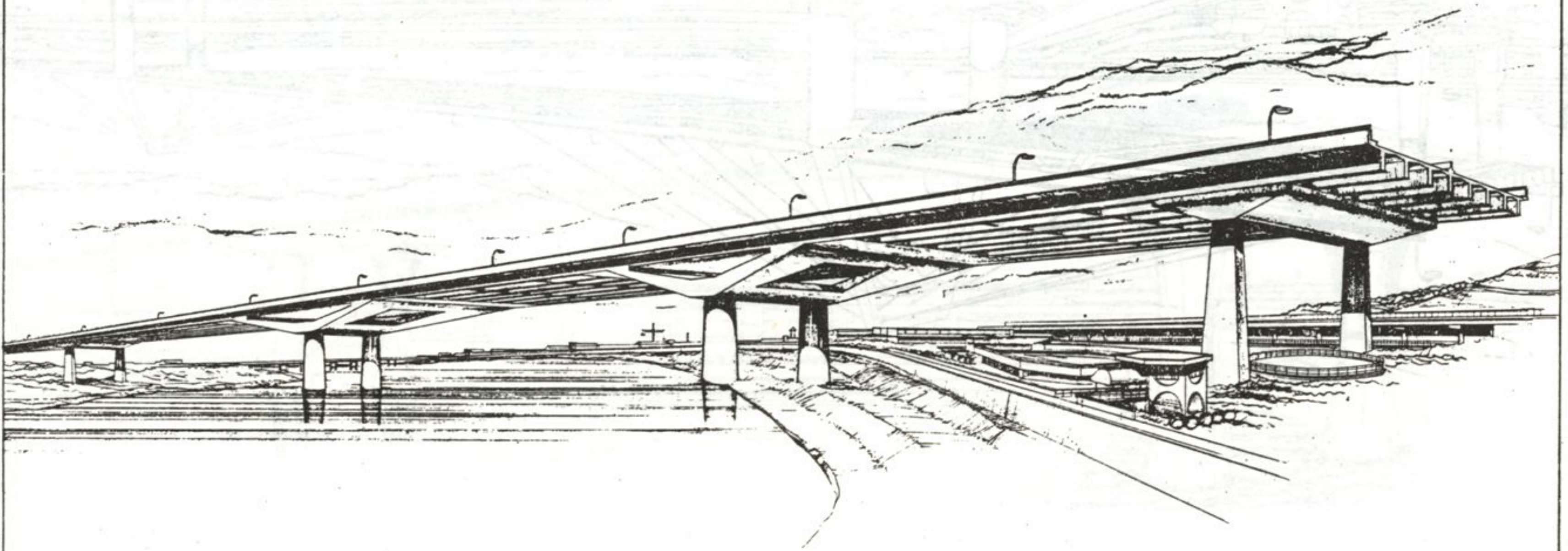


T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-72003
10 NOV 72
T.Y. LIN
BERMUDEZ

PERSPECTIVE SCHEME. 3
YUAN SHAN BRIDGE
TAIWAN AREA FREEWAY CONST. BUREAU

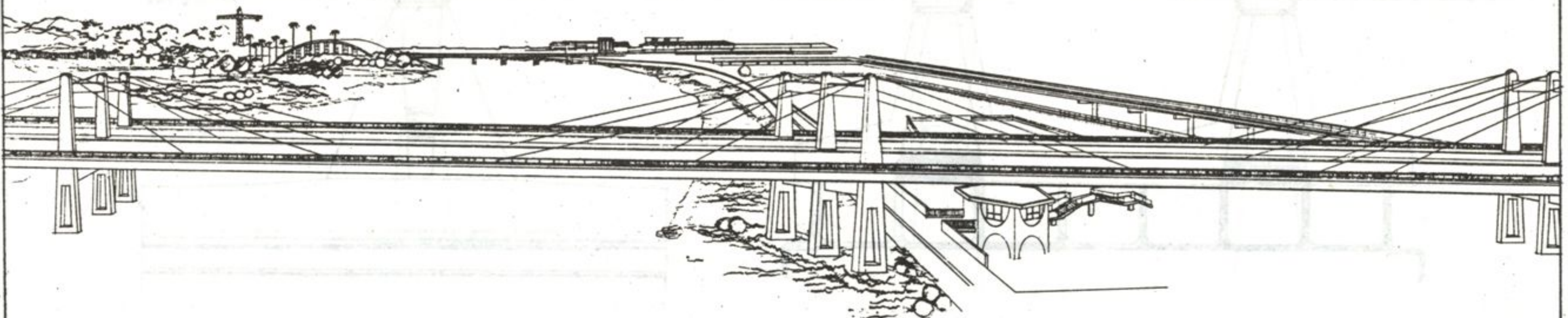
P-9



T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-72003	PERSPECTIVE SCHEME 4
8 NOV 1972	YUAN SHAN BRIDGE
Y.C. YANG	TAIWAN AREA FREEWAY CONST. BUREAU
BERMUDEZ	

P-10

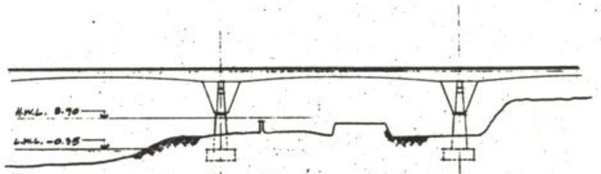


T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

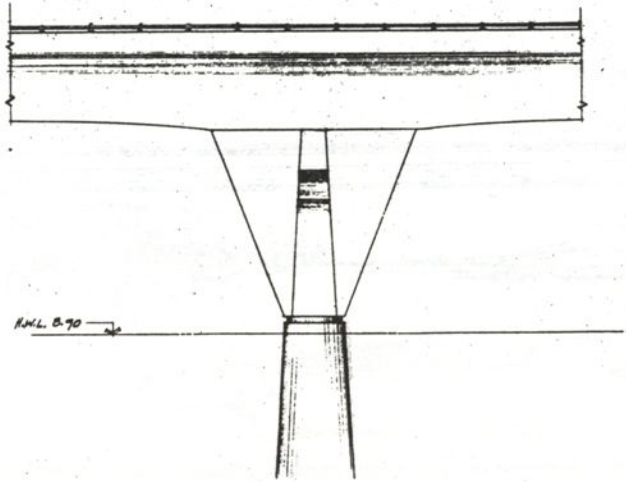
T-72003
SEPT. 17, 1972

SCHEMATIC STUDY
YUAN SHAN BRIDGE
TAIWAN AREA FREEWAY CONST. BUREAU

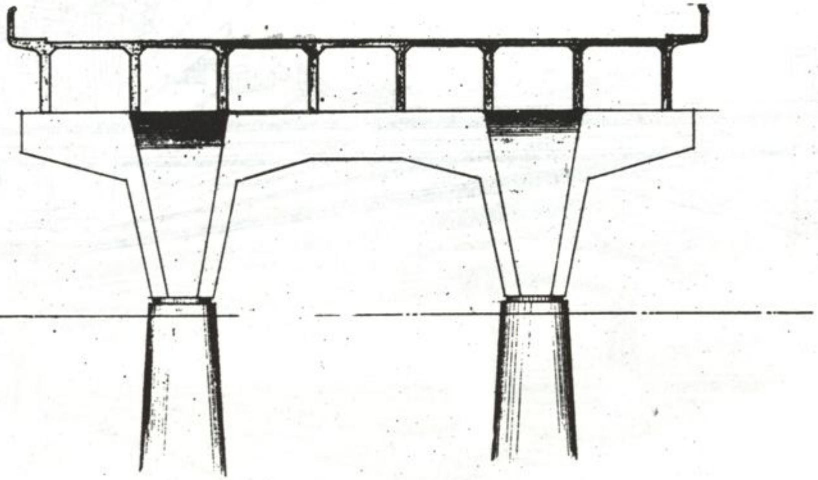
P-11



PARTIAL ELEVATION
1:600



PIER SUPPORT- SIDE ELEVATION
1:100

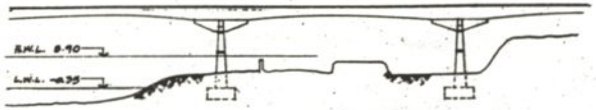


PIER SUPPORT - FRONT ELEVATION
1:100

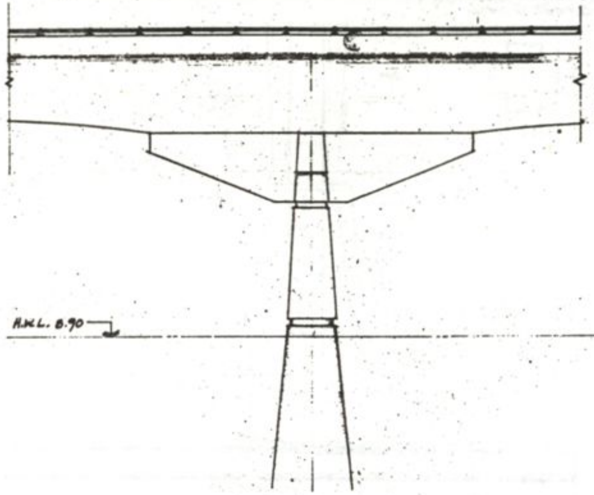
T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-72003
OCT 17,1972
Y.C. YANG

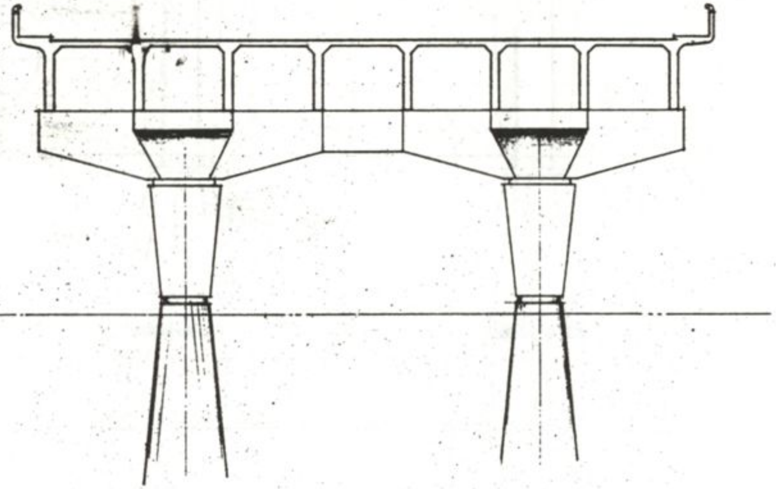
PIER SUPPORT STUDY
YUAN SHAN BRIDGE
TAIWAN AREA FREEWAY CONST. BUREAU
P-12



PARTIAL ELEVATION
1:1000



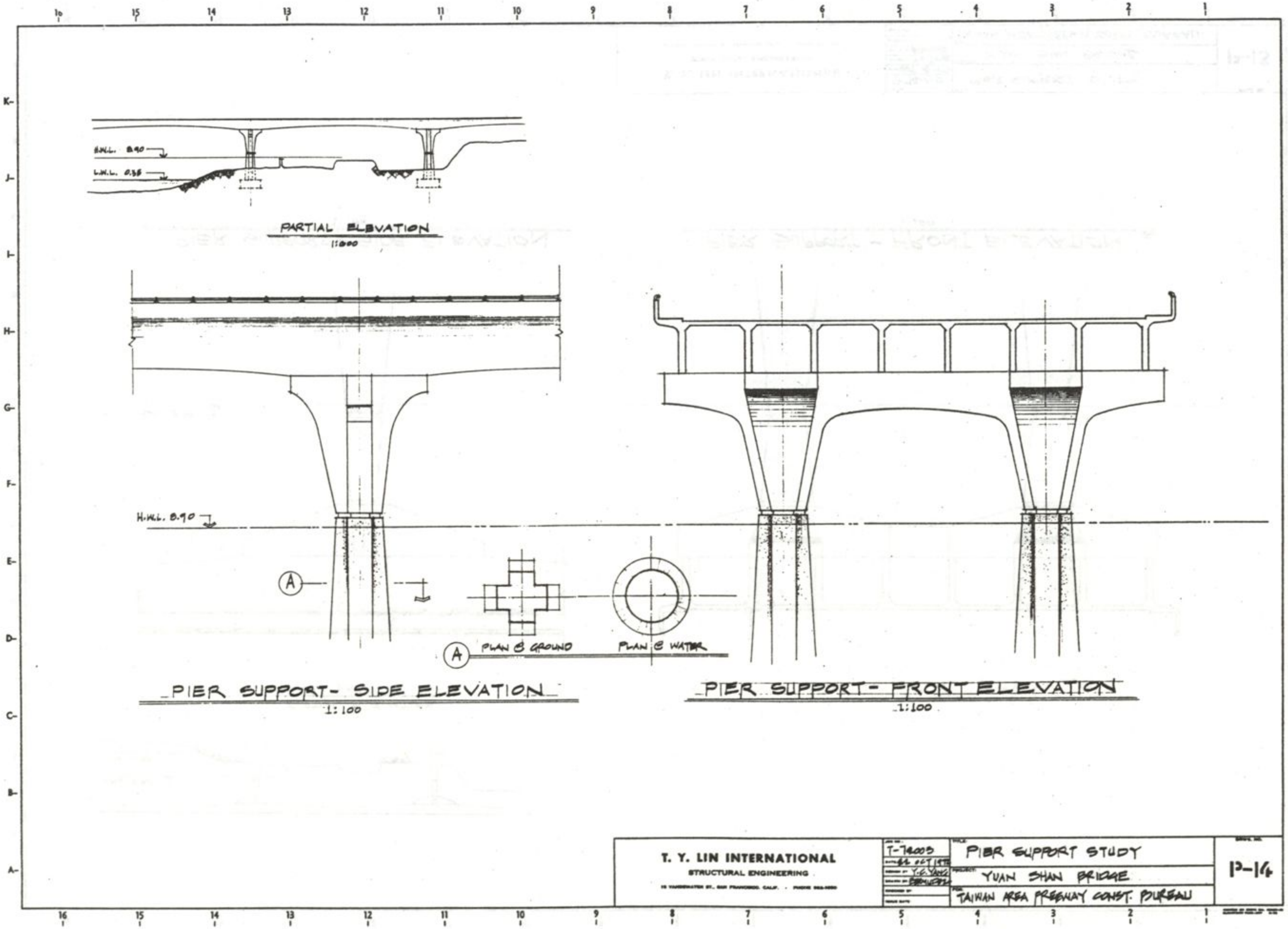
PIER SUPPORT - SIDE ELEVATION
1:100



PIER SUPPORT - FRONT ELEVATION
1:100

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-72-005	PIER SUPPORT STUDY		P-13
SEPT 10, 1972			
Y.D. YANG	YUAN SHAN BRIDGE		
[illegible]	TAIWAN AREA FREEWAY CONST. BUREAU		



PARTIAL ELEVATION
1:1000

PIER SUPPORT- SIDE ELEVATION
1:100

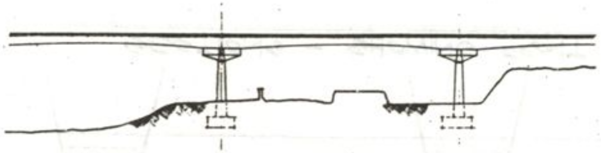
PIER SUPPORT- FRONT ELEVATION
1:100

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-74003
24 OCT 197
Y.C. YANG

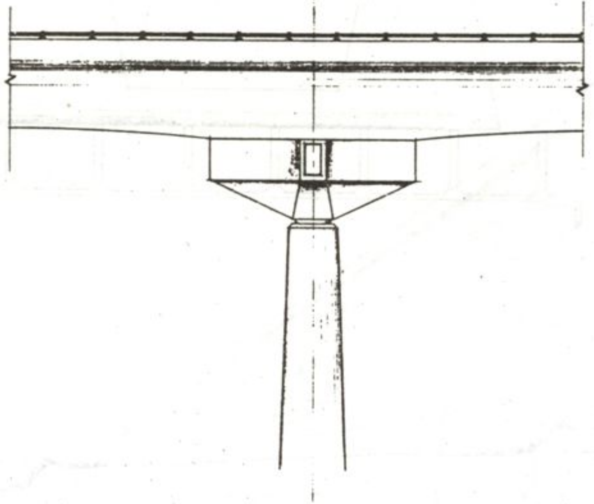
PIER SUPPORT STUDY
YUAN SHAN BRIDGE
TAIWAN AREA FREEWAY CONST. BUREAU

P-14



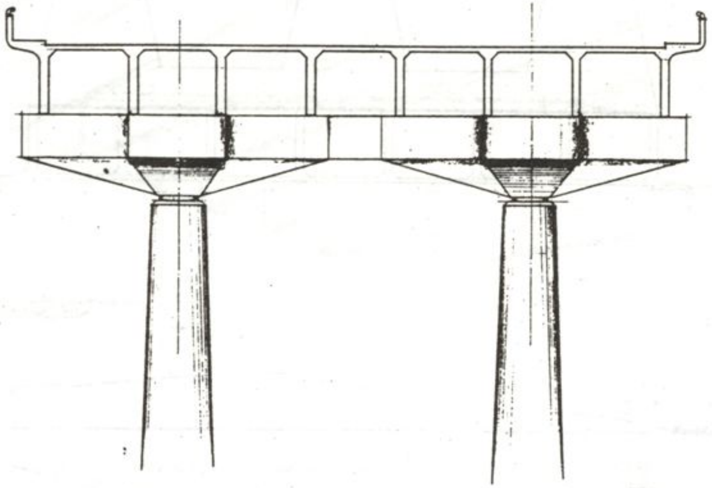
PARTIAL ELEVATION

1:600



PIER SUPPORT - SIDE ELEVATION

1:100



PIER SUPPORT - FRONT ELEVATION

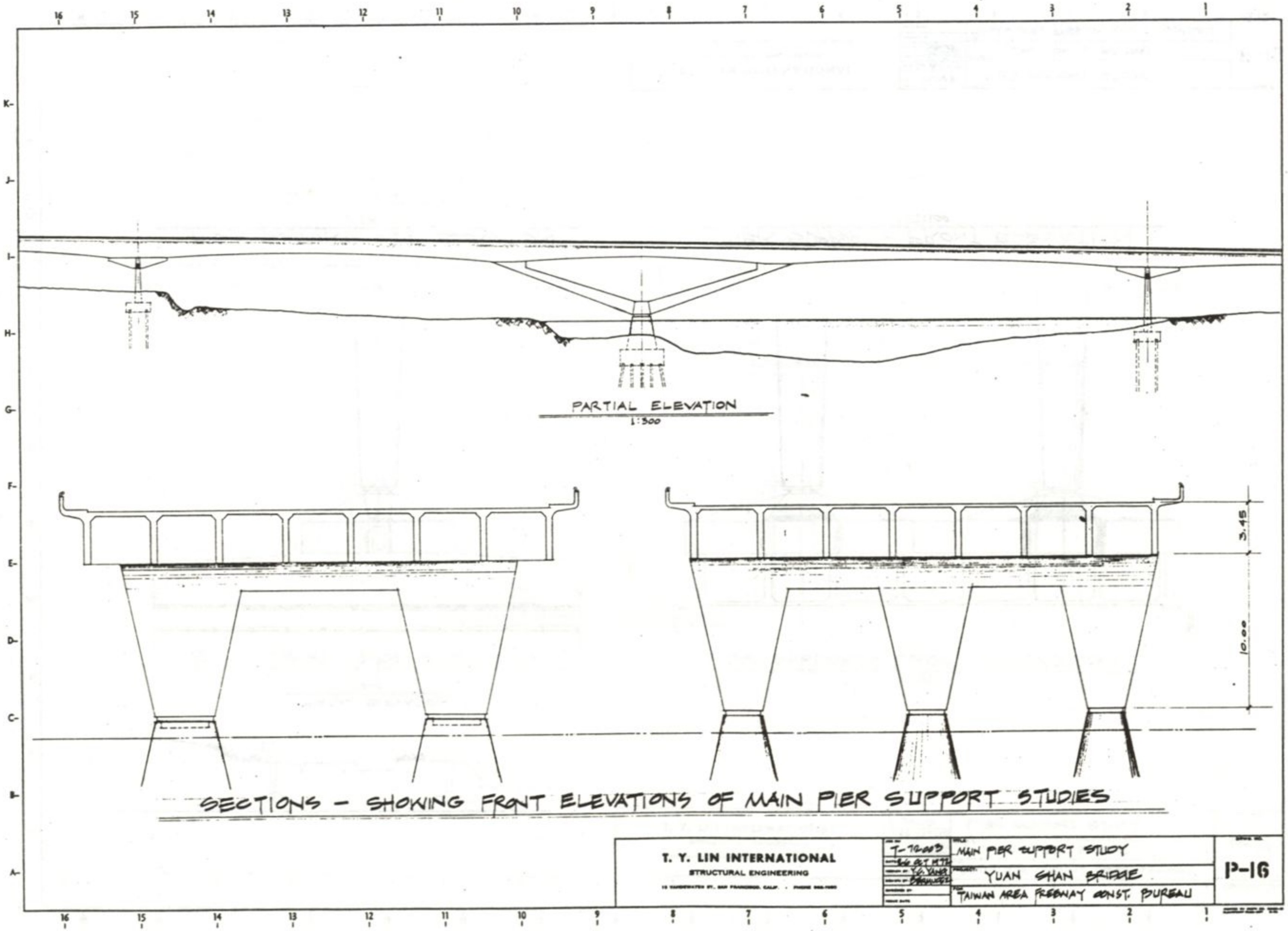
1:100

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

JOB NO.	T-72003
DATE	24 OCT 1972
DESIGNED BY	Y.C. YANG
DRAWN BY	[illegible]
CHECKED BY	
ISSUE DATE	

TITLE: PIER SUPPORT STUDY
PROJECT: YUAN SHAN BRIDGE
FOR: TAIWAN AREA FREEWAY CONST. BUREAU

DRWG. NO. P-15



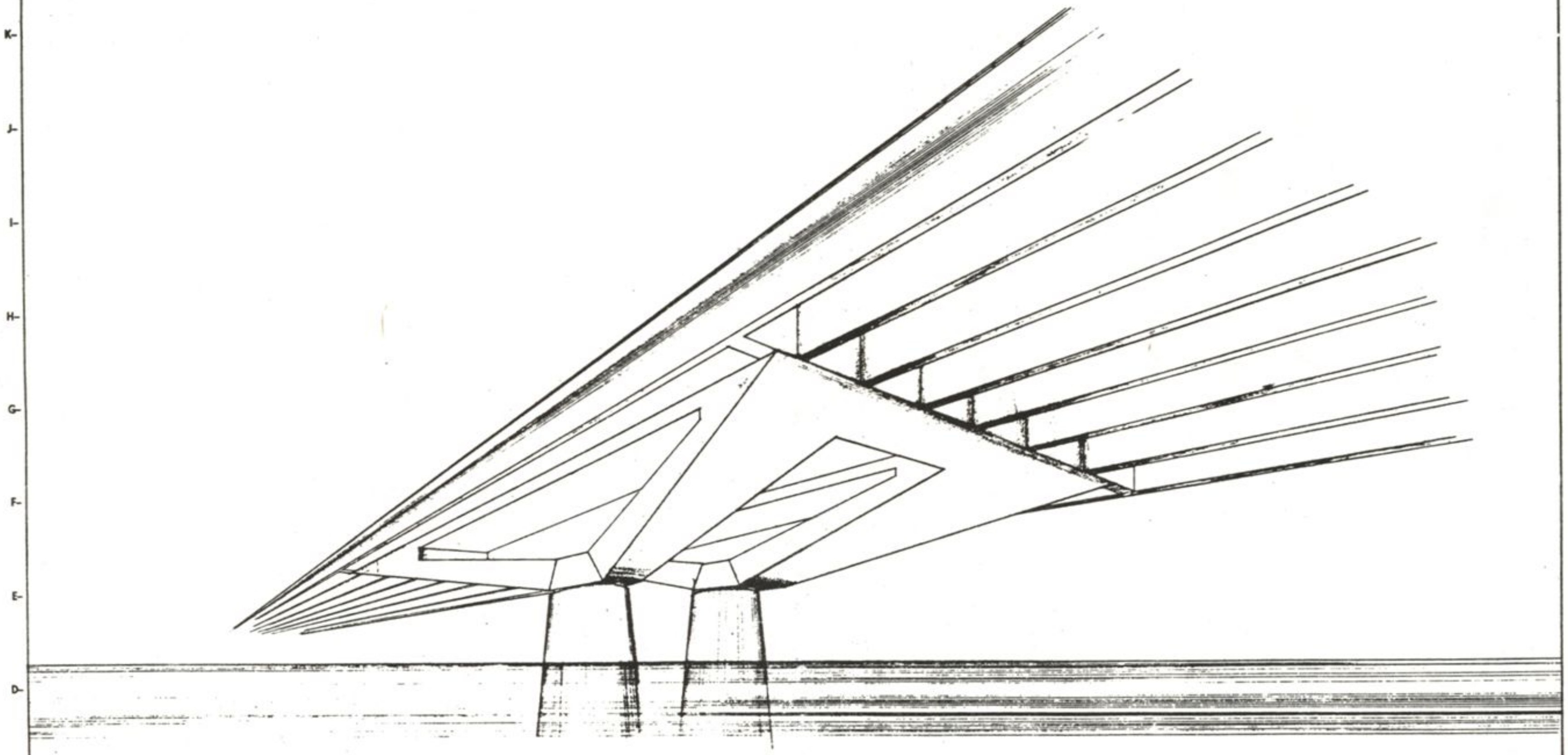
SECTIONS – SHOWING FRONT ELEVATIONS OF MAIN PIER SUPPORT STUDIES

T. Y. LIN INTERNATIONAL
STRUCTURAL ENGINEERING

T-72003
26 OCT 1972
Y.C. YANG

MAIN PIER SUPPORT STUDY
YUAN SHAN BRIDGE
TAIWAN AREA FREEWAY CONST. BUREAU

P-16



T. Y. LIN INTERNATIONAL

STRUCTURAL ENGINEERING

T-72003
4 NOV 1972
Y.C. YANG

MAIN PIER SUPPORT STUDY

YUAN SHAN BRIDGE

TAIWAN AREA FREEWAY CONST. BUREAU

P-17

附錄二　圓山橋基礎研究報告

REPORT ON REVIW OF FOUNDATIONS CONDITIONS
PROPOSED YUAN-SHAN BRIDGE
TAIPEI, TAIWAN

March 28, 1973

T. Y. LIN INTERNATONAL

LEE AND PRASZKER

Consulting civil Engineers

LEE AND PRASZKER
CONSULTING CIVIL ENGINEERS
147 NATOMA ST AT NEW MONTGOMERY
SAN FRANCISCO 94105
(415) 392-4866

CHARLES H LEE (1883-1967)
F. ASCE

MICHAEL PRASZKER
F. ASCE

March 28, 1973

T. Y. Lin International
15 Vandewater Street
San Francisco, California

Regarding: Report On Review of Foundation Conditions
Proposed Yuan Shan Bridge
Taipai, Taiwan

Gentlemen:

We are pleased to transmit herewith four (4) copies of the above captioned report. To avoid repetition no summary of the findings are attempted as the report has been arranged in proper sequence dealing with each item separately.

Should you have any questions regarding the contents please feel free to call upon us. We wish to assure you of our appreciation for having placed your trust in us with this very challenging assignment.

LEE AND PRASZKER

Michael Praszker

MP/J
Encl.

I-N-D-E-X

LIST OF DIAGRAMS

APPENDICES

Note: Test data by George Lab not included as they relate to overburden soils

REPORT ON REVIEW OF FOUNDATION CONDITIONS
PROPOSED YUAN SHAN BRIDGE
TAIPAI, TAIWAN

1. INTRODUCTION

This report presents results of soil and foundation studies together with conclusions and recommendations for design criteria of foundation support for the proposed bridge sturcture.

Soil borings along the approximate center line of the bridge had been made by others for 4 trial alignments inclusive of the final alignment. Test boring logs of only the borings of the final alignment were made available to this office for review, as were corresponding various laboratory test results.

The review also included a trip to Taiwan by the writer of this report and an on-site inspection of a limited number of soil samples. Conferences were held with various departments in Taipai, and availability of local equipment was explored for foundation installation.

The owner of the proposed bridge is the Taiwan Area Freeway Construction Bureau, and the Structural Engineers are T. Y. Lin International.

2. HISTORY

The proposed bridge had been in the planning stages for several years, and soils explorations were made progressively with each trial alignment. The sets of borings corresponding to each trial alignment were designated by B, Y, V and finally F. Only the F series of borings are shown in plan location in Diagram 3. There were 27 borings made in connection with the F alignment. Essentially all the borings penetrated into the underlying rock which varied in depth from zero (exposed) at approximately midway of the bridge to 160 feet at one extreme end of the river.

Based upon laboratory tests made by local engineers it was initially planned to use open shaft cast-in place concrete piles, which, as in the case of previous projects, were made by the method of reverse circulation. This type of pile is installed by jetting a large diameter shaft and floating the cuttings in a bentonite slurry which is continuously pumped out and screened while the bentonite is recirculated. These piles were to penetrate through a maximum of 160 feet of softer soils and were to be supported by point bearing on the underlying rock. There appears to have arisen a question with regard to the allowable, or safe, bearing capacity of the rock on which the piles were to bear; the limitation of future settlements imposed by the Structural Engineers largely controlled such bearing capacities and it was at this juncture that this office was retained by the Structural Engineers to assist them in the design criteria of the support of bridge foundations.

3. PROPOSED BRIDGE

The proposed Yuan Shan Bridge is to be located at the central part of Taipai City, crossing the meandering keelung River at two locations, as shown in plan on Diagram 1. The bridge terminals are to connect to elevated freeway sections on either side of the Keelung River. The total length of the bridge is to be 671 meters(2,200 feet) with six (6) traffic lanes and shoulders for a total width of 34.6 meters(114 feet). Spans between supports will be 75 meters(246 feet), 67.5 meters (222 feet) and 43 meters(141 feet). The superstructure will be a prestressed concrete box cantilever type. Foundation loads quoted by the Structural Engineer are as follows:

Piers A & B 75 M. Cantilever Section 6,600 Metric Tons=14,520^k
Pier D 75 M. Cantilever Section 6,750 Metric Tons=14,850^k
Pier C 67.5M. Cantilever Section 5,740 Metric Tons=12,628^k
Pier E 43 M. Cantilever Section 3,900 Metric Tons= 8,580^k

The above loads are dead load plus adjusted live load and do not include earthquake forces.

The maximum depth of the box structure will be 6.25 meters and will occur over the longest cantilever section. This depth will vary with spans and within a span as shown on Typical Sections, Diagram 8-1.

Each section will be supported by 2 hollow circular columns with diaphragms between them, the columns resting upon a piled concrete cap.

4. SCOPE OF SERVICES

Ordinarily it is the custom and practice of soil mechanics and foundation engineers to plan and lay out a test boring program in cooperation with the Structural Engineers. This includes the determination of:

a) Number and location of borings.
b) Depth of individual borings and sampling at critical horizons.
c) Laboratory testing of materials within critical horizons where foundation support is taken.
d) Evaluation of field and laboratory test results with application to foundation design criteria.

In the case of our current assignment, items a through c had been planned, and virtually completed, prior to the engagement of this office. Test boring logs with description of soils penetrated, standard penetration numbers and some other data were made available to this office in conjunction with the latest test boring program. Laboratory test results of the kind deemed necessary by the investigators were also made available.

All the above data were reviewed and soil profiles were prepared from the F series of test boring logs. Soil strength characteristics of the bearing material, which in some places is in excess of 150 feet below ground surface, were evaluated by the writer on the basis of additional shear tests which were made during the writer's stay in Taipai (between February 6 and February 10, 1973) and visual inspection of soil cores.

Samples of the non-bearing soils were no longer available as they had.

been disposed of. However, a great number of tests had been made up on this material, while only a few routine tests had been made upon the underlying rock which constitutes the bearing strata.

Based on all the above data, analyses were made for the determination of safe bearing pressures of the underlying rock in which support would be taken by either point bearing cast-in-place piles or by driven piles which would distribute the loads in friction within the embedded part of the pile. Pile carrying load capacities were evaluated together with horizontal load capacities as required for earthquake forces.

5. EVALUATION OF SOIL BORINGS AND LABORATORY TESTS

A. Non-Rock

The numerous tests made on this material comprise:

a) Dry Density and Moisture Content (Volumetric)
b) Gradtion
c) Atterberg Limits
d) Unconfined Compressive Strength
e) Direct Shear Test

All the test results lead to the conclusion that the soils overlying bedrock are heterogeneous in nature and vary in composition from relatively soft clays, silts, to sands and gravels in a silt or clay matrix. There may also be present gravel lenses at unpredictable horizons. This non-rock material is not suitable to support heavy concentrated loads due to:

1. Heterogeneity with respect to bearing capacity.
2. Resulting settlements and differential settlements.
3. Relatively low bearing capacities.

All the testing of the overburden soils in conjunction with the F borings were made by George Soil Mechanics Laboratory Ltd. of Taipai. A summary of all test boring logs and laboratory test results which were made available to this office is presented herewith. The laboratory test data by George Soil Mechanics Laboratory Ltd. are not included in this report as they pertain to the overburden only.

Test boring logs of the F series are presented on the Soil Profile in Diagram 3.

B. Bedrock

All but three of the 27 borings listed in this report penetrated into rock, which outcrops at Pier C and dips thence to the east and west, submerging under the Keelung River. The depth to rock below existing ground surface at the extremeties of the proposed bridge is approximately 130 to 160 feet.

Relatively few laboratory tests were made on rock samples as indicated in the Summary of Test Borings and Soil tests, Diagram 4. The few tests that were made on rock samples were confined to volumetric, sieve analyses and unconfined compressive strength. It was difficult to evaluate rock strength characteristics from these tests as no triaxial test or direct shear test had been made.

Judging from the rock description in boring logs, together with standard penetration tests and a limited number of laboratory tests, it could be concluded that the rock strength characteristics vary from relatively low to fairly high values. Visual examination of rock samples by the writer in Taipai confirmed that the rock varied from fairly hard sandstone to friable shale and mudstone. A few typical samples were selected by the writer and subjected to direct shear tests for the weaker materials, and triaxial tests for the firmer materials. Test results are summarized herewith and test data are attached in Appendix II of this report.

SUMMARY OF SHEAR TESTS ON ROCK SAMPLES

			SHEAR TEST RESULTS				
			UNCONFINED		TRIAXIAL		
TEST BORING	SAMPLE DESCRIPTION	DEPTH METERS	ø^0	C (kg/cm^2)	ø^0	C (kg/cm^2)	REMARKS
F-31	Yellow-brown, fairly cemented, sandstone	27.6-27.8			26	10	
F-31	" "	27.10-27.45			23	28	
F-28	" "	5.2-8.2			26	25	
F-12	" "	54.2-57.5			N.A.	N.A.	Two concentric circles

F-18	Dark brown mudstone or shale	29.8-29.95	20	.65	Remolded and compacted to approximate in-situ density
F-26	Yellow-brown sandstone	8.9-9.9	29.5	.85	
F-26	" "	10.35-11.35	29.5	.48	
F-26	" "	14.25-16.00	29°	.42	
F-33	" "	29.00-29.40	32.5	.20	

One the basis of the above examinations and tests it is concluded that the soil strength characteristics of the underlying rock are as follows:

a) Fairly hard rock consisting of fine grained sandstone with internal angles of friction on the order of 26° and cohesion of 2000 PSF.

b) Weakly cemented mudstone consisting of silt with traces of clay particles, fairly soft, with an angle of internal friction of 20° and cohesion on the order of 1000 PSF.

6. SOIL PROFILES AND ROCK BEARING CAPACITIES

A. Determination of Bedrock Elevations

The soils overlying bedrock are generally classified as clayey to sandy silts with occassional lenses of gravel or sandy gravel. The standard penetration tests in these soils vary from less than 10 to the low 30's. Only occassionally does the blow count rise to above 50. Atterberg Limits and Sieve Analyses made on these materials (except for the gravel) confirm these classifications. The overburden is, therefore, not suitable to support heavy concentrated loads without excessive settlements.

The field classifications of the overburden appear to become less distinct upon approaching the underlying bedrock surface, so that from the soil description above it is sometimes difficult to differentiate between overburden and bedrock.

With the aid of the N-numbers listed alongside the boring logs the surface of bedrock elevation was determined and noted on Diagram 4. The individual test borings were also plotted horizontally and vertically on the Soil Profile, Diagram 3. The bedrock surface line was then drawn on this profile as a result of studies of soil description together with respective blow counts.

It is noted that the blow counts corresponding to rock penetration as listed in the Summary of Test Borings are nowhere less than 100, which is commensurate with common experience in weathered rock of the type described. It is also noteworthy that the blow counts directly above the rock line shown on Diagram 4 are radically lower than 100, so that the line has reasonable validity. Nevertheless, allowance for some local deviations from the rock line should be made,especially if large interpolations or extrapolations are involved. An approximation of these variations has been attempted by drawing rock elevation contours on Diagram 3.

B. Allowable Bearing Pressures in Rock

The rock in which foundation support is to be taken is known to vary in its composition and strength characteristics. This has been verified by a)blow count, b) direct and triaxial shear tests, and c) by visual inspection of the rock cores.

The triaxial test results from samples selected by the writer for testing, together with visual observations, confirm the variation of the rock from fairly cemented fine grained sandstone to thinly bedded shale and mudstone.

Since it is not practical to use variable soil strength characteristics for the bearing rock to support a certain foundation, the lowest strength characteristics for the submerged rock must be used, namely Ø = 20^{0} and c = 1 KSF.

7. FOUNDATION TYPES

A, General

It is obvious that all foundation columns must take support in the underlying rock which varies in depth from zero at Pier C, at approximate midpoint, to 50 meters (164 feet) at Pier N to the west and 40 meters (131 feet) at Pier S to the east.

Depth to rock for intermediate piers are shown on Diagram 3.

The safe bearing capacity of the rock varies with its type (sandstone, shale or mudstone) and it also varies with the degree of weathering of each rock type. It is proposed to design all supporting piles on the basis of $\phi = 20^0$ and c=1 KSF and to embed the piles sufficiently to prevent slippage of pile tips in the direction of the rock slope. Furthermore, it is appropriate to provide some pile embedment to ensure approximately uniform bearing on the rock. The safe bearing capacity for the weakest rock then becomes a function of the surcharge (overburden depth)which has been evaluated with a safety factor of 2.5 on Diagram 5. It is, therefore, recommended that all piers,except for Pier C, be supported by piles which are of sufficient dimensions and of sufficient structural adequacy to support dead and live loads attributed to each individual pile both vertically and horizontally. Pier C may be supported directly in the outcropping rock.

Basically two types of piles will be discussed here, but not to the exclusion of other types of piles that satisfy all the structural and other requirements set forth in this report.

B. Cast-in-Place Concrete Piles

This type of pile is locally used for the support of heavy structures in deep bearing strata overlain by soft materials. The method used for its installation is known as "The Reverse Circulation." This method consists of churning or jetting a circular shaft using circulation mud to keep the shaft from squeezing and also to float the cuttings which are pumped to the surface. This process is normally carried to rock or gravel surface beyond which the soil cannot be sufficently churned up for floation in the circulating mud. It is reported that such piles have been installed with tremied concrete up to 200feet in depth and with diameters up to 5 feet.

The use of such piles has been considered for the subject Project; however, due to some uncertainities which are listed below, their adoption has to be questioned, apart from their economic merits or demerits:

a) The piles are exclusively point bearing and the true nature and cross-sectional area of the point cannot be verified after installation.

b) The true bearing pressure at the point is, at best, $\frac{P}{A}$ where p is the pile load and A is its cross sectional area. Even if A were true, the value of $\frac{P}{A}$ should not exceed the allowable value based on $\phi = 20^0$ and c = 1 KSF. For piles resting on top of the rock with overburden of 150 feet the maximum allowable bearing pressure is 25 KSF which limits the load capacity of a 5 foot diameter pile to 500 kips.

c) The load bearing capacity for piles with varying(lesser) overburden would have to be correspondingly decreased or the pile diameter would have to be increased.

d) The continuity of the concrete in the shaft cannot be verified other than by core drilling.

If all the above questions could be positively answered, and economy warranted it, an indispensable requirement of embedding the pile at least 5 feet into rock still remains. Without such embedment the pile tip upon full load application could slip laterally on an inclined rock surface.

With regard to cost of piles generally, it is proposed to measure the economic preference of a pile by cost per ton of support. This is simply computed by dividing the cost of a certain pile installed by the tons it carries. An additional economic factor is the pile cap whose cost varies with the number of pile in a cap.

Inquiries conducted by the writer in Taipai indicated that for the subject project the cost of a cast-in-place concrete pile, 150 feet long, would be on the order of $20 to $25 per ton (2 kips), thus a 400 ton (800 kips) pile embedded 5 feet into rock would cost $8,000.00 to $10,000.00, installed.

C. Driven Piles

This type of pile may generally consist of either timber, concrete or steel, or a combination of all. Its main common feature is that in its process of driving it also registers the quality of soil it penetrates.

Inquiry in Taipai by the writer indicated that there is available driving equipment to drive piles up to 100 feet in length and that heavy driving hammers are available to drive heavy sections through overburden into weathered rock.

Due to the appreciable depth (150+ feet) through whcih some piles must be driven (Piers N and S) it will be necessary to splice at least 2 sections to make up the length of pile required. Other piles may be driven in a single section. It is recommended that a small displace-ment pile be used to attain maximum penetration in the rock. If a large section is used it must be equipped with a small displacement tip section to accomplish the same result.

Of the piles most suitable for the subject project a prestressed concrete pile with a H-steel tip or a closed end steel pile with a H-steel tip is recommended.

It is proposed to use a 200^T (400^k) load carrying pile driven into the underlying rock. The advantages and some disadvantages of such a pile are as follows:

1. It records the soft and hard strata inclusive of the bearing rock by blow count and,upon continuous driving, it also determines the pressence or absence of softer soil within short reaches of its tip.
2. It distributes the load by friction on its sides in the bearing stratum so that the maximum bearing pressure in the rock is largely diluted.(see Diagram 6)
3. It allows for adjustment of greater or smaller penetration in the rock depending upon the soundness of the rock it penetrates.
4. Speed of installation.
5. Splicing and supervision of driving may increase the cost per pile.
6. Number of driven piles required may be twice the number of cast-in-place concrete piles, but since the pile caps are large by other requirements, this effect may not be detrimental.
7. Piles will have to be cut-off by mechanical means, as few piles could be driven to pre-determined length.

Inquiry in Taipai indicates that the cost of a 200^T (400^k) pile installed may be on the order of \$1,000.00, which when divided by 200 tons, yields approximately \$5.00 per ton. Adding extra cost for splicing and steel tip of approximately \$300.00 per pile would bring the cost per ton to $6.50. This is much less than the estimated $20 to $25 for the cast-in-place concrete pile.

The diameter of the prestressed concrete pile need be only a minimum to satisfy structural compression member requirement. However, a steel tip dimension of 14" BP H 127 has been assumed.

D. Load Capacity For Driven Piles and Required Penetration in Rock

The desired pile load capacity of 200^T (400^k) has been analyzed for resulting maximum soil pressures and for the required depth of penetration in the rock. Assuming $\phi = 20^o$ and c = 1 KSF the required depth of embedment in the rock is approximately 9,5 feet to support the pile in friction (thickness of overburden 150 feet). The resulting maximum bearing pressures at the tip horizon of the pile will be on the order of 28 KSF, which is less than the allowable for the rock horizon at the pile tip. This penetration may be less if the rock is firmer than assumed. However, it is recommended that the length of the tip be limited to 7 feet, and, should additional penetration be required where the rock happens to be mudstone and soft, the concrete section of the pile will be forced into the soft rock until adequate driving resistance is obtained. The method of analyses together with the mechanics thereof is presented in Appendix I.

E. Minimum Spacing

Analytical results presented in Appendix I indicate that minimum spacing between centers of piles need only be 3' -6". However, larger spacing may be desirable and would also be beneficial.

F. Lateral Loads and Resulting Maximum Moments and Deflections

The moments and deflections resulting from applying a lateral load to a pile have been analyzed using the non-dimensional approach by M. T. Davisson and H. L. Gill, ASCE Journal of the Soil Mechanics and Foundation Division, May 1963.

A conservative coefficient of subgrade reaction (k_h) of 500 psi has been used, which corresponds to a medium clay. The analysis presented in Appendix I has been carried out for a 16" x 16" concrete pile for both free-head and fixed head conditions. Corresponding deflections have also been plotted for various lateral loads which are shown in Diagram 6.

It is desirable to provide sufficient embedment of piles in the pile caps to achieve "fixed head" conditions, and it is generally assumed that an embedment of one diameter provides sufficient fixity to meet the assumed conditions. It is noted that the comparatively high negative moments for "full fixity" never develop, as there is some rotation at the joint. The provision for negative moments equivalent to the computed positive moments for "fixed head" conditions is deemed adequate.

Diagram 7 shows that a laterally applied load of 40^k at the ground surface will result in maximum moment (fixed head) of approximately 30k' with a corresponding deflection of approximately 3/4". Linearity may be assumed in the extrapolation of these figures. It is also noted that further lateral support may be obtained from battering piles, but it is recommended that batters not exceed 1:12.

G. Pier "C" - Spread Footing In Rock

Pier C will be founded in the outcropping rock at Sta. 1+224.00 as shown in attached Diagram 8, from which it is seen that the right hand side of the substructure will be in excavation while the extreme left section will partly overhang the rock slope.

The rock at this location consists of sandstone whose minimum strength parameters correspond to the triaxial shear test results with $\phi = 26^o$ and c = 2000 psf., through visual observations by the writer, proved the exposed rock is superior to that taken and tested from depths 5, 27, and 54 meters at test borings 30,31 and 12, respectively. It is, therefore, proposed to found this pier in the rock by spread footings, or a single strip footing, with rock strength parameters of $\phi = 26^o$ and c = 2 KSF. The minimum embedment below the lowest adjacent grade is to be 5 feet. The footing near the slope must be sufficiently embedded so as not to be affected by the toe of the rock slope, as shown on Diagram 8.

On the basis of the rock strength parameters quoted above a safe bearing pressure of 25 kips per square foot may be used. The choice between a continuous strip footing and two separate spread footings is one of economy, depending upon the cost of additional excavation and concrete, in the case of a continuous strip footing.

As seen from Diagram 8 the excavation side of the rock must be carried beyond the box substructure and laid back at a safe angle so as not to endanger the pier structure.

It is recommended that the footing excavations be swept clean, free of loose rock and soil prior to placing of steel and pouring of concrete.

8. PILE DRIVING EQUIPMENT AND DRIVING CRITERIA

A. Equipment

It is recommended that fixed leads be used and that swing leads not be permitted. The leads must be capable of rotating slightly to accommodate the desired batter. The rig should preferably be crawler mounted for better stability. The minimum rated energy per single blow of the hammer should be on the order of 56,000 ft. - lbs. corresponding to a Delmag 30 or a Kobe 42. Use of followers is limited to 10 feet. All equipment shall be submitted to the Engineer for approval prior to mobilization.

B. Driving Criteria and Probe Piles

Driving criteria shall be established in the field following the installation of probe piles. These will be installed at each pier prior to ordering the bulk of the piles. At least 10 probe piles shall be driven at each pier and the probe piles shall be driven with the same equipment as is planned for the bulk of the piles.

It is desired to embed piles at least 7 feet in the underlying rock, the surface elevations of which is shown in profile on Diagram 3. However, probe piles should be ordered at least 15 feet in excess of the measured depth to rock. Driving resistance, blows per foot, shall be recorded for the whole length of pile with particular emphasis on the point near the rock surface when driving resistance suddenly increases,

Piles may be stopped short of the required 7 foot penetration if the resistance to driving exceeds 2.5 times that of the ENR formula for the last 6 inches of penetration. On the other hand, if resistance to driving has not reached the minimum blows per foot according to the ENR formula, driving shall continue until the required blow count is attained.

At least one of the probe piles at each pier shall be load tested as described below.

C. Splicing

Where piles to be driven exceed the practical limit of length for a single section, splicing will be necessary. It is assumed that the practical limit is approximately 80 feet, so that two 80 foot sections will have to be spliced to make up the maximum required pile length of 160 feet. However, where an 80 foot section is to be spliced to a shorter section, it is recommended that the shorter section be placed at the bottom; this is in order to gain maximum confinement around the splice as the pile is driven into the bearing stratum.

The splice itself may be any standard or locally available splice such as the bayonette type or dowel type. However, the use of a plate to plate welded splice should be investigated. This type simply consists of two steel plates, each doweled and fixed to one end of each section to be spliced. A full v-weld is then provided along the periphery of the two plates.

The difficulty with plates to be overcome in the field is the lack of squareness of the two plates prior to welding; the first driven section may twist slightly, rotating the end to which one plate is fixed. To avoid this, it is recommended that the first driven section be predrilled to 3/4 its length with a flight auger of diameter equal to the side of the pile. This will assure the maximum plumbness of the driven section and the undamaged condition of the plate. To further ensure plate to plate contact after splicing, it is recommended that both plates be covered by a thin layer of epoxy prior to welding.

D. Load Tests

The piles to be load tested (one in each pier) shall be selected by the Soil Engineers. Tests shall be performed in accordance with ASTM Test Description D1143. The contractor shall provide all the testing equipment for this test inclusive of gauges and dials.

Anchorage piles need not be wasted as they can be used as ordinary bearing piles after the test. This will require driving a sufficient number of specially (tension) reinforced bearing piles around the pile to be test loaded. It is assumed that the anchorage piles and the test load pile will be driven at their design locations. Ordinary piles included in the pile cap as part of the column load support are referred to as "pay piles" and such piles adjacent to the load test pile may be used, wholly or partly, as anchorage piles, provided they are not damaged and are re-tapped after use. The test piles may also be used as a pay pile, provided, too, they are undamaged and have not been loaded beyond the "elastic zone."

Criteria to be applied in evaluating load carrying capacity of pile that has been loaded to at least twice the design load is to be based on U. B. C. Code Section 2908, 1970 edition, Method 2 or 3, except that Method 2 shall be modified as follows:

Method 2: It shall not exceed one half of the load at a point on the load vs. settlement curve which causes a net rate of settlement of 1/100" per ton of test load which has been applied for 40 hours.

Method 3: It shall not exceed one half of that load under which, during a 40 hour period of continuous load application, no additional settlement takes place.

Satisfactory completion of the load tests shall constitute confirmation of the performance ability of the pile and the bulk of the piles may then be ordered based upon the probe pile lengths.

9. LIMITATIONS

This report necessarily assumes uniform variation of soils between test borings. If any unusual conditions, such as soft pockets or unusually high groundwater are encountered when making excavations, the owner or his representative should notify the Soil Engineer immediately, so that supplementary recommendations can be made.

This report is issued with the understanding that it is responsibility of the owner or this representative to ensure that the applicable provisions of the recommendations contained herein are called to the attention of the Architect and the Structural Engineers and incorporated into the plans and that the necessary steps are taken to see that the contractor and sub-contractors carry out such provisions in the field.

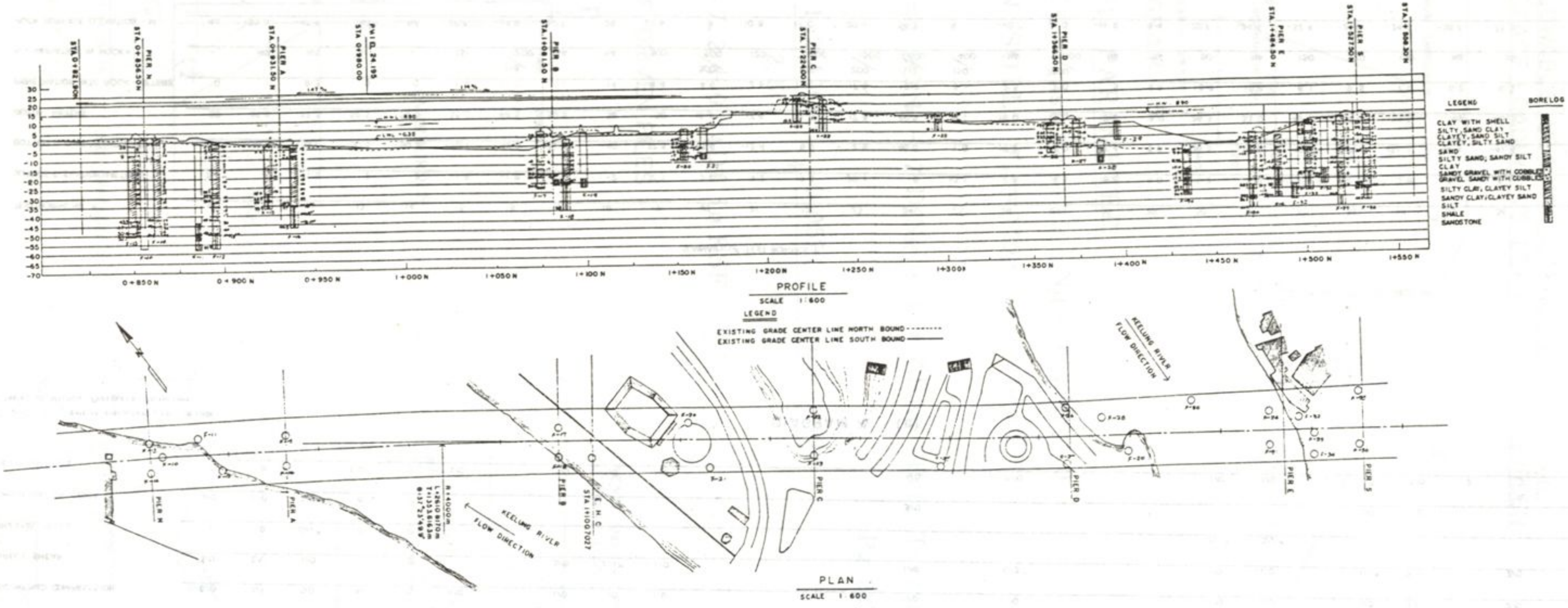

PIER N
STA. 0+836.30 N
PIER A
STA. 0+931.30 N
PW1 EL 24.195
STA. 0+980.00
PIER B
STA. 1+081.30 N
PIER C
STA. 1+224.00 N
PIER D
STA. 1+366.30 N
PIER E
STA. 1+484.30 N
PIER S
STA. 1+527.50 N
STA. 1+566.30 N
30
25
20
15
10
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
-50
-55
-60
-65
-70
0+850 N
0+900 N
0+950 N
1+000 N
1+050 N
1+100 N
1+150 N
1+200 N
1+250 N
1+300 N
1+350 N
1+400 N
1+450 N
1+500 N
1+550 N
LEGEND
CLAY WITH SHELL
SILTY, SAND CLAY
CLAYEY, SAND SILT
CLAYEY, SILTY SAND
SAND
SILTY SAND, SANDY SILT
CLAY
SANDY GRAVEL WITH COBBLES
GRAVEL SANDY WITH COBBLES
SILTY CLAY, CLAYEY SILT
SANDY CLAY, CLAYEY SAND
SILT
SHALE
SANDSTONE
BORE LOG
PROFILE
SCALE 1:600
LEGEND
EXISTING GRADE CENTER LINE NORTH BOUND
EXISTING GRADE CENTER LINE SOUTH BOUND
KEELUNG RIVER
FLOW DIRECTION
PIER N
PIER A
PIER B
PIER C
PIER D
PIER E
PIER S
E.H.C.
STA. 1+100.7027
PLAN
SCALE 1:600

DIAGRAM 3

NUMERICAL SUMMARY OF LABORATORY TESTS

TEST BORING NO. F	10	11	12	13	14	15	16	17	18	19	20	21	22	23
STANDARD PENETRATION TESTS[1]	21/0	11/3	18/1	20/0	23/0	9/1	13/3	10/0	10/0	10/1	7/3	9/0	1/3	0/8
DRY DENSITY & WATER CONTENT	18/0	10/2	17/1	12/0	7/0	6/0		4/0	6/0	2/0		5/0		
ATTERBERG LIMITS (L.L., P.L., P.I.)	8/0	5/1	8/0	6/0	6/0	5/0		4/0	5/0	3/0		2/0		
UNCONFINED COMPRESSION	8/0	3/1	7/0	1/0	1/0	1/0		1/0	1/0	1/0				
DIRECT SHEAR	3/0	2/0	3/0	0	0			3/0	2/1*	1/0				
TRIAXIAL TESTS	0	0	0/1*	0	0									
CONSOLIDATION TESTS	3/0	2/0	3/0	0	0									
SIEVE ANALYSIS	0	0		2/0	1/0	1/0		1/0	1/0	1/0				

TEST BORING NO. F	24	25	26	27	28	29	30	31	32	33	34	35	36
STANDARD PENETRATION TESTS[1]	11/3	2/1	9/0	6/3	2/0	0/0	4/1	6/0	13/1	5/1	11/0	19/2	13/1
DRY DENSITY & WATER CONTENT	4/1		4/0		2/0		2/0	3/0		2/0	10/0	2/0	5/0
ATTERBERG LIMITS (L.L., P.L., P.I.)	3/0		1/0		0/0		2/0	4/0		2/0	3/0	4/0	5/0
UNCONFINED COMPRESSION	1/0		1/0				1/0	1/0		1/0	3/0	1	2/0
DIRECT SHEAR	1/0		0/3*				1/0	1/0		1/1*	2/0	1	2/0
TRIAXIAL TESTS							0/1*	0/2*					
CONSOLIDATION TESTS	3/0									3/0	0		
SIEVE ANALYSIS	1/0		2/0				1/0	1/0		1/0	3/0	2	

DIAGRAM 4 (a)

1. 21/0 = 21 Tests in overburden, zero in rock.

* Tested on Michael Proszker's Instruction

SUMMARY OF TEST BORINGS F

TEST BORING NO. F	10	11	12	13	14	15	16	17	18	19	20	21	22	23
SURFACE ELEVATION (METERS)	2.2	-1	1.8	2	2	-1.8	-1.8	-1.2	-2	2.5	3.2	4.2	19.5	20.1
BOTTOM ELEVATION (METERS)	-57.8	-56.8	-55.7	-50.5	-50.5	-35.1	-46.3	-31.7	-40	-27.5	-13.6	-13.4	6.0	2.6
DEPTH - METERS	60	55.8	57.5	52.5	52.5	33.3	44.5	30.5	38	30	16.8	17.6	13.5	17.5
PENETRATION INTO ROCK - METERS	0	1	8.3	0	0	7.4	11.7	1	14	4.6	4.6	2.6	11.0	17.5
N - VARIATION IN ROCK*	-	140	165	-	-	131	116/1200	NA	NA	300	300/600	120	200/1500	200/1500
ROCK SURFACE ELEVATION - M	NA	-55.8	-47.4	NA	NA	-27.7	-34.6	-30.7	-26	-22.9	-9	-10.8	17.0	20.1

TEST BORING NO. F	24	25	26	27	28	29	30	31	32	33	34	35	36
SURFACE ELEVATION (METERS)	-6	10.1	2.8	4.5	-5.2	4.0	-10.2	-6.8	1.6	2.9	3.8	4.1	4.1
BOTTOM ELEVATION (METERS)	-46.5	1.0	-13.2	-15.0	-17.4	-5.6	-36.5	-39.8	-40.5	-34.1	-31.5	-44.9	-41.4
DEPTH - METERS	40.5	9.1	16.0	19.5	12.2	9.0	26.3	33.0	42.1	37.0	35.3	49	45.5
PENETRATION INTO ROCK - METERS	12.2	5.1	2.5	7.0	.5	9.6	13.8	15.4	8.1	9.5	7.0	6.2	6.3
N - VARIATION IN ROCK*	112/600	150/400	NA	300/1000	NA	NA	90	NA	150	320	150	NA	150
ROCK SURFACE ELEVATION - M	-34.3	6.1	-10.7	-8.0	-16.9	3.4	-22.7	-24.4	-32.4	-24.6	-24.5	-38.7	-35.4

DIAGRAM 4 (b)

*Blows for 30 cm. penetration
Not available

ALLOWABLE BEARING CAPACITY FOR ROCK SURFACE LOADING VS DEPTH OF OVERBURDEN

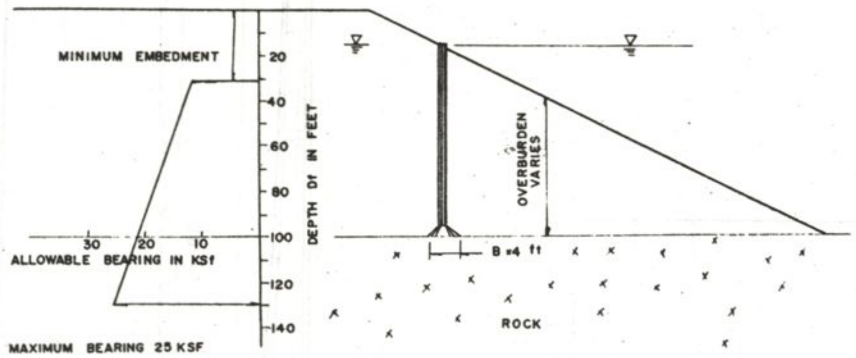


DIAGRAM 5

ASSUMED SOIL PARAMETER

ROCK: $\phi : 20^\circ$, $C = 1$ksf, $Nc = 15$, $Nq = 6$, $Nf = 0$

OVERBURDEN SOIL: $\gamma_s = 120$pcf, $\gamma_b = 57$pcf

S.F. = 2.5

$2.5q_{dr} = 1.2C \cdot Nc + \gamma\, DfNg + 0.6\,\gamma\, BNq$ Ref. Terzaghi & Peck Pg 223 1967ed.

$$q_{all} = \frac{1.2(1)(15)}{2.5} + \frac{0.057(D_f)(6) + 0}{2.5}$$

$$q_{all} = 7.2 + 0.14Df \text{ in KSF}$$

LOAD CAPACITIES OF DRIVEN PILES

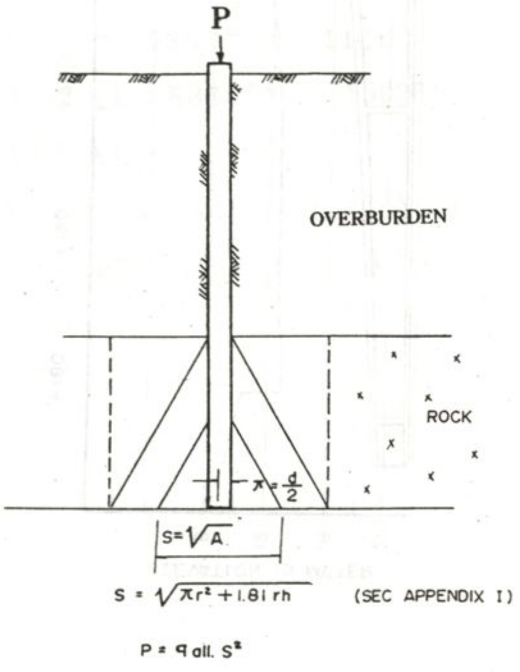


DIAGRAM 6

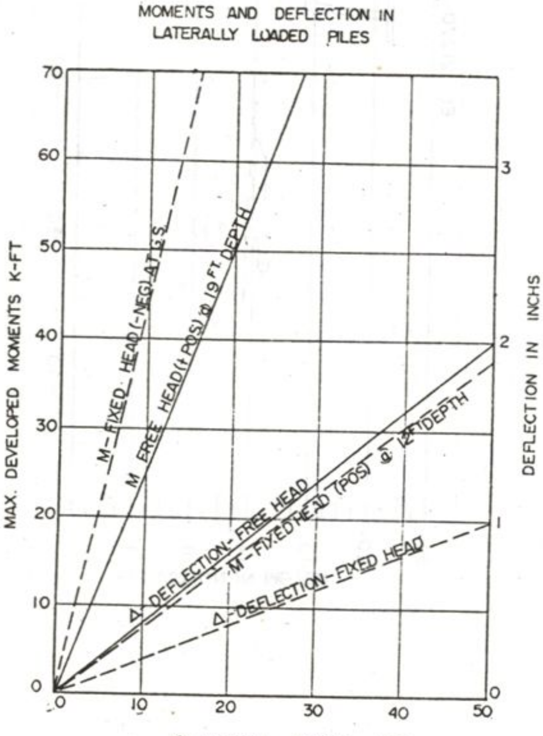
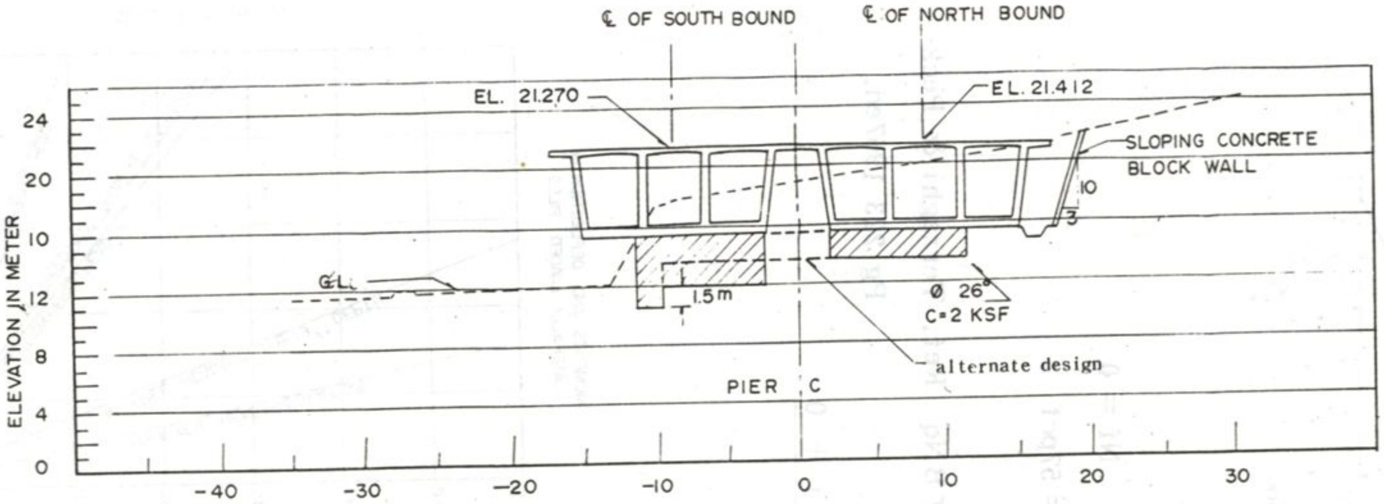
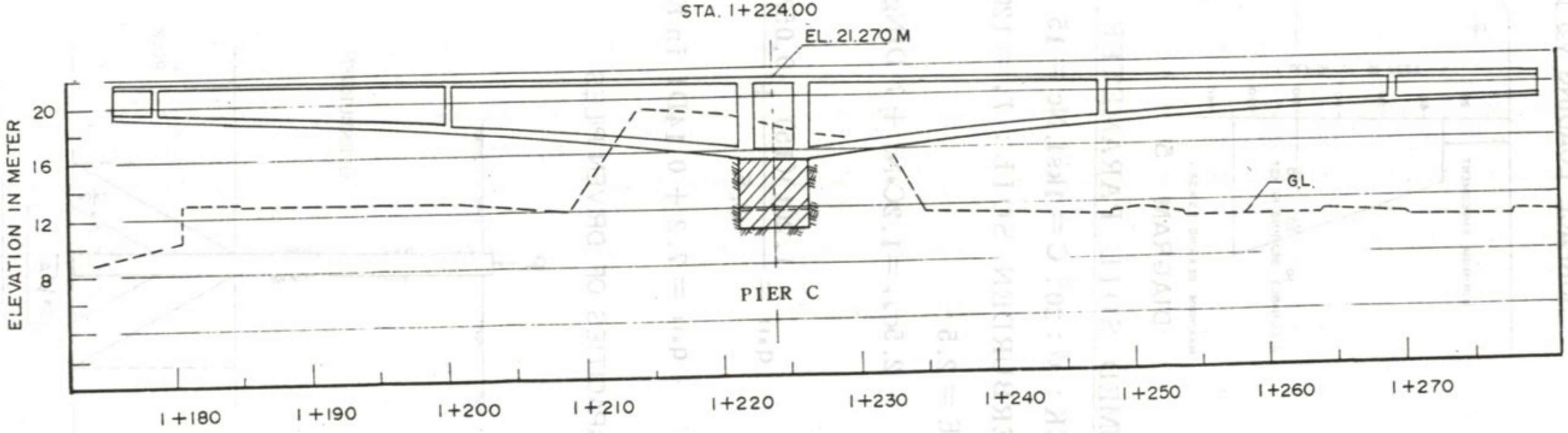


DIAGRAM 7



SECTION AT STA. 1 + 224.00 (PIER C)

(a)



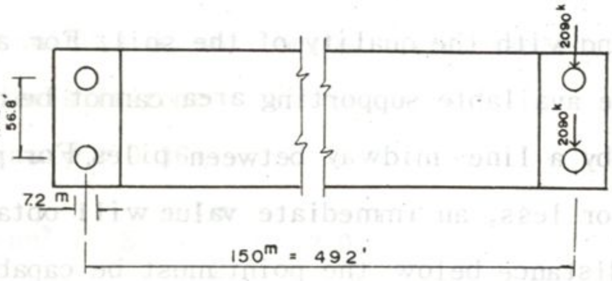
CENTERLINE OF SOUTH BOUND

(b)

DIAGRAM 8

APPENDIX I

A TYPICAL LOADS



PIER 1.

VERTICAL

D.L. = 6300^{Tm} = 13860^{K}

L.L. = 530^{Tm} = 1166^{K}

TOTAL : 6830^{Tm} = 15026^{K}

LATERAL

V = 0.15 (D.L. + L.L.) = 0.15 (15,026) = $2{,}254^{K}$

MAX MOMENT = 10,000 Ton-meters = 72,182 h-ft.

LOAD CARRYING CAPACITY OF FRICTION PILES EMBEDDED IN BEARING STRATUM

1 REQUIREMENTS

①Each pile must be Capable of carryiny its load as a compression member.

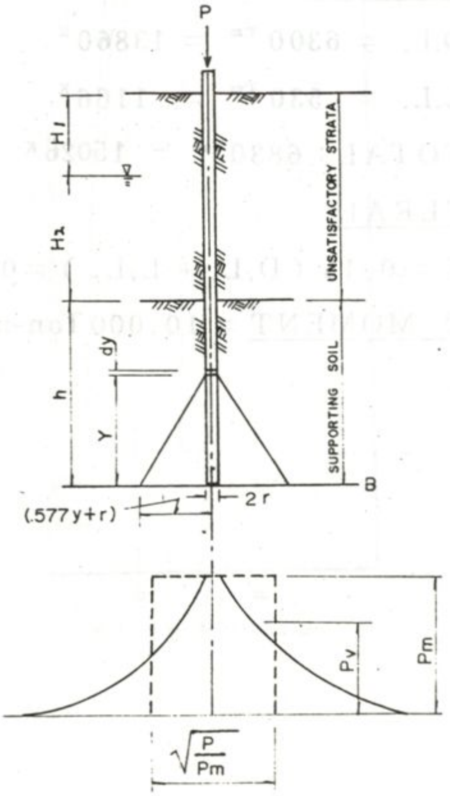
②Each pile must be driven far enough into the soil to transmit its load to the soil by bearing on its point and shear on its periphery without excessive motion relative to the soil.

③The soil at the horizon of each pile point must be capable of carrying the full pile load on the area available for that purpose with a suitable factor of safety against failure. For a single pile the area available for support will vary with the length of embedment of the pile in the supporting soil and with the quality of the soil. For an interior pile in a large group, the available supporting area cannot be much larger than the area bounded by a line midway between piles. For piles in small groups, say four or less, an immediate value will obtain.

④The soil at any distance below the point must be capable of carrying the full pile load on the larger area available because of the lateral spread of the pile loads, again with a suitable factor of safety against failures.

Ref. : Pile Foundations for Building
by john W. Dunham ASCE
JAN. 1954.

2 ISOLATED PILES IN ϕ−C MATERIAL



①Assume full passive pressures on the embedded pile section in supporting soil.

②P = Pile load

③ h = Required embedment in supporting soil for frictional resistance only.

④ H_1 = Depth to ground water surface.

⑤ $H_1 + H_2$ = Depth to surface of supporting soil.

⑥ Pm = Max. pressure at tip of pile.

⑦ a = Cross sectional area of embedded pile.

⑧ Load carried by fricfion = $(P - P_m a)$

⑨ Friction area of pile in supporting soil = o = $2\pi rh$.

3 REQUIRED EMBEDMENT IN SUPPORTING STRATUM

$$h = \frac{P - P_m a}{o〔(\gamma_1 H_1 + \gamma_b H_2)(\tan^2(45^\circ + \frac{\phi}{2}) + \tan\phi + c〕}$$

γ_1 = Unit wt. of soil above water table.

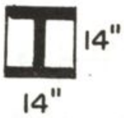
γ_b = Bouyant unit wt. of soil below water table = 60 pcf

ϕ = 20° (for siltstone)

$$\tan\phi = 0.364$$

$$\tan^2(45^\circ + \frac{\phi}{2}) = 2.03$$

C = 1 ksf

O = 

$\frac{4 \times 14''}{12} = 4.67\ ft^2/ft$

$P = 360^K$

Pm= Try 20 ksf

$$a = \frac{14'' \times 14''}{144} = 1.4\ ft^2$$

H_1 = 0 (Assume water table at surface)

H_2 = 150 ft. Max.

$$h = \frac{360 - 20(1.4)}{4.7〔(0 + 0.060 \cdot 150)(2.03)(0.364) + 1〕} = \frac{332}{4.7(6.7 + 1)}$$

h = 9.2ft. Say $9\frac{1}{2}$ ft.

4. EVALUATION OF MAX. SOIL PRESSURE AT POINT HORIZON OF PILE EMBEDDED 150 FT. (H_2) IN NON-SUPPORTING SOIL AND 9½ FT (h) IN SUPPORTING SOIL

$$dP_m = \frac{P - P_m \pi r^2}{h} \cdot dy \frac{1}{\pi [(.577y + r)^2 - r^2]}$$

$$dP_m = \frac{P - P_m \pi r^2}{\pi h} \cdot \frac{dy}{(0.577y + r)^2} \quad \text{(sacrifice)}$$

Neglect $- p_m \pi r^2$

$$dP_m = \frac{P}{\pi h} \cdot dy \frac{1}{(0.577f + \gamma)^2}$$

$$p_m = \frac{P}{\pi h} \int \frac{1}{(.577y + r)^2} \cdot dy$$

$$p_m = \frac{P}{3.14 r^2 + 1.81 rh} = \frac{P}{a + 1.81 rh}$$

$a = 1.4 \text{ ft}^2 \qquad h = 9\frac{1}{2} \text{ ft} \qquad P = 360^K$

$r = 0.67$ ft. (Equiv)

$$P_m = \frac{360^K}{1.4 + 1.81 (.67) (9.5)}$$

$$P_m = \frac{360^K}{1.4 + 11.5}$$

$P_m = 28$ ksf $>$ 20 ksf O.K.

Ref : Pile Foundation For Buildings by J.W. Dunham M. ASCE JAN. 1954

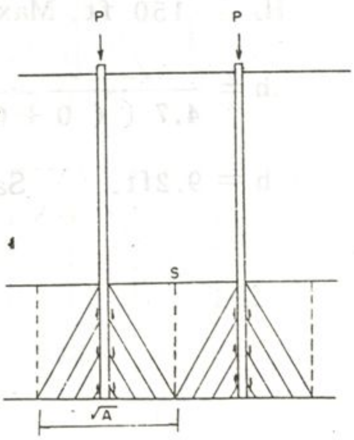
5. EVALUATION OF EFFECTIVE LOADED AREA UNDER INDIVIDUAL PILE TIP AND MIN. SPACINGS

$$\sqrt{A} = \sqrt{\pi r^2 + 1.81 rh}$$

$$= \sqrt{1.4 + 1.81 (0.67) (9.5)} = \sqrt{13.0}$$

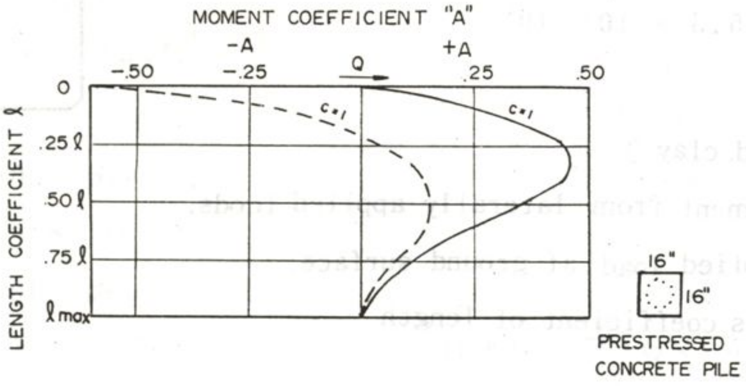
$$= 3.6'$$

$\therefore$ Min spacing $= 3' - 6''$



B. MOMENTS AND DEFLECTIONS IN PILES DUE TO LATERAL LOADS

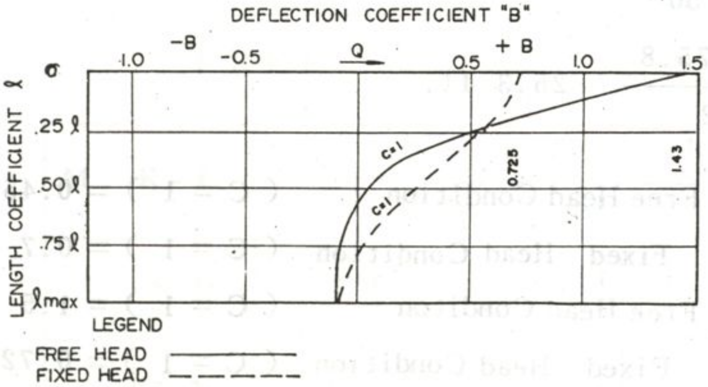
1. MOMENTS



$$M = QA\sqrt[4]{\frac{EI}{k_h}} = QAR$$

$$\ell max = 4R \qquad R = \sqrt[4]{\frac{EI}{k_h}}$$

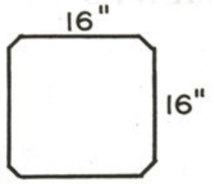
2. DEFLECTIONS



$$\text{DEFLECTION } \triangle = BQ\frac{R^3}{EI} = \frac{BQ}{(E)^{1/4}\ K}$$

Ref : Laterally loaded pills in a layered soil system.
M.T. Davisson & H.L. Gill

C. LATERAL LOAD CAPACITIES FOR 16in × 16in PRESTRESSED CON. PILES



$E = 3 \times 10^6$ psi (conc.)

$$I = \frac{1}{12}(16)^4 = 5.5 \times 10^3 \ \text{in}^4$$

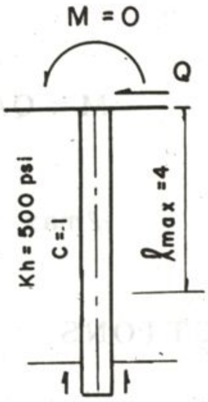
K_h = 500psi (Med. clay)

M = Resulting moment from laterally applied loods.

Q = Laterally applied load af ground surface.

ℓ = Dimensionless coefficient of length

$$M = Q\triangle\sqrt[4]{\frac{EI}{K_h}} \qquad \triangle = \text{Moment Coeff.}$$



$$M = Q\triangle R \qquad R = \sqrt[4]{\frac{EI}{K_h}}$$

$$\frac{L}{R} = \ell_{max} = 4 \quad L = 4\ R$$

$$R = \sqrt[4]{\frac{3.0 \times 10^6 \times 5.5 \times 10^3}{500}} = 75.8 \text{ in}$$

$$L = 4\ R = \frac{4 \times 75.8}{12} = 25.3 \text{ ft.}$$

$A = M_{ly}$ for Free Head Condition (C = 1) = 0.45

$A = M_{ly}$ for Fixed Head Condition (C = 1) = 0.7

$B = Y_{ly}$ for Free Head Conditon (C = 1) = 1.5

$B = Y_{ly}$ for Fixed Head Condition (C = 1) = 0.725

$$\triangle_{fixed} = \frac{BQ}{(EI)^{1/4} \cdot (Kh)^{3/4}} = \frac{0.725\ Q}{359 \times 107} = 0.019\ Q$$

(Qin kips)

$$\triangle_{free} = \frac{1.5\ \ Q}{359 \times 107} = 0.039\ Q$$

(Qin kips)

D. RESULTING MOMENTS AND DEFLECTIONS FOR VARIOUS VALUES OF " Q "

$M = Q \triangle R$

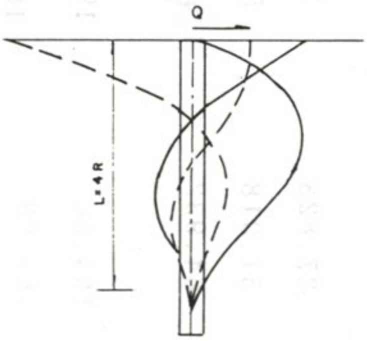
$$\triangle = \frac{B Q}{(E I)^{1/4} \cdot K_h^{3/4}}$$

Assume :

(1) K_h = 500 psi

(2) R = 75.8 in

(3) K = 500 psi



DEFLECTION AND MOMENTS

$M = Q \triangle R$

$$\text{Free } +M_{l,} = Q \times 0.4 \times \frac{75.8}{12} = 2.53 \text{ Q k-ft}$$

$$\text{Fixed } -M_{l,} = Q \times 0.7 \times \frac{75.8}{12} = -4.42 \text{ Q k-ft}$$

$$\text{Fixed } +M_{l,} = Q \times 0.12 \times \frac{75.8}{12} = 0.76 \text{ Q k-ft}$$

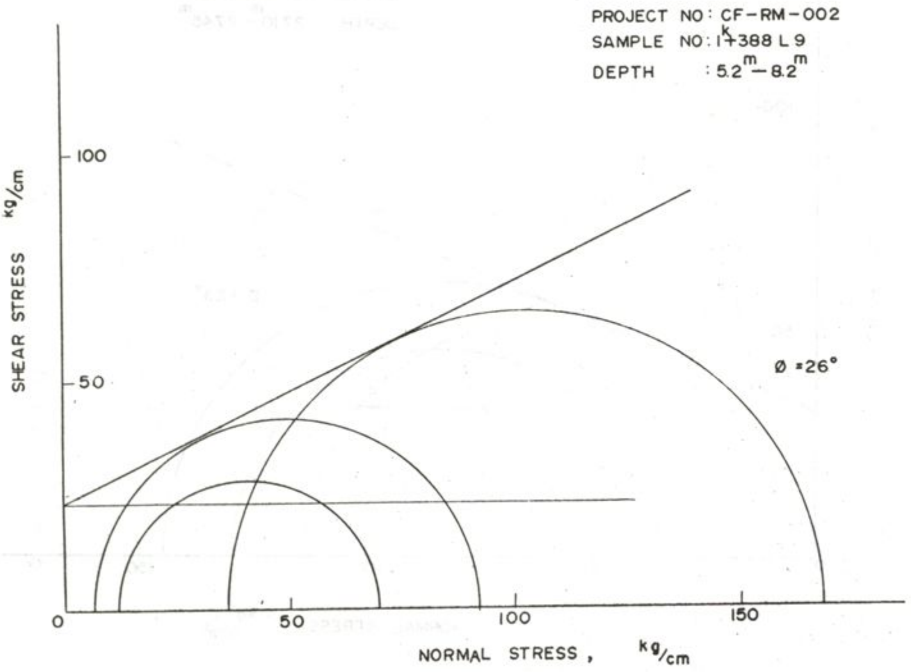
Qin Kips	FREE		Fixed		
	M K-ft	△"	M K- ft		△"
			G.S.	@ 12'	
1	+ 2.53	.04	− 4.4	+ 0.76	.02
10	+ 25.3	0.4	− 44.	+ 7.6	0.2
20	+ 50.6	0.8	− 88.	+ 15.2	0.4
30	+ 75.9	1.2	− 132	+ 22.8	0.6
40	+ 101.2	1.6	− 172	+ 30.4	0.8

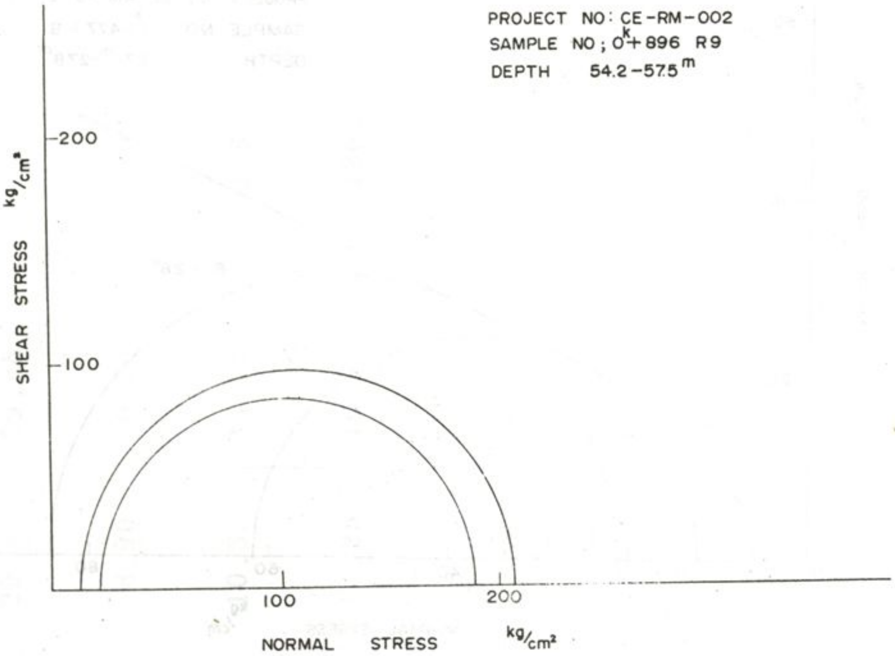
SUMMART OF TRIAXIAL TESTS ON ROCK CORES

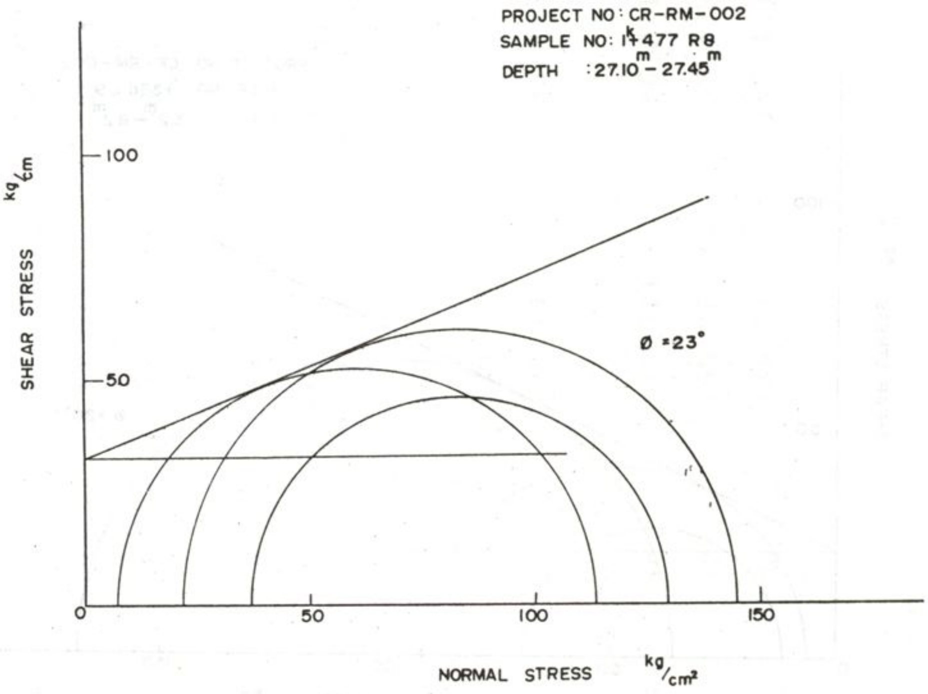
SAMPLE No.	DEPTH M	DIAMETER CM	HEIGHT CM	MASS gm	P_3 kg / cm²	(p_1-P_3)max. kg/ cm²	(P_1-P_3)min. kg / cm²
1^{K} + 477R8	27.6---27.8	5.29	9.255	396.73	6	27.828	27.832
F−31					12	51.018	50.01
					21	64.932	63.0
o^{K} + 896 R9	54.2 ---57.5	5.289	10.84	521.70	12	194.868	194.86
F−12					21	167.03	167.03
1^{K} + 477 R8	27.10---27.45	5.379	10.335	486.09	6	107.654	107.118
F−31					21	125.596	124.971
					36.5	94.197	93.728
1^{K} + 388 L−9	5.2---8.2	5.385	9.02	440.50	6	89.514	86.071
F−28					12	60.87	58.529
					36.5	138.747	138.411

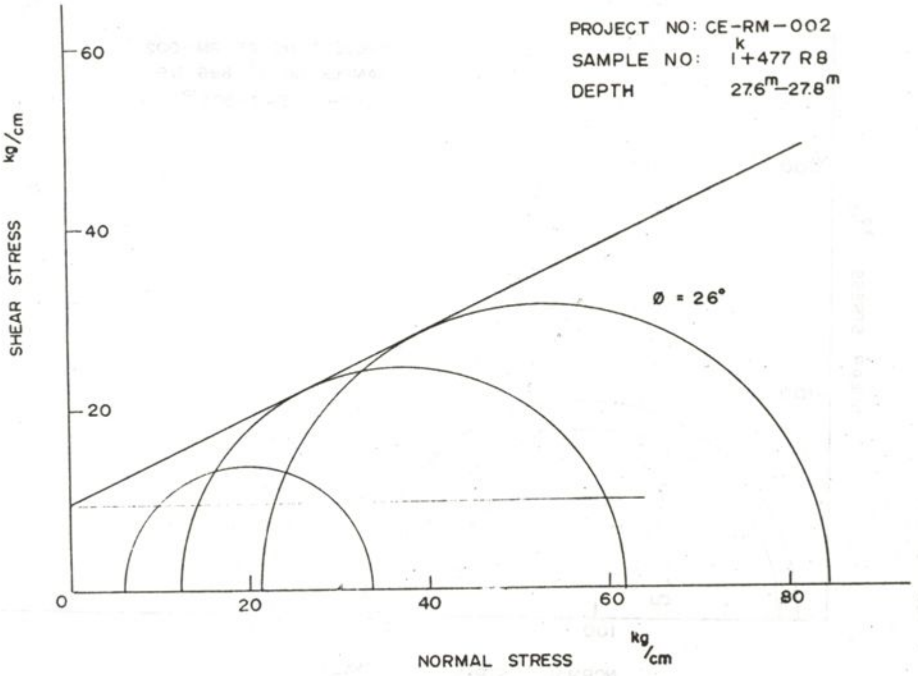
※ UNCORRECTED FOR CHANGE IN LENFTH OR CROSS SECTION AREA

※※ CORRECTED FOR L/D RATIO

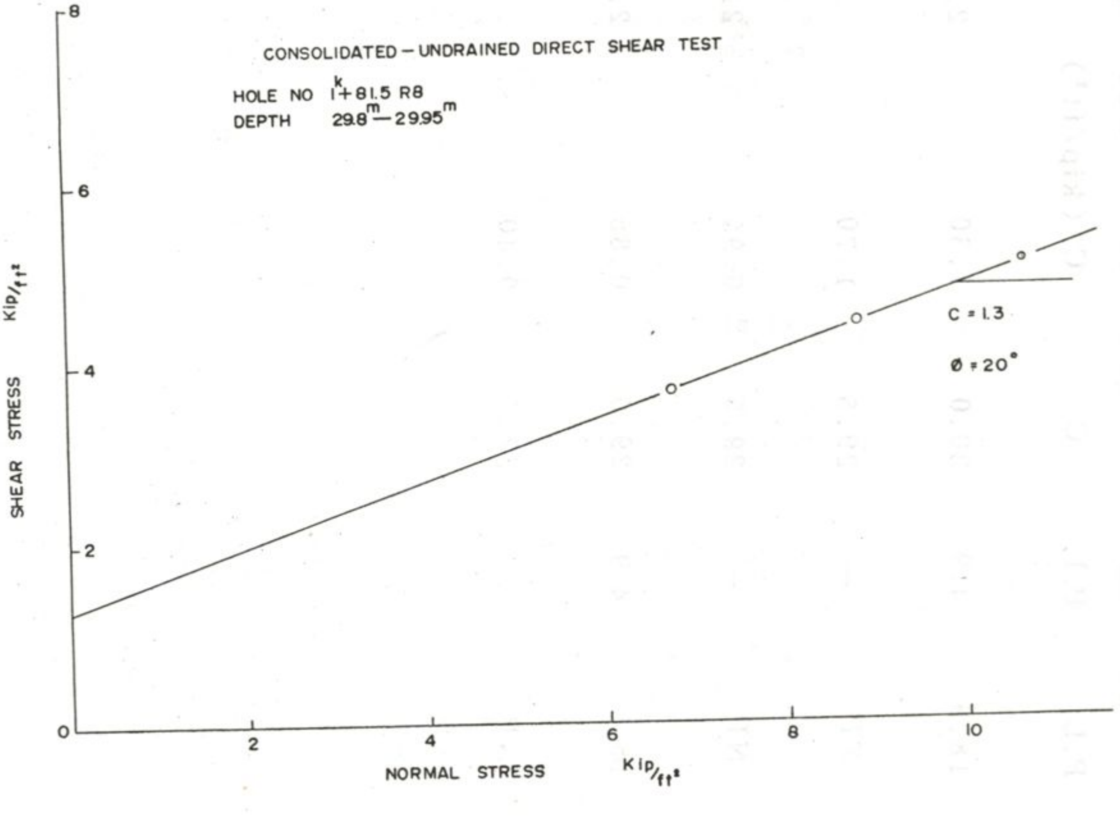

PROJECT NO: CF-RM-002
SAMPLE NO: 1k+388 L9
DEPTH : 5.2m – 8.2m
100
50
SHEAR STRESS kg/cm
Ø =26°
0
50
100
150
NORMAL STRESS, kg/cm

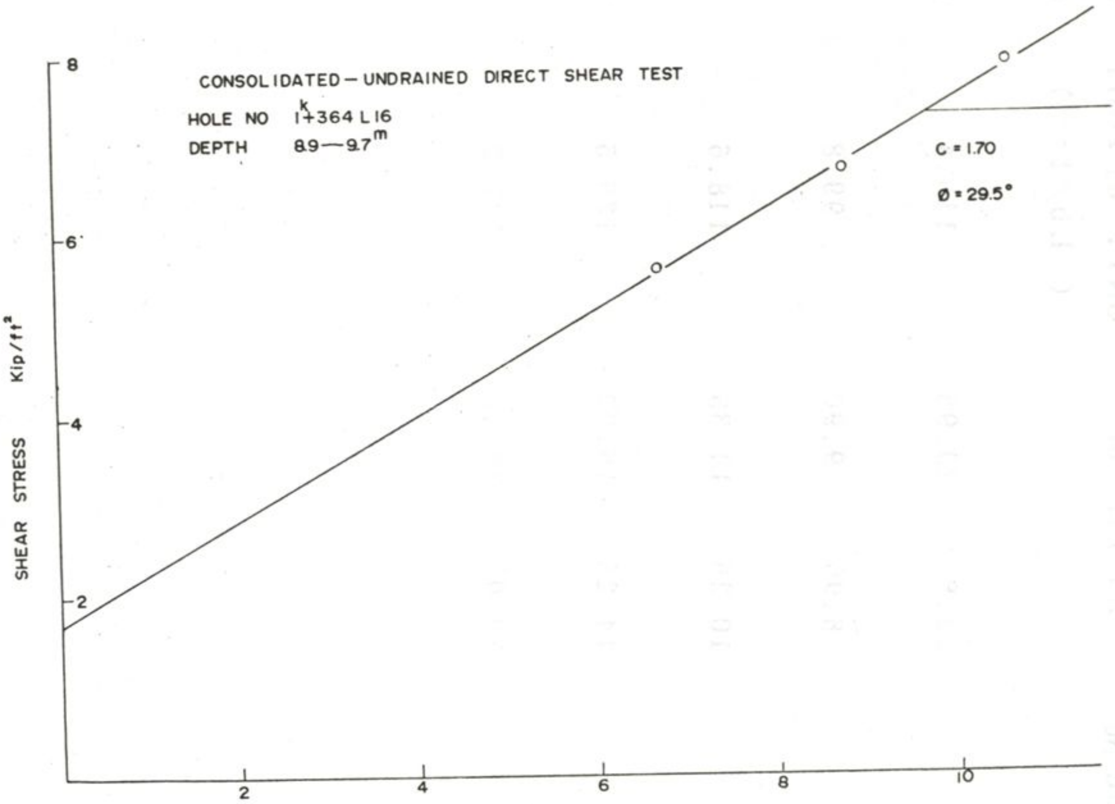

PROJECT NO: CE-RM-002
SAMPLE NO; 0k+896 R9
DEPTH 54.2-57.5m
200
100
SHEAR STRESS kg/cm²
100
200
NORMAL STRESS kg/cm²

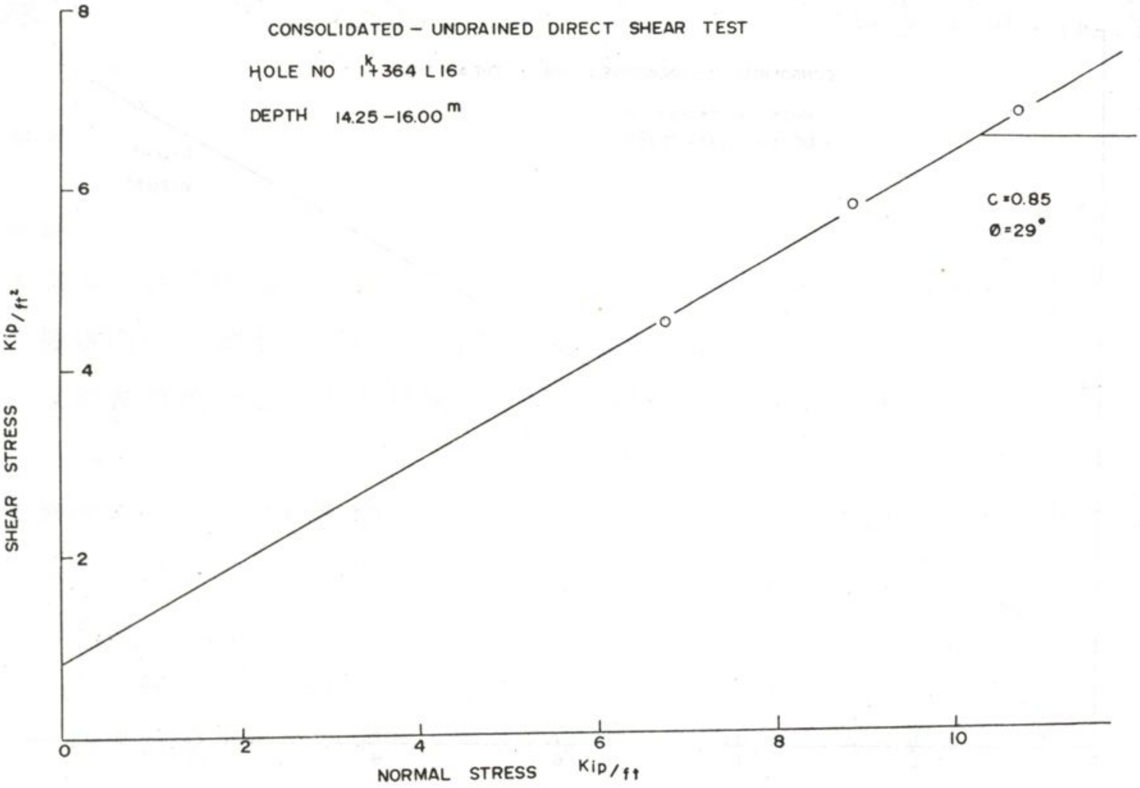

PROJECT NO: CR-RM-002
SAMPLE NO: 1^k+477 R8
DEPTH : 27.10^m - 27.45^m
SHEAR STRESS kg/cm
100
50
Ø = 23°
0
50
100
150
NORMAL STRESS kg/cm²

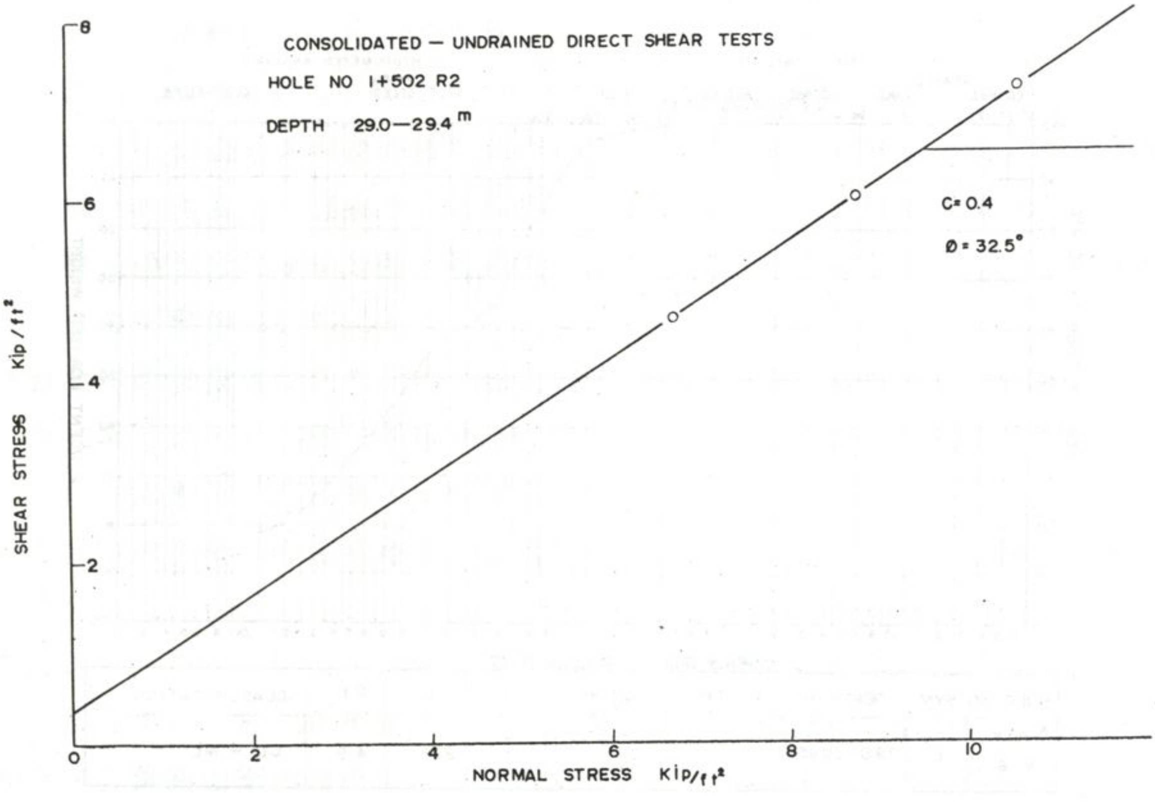

PROJECT NO: CE-RM-002
SAMPLE NO: 1^k+477 R8
DEPTH 27.6^m–27.8^m
SHEAR STRESS kg/cm
60
40
20
Ø = 26°
0
20
40
60
80
NORMAL STRESS kg/cm

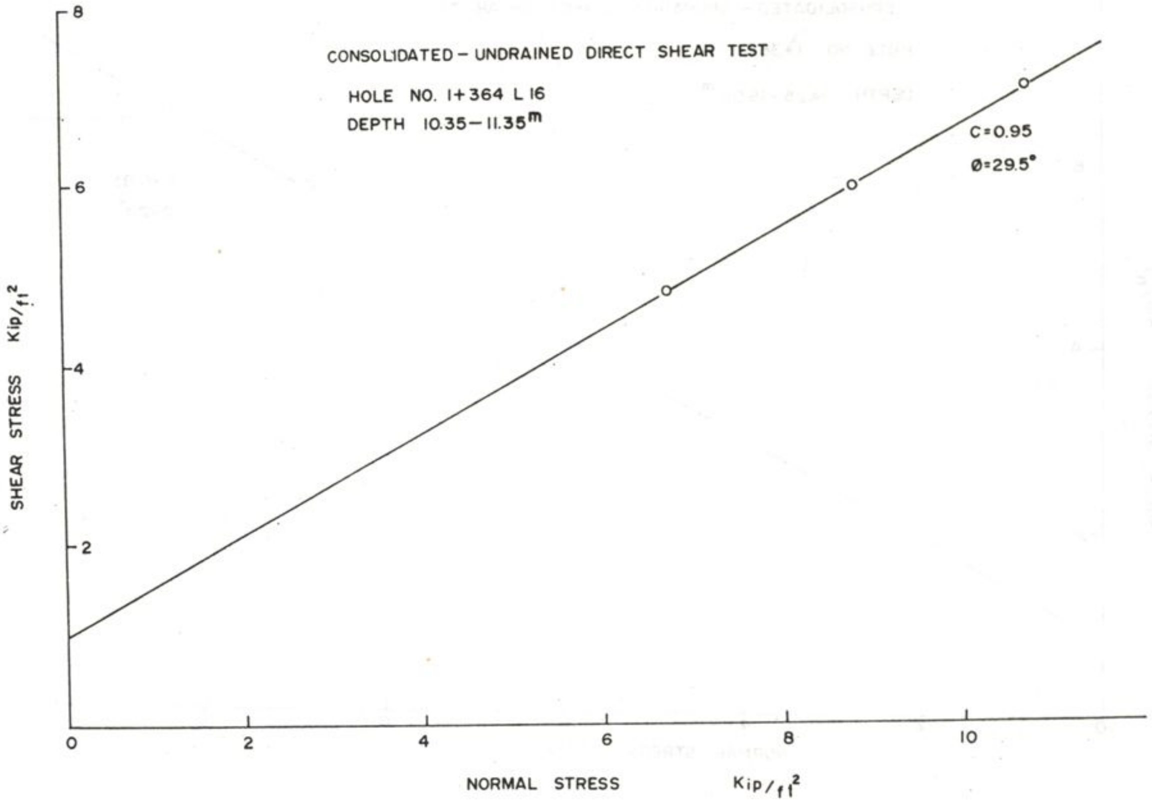
SAMPLE No.	DEPTH (m)	UNIT WEIGHT	ATTERBERG LIMIT (%)			DIRECT SHEAR TEST		S.G.
		(Lb/ft³)	L.L.	P.L.	P.I.	C	C. (kip/ft²)	
	29.8 — 29.95	115.4	23.8	18.9	4.9	20.0	1.30	2.67
	8.90 — 9.90	99.8	—	NP	—	29.5	1.70	
	10.35 — 11.35	118.6	—	NP	—	29.5	0.95	2.55
	14.25 — 16.00	126.5	23.8	18.9	4.9	29.0	0.85	2.52
	29.00 — 29.40	147.9	—	NP	—	32.5	0.40	2.62

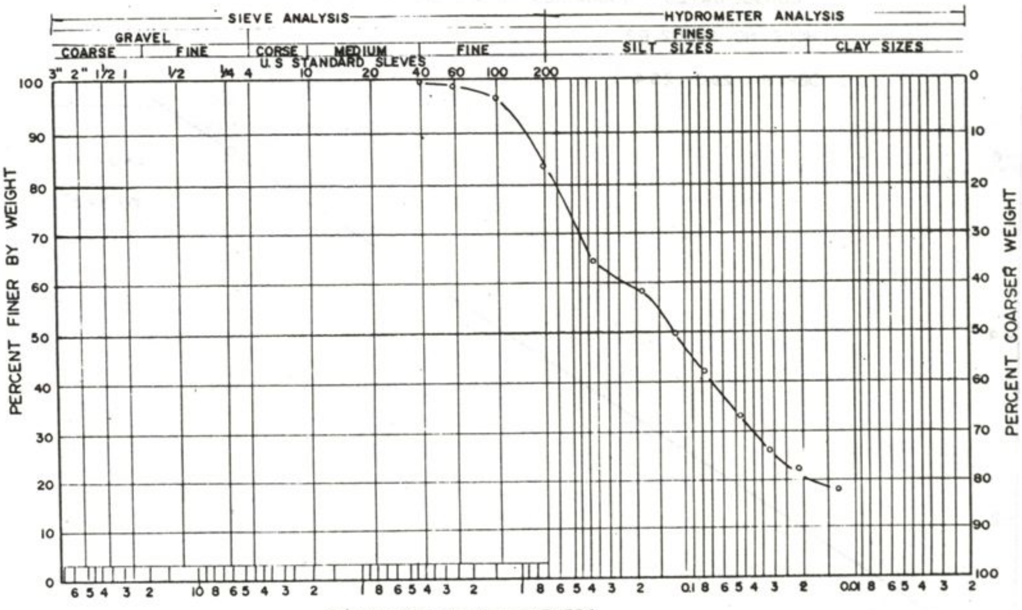

CONSOLIDATED – UNDRAINED DIRECT SHEAR TEST
HOLE NO 1k+81.5 R8
DEPTH 29.8m – 29.95m
C = 1.3
Ø = 20°
SHEAR STRESS Kip/ft²
NORMAL STRESS Kip/ft²
0
2
4
6
8
10


CONSOLIDATED – UNDRAINED DIRECT SHEAR TEST
HOLE NO 1k+364 L16
DEPTH 8.9 – 9.7m
C = 1.70
Ø = 29.5°
SHEAR STRESS Kip/ft²
NORMAL STRESS Kip/ft²
2
4
6
8
10


CONSOLIDATED – UNDRAINED DIRECT SHEAR TEST
HOLE NO 1k+364 L16
DEPTH 14.25 – 16.00 m
C = 0.85
Ø = 29°
SHEAR STRESS Kip/ft²
NORMAL STRESS Kip/ft
0
2
4
6
8
10


CONSOLIDATED – UNDRAINED DIRECT SHEAR TESTS
HOLE NO 1+502 R2
DEPTH 29.0 – 29.4 m
C = 0.4
Ø = 32.5°
SHEAR STRESS Kip/ft²
NORMAL STRESS Kip/ft²
0
2
4
6
8
10





HOLE NO	SYM	DEPTH (M)	D_{10} (MM)	D_{60} (MM)	Cu	L.L.	P.I	CLASSIFICATION
K 1+81.5 R. 8	O	29.8 – 29.95				23.8	4.9	CL – ML

附錄三　高速公路圓山至內湖段水工模型試驗報告

一、結論與建議

(一)結論

1. 高速公路圓山橋其橋墩於中山橋瓶頸河段兩次跨越基隆河，原設計橋墩有主橋墩四座位於主要河槽中，兩端並有實體橋台兩座，位於洪水河槽中，其中以兩實體橋台之阻水最爲嚴重，已照試驗成果改爲透水橋柱，以減輕橋樑之阻水。

2. 修改後之高速公路橋橋墩布置，對基隆河水流之影響較原設計者爲輕，惟仍產生抬高水位、增加流速及改變流向等不利影響，其程度如下：

⑴水位之抬高：

在 20 及 200 年頻率洪水下，氾濫水深分別增加約 25 及 10 公分。防洪計畫完成後將提高上游大直堤防計畫洪水位及堤頂標高 15 公分左右。

⑵流速之增加：

橋墩段之流速劇變，圓山鐵路橋右岸堤脚處，流速由 1.24 秒公尺增至 2.96 秒公尺，將危及橋墩及附近堤防之安全；上游河段則除局部地區外，流速變緩，可能引起淤積，而影響洪水之排洩。

⑶流向之改變：

中山橋至圓山鐵路橋間最爲顯著，尤以橋墩N，將主流挑向右岸，形成 40 度偏角，直接冲向鐵路橋右岸堤脚處，致壓力強度由原承受之靜水壓每平方公尺 5.3 公噸，再增加動量壓力強度每平方公尺 0.66 公噸；總外力則增加 24 %以上。

3. 台北市政府擬議中之松江橋，於高速公路橋上游跨越基隆河，橋墩跨距僅 23 至 35 公尺，有橋墩 23 排共 46 座，洪水期橋墩阻水，抬高上游洪水位，一旦爲流木及浮滓阻塞，勢將發生災害。

4. 台北市政府擬議中之承德橋，因橋墩阻水致抬高上游段至大直橋間水位約 5 公分。

5. 高速公路松江交流道，主道及南下進流匝道部分塡土路基伸入河床，如將其改爲高架式，於 200 年頻率洪水時可減輕上游洪水位約 5 公分。

6. 高速公路圓山至內湖段完成後，基隆河自民生東路末端至圓山橋間，尙有台北機場東端、高速公路路基涵洞及大直高架段等三缺口，無法達成台北機場及市區之防洪目的。

7. 基隆河在5年頻率等中度洪水時，主流由機場東端缺口漫流，經濱江街、高速公路大直高架孔，滙還原河槽；漫流流速瞬時間達5秒公尺左右，基隆河有改道之虞。

8. 松江、大直兩堤防興建後對高速公路在水理上尙無特殊之影響。

9. 圓山高速公路橋主橋墩A、B、D、E施工之圍堰築島，橋墩A因位於主流河槽顯著抬高上游洪水位，迴水範圍達松山附近，在2年頻率洪水時，抬高上游水位約 20 公分；5年頻率時達 28 公分。至其他圍堰對洪水位影響尙不顯著。

㈡建議

1. 圓山至內湖段高速公路完工後，台北機場東端、高速公路大直高架段及路基涵洞，仍形成洪水泛濫之缺口，在中等五年頻率洪水時，除無法達成防洪目的外，甚至有改變河道，機場附近形成流路之虞，防洪工程宜配合高速公路及早實施，以策安全。

2. 高速公路原設計之圓山橋二實體橋台及 13 、 14 二橋墩已加改善，松江交流道主道及南下進流匝道伸入河床之塡土路基則宜部分改爲高架，以改善其對基隆河之水流影響。

3. 高速公路之興建，影響圓山堤防再春游泳池段、鐵路橋右岸堤脚、橋墩及河濱公園河岸等處之安全，上述各處宜嚴加保護，以策安全。

4. 中山橋下游右岸堤防，因高速公路圓山橋橋墩之影響，產生流向偏角之負荷力，堤防之安全，除應考慮靜水壓 5.3 公噸／平方公尺外，亦加考慮此外力在堤防上所致之動量 0.66 公噸／平方公尺，以策安全。

5. 高速公路圓山橋橋墩施工，以橋墩A之圍堰築島對洪期水流影響顯著，應於枯水期進行外，其他各橋墩之施工，宜綜合硏究施工順序與洪水季之關係妥爲安排，以免危及圓山堤防及上游地區之安全。

6. 台北市政府擬議中之松江橋，橋墩密集，影響水流，宜加改善。

7. 台北市政府擬議中之承德橋，對基隆河之水流尙不致構成嚴重影響，可照原址辦理有關規劃事宜。

二前　　言

高速公路圓山至內湖段有下列問題與防洪問題有關：

㈠圓山橋兩次跨越基隆河，部分橋墩位於堤防上，影響堤防之安全；部分橋墩則位於河流中，阻礙洪水之渲洩，並影響河性之改變。

㈡大直橋段約 1.13 公里，路基採高架橋式，需賴防洪措施予以保護，在防洪堤

防未興建前，高架橋孔成爲洪水泛濫之主要孔道，影響濱江街及松山機場之安全。

㈢大直橋至內湖橋段約1.6公里，路基採填土方式，其路面高度低於防洪計畫標高，又公路下涵洞（如人行道、地下道工程）未加設防洪措施，公路完成後將成一缺口，影響鄰近地區颱洪期之安全。

㈣圓山交通道部分路基深入預定堤線內，影響堤線布置。

㈤圓山橋主橋墩之施工需採用圍堤，將影響施工期水流之改變，施工程序及洪期保護措施有待檢討。

上述諸水理問題無法以一般水力學理論作解析，交通部台灣區高速公路局有鑒於此，乃委託本會利用淡水河水工模型。藉模型相似律作比較試驗。

另外台北市政府擬議中之松江、承德二橋分別於中山橋上、下游跨越基隆河，爲瞭解其對基隆河水流之影響，亦於本試驗中一併檢討，以供各工程規劃參考。

三試驗計畫

㈠試驗目的

本試驗之目的在利用淡水河水工模型，研究高速公路圓山一內湖段工程對洪水及防洪工程之影響，並一併檢討台北市政府擬議中之松江、承德二橋之阻水影響，以供有關工程規劃或施工之參考。

㈡試驗項目

本試驗之項目包括下列各項：

1. 圓山橋不同橋墩布置、松江橋及承德橋之阻水影響。
2. 圓山至大直段堤線變更及堤防設計資料之蒐集。
3. 圓山一內湖段高速公路完成前後泛濫情況之比較。
4. 大直一內湖段路基與洪水位之比較。
5. 大直、松山堤防與高速公路之配合及其相互影響。
6. 圓山橋下游圓山鐵路橋段流速之變化。
7. 圓山橋橋墩施工期圍堤對水流之影響。

㈢試驗流量

試驗流量分20年及200年頻率洪水兩種，20年頻率洪水配合淡水河外海高低潮位1.20公尺及－1.20公尺等兩種，200年頻率洪水則配合1.91公尺及－1.50公尺兩種，其流量配合如表1.所示。並根據初步試驗成果，淡水河在洪水時，外海潮位影響僅及土地公鼻，對基隆河水流影響不顯著，是以

不同潮位之配合試驗，僅於圓山橋阻水影響辦理外，其他試驗係均在高潮下辦理。

表 1.　淡水河各支流流量分配

頻　　率	流　量　（　秒　立　方　公　尺　）				潮位（公尺）	
年	大漢溪	新店溪	基隆河	淡水河	高　　潮	低　　潮
200	11,000	10,800	3,200	25,000	1.91	－ 1.50
20	6,100	7,500	2,400	16,000	1.20	－ 1.20

㈣模型設計

本試驗係利用既有之淡水河定床全模型辦理，模型範圍上游自大漢溪之板橋樹林間鐵路橋、新店溪之秀朗橋及基隆河之汐止起，下游至淡水河〇〇〇斷面外 4.5 公里外海止。

模型水平比尺為1／300　，垂直比尺為1／50　，不等比為6　，其他比尺之換算均以福氏律（ Froude Law　）為準。詳如表 2.所示。

※請參閱淡水河水工模型試驗報告，經濟部水資源統一規劃委員會。

03 －試－ 07　　中華民國五十五年四月。

表 2.　模型與原體各項比例

比例＼項目	水　　位	流　　速	時　　間	流　　量	糙　　率
關係公式	$1/Y$	$1/Y^{1/2}$	$Y^{1/2}/X$	$1/XY^{3/2}$	$X^{1/2}/Y^{2/3}$
比　　例	1／50	1/7.07	1/42.5	1/106,000	1.27／1

註：X：原體與模型橫比 300

　　Y：原體與模型豎比　50

四圓山橋對水流之影響

基隆河中山橋上下游約1公里之蜿蜒河段，除現有之中山橋、鐵路橋外，現施工中之高速公路橋兩度跨越該河段（圖1.）根據水工試驗知該橋對於基隆河水流之影響有三，即抬高水位、增加流速及改變流向，茲分述如下：

㈠抬高水位

高速公路圓山橋兩度跨越基隆河，原設計橋墩有主橋墩四座位於主要河槽中，兩端並有實體橋台兩座，位於洪水河槽中，橋墩阻水，以兩實體橋台爲最劇，後將該二實體橋台改爲圓形墩柱（圖2.），並將13、14號二橋墩（位於原設計下游實體橋台後）之墩形由矩形改爲圓形（圖2.）阻水現象獲得改善。

按洪水位抬高之影響，在現階段基隆河堤防未完成時，爲泛濫水深之加劇；在防洪計畫完成堤防修築時，爲提高計畫堤防之高度，分述如下：

1.現況防洪布置下，泛濫水深加劇情形：

現況防洪及原擬高速公路圓山橋布置下，基隆河主要控制點，在20年及200年頻率洪水情況下，洪水位之試驗成果如表3.所示。

表3.　現況防洪佈置下，圓山橋之阻水影響

洪水頻率：年 基隆河流量（秒立方公尺）	潮別 外海潮位（公尺）	布置	洪水位（公尺）			
			中山橋	大直橋	內湖	松山
20 （2,400）	高潮 （1.20）	現況	6.17	6.63	6.72	7.25
		原設計圓山橋完成	6.58	6.95	6.93	7.31
		水位差	+0.41	+0.32	+0.21	+0.06
	低潮 （-1.20）	現況	6.17	6.63	6.72	7.25
		原設計圓山橋完成	6.58	6.95	6.93	7.31
		水位差	+0.41	+0.32	+0.21	+0.06
200 （3,200）	高潮 （1.91）	現況	8.78	9.13	9.19	9.34
		原設計圓山橋完成	8.93	9.26	9.27	9.34
		水位差	+0.15	+0.13	+0.08	± 0
	低潮 （-1.50）	現況	8.78	9.13	9.19	9.34
		原設計圓山橋完成	8.93	9.26	9.27	9.34
		水位差	+0.15	+0.13	+0.08	± 0

由表知：在洪水情況下，外海潮位之影響未及基隆河中山橋。原擬高速公路圓山橋將增加上游泛濫水深，其影響情況，洪水頻率愈低程度愈大，在 20 年頻率情形下，影響範圍溯及松山長壽橋附近，影響幅度在中山橋附近，泛濫深度約增加 40 公分，大直附近增加約 30 公分，內湖附近約 20 公分，至松山附近，亦增加 5 公分左右。在 200 年頻率洪水情況下，影響範圍達內湖附近，泛濫深度增加程度均在 15 公分以下。

圓山橋橋墩修改後，在上述布置及流量下，洪水位之試驗成果如表 4. 所示。

表 4. 現況防洪布置下，圓山橋修改後之阻水影響

洪水頻率：年 基隆河流量 （秒立方公尺）	布置	洪水位 （公尺）			
		中山橋	大直橋	內湖	松山
20 (2,400)	現況	6.17	6.63	6.72	7.25
	圓山橋完成	6.40	6.85	6.93	7.31
	水位差	+ 0.23	0.22	+ 0.21	0.06
200 (3,200)	現況	8.78	9.13	9.19	9.34
	圓山橋完成	8.84	9.22	9.23	9.34
	水位差	+ 0.06	0.09	+ 0.04	± 0

由表知：修改後之高速公路圓山橋，亦增加上游泛濫水深，惟其影響程度較原設計者輕。即洪水頻率愈低影響程度愈大。在 20 年頻率情形下，影響範圍溯及松山長壽橋附近，影響幅度在中山橋附近，泛濫深度約增加 25 公分，至內湖附近，則增加 20 公分，松山附近則為 5 公分左右。在 200 年頻率洪水情況下，影響範圍僅達內湖附近。泛濫深度增加程度均在 10 公分以下。

2. 防洪計畫完成時，計畫洪水位之提高情形：

計畫防洪及原擬高速公路圓山橋布置下，基隆河主要控制點，在 20 年及 200 年頻率洪水情況下，洪水位之試驗成果如表 5. 所示。

表 5. 計畫防洪布置下，原擬圓山橋之阻水影響

洪水頻率：年 基隆河流量（秒立方公尺）	布置	洪水位（公尺）			
		中山橋	大直橋	內湖	松山
20（2,400）	防洪計畫完成	6.01	6.50	6.87	7.38
	圓山橋完成	6.25	6.77	7.03	7.45
	水位差	+ 0.24	+ 0.27	+ 0.16	+ 0.07
200（3,200）	防洪計畫完成	8.59	8.93	9.02	9.48
	圓山橋完成	8.83	9.15	9.22	9.64
	水位差	+ 0.24	+ 0.22	+ 0.20	+ 0.16

由表知：防洪計畫如採200年頻率洪水為設計洪水，圓山橋之興建，將提高上游堤防之計畫洪水位，大直堤防計畫洪水位抬高 25 公分左右，松山堤防亦將抬高 16 公分以上。

圓山橋橋墩修改後，在上述防洪布置及流量下，洪水位之試驗成果如表6.所示。

表 6. 計畫防洪布置下，圓山橋修改後阻水影響

洪水頻率：年 基隆河流量（秒立方公尺）	布置	洪水位（公尺）			
		中山橋	大直橋	內湖	松山
20（2,400）	防洪計畫完成	6.01	6.50	6.87	7.38
	圓山橋完成	6.20	6.73	6.99	7.41
	水位差	+ 0.19	+ 0.23	+ 0.12	+ 0.03
200（3,200）	防洪計畫完成	8.59	8.93	9.02	9.48
	圓山橋完成	8.73	9.03	9.15	9.54
	水位差	+ 0.14	+ 0.10	+ 0.13	+ 0.06

由表知：圓山橋之橋墩雖經修改，仍將提高上游堤防之計畫洪水位，惟其程度較原設計者爲輕，其幅度在 15 公分以下。

㈡增加流速

高速公路圓山橋興建後，在橋墩段產生局部流速之劇變，上游河段則除局部地區外流速變緩。其影響在橋墩河段可能危及附近防洪構造物之安全，並引起河床局部之劇烈冲刷。在上游河段則可能引起淤積，而導致排洪量之不足。其中尤以圓山鐵路橋及河濱公園河段處有二處流速變化劇烈處，該二處流速之變化詳如表 7.所示。

表 7.　圓山橋對流速之影響

布　置		洪水頻率年	流速（秒公尺）	
			鐵路橋右岸堤脚	河濱公園轉角
現況防洪工程	無圓山橋	20	1.24	1.87
		200	1.87	1.18
	原設計圓山橋	20	1.87	1.93
		200	2.45	1.11
	修改後圓山橋	20	1.81	1.68
		200	2.06	1.05
計劃防洪工程	無圓山橋	20	1.49	2.81
		200	2.12	3.10
	原設計圓山橋	20	3.03	2.19
		200	3.18	3.18
	修改後圓山橋	20	2.96	2.31
		200	2.81	3.18

由表知：該局部流速之變化約爲 2 ～ 3 倍，按水流之冲刷力與流速之平方成比例關係，則該等局部地區之冲刷力將變大 4 ～ 9 倍，附近構造物，如圓山堤防再春游泳池段，鐵路橋右岸堤脚、橋墩及河濱公園河岸等處均需嚴加保護，以策安全。

3.改變流向

高速公路圓山橋修建後，對於中山橋至圓山鐵路橋間流向之改變影響最大，橋墩修建後，主流直接冲向右岸，尤以橋墩N（原設計爲下游之實體橋

台），將主流挑向右岸，形成 40 度角，直接鐵路橋右岸堤脚處，此偏角將增加堤防之荷力。按一般堤防設計時僅考慮靜水壓，此流向偏角產生之荷力將爲堤防承受之額外外力，設計時宜考慮流量在堤防上動量改變所致外力之大小，按動量在堤防上之作用力如附圖之示意圖所示，各外力間之關係爲：

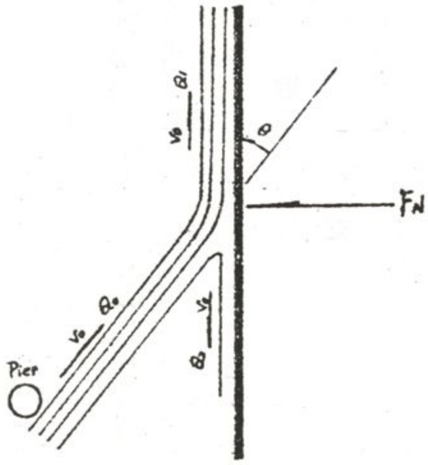


圖 3. 動量在堤防上作用力

$$F_N = \frac{W}{g} Q_o \cdot V_o \cdot \sin\theta$$

W = 1000 公斤／立方公尺，g = 9.8 公尺／秒^2

V_o = 3.18 公尺／秒，Q_o = 5.3 × 1 × 3.18 = 16.854 立方公尺／秒

θ = 40°，$\sin\theta = 0.643$

利用該式知 F_N 之作用力 3.515公噸，即每平方公尺 0.66 公噸堤防之設計除一般靜水壓外，亦應考慮此項額外之力，以策安全。

五松江大橋及承德橋對於水流之影響

市府擬議中之松江大橋位於新生南路北端，穿越圓山高速公路橋主道橋孔，並於圓山橋主橋樑上游處跨越基隆河，橋墩設計採二併排圓墩，跨距淺槽採 23 ～ 26 公尺，主槽採 35 公尺。承德橋則位於鐵路橋下段 150 公尺，銜接承德路。其布置均如圖 1.所示。爲檢討兩橋對基隆河水流之影響。乃以防洪計畫完成及修訂圓山高速公路橋布置等二種情況爲共同比較之條件，試驗 20 年及 200 年頻率洪水下兩橋之阻水影響，玆將成果分述如下：

㈠單獨橋樑阻水影響

松江大橋與承德橋單獨時：洪水位之試驗成果如表 8.所示，由表知：20 年及 200 年頻率洪水下，松江大橋抬高上游洪水位之幅度均在 10 公分以下，其迴水範圍達上游之松山；承德橋之阻水迴水則及大直橋，抬高上游洪水位之幅度則在 5 公分以下。

㈡二橋共同存在之阻水影響

松江橋與承德橋二橋共同存在阻水影響之試驗成果亦如表 8.所示，由表知，二橋之橋墩阻水，抬高上游水位，在 20 年頻率及 200 年頻率洪水下，抬高量均約爲 10 公分。如防洪計畫採 200 年頻率洪水爲計劃流量，則上游大直、

表 8. 松江大橋及承德橋之阻水影響

洪水頻率：年 基隆河流量 （秒立方公尺）	布置	洪水位（公尺）差（公尺）								備註
		中山橋	差	大直橋	差	內湖	差	松山	差	
20 （2,400）	修訂圓山高速公路橋	6.20	—	6.73	—	6.99	—	7.41	—	比較標準
	＋松江大橋	6.22	＋0.02	6.81	＋0.08	7.03	＋0.04	7.43	＋0.02	
	＋承德橋	6.25	＋0.05	6.74	＋0.01	6.99	± 0	7.41	± 0	
	＋松江大橋 ＋承德路	6.25	＋0.05	6.82	＋0.09	7.03	＋0.04	7.43	＋0.02	
200 （3,200）	修訂圓山高速公路橋	8.73	—	9.03	-	9.15	—	9.54	—	比較標準
	＋松江大橋	8.79	＋0.06	9.11	＋0.08	9.17	＋0.02	9.55	＋0.01	
	＋承德橋	8.77	＋0.04	9.04	＋0.01	9.15	± 0	9.54	± 0	
	＋松江大橋 ＋承德橋	8.79	＋0.06	9.12	＋0.09	9.17	＋0.02	9.55	＋0.01	

、松山提防之計畫洪水位及提頂高度應採表 8.或圖 3.所示者爲準，高速公路大直至內湖段之路面或防水欄柵之設計標高亦應比照提高之。因鑑於試驗係在清水下辦理，而實際洪水期河川尙有流木或浮滓，在基隆河圓山段之約 1 公里河段各橋均完成後，河床中約有橋墩 80 座，尤以松江橋橋墩跨距甚小，一旦阻塞流木或浮滓，則流水斷面將完全阻塞，近乎形成攔水壩，勢將大量抬高上游洪水位，造成災害，故松江大橋之橋址，似以另覓新址爲佳。至於承德橋，則其影響程度有限，修建時可採原址。

六松江交流道對水流之影響

高速公路松江交流道位於基隆河濱江公園對岸河床，部分主道及匝道以塡土路基伸入計畫堤線以內，圖 4.，洪水期阻礙水流、抬高水位，如將此二塡土路基改爲高架路基，則可減輕其影響。表 9 示防洪計畫完成之布置情況下，上述二塡土路基之阻水影響。

表 9. 伸入河床塡土路基改爲高架之效果

洪水頻率：年 基隆河流量（秒立方公尺）	布置	洪水位（公尺）			
		中山橋	大直橋	內湖	松山
20 （2,400）	原設計塡土路基	6.20	6.73	6.99	7.41
	改爲高架路基	6.20	6.68	6.97	7.41
	水位差	± 0	− 0.05	− 0.02	± 0
200 （3,200）	原設計塡土路基	8.73	9.03	9.15	9.54
	改爲高架路基	8.69	8.97	9.10	9.52
	水位差	− 0.04	− 0.06	− 0.05	− 0.02

由表知：在 200 年頻率洪水情況下，高速公路圓山橋上游伸入河床之實塡路基，抬高水位之影響範圍達松山附近，惟其幅度均在 10 公分以下，換言之，該實塡路基如改爲高架式，則具有減輕洪水位之效果。

七高速公路圓山～內湖段之水理

高速公路圓山至內湖段完成後，基隆河自民生東路末端至高速公路圓山橋間尙有台北機場東端、高速公路路基涵洞及大直高架段等三缺口，無法達成機場及市區

防洪之目的，玆將該缺口水流情形說明如下：

㈠五年頻率以下洪水

基隆河水流在2年頻率洪水，流量1,200秒立方公尺情況下，水流漫溢河岸，從高速公路大直高架段較低地帶侵入機場，洪水增至5年頻率流量1.700秒立方公尺時，機場東端無堤防之缺口及高速公路路基涵洞亦分別侵入洪水，瞬間流速在路基涵洞處及機場東端缺口達5秒公尺左右，台北現有防洪牆保護以外地區皆被洪水淹沒，基隆河主要水流部分由機場東端缺口，經濱江街，高速公路大直高架孔與河道主流會流而下，惟其流速不大，均在0.5秒公尺以下。

㈡5年頻率以上洪水

基隆河水流超過5年頻率洪水時，現有防洪牆失去效用，洪水自松江路防洪牆缺口漫入市區，上述機場東端缺口，濱江街，高速公路大直高架孔之洪流幾成襲流現象，流速約在1秒公尺左右。

至於台北機場及濱江街等地區之冲刷，因限於本試驗係在定床下辦理，尚無法判斷其影響。

八松山、大直兩堤防興建後與高速公路之影響

高速公路圓山內湖段完成後，因部分路段採高架路基，且塡土路基之大直～內湖段除留有涵洞外，路面標高未達200年頻率洪水之設計標準，故不能利用爲大直左岸堤防。今因高速公路已經施工，堤防之興建當在高速公路完成後，且堤線位於高速公路外，堤防興建後高速公路已在其保護內，故除應注意兩構造物在結構上之安全外，水理上尙無特殊影響。

九圓山橋橋墩施工圍堤之水理

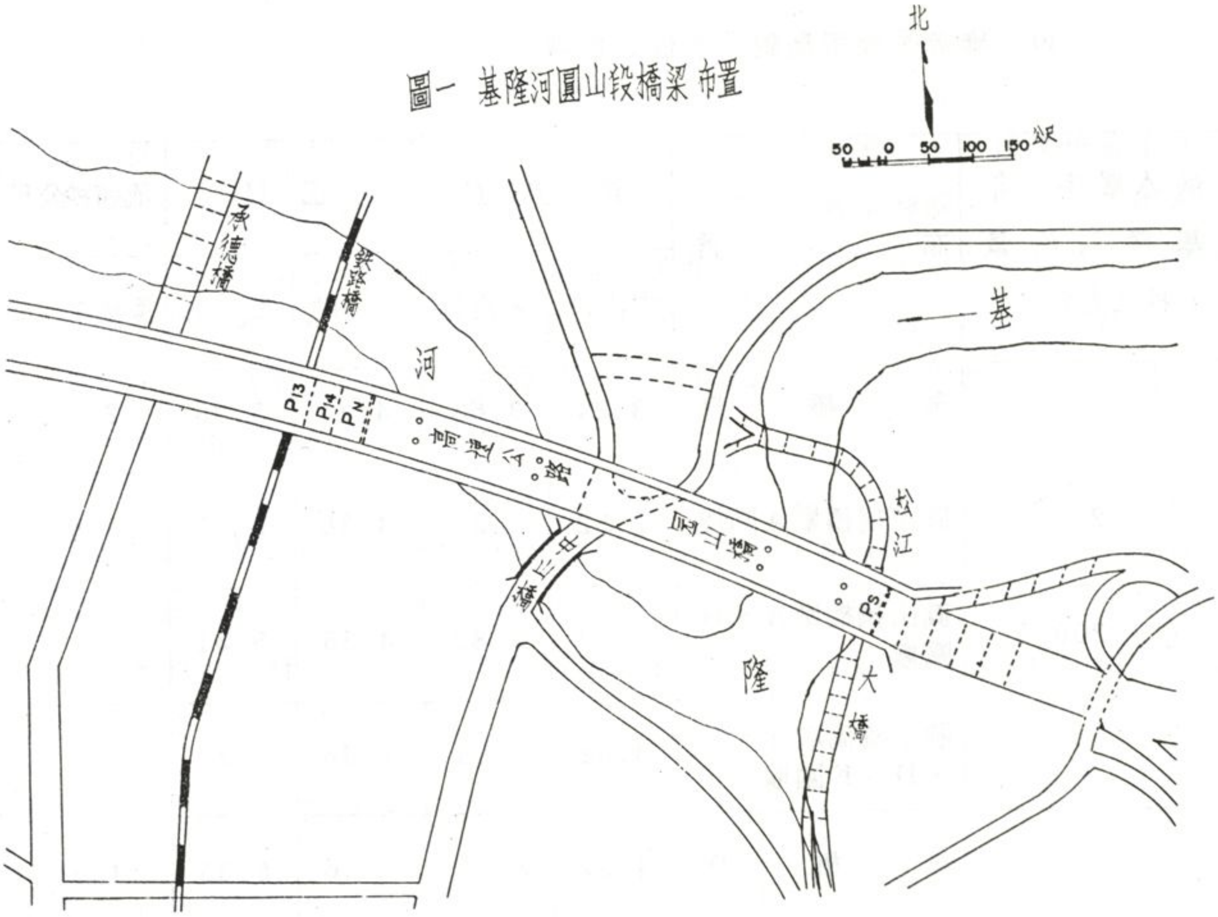
高速公路圓山橋主橋墩A、B、D、E位於基隆河主河槽中，各該墩礎於施工期間，必須採取圍堰築島措施爲瞭解此圍堰築堤影響，乃以5年及2年頻率洪水，試驗各墩圍堰之影響，試驗成果如表 10所示：

表10. 橋墩圍堰築島對洪水位之影響

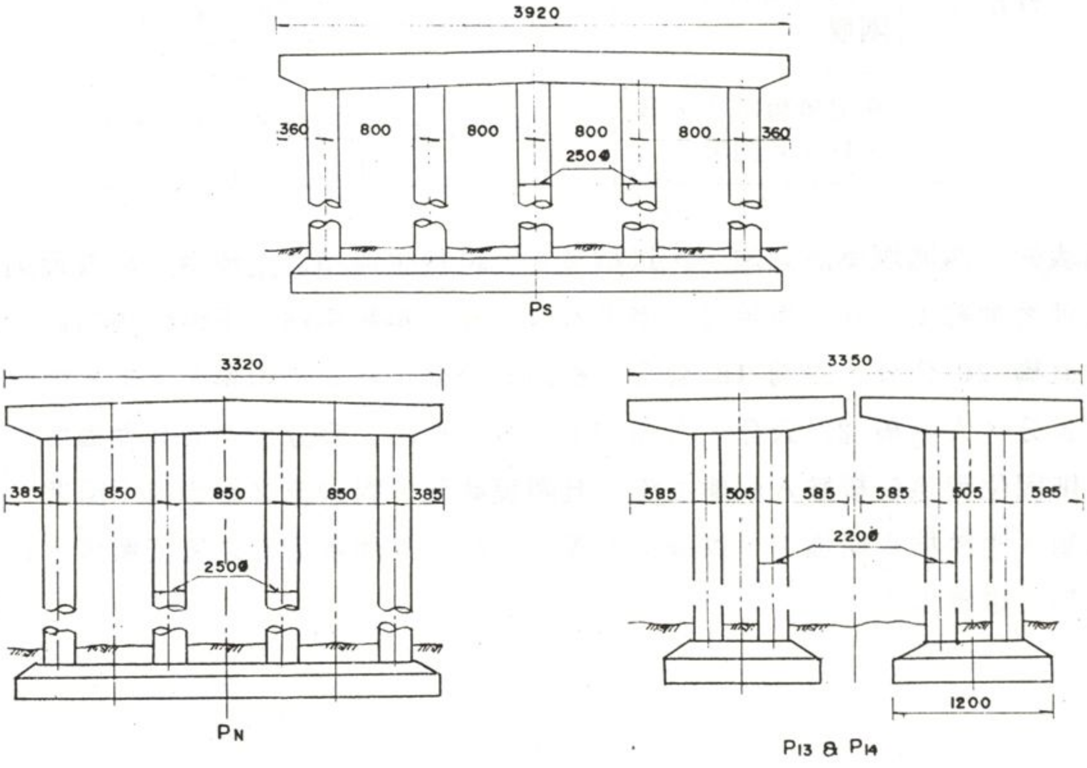
洪水頻率：年 基隆河流量（秒立方公尺）	布置	洪水位（公尺）				流速秒公尺
		中山橋	大直橋	內湖	松山	築島A河岸
2 （1,200）	無橋墩	3.31	3.63	4.24	5.25	2.19
	圓山橋橋墩A圍堰	3.52	3.83	4.36	5.31	3.61
	圓山橋橋墩A、B圍堰	3.52	3.83	4.36	5.31	
	圓山橋橋墩A、B、D、E圍堰	3.52	3.83	4.36	5.31	
5 （1,700）	無橋墩	4.52	4.93	5.46	6.35	1.81
	圓山橋橋墩A圍堰	4.80	5.16	5.59	6.38	3.03
	圓山橋橋墩A、B圍堰	4.80	5.16	5.59	6.38	
	圓山橋橋墩A、B、D、E圍堰	4.80	5.16	5.59	6.38	

由表知：四圍堰築島之施工，以橋墩A因位於主河槽之主流中，影響最劇，除局部流速增加約1倍外，復抬高上游洪水位，在2年頻率洪水下中山橋抬高 21 公分，大直橋 20 公分，內湖 18 公分，松山6公分；在5年頻率洪水下，中山橋抬高 28 公分，大直橋 23 公分，內湖 13 公分，松山3公分。至於其他橋墩之圍堰築島，則因位置遠較橋墩A偏離主流，且圍堤較低，對水流之影響尚不顯著。故施工時橋墩A應於枯水期施工，以避免影響洪水位，其他各橋墩之施工順序，在水理上受季節之影響尚不大。

圖一 基隆河圓山段橋梁布置

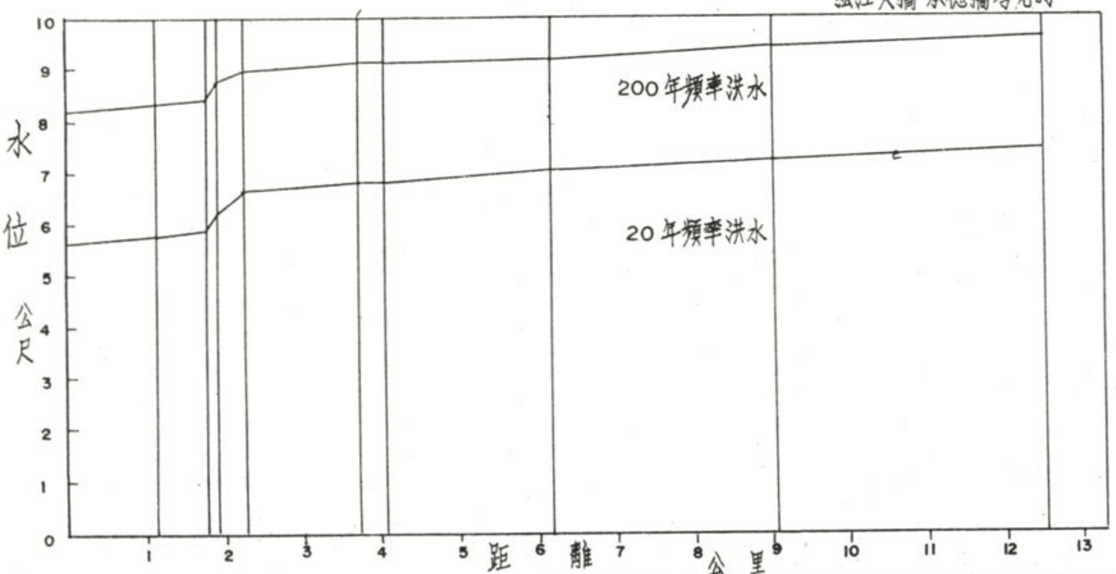


圖二 圓山橋P_S P_N P_{13}及P_{14}橋墩修正



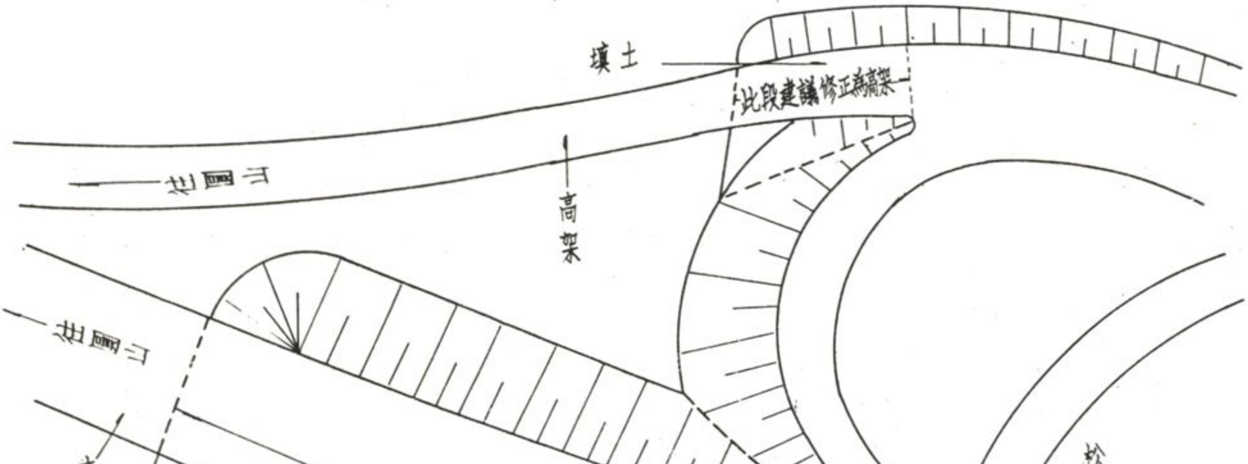
圖三 基隆河水位縱剖面

布置：防洪計畫、圓山－內湖高速公路
松江大橋、承德橋均完成



測點	水位公尺	
	20年	200年
百齡橋	5.63	8.20
基15L	5.72	8.29
基16	5.89	8.41
中山橋	6.25	8.79
基17R	6.68	8.99
大直橋	6.81	9.11
基20R	6.81	9.11
基24R 內湖	7.03	9.19
基29R	7.22	9.41
基35R 松山	7.43	9.55

圖四 松江交流道填土路基伸入河內



附錄四 (1)細部設計要領

1 INTRODUCTION

1.1 Objectives

The purpose of this study is to provide the necessary data for the construction of Yuan Shan Bridge using Freyssinet system. This calculation note covers the following items:

(1) Modification of tendon layout, anchorage details, etc. in connection with prestressing work with the Freyssinet cables, giving the same prestressing forces and moments as these given in the original design

(2) Preparatory calculation for the necessary control of prestressing at the site, such as relationship between gage pressure and elongation

(3) Calculation of elastic deflection due to dead weight of each segment and wagon and to prestressing forces during the cantilever construction, and deferred deflection due to effect of shrinkage and creep of concrete

1.2 Sequence of Construction and Construction Schedule

In order to make the above preparations, the sequence of construction and construction period were assumed as shown in Figs.1.1 through 1.4 and Table 1.1, on the following assumptions:

(1) A sufficent length of girder near the top of each pier shall be concreted on falsework to provide enough space for placing two wagons (see Fig.1.5). The end section of the box girder shall be rightangled to the axis of girder.

(2) One wagon shall be placed on the North bound (or South bound) and cantilever construction shall be carried out as far as third segment and then the other wagon shall be placed on the South bound (or North bound) (see Fig.1.5).

(3) The former shall be dismantled when the final segment is concreted and prestressed to permit the continuous operation of the latter. The

latter shall be pushed back to the top of pier to be dismantled after having completed the final segment of the cantilever.

(4) The both bounds shall be connected with the cast-in-place concrete deck slab and crossbeams.

(5) The remaining central segment of the span shall be concreted on falsework suspended from the both ends of the cantilevers.

1.3 Layout of Tendons

1.3.1 Principal tendons

Modification of principal tendons, including additional and joinning tendons as well as the tendons in diaphragms was carried out to give the same prestressing forces and moments as those given by the original design, on the basis of making use of Freyssinet 12T 13 cables.

1.3.1.1 Prestressing Tendon and Steel

The mechanical properties of prestressing steel and its allowable stress are given in Table 1.2.

Table 1.2 Seven-wire Stress-relived Strand

Mechanical Property	Nominal Diameter (mm)	12.7
	Cross-sectional Area (mm^2)	1184.52
	Unit Weight (kg/m)	9.288
	Tensile Strength (kg/mm^2)	190
	0.2% Proof Stress (kg/mm^2)	160
Allowable Stress (kg/mm^2)	Temporary Overstressing for Short Period ($0.8 f_s'$)	152
	At transfer to Concrete ($0.7 f_s'$)	133
	At Service Load after Losses ($0.6 f_s'$)	114

Note: 1) The cross-sectional area of a Freyssinet 12T13 cable is equal to 1,184.5 mm^2.

2) The internal diameter of duct is 65mm.

1.3.1.2 Steel Stress Distribution

The steel stress distribution at transfer along each cable was estimated on the assumption that the minimum steel stress always reaches a fixed value shown as follows:

Length of Cable (m)	Min. Steel Stress (kg/mm^2)
$\geqslant 70$	115
< 70	120

The preliminary stress calculation proved that the necessary jacking stresses to compensate for the losses of prestress do not exceed the allowable stress for the temporary over-stressing, provided that the minimum steel stresses remain at the above values.

The losses of prestress due to friction and slipping within the anchorages were taken into account on the following assumptions:

(1) Loss due to friction was estimated according to the AASHO (1973) Article 1.6.7. (A) using the following values,

$$K = 0.0020 \qquad \mu = 0.3$$

(2) Loss due to slipping within the anchorage was estimated on the basis of 12mm pull-in which is defined by the Provision of Freyssinet System.

(3) Loss of prestress due to creep and shrinkage of the concrete and to relaxation of the prestressing steel was estimated applying the FIP-CEB International Recommendations for Design and Construction of Concrete Structures.

(a) The creep coefficient ϕ can be given as:

$$\phi = k_c k_d k_b k_c k_t$$

where:

k_c depends on enviromental conditions. For 70% of relative humidity of air k_c is taken as 2.3.

k_d depends on the hardening of the concrete at the age of loading. When the concrete is subjected to loading at 7 days of age, k_d is taken as 1.4.

k_b depends on the composition of the concrete. For the concrete having 400kg/m^3 cement and 42% water-cement ratio, is tadin as 0.9.

k_e depends on the theoretical thickness of the member (tne quotient of the area of the section divided by the semi-perimeter in contact with the atmosphere). For the section used for this bridge, the theoretical thickness is equal to 0.4m and k_e is taken as 0.71.

k_t covers the development of the deferred deformation with time. For the final state k_t can be taken as 1.0

Applying the above figures, the basic coefficient ϕ is given as follows:

$$\phi = 2.3\times1.4\times0.9\times0.71\times1.0 = 2.06 \fallingdotseq 2.1$$

(b) The shrinkage deformation ε_s is determined by the product of five partial coefficients:

$$\varepsilon_s = \varepsilon_c\, k_b\, k_e\, k_p\, k_t$$

where:

ε_c depends on the enviroment, here 27.5×10^{-5}.

k_b depends on the composition of the concrete, here, 0.9.

k_e depends on the theoretical thickness of the member, here, 0.55.

k_p depends on the geometric percentage ρ of longitudinal reinforcement of area A with respect to the cross-sectional area of the member B. Here ρ is equal to 0.394% and k_p can be taken as:

$$k_p = \frac{100}{100+n\rho} = \frac{100}{100+20\times0.394} = 93$$

k_t defines the development of shrinkage as a function of time. For the final stege k_t is equal to 1.0.

Applying the above figures, the shrinkage of the concrete is taken as:

$$\varepsilon_s = 27.5 \times 10^{-5} \times 0.9 \times 0.55 \times 0.93 \times 1.0 = 12.7 \times 10^{-5} \fallingdotseq 13 \times 10^{-5}$$

(c) The relaxation of steel was taken as 16% of an initial stress equal to 80% of the tensile strength. The law of relaxation can be obtained by assuming that the changes follow a parabolic law

from a value of $\sigma_{po} = 50\%$ of tensile strength and have a horizontal tangent at this point as shown in the following figure.

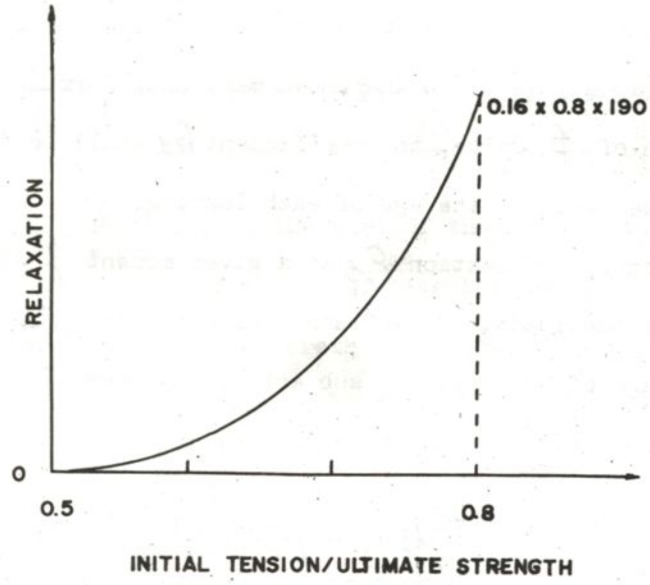


Fig. 4-1-1

The value of the apparent relaxation of the steel may be obtained from the following expression:

$$\Delta\sigma_{ap} = \Delta\sigma_{ap,\infty} \left[1 - 3 \frac{\Delta\sigma_{p,\varphi+s}}{\sigma_{po}} \right]$$

in which:

$\Delta\sigma_{ap,\infty}$: pure relaxation of the steel estimated above.

1.3.1.3 Deferred Deformation of Structure Constructed by Segmental Constrection

The concrete stresses at the level of C.G.S in any segment increase with the process of segmental construction. Take the prestress in the first segment, for example, the first prestress will be applied at 10 days of age and the second prestres is induced at 20 days of age by tensioning the tendons anchored at the end face of the second segment and the same procedure will be continued until the tendons anchored in the final segment are prestressed. Thefore the prestress is applied every 10 days. The deferred deformation due to the creep of the concrete has different magnitude according to the value of the prestress to be applied and the concrete age when the prestress is applied. In the light of this fact, the value of creep coefficient ϕ shall be changed according to the process of the construction considering the age of the concrete when the load is applied. In the above mentioned formula for the estimation of ϕ value, the coefficient k_d shall be determined on the basis of the age of each loading.

For the creep deformation ε_ϕ at a given moment z after application of the loads, the influence of a stress σ_{cj}, applied at the age of j days and subject at any moment i to variation in intensity σ_{ci}, may be expressed as:

$$\varepsilon_\varphi = \frac{k_i k_b k_e}{E_c}\left[\sigma_{cj} k_{dj} \cdot k_{t(z-j)} + \sum \sigma_{ci} k_{di} \cdot k_{t(z-i)}\right]$$

where:

$$k_{dj} = \frac{9.69}{4.4+\sqrt{j}} \quad \text{for normal cement}$$

Although this method of superposition is theoretically very correct, its practical application to the solution in the case of the segmental construction is very complicated because of too many variables. Therefore a mean value of the final creep coefficient could be used in the practical calculation and this mean value for a given segment may be expressed as follows:

$$\text{Mean value of } \phi = \frac{k_i k_b k_e \sum \sigma_{cj} k_{dj}}{\sum \sigma_{cj}}$$

The same method could be applied to the estimation of the rate of development of the creep deformation:

Mean value of $$k_t = \frac{k_i k_b k_e \Sigma \sigma_{cj} \cdot k_{dj} k_t(z-j)}{\Sigma \sigma_{cj} h_{dj}}$$

The mean deformation due to the shrinkage of the concrete in the estimation of losses of prestress could be estimated, following same method as the above-mentioned one.

Mean value of $$\varepsilon_s = \frac{\varepsilon_c k_b k_e k_p \Sigma [k_{tz} - k_{tj}]}{[k_{tz} - k_{t1}]}$$

where:

$[k_{tz} - k_{tj}]$ covers the part of the deformation due to shrinkage in an interval of time $(z-j)$.

$[k_{tz} - k_{t1}]$ covers the part of the deformation due to shrinkage in an interval of time between z and first loading.

If mean shrinkage of the concrete is estimated by this method, it became too small to risk the underestimation of the deformation due to shrinkage. For safery, it was supposed in the estimation of losses of prestress that the total shrinkage expected after the first loading shall be taken into account in the caloulation. The first loading on the concrete of any segment is assumed to be applied at 10 days of age and the deformation due to shrinhage before the application of the first prestressing could be deducted in calculating the loss of prestress due to the shrinkage of concrete. The first prestressing is carried out at 10 days of age and in this case k_t is taken as 0.1. Therefore, the value of deformation due to shrinkage in the calculation is taken as:

$$\varepsilon_s = 13 \times 10^{-5} \times (1.0 - 0.1) = 12 \times 10^{-5}$$

In conclusion, the loss of prestress due to the creep and shrinkage of the concrete shall be estimated as follows:

$$\Delta\sigma_{p.\varphi+s} = n \varphi_{mean} \sigma_c + E_p \varepsilon_s$$

where

n ratio of modulus of elasticity

φ_{mean} mean creep coefficient for a given segment

σ_c concrete stress at the level of centroid of steel

E_p modulus of elasticity for steel (20×10^{5})

ε_s shrinkage of concrete (12×10^{-5})

The effect of the induced moment on the loss of prestress should be considered for this bridge, because the deformation due to the creep is restrained after the completion of the statically indeterminate structure. This induced moment increases with time from zero to its final calculated value, therefore the estimation of the loss of prestress due to this induced moment could be carried out on the basis of its mean value. Moreover this induced moment will be applied to each segment which has different age of concrete. Therefore, a mean creep coefficient shall be estimated which could give the same deflection as that calculated considering the difference in the age of the concrete of each segment.

$$\text{Mean } \phi = \frac{\delta_{\phi}}{\delta_{e}}$$

where:

Mean ϕ is a mean creep coefficient expected after the completion of structure

δ_{ϕ} is creep deflection calculated on the assumption that there is no change in structural system and each segment has different age of concrete.

δ_{e} is elastic deflection just after the completion of structure

In conclusion, the estimation of the loss of presterss due to the induced moment shall be carried out on the basis of the mean value of the induced moment and the reduced mean value of ϕ.

1.3.1.4 Spacing of Anchorage and Reinforcement in Anchorage Zone

The spacing of anchorage and reinforcement in anchorage zone shall be determined in accordance with the AASHO (1973) Article 1.6.15 and the Provision of Freyssinet System.

1.3.2 Vertical Prestressing Bar

1.3.2.1 Prestressing Bar

The prestressing steel bars to be used for the vertical prestressing shall have the following properties.

Table 1.3 Prestressing Steel

Mechanical Property	Nominal Diameter (mm)	23
	Cross-sectional Area (mm^2)	404.8
	Tensile Strength (kg/mm^2)	110
	0.2% Proof Stress (kg/mm^2)	95
Allowable Stress (kg/mm^2)	Temporary Overstressing for short Period	88
	At Transfer to Concrete	77
	At Service Load after Losses	66

Note: Tha internal diameter of duct is 35mm.

1.3.2.2 Prestress

The effictive steel stress was estimated on the basis of the initial tensioning stress taking account all losses due to friction and to creep and shrinkage of the concrete. The initial minimum steel stress was assumed to be $65kg/mm^2$ Loss due to the elastic deformation shall be compensated during tensioning as

described in Chapter 3.

1. 3. 2 3. Spacing of Anchorage and Reinforcement in Anchorage Zone

The Recommendations of AASHO (1973) shall be applied.

1.4 Preparatory Calculation for Control of Stressing Tendons at Site

1.4.1 Principal Tendons in Girder and Diaphragms

The control of stressing of cable shall be carried out by the measurment of gage pressures as well as elongations at all times.

In measuring the elongation of cable, a mark shall be drawn on one strand at a distance 30cm from the surface of anchorage. The displacement of this mark shall be measured to estimate the elongation of cable.

In preparing the chart for control of stressing of cable, two curves representing the relationships between gage pressures and elongations were presented using two different combinations of K and μ . The ratio of K to μ was taken as

$$K/\mu = 0.007$$

giving the same ratio as that between K and μ described in 1.3.1.2 (1)

Table 1.4 Values of K and μ Used in Calculation

	K	μ
Case 1	0.0007	0.10
Case 2	0.0028	0.40

In calculation of the elongation of cable during stressing, the modulus of elasticity of the prestressing steel was taken as

$$E_p = 20 \times 10^5 \ Kg/cm^2$$

and also the followings were taken into account:

(1) The end stressing force shall be increased to compensate the loss due to elastic deformation.

(2) The friction loss in jacking equipment and anchorage was assumed to be 4%.

The following figure shows a typical chart for the control of stressing. In marking a measured result on the diagram, probable value of friction μ could be easily estimated on a specific cable and by limiting the variation of this estimated μ value within a certain value, the control of stressing could be carried out at site.

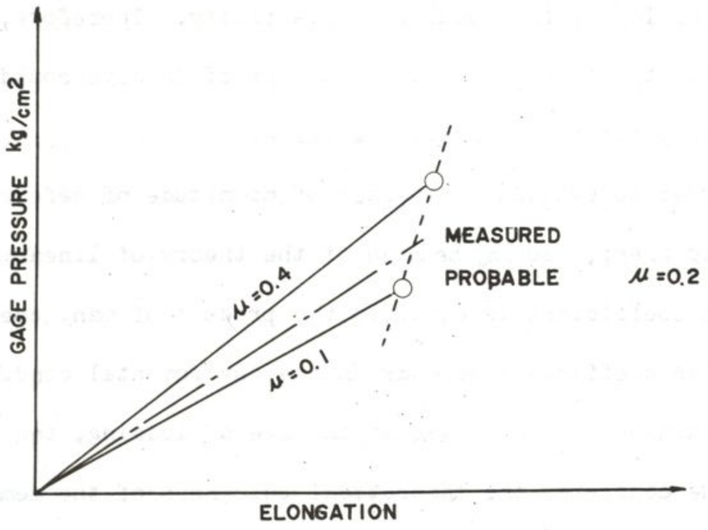


Fig. 4-1-2

1.4.2 Vertical Prestressing Bar

For the prestressing bars, only one curve representing a relationship between gage pressures and elongations was given assuming the following values for K and μ :

$$K = 0.0003 \qquad \mu = 0.20$$

In this calculation the loss of prestress due to elastic deformation was compensated by increasing the end steel stress and the modulus of elasticity of prestressing bar was taken as

$$E_p = 20 \times 10^5 \text{ Kg/cm}^2$$

1.5 Calculation of Deflection

1.5.1 General

In computing the deflection of a prestressed concrete bridge constructed with the segmental construction method, there are many difficulties and uncertainties about the concrete properties.

(1) Compressive strength and modulus of elasticity of the concrete will increase with the age, but as the modulus of elasticity is supposed to be proportional to the squareroot of the compressive strength of the concrete, the variation in the modulus of elasticity is small as compared with that in compressive strength. An increase of 30% in the compressive strength, for example, results in

that of 14% in the modulus of elasticity. Therefore, modulus of elasticity of the concrete at an age of 28 days could be used in the computation as an average value.

(2) In order to evaluate the order of magnitude of deferred deflections due to creep, use may be made of the theory of linear creep. The creep coefficient is equal to the product of many coefficients, such as coefficient depends on the enviromental conditions, the hardening of the concrete at the age of loading, the composition of the concrete, the theoretical thickness of the member and the development of the deferred deformation with time.

In computing the creep deflection expected during the segmental construction, the devlopment of creep deformation with the time could be taken into consideration using the previously given formula step by step matching with the construction schedule. This calculation could be simplified using a concept of mean creep coefficient and mean devlopment rate of creep.

These calculation methods is only of an academic interest, because of too many uncertainties about the creep properties of the concrete. Therefore, for the practical purpose it could be thought that the effect of creep on the deflection could be neglected during the segmental construction and the deflection due the the creep of concrete may make its appearance only after the completion of the structure on the basis of a constant value, 2.5, of creep coefficient along the member.

1.5.2 Various Deflections

1.5.2.1 Deflection during Construction

(1) Elastic deflection due to own-weight of girder already constructed

(2) Elastic deflection due to own-weight of wagon placed for concreting the next segment

(3) Elastic deflection due to own-weight of the segment newly concreted

(4) Elastic deflection due to application of prestress on the newly placed segment

1.5.2.2 Deflection after Completion of Bridge

(1) Elastic deflection due to weight of curb, railing, pavement and the other permanent loads except the dead-weight of the girder

(2) Creep deflection due to the loss of prestress

(3) Creep deflection due to permanent loads

(4) Creep deflection due to induced moment

1.5.2.3 Additional Camber

In order to avoid an unpleasant profile of the bridge in case of unexpected excessive creep due to any unforseen causes, an additional camber shall be provided as shown in the following figure.

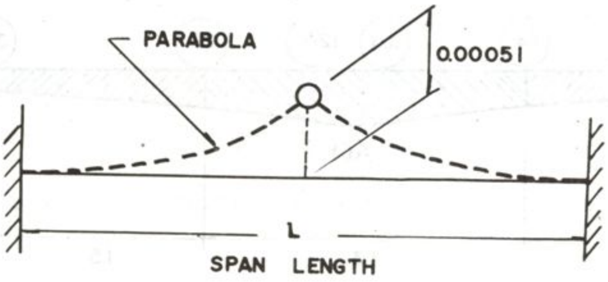


Fig. 4-1-3

2 TENDON LAYOUT

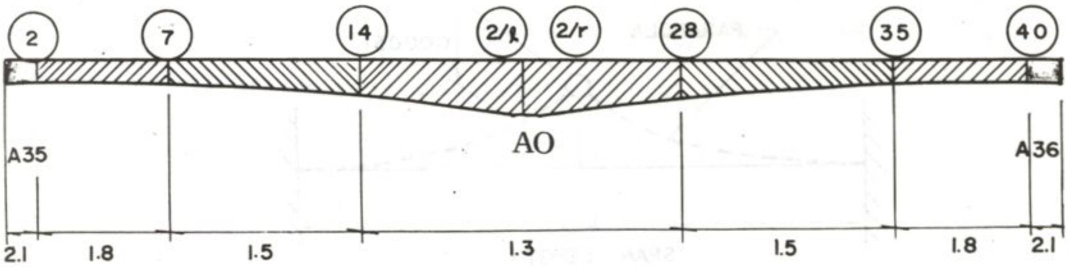
2.1 General

The results of calculation given in this Chapter clearly show that new layout of tendons using the Freyssinet 12T13 cables shown in the drawings, can satisfy all the requirments of prestressing after losses given by the original design.

2.2 Principle Tendons and Additional Tendons

As already stated in Paragraph 1.3.1.3, the mean creep coefficient shall be used in the calculation of the losses due to the creep of the concrete on the assumption that each partial prestress on any segment is to be applied every 10 days.

This mean creep coefficient was estimated at every segment according to the process of the construction and these estimated values were grouped in four, for convenience and safety, as shown in the following figure.



MEAN CREEP COEFFICIENT FOR EACH ZONE

Fig. 4-1-4

mean creep coefficient for each zone

In calculating the concrete stress acting on the j - th segment due to stressing the cables anchored at the end face of this segment, their effects shall be neglected because of the insufficient length of this segment to make the spreading of the prestress effective over this segment.

The losses of prestress at several sections were computed on the basis of the above mean creep coefficients, shrinkage of 12×10^{-5} and the apparent relaxation of the steel described in paragraph 1.3.1.2 . The results are shown in the following table.

section	Loss of steel stress (kg/mm^2)		
	Creep, shrinkage	Relaxation	Total
2	530	500	1030
7	1220	470	1690
11	1330	420	1750
14	1480	360	1840
18	1400	330	1730
21ℓ	1170	330	1500
21r	920	360	1280
23	1190	350	1540
28	960	430	1390
30	1090	440	1530
34	880	530	1410
40	490	510	1000

Note: The value of n was taken as 7.

From the above calculations, the loss of steel stress due to the creep, shrinkage of the concrete and the relaxation of the steel could be estimated at $1750kg/cm^2$ (25,000psi) for simplicity, irrespectiv of the complicated segmental construction. This value was adopted in the original design.

The calculation of the steel stress was carried out using the electronic computer, at each joint according to the profiles of each cable. The instantaneous losses which can occur before and at the moment the steel is locked off, as well as the deferred losses of tension, $17.5kg/mm^2$, were taken into account in the calculation and the results were compared with the values given by the original design to check the safety.

2.3 The Tendons placed in Tie Beam and Joinning Tendons

The effect of prestressing the tendons placed in the tie beam at each support was checked with the same method as that described in Paragraph 2.2 to satisfy the requirments of prestressing given by the original design.

These tendons consist of two kinds of layouts as shown in the following figure.

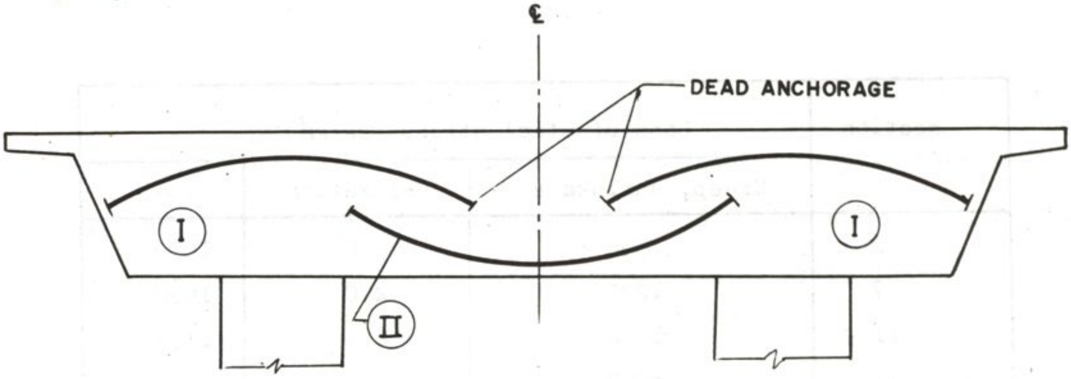


Fig. 4-1-5

The tensile stresses of steel at transfer were calculated to give the following values at the specified sections of each tendon.

Tendon (I) 91kg/mm² at dead anchorage

Tendon (II) 110kg/mm² at the center

The losses of steel stress due to shrinkage and creep of the concrete and to relaxation of the steel were taken as 17.5kg/mm²

The specified joinning cable with 148t tensioning capacity can be easily replaced by the Freyssinet 12T13 cable with 152t effective tensioning capacity, without any changes in its layout.

2.4 Vertical Tendons

The effective tension in prestressing bar was estimated on the basis of the figures stated in Chapter 1 and the necessary bar spacings were calculated to give the required tensioning force every one meter long.

3 Preparatry Calculation for Control of Stressing Tendons at Site

As already stated in Paragraph 1.4, the control of stressing of cable shall be carried out by the measurment of gage pressures as well as elongations at all times. In preparing the chart for control of stressing of cable, two curves representing the relationanips between gage pressures and elongations were prepared.

The caloulation was carried out with the electric computer for the principale and additional tendons, and for the other tendons the manusl calculation was made.

4 Deflection

4.1 Elastic Deflections under Various Loading Conditions

4.1.1 General

All elastic deflections under various loading conditions were calculated by electronic computer on the assumption that the both bounds of the bridge are completely separated during the construction even on the intermediate support. An unbalanced moment acting on one bound of the bridge could be transmitted to the other bound by the torsional rigidity of the cross-beam placed on the intermediate support. But for simplicity, the above assumption was adopted in the calculation of the elastic deflection assuming that an unbalanced moment acting on one bound of the bridge is resisted by the column supporting that bound.

4.1.2 Deflection during Segmental Construction

4.1.2.1 Deflection due to Weight of Concrete of Segment ⓘ which is newly concreted

The weight of concrete is supported by wagon and its effect on the deflection of the cantilever which was already constructed, can be represented by the shearing force and moment acting on the end section of the cantilever, as shown in the following figure.

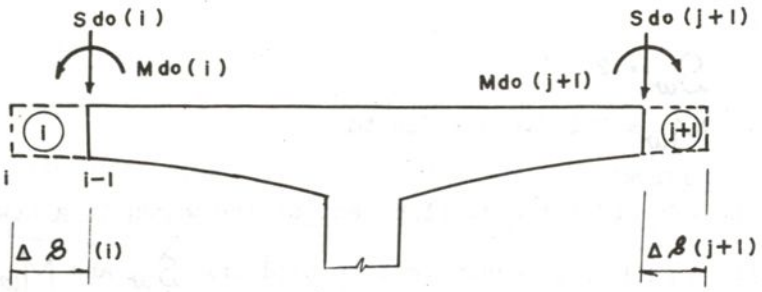


Fig4-1-6

$$S_{do(i)} = \frac{1}{2} \left\{ W_{do(i-1)} + W_{do(i)} \right\} \Delta \mathcal{S}_{(i)}$$

$$M_{do(i)} = \frac{1}{6} \left\{ 2 W_{do(i)} + W_{do(i-1)} \right\} \Delta \mathcal{S}_{(i)}^{2}$$

where:

$$W_{do}(i) = 2.4\,A_{(i)} \quad (t/m)$$

$A_{(i)}$ = sectional area of section i (m^2)

$\Delta\ell_{(i)}$ = length of segment i (m)

4.1.2.2 Deflection due to weight of wagon

The deflection due to the weight of wagon was calculated considering the shearing force and moment acting at the end face of cantilever which was already constructed. The shearing force, S_w and the moment M_w to be taken into account are estimated as follows:

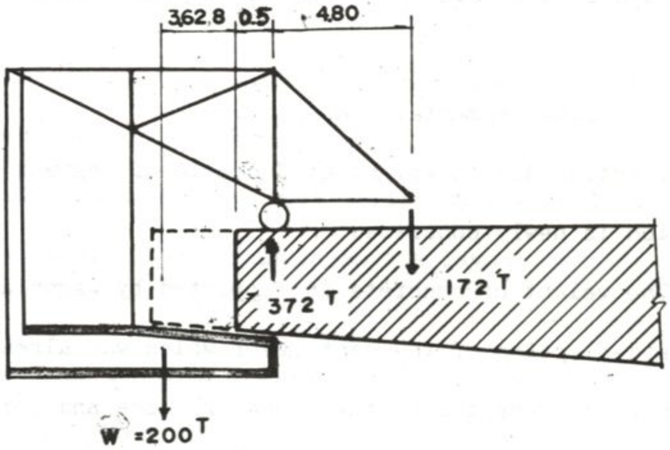


Fig.4-1-7

(weight of wagon)

S_w = 200 t

M_w = 200×3.628 = 726 tm

The effect of the displacement of the wagon to a new position, ($i+1$), is equivalent to applying S_w and M_w at the end section, (i), and to removing these at the section, ($i-1$).

4.1.2.3 Deflection due to its own Weight of a Segment added to part already constructed.

The weight of the segment which was carried by the wagon before hardening, is carried by a cantilever consisting of this segment and the part already constructed, after applying the prestress.

In calculationg the deflection due to the weight of segment, i, the shearing force and moment should be applied at the section, $i-1$, as described in paragraph 4.1.2.1.

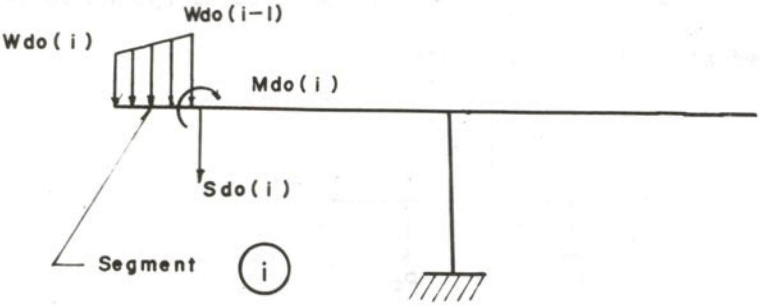


Fig.4-1-8

4.1.2.4 Deflection due to prestress of Prinipal Tendons

The deflection due to prestress was computed on the basis of the prestressing forces at transfer and the eccentricity of tendons, described in Chapter 2.

The effect of the prestressing moment acting on the segment which involves the tendons to be tensioned was taken into account in computing the deflection due to tensioning of these tendons.

4.1.2.5 Deflection due to weight of Cast in Place Concrete Deck slab and Cross-beams

The weight of cast-in place concrete deck slab and cross-beams will be loaded on the cantilever structure before placing the final segment in which a central hinge will be placed. The loading condition can be shown as in the following figure.

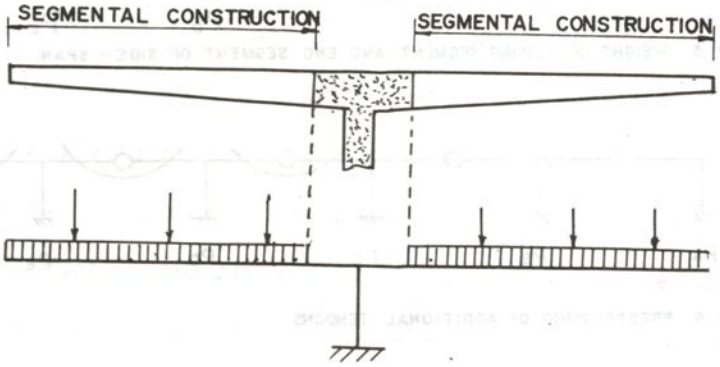
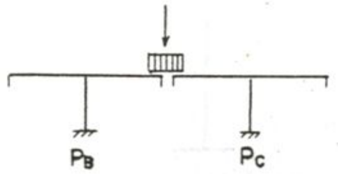


Fig.4-1-9

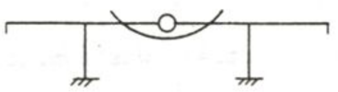
4.1.2.6 Deflections due to weight of Closing Segment and to Prestress of Additional Tendons

The deflections due to these loading conditions should be computed according to the sequence of construction considering the structural system which is completed at each loading state.

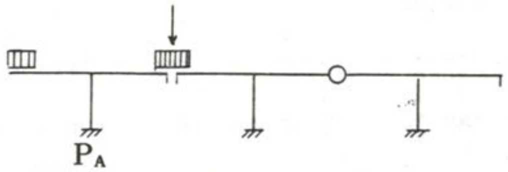
The followingn figure shows the structural system to be considered at each loading state.



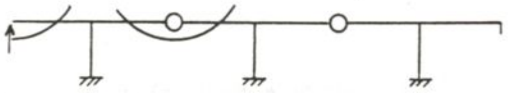
STEP 1 WEIGHT OF CLOSING SEGMENT AND CROSS—BEAM



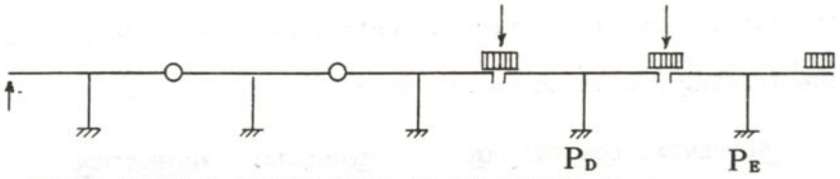
STEP 2 PRESTRESSING OF ADDITIONAL TENDONS



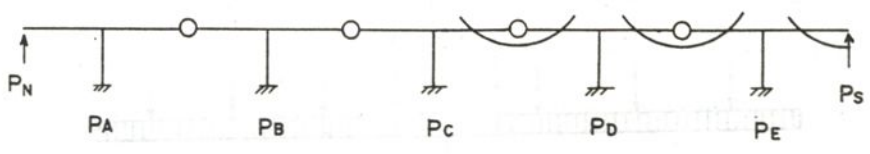
STEP 3 WEIGHT OF CLOSING SEGMENT AND CROSS-BEAM.
WEIGHT OF THE END SEGMENT OF THE SIDE-SPAN.



STEP 4 PRESTRESSING OF ADDITIONAL TENDONS



STEP 5 WEIGHT OF CLOSING SEGMENT AND END SEGMENT OF SIDE-SPAN



STEP 6 PRESTRESSING OF ADDITIONAL TENDONS

Fig.4-1-10

4.1.3 Deflection due to Weight of Curb, Railing, Pavement and Other Permanent Loads

The deflection due to these loadings except the dead weight of the bridge can be computed applying the uniform loading over the completed structural system.

4.2 Deferred Deflection

4.2.1 Deferred deflections due to Prolonged Loads

(a) For the dead weight of the girder applied up to the end of the segmental construction time, the above elastic deflections should be increased by a factor of 2.5 for simplicity and safety.

(b) For long-term loads applied after the completion of the bridge, the value of this factor should be reduced to 1.5.

4.2.2 Deferred Deflection due to Prestressing

This deflection should be computed as follows:

Deferred Deflection

=(Elastic Deflection due to Prestressing)×2.5

-(Elastic Deflection due to Deferred Losses of steel stress) × 1.25

4.2.3 Deferred Deflection due to Induced Moment

The elastic deflection due to the induced moment should be increased by a factor of 1 and the half of this deflection is taken as the deferred deflection due to the induced moment.

4.3 Required Camber after Completion of Segmental Construction

In order to give the bridge a required profile of the deck surface, after all the deferred deformations will occur, each cantilever shall be provided with the necessary camber just after its completion.

The various deflections expected during the construction and after the completion of the bridge are illustrated in Fig. 4.1 considering the sequence of construction and the construction periods elapsed among the various stages. Assuming that the final profile of the deck surface will coincide with that of the additional camber, the required cambers of each section after the completion of each segmental construction are diagrammatically shown in this figure. Special attension shall be paid to get the same level for the both ends of the consecutive cantilever just before placing the central segment. Thefore, the rate of development of the creep shall be taken into account in computing the deferred deflection expected just before coupling the both ends of the cantilevers.

Neglecting the effect of the creep of the concrete during the segmental construction, it was assumed that the deferred deflection will occur just after the completion of each cantilever. The rate of development of the creep could be computed for m months as follows:

$$k_t = \frac{(0.75+m)m}{1+(13.9+m)m}$$

Based on the above principle of the calculation, the required cambers were computed just after the completion of the segmental construction for each cantilever.

4.4 Required Position of each Segment during Construction

The position of each segment shall be fixed so as to obtaine the canber of each cantilever specified in paragraph 4.3.

In fixing the front face of the formwork of segment (i), its position shall be determined by summing up the following deflections computed at section i.

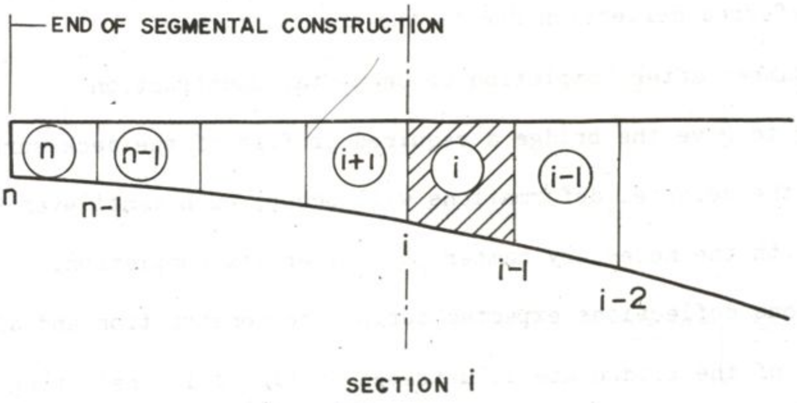


Fig. 4-1-11

a) Deflections due to weights of segments (i), (i+1), ..., (n)

b) Deflections due to loading and unloading of wagon on segments (i+1), (i), (i+1)

............., (n-1)

c) Deflections due to tensioning tendons anchored at every front face of segments (i), (i+1), ..., (n)

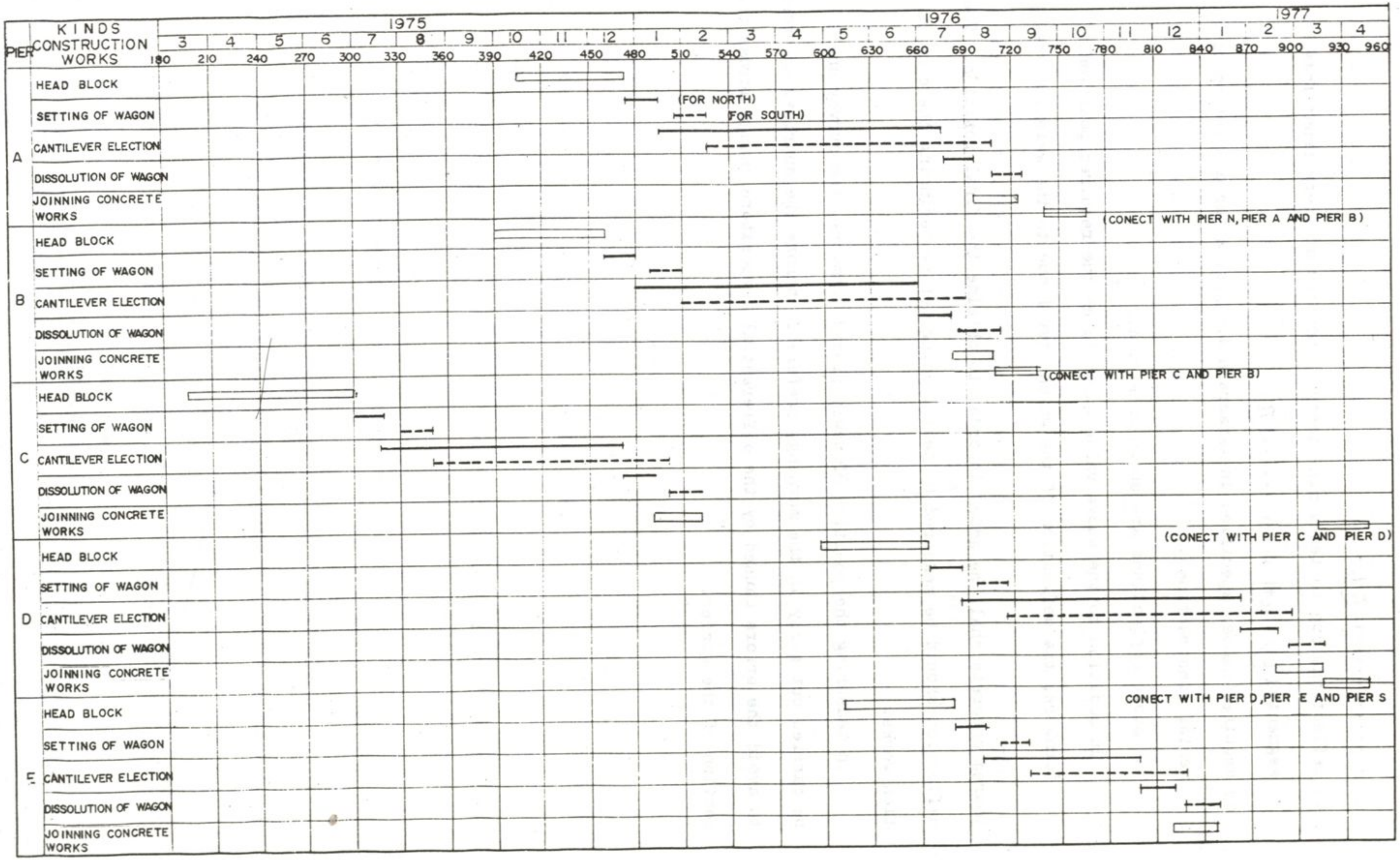
d) Required camber specified in paragraph 4.3 just after completion of segmental construction

The above deflections are shown in Fig. 4.2

In addition to the above value which fixes the required position of formwork, the deflection of the wagon itself due to the weight of newly poured concrete shall be added in order to obtaine the final position. The deflection should be measured by test loading before beginning the construction works.

In measuring the position of newly placed formwork, the survey shall be carried out early in the morning, preferably before the sunrise, in order to avoid the errors caused by the differential temperature change across the section of the girder.

TAB. 1-1 YUAN SHAN BRIDGE CONSTRUCTION SCHEDULE (FOR CALULATION OF CAMBER)



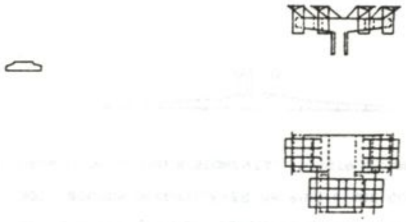
SEQUENCE OF CONSTRUCTION

STAGE 1 CONSTRUCTION OF FIRST SEGMENT ON PIER C

FIRST PART OF GIRDER NEAR PIER C IS CONCRETED ON FALSEWORK.

AND THEN ONE WAGON IS PLACED ON THE NORTH BOUND (OR SOUTH BOUND)

STAGE 2 COMMENCEMENT OF SEGMENTAL CONSTRUCTION.



STAGE 3 PREPARATION FOR CONCRETING FIRST SEGMENT ON PIER A AND B

WHILE SEGMENTAL CONSTRUCTION FROM PIER C IS CARRIED OUT SUCCESSIVELY

FALSEWORKS OF FIRST SEGMENT ON PIERS A AND B ARE UNDER PREPARATION

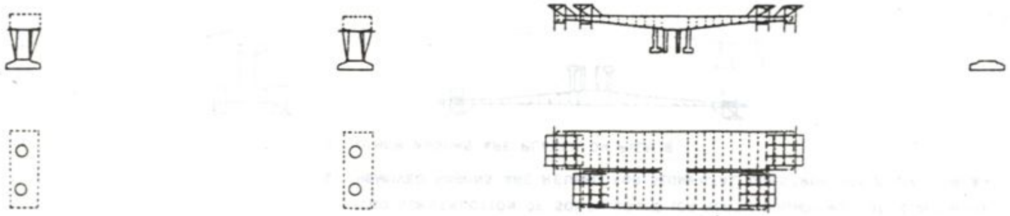


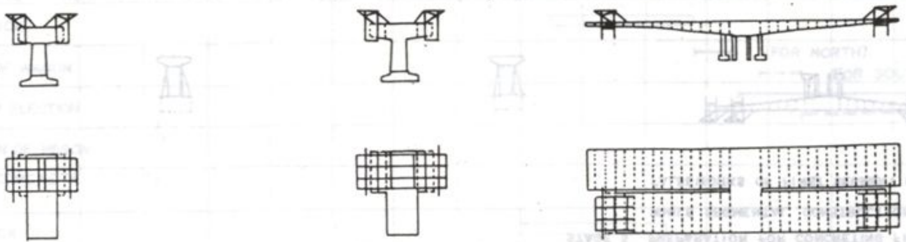
Fig.4-1-12

STAGE 4 CONSTRUCTION OF FIRST SEGMENT ON PIERS A AND B

1 THE WAGON FOR THE NORTH BOUND (OR SOUTH BOUND) IS DISMANTLED WHEN THE FINAL SEGMENT IS COMPLETED AND CONSTRUCTION OF SOUTH BOUND (OR NORTH BOUND) WILL BE CONTINUED.

2 REMOVED WAGONS ARE REUSED FOR SEGMENTAL CONSTRUCTION FROM PIER A.

3 OTHER WAGONS ARE PLACED ON PIER B.

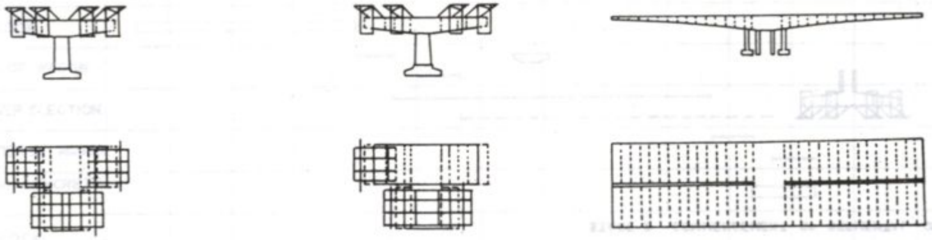


STAGE 5 COMMENCEMENT OF SEGMENTAL CONSTRUCTION ON PIERS A AND B
CONNECTION OF BOTH BOUND OF CANTILEVERS ON PIER C

1 AFTER COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER C, REMOVED WAGONS ARE REUSED FOR CONSTRUCTION FROM PIER A

2 BOTH BOUNDS OF CONTILEVER ON PIER C ARE CONNECTED AFTER PLACING SLAB AND CROSS-BEAM.

3 FROM PIERS A AND B SEGMENTAL CONSTRUSTION WILL BE CONTINUED.



STAGE 6 PREPARATION FOR CONCRETING FIRST SEGMENT ON PIERS D AND E
WHILE SEGMENTAL CONSTRUCTION FROM PIERS A AND B IS CARRIED OUT SUCCESSIVELY
FIRST PARTS OF GIRDER NEAR PIERS D AND E IS CONCRETED ON FALSEWORK.

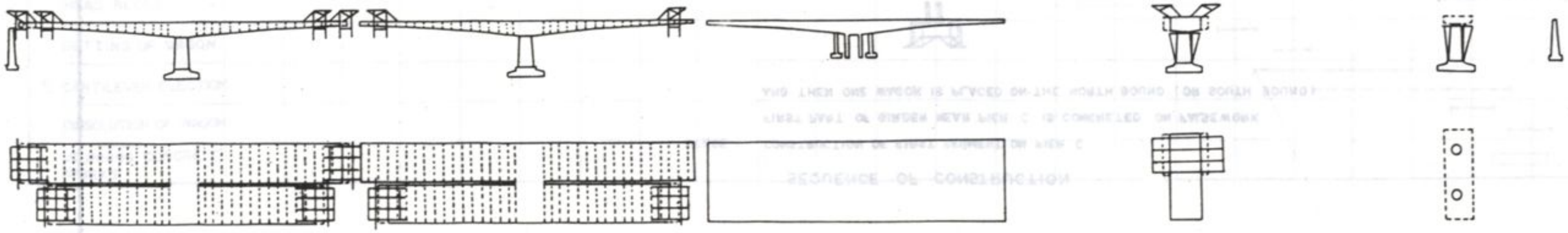
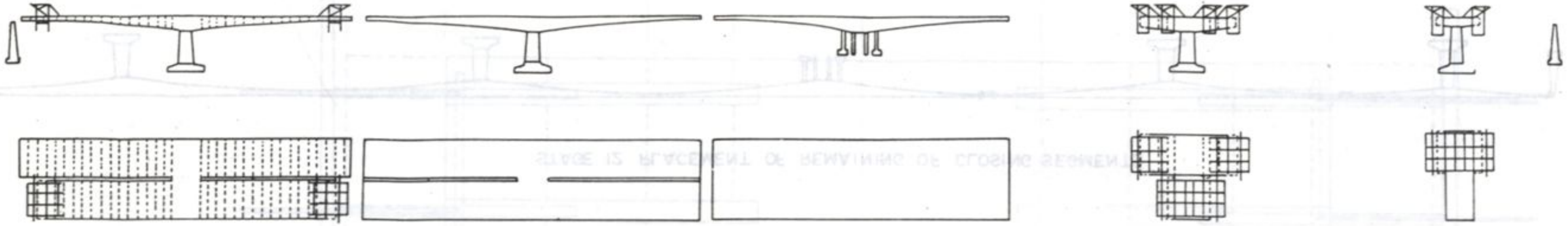


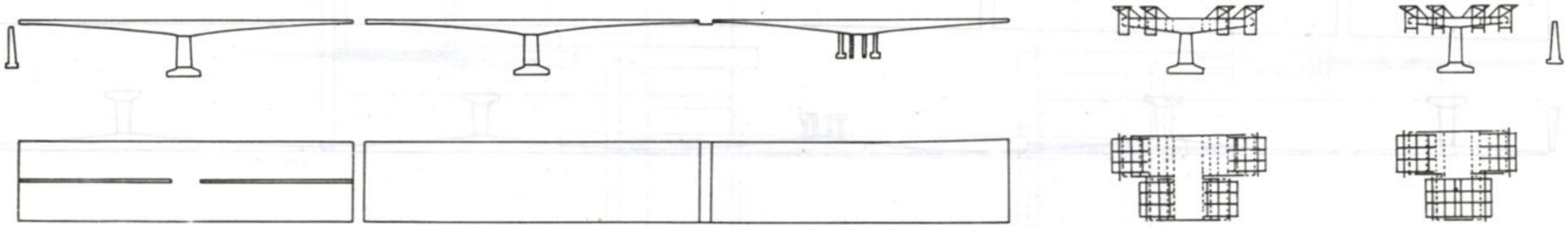
Fig.4-1-13

STAGE 7 COMMENCEMENT OF SEGMENTAL CONSTRUCTION ON PIERS D AND E



STAGE 8 PLACEMENT OF CLOSING SEGMENT BETWEEN TWO CONTILEVERS ON PIERS B AND C

CLOSING SEGMENT BETWEEN BOTH ENDS OF COTILEVERS ON PIERS B AND C IS PLACED AND ADDITIONAL TENDONS ARE TENSIONED.



STAGE 9 PLACEMENT OF CLOSING SEGMENT BETWEEN TWO CONTILEVERS ON PIERS A AND B

PLACEMENT OF CLOSING SEGMENT AT PIER N.

CLOSING SEGMENTS ARE PLACED WHILE SEGMENTAL CONSTRUCTIONS FROM PIERS D AND E ARE CONTINUED.

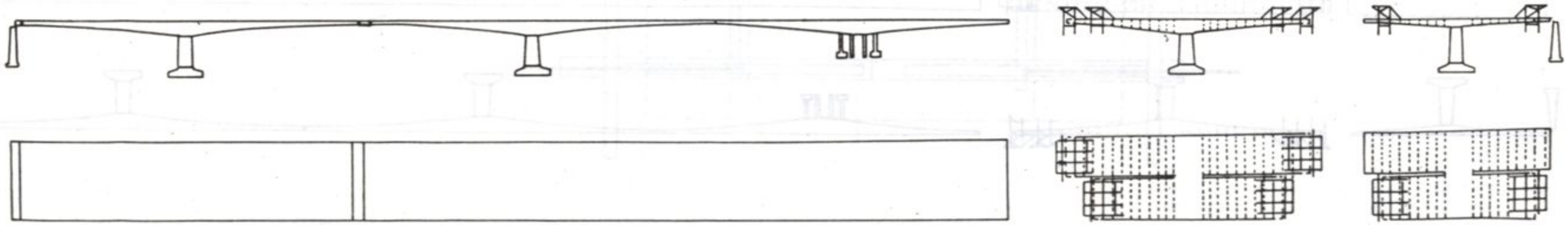
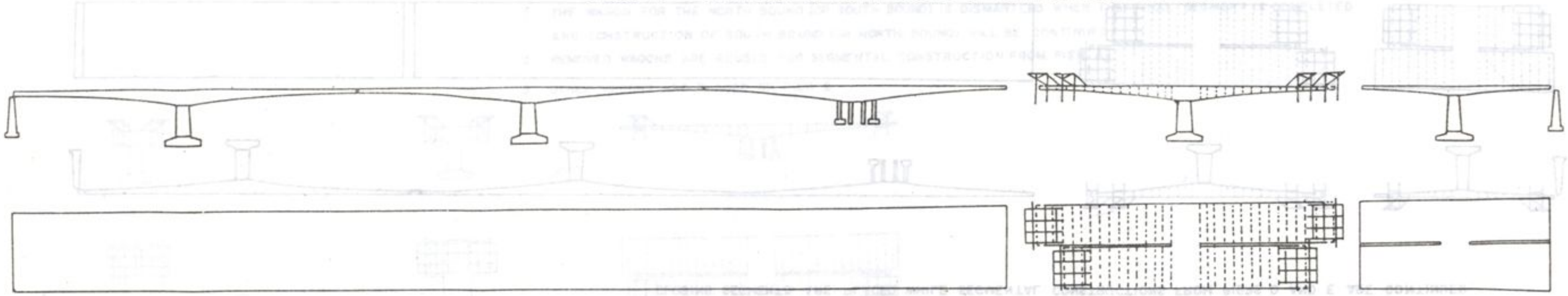
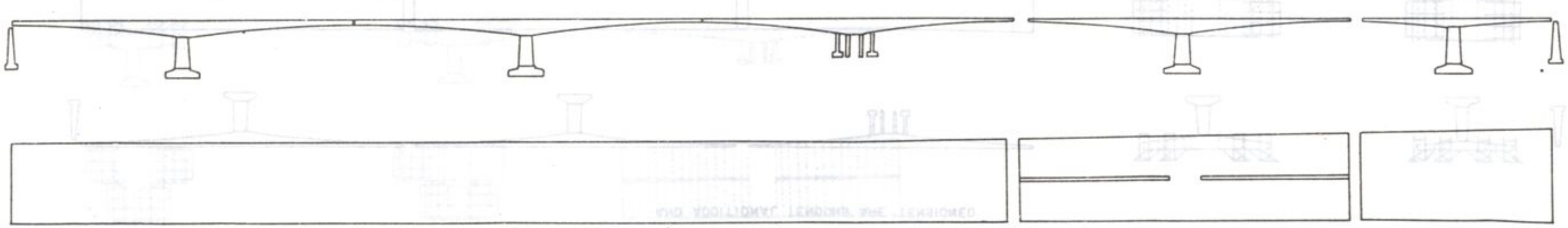


Fig.4-1-14

STAGE 10 CONNECTION OF BOTH BOUND OF CANTILEVERS ON PIER E



STAGE 11 CONNECTION OF BOTH BOUND OF CANTILEVERS ON PIER D



STAGE 12 PLACEMENT OF REMAINING OF CLOSING SEGMENTS

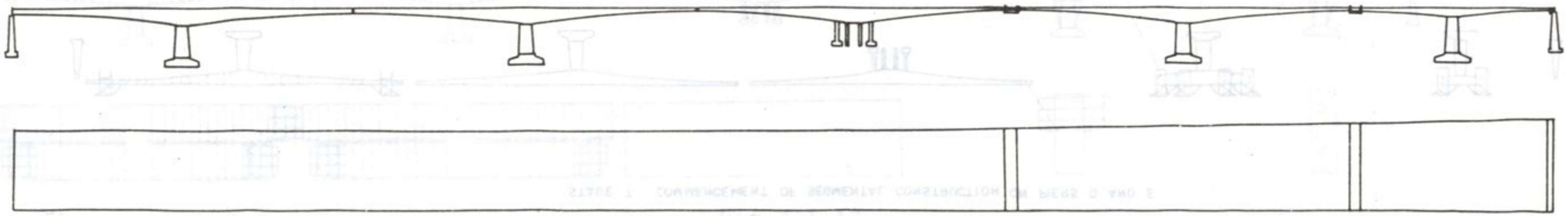


Fig.4-1-15

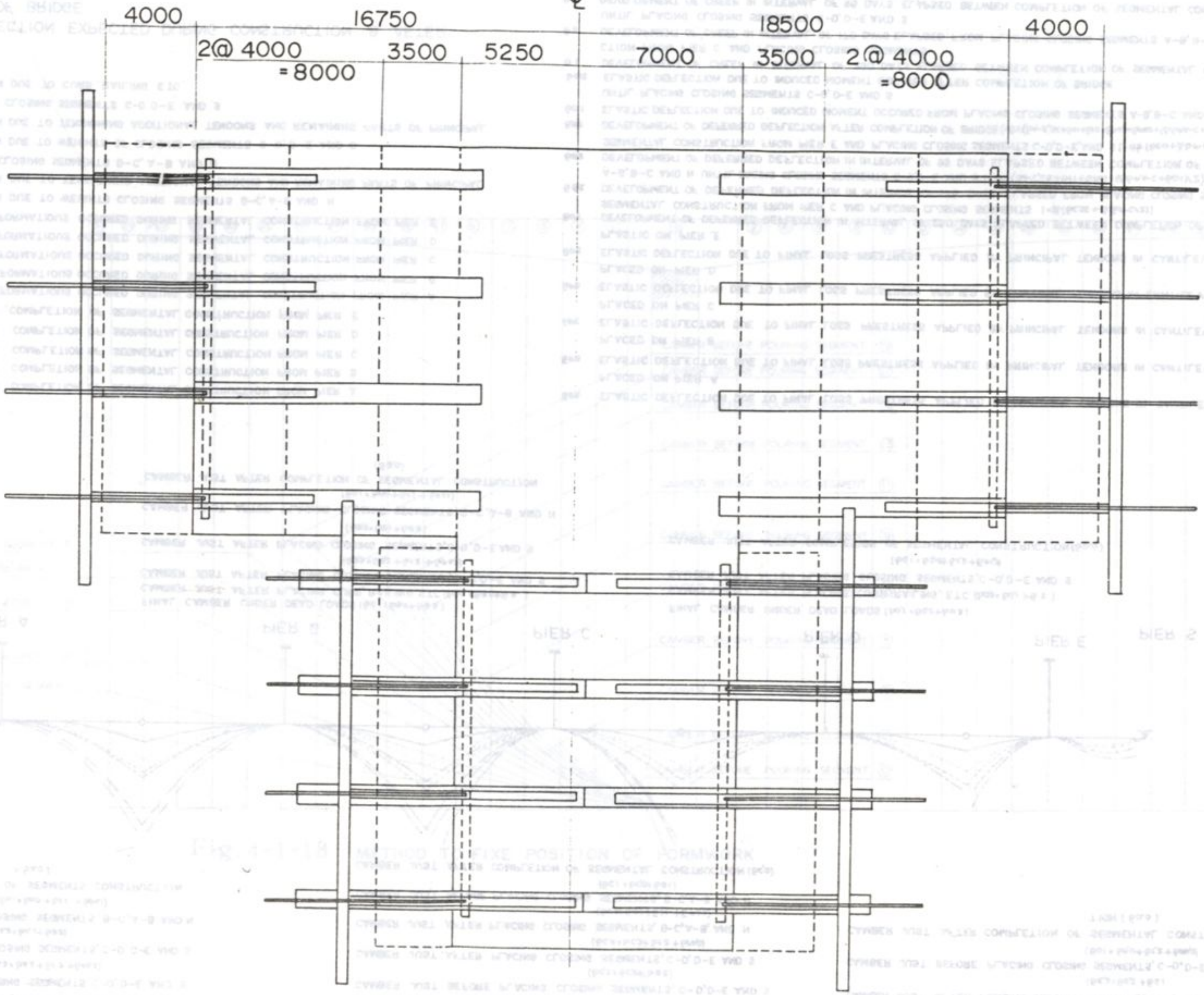
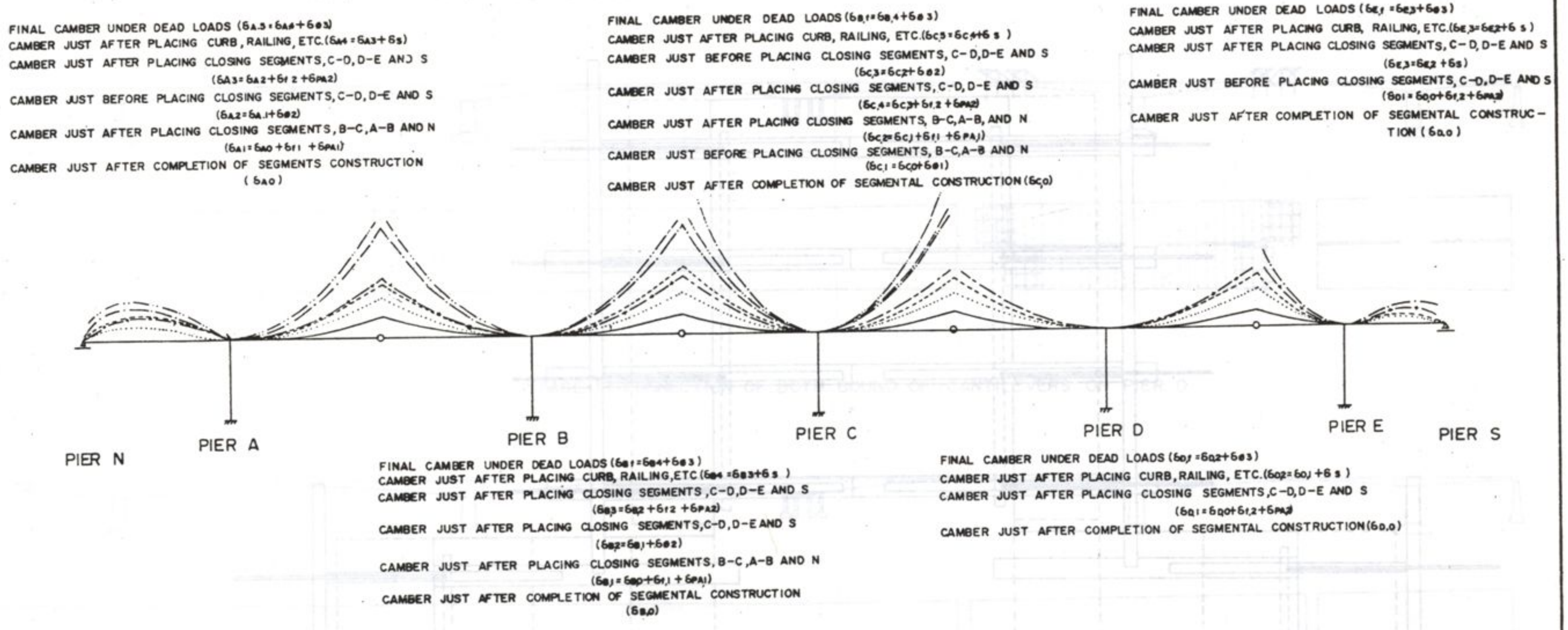


Fig.4-1-16　NECESSARY SPACE OF EACH WAGON SHOWN AT PIER E



δ_{A0}	CAMBER JUST AFTER COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER A
δ_{B0}	CAMBER JUST AFTER COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER B
δ_{C0}	CAMBER JUST AFTER COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER C
δ_{D0}	CAMBER JUST AFTER COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER D
δ_{E0}	CAMBER JUST AFTER COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER E
$\delta_{A,SC}$	SUM OF ELASTIC DEFORMATIOUS OCCURED DURING SEGMENTAL CONSTRUCTION FROM PIER A
$\delta_{B,SC}$	SUM OF ELASTIC DEFORMATIOUS OCCURED DURING SEGMENTAL CONSTRUCTION FROM PIER B
$\delta_{C,SC}$	SUM OF ELASTIC DEFORMATIOUS OCCURED DURING SEGMENTAL CONSTRUCTION FROM PIER C
$\delta_{D,SC}$	SUM OF ELASTIC DEFORMATIOUS OCCURED DURING SEGMENTAL CONSTRUCTION FROM PIER D
$\delta_{E,SC}$	SUM OF ELASTIC DEFORMATIOUS OCCURED DURING SEGMENTAL CONSTRUCTION FROM PIER E
δ_{f1}	ELASTIC DEFLECTION DUE TO WEIGHTS CLOSING SEGMENTS B-C, A-B AND N
δ_{PA1}	ELASTIC DEFLECTION DUE TO TENSIONING ADDTIONAL TENDONS AND REMAINING PARTS OF PRINCIPAL TENDONS PLACED IN CLOSING SEGMENTS B-C, A-B AND N
δ_{f2}	ELASTIC DEFLECTION DUE TO WEIGHTS OF CLOSING SEGMENTS C-D, D-E AND S
δ_{PA2}	ELASTIC DEFLECTION DUE TO TENSIONING ADDITIONAL TENDONS AND REMAINING PARTS OF PRINCIPAL TENDONS PLACED IN CLOSING SEGMENTS C-D D-E AND S
δ_{s}	ELASTIC DEFLECTION DUE TO CURB, RAILING ETC.

δ_{PA}	ELASTIC DEFLECTION DUE TO FINAL LOSS PRESTRESS APPLIED BY PRINCIPAL TENDONS IN CANTILEVER PLACED ON PIER A
δ_{PB}	ELASTIC DEFLECTION DUE TO FINAL LOSS PRESTRESS APPLIED BY PRINCIPAL TENDONS IN CANTILEVER PLACED ON PIER B
δ_{PC}	ELASTIC DEFLECTION DUE TO FINAL LOSS PRESTRESS APPLIED BY PRINCIPAL TENDONS IN CANTILEVER PLACED ON PIER C
δ_{PD}	ELASTIC DEFLECTION DUE TO FINAL LOSS PRESTRESS APPLIED BY PRINCIPAL TENDONS IN CANTILEVER PLACED ON PIER D
δ_{PE}	ELASTIC DEFLECTION DUE TO FINAL LOSS PRESTRESS APPLIED BY PRINCIPAL TENDONS IN CANTILEVER PLASTIC ON PIER E
$\delta_{\phi 1}$	DEVELOPMENT OF DEFERRED DEFLECTION IN INTERVAL OF 250 DAYS ELAPSED BETWEEN COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER C AND PLACING CLOSING SEGMENTS [$=\phi_1(\delta_{C,SC}+\Delta\delta_{P-C}/2)$]
$\delta_{\phi 2}$	DEVELOPMENT OF DEFERRED DEFLECTION IN INTERVAL OF 175 DAYS ELAPSED FROM PLACING CLOSING SEGMENTS A-B, B-C AND N UNTIL PACING CLOSING SEGMENTS C-D, D-E AND S [$=\phi_2\{\delta_{A-C,CS}+\delta_{f1}+\delta_{PA1}+(\Delta\delta_{PA-C}+\delta_{ci1})/2\}$]
$\delta_{\phi 2}'$	DEVELOPMENT OF DEFERRED DEFLECTION IN INTERVAL OF 95 DAYS ELAPSED BETWEEN COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER E AND PLACING CLOSING SEGMENTS C-D, D-E, AND S [$=\phi_2'(\delta_{E0}+\Delta\delta_{P-E}/2)$]
$\delta_{\phi 3}$	DEVELOPMENT OF DEFERRED DEFLECTION AFTER COMPLETION OF BRIDGE($=\phi_3\{\delta_{A-E,SC}+\delta_{f1}+\delta_{f2}+\delta_{PA1}+\delta_{PA2}+(\Delta\delta_{PA-E}+\delta_{ci2})/2\}$)
δ_{ci1}	ELASTIC DEFLECTION DUE TO INDUCED MOMENT OCCURED FROM PLACING CLOSING SEGMENTS A-B, B-C AND N UNTIL PLACING CLOSING SEGMENTS C-D, D-E AND S
δ_{ci2}	ELASTIC DEFLECTION DUE TO INDUCED MOMENT OCCURED AFTER COMPLETION OF BRIDGE
ϕ_1	DEVELOPMENT OF CREEP IN INTERVAL OF 250 DAYS ELAPSED BETWEEN COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER C AND PLACING CLOSING SEGMENTS
ϕ_2	DEVELOPMENT OF CREEP IN INTERVAL OF 175 DAYS ELAPSED FROM PLACING CLOSING SEGMENTS A-B, B-C AND N UNTIL PLACING CLOSING SEGMENTS C-D, D-E AND S
ϕ_2'	DEVELOPMENT OF CREEP IN INTERVAL OF 95 DAYS ELAPSED BETWEEN COMPLETION OF SEGMENTAL CONSTRUCTION FROM PIER E AND PLACING CLOSING SEGMENTS C-D, D-E AND S
ϕ_3	DEVELOPMENT OF CREEP AFTER COMPLETION OF BRIDGE

VARIOUS DEFLECTION EXPECTED DURING CONSTRUCTION & AFTER COMPLETION OF BRIDGE

Fig.4-1-17 VARIOUS DEFLECTION EXPECTED DURING CONSTRUCTION & AFTER COMPLECTION OF BRIDGE

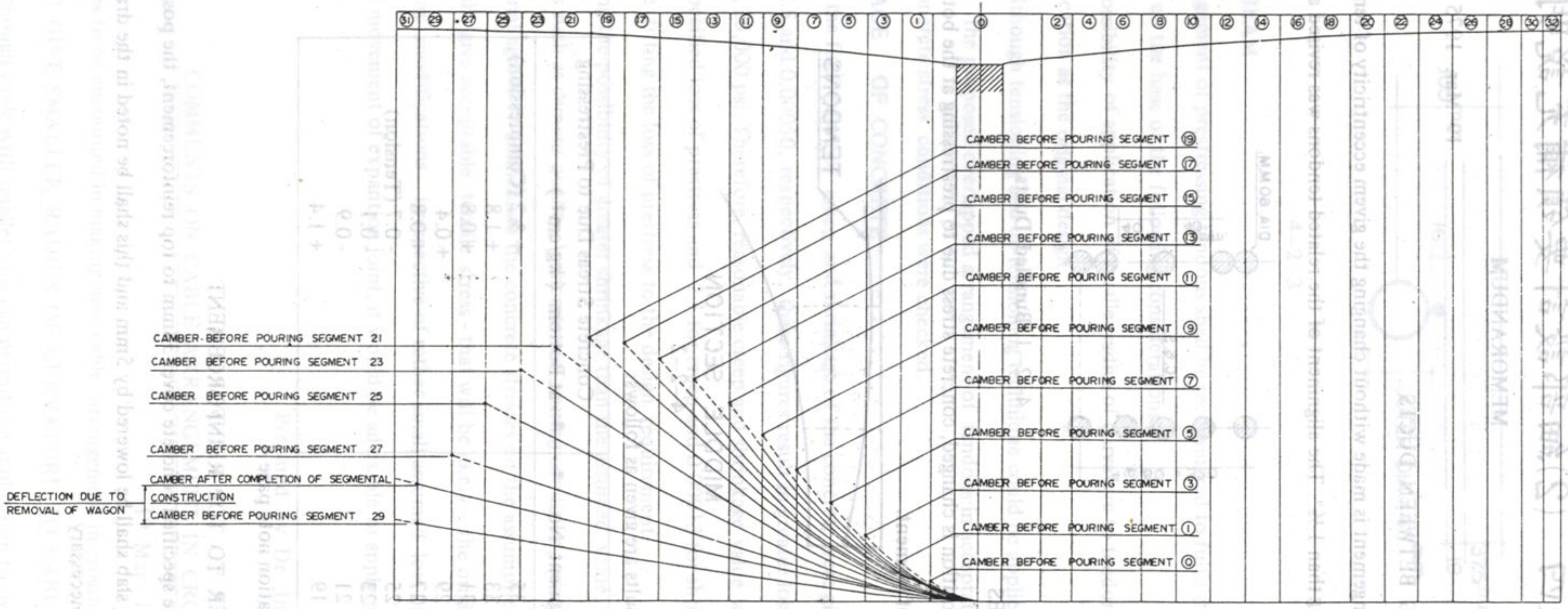


Fig.4-1-18 METHOD TO FIXE POSITION OF FORMWORK

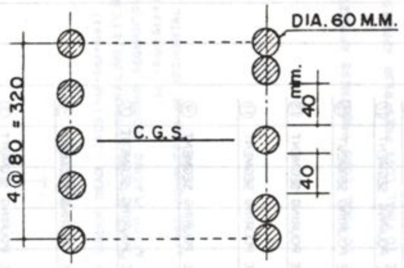
附錄四 (2)細部設計要領補充説明

MEMORANDUM

19 Mar. 1975

A) CLEAR SPACING BETWEEN DUCTS

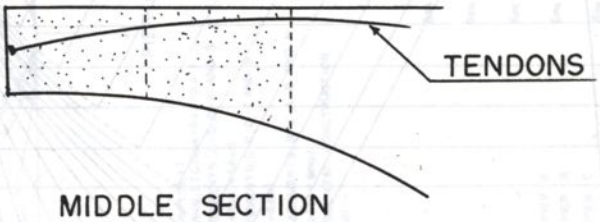
Bundled duct arrangement is made without changing the givem eccentricity of tendons, in order to give greater Spacing than 1-½". The alignment of the related tendons was revised as shown in sheet No. 32-34.



4–2–1 Bundled Ducts.

B) LOCAL STRESSES

Since anchorage position is changed, concrete stress due to prestressing at the bottom was checked at middle sectio of segment.



MIDDLE SECTION

4–2–2

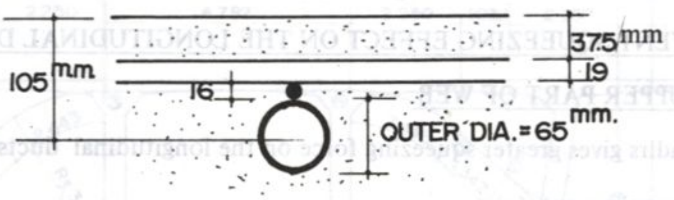
The Calculated results are given as follows:

Segment No. *	* Concrete Streas Due to Prestressing at Bottom (kg/cm^2)
35	+ 3.2 (Compression)
33	+ 1.8
31	+ 0.8
29	+ 0.4
27	+ 0.8
25	- 0.7 (Tension)
23	- 0.1
21	- 0.9
19	+ 1.4

* reffer to calculation note page 4

C) MINIMUM COVER TO TOP REINFORCEMENT

In order to give the specified concrete cover 38mm to top reinforcement, the position of duct placed in the deck slab shall be lowered by 5mm and this shall be noted in the drawings. Stress calculation is not necessary.



4–2–3

D) CROSS BEAM

The arrangement of prestressing tendons shall be reconsidered to fulfil the basic design conception in Tokyo on the base of the following considerations:

(a) Possibility of application of smaller radius of curvature to tendon, keeping the same anchor location as the original design.

(b) Although tensioning and de-tensioning technique could be applied to reversely curved tendons the proposed overlapped arrangement of tendons in acceptable provided that the local concrete stress conditions were checked.

E) SHRINKAGE OF CONCRETE

Although the final values of creep and shrinkage of the concrete were given in the design note to be equal to 2.5 and 0.00050, respectively, these figures result in greater loss of prestress than the specified value, 25,000 psi. Therefore, reasonable creep and shrinkage value was estimated in order to obtain the specified loss of prestress and 2.1 for creep and 0.00013 for shrinkage were thought to be reasonable fitting the loss of prestress to the design requirement.

Considering the possibility of greater shrinkage than the assumed value, affected by various actual conditions such as cement quality, concrete mix, curing condition, relative humidity, etc. T. Y. Lin International agreed to recalculate the concrete stresses on the assumption of greater shrinkage value If the results give acceptable tensile stress - that will be likely -, the original design could be adopted without any modifications irrespective of rather small assumed loss of prestress. In calculating the horizontal movement of expantion joint, it would be advisable to apply greater value of shrinkage, for safety.

Prepared by Dr. Inomata

COMMENTS ON CABLE ARRANGEMENT IN CROSS–BEAM

19 Mar. 1975

Defficulties to be encounted in making new cable arrangement in crossbeam:

(a) TO GIVE SMALLER RADIUS OF CURVATURE TO CABLES

Although cable with smaller unit can permit the application fo smaller radius, the number of cables to be placed in the crossbeam will increase, resulting in congestion of ducts. This will result in the difficulties of cagle arrangement as well as concreting.

(b) **TO PREVENT SQUEEZING EFFECT ON THE LONGITUDINAL DUCTS PLACED IN THE UPPER PART OF WEB**

Smaller radirs gives greater squeezing force on the longitudinal ducts and this is inconsistent with the requirement (a).

The most practicla solution seems to apply the necessary vertical prestressing in the cross-beam with prestressing bars.

Calculation of effective prestressing force due to tendous placed in tie beam

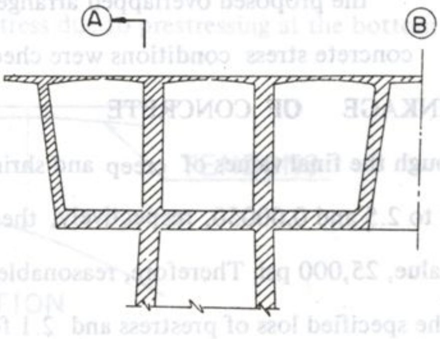
Pier A&B

Friction coefficient $- U = 0.20, K = 0.00133$

pull - in — 13 mm

Initial stress — $\sigma pi = 140$ kg/mm²

Critical section



Section A

$$\sigma pe = \frac{107.6 + 103.3 + 115.2}{3} - 17.5 = 91.2 \text{ kg/mm}^2$$

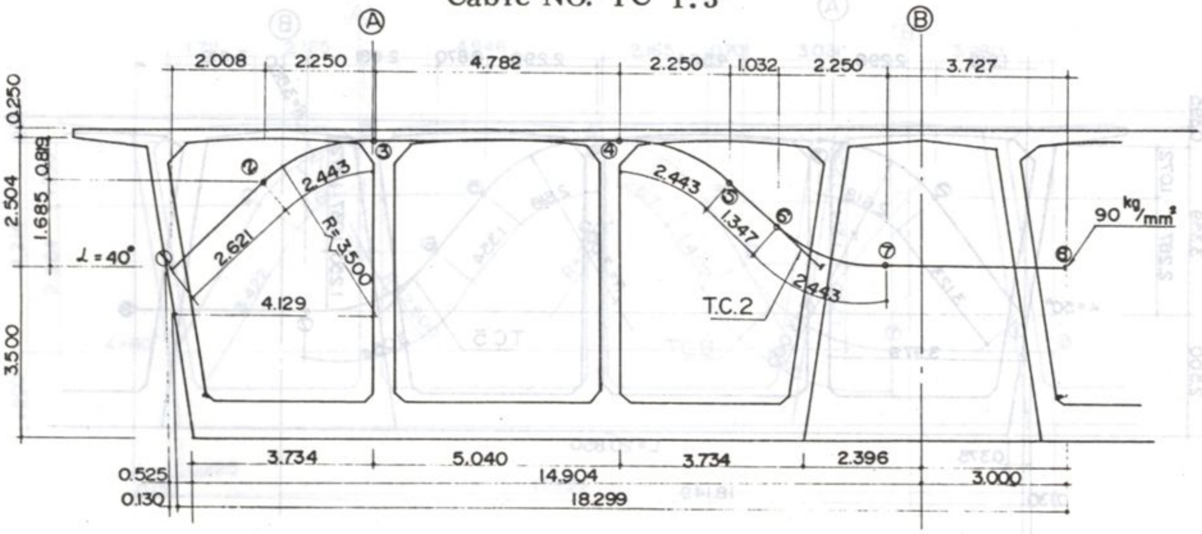
$$Pe = N, Ap \cdot \sigma pe = 9 \times 1184.52 \times 91.2 \times 10^{-3} = 972 \text{ t} \approx 1000\text{t}$$

Section B

$$\sigma pe = \frac{94.4 + 92.2 + 91.3}{3} - 17.5 = 75.1 \text{ kg/mm}^2$$

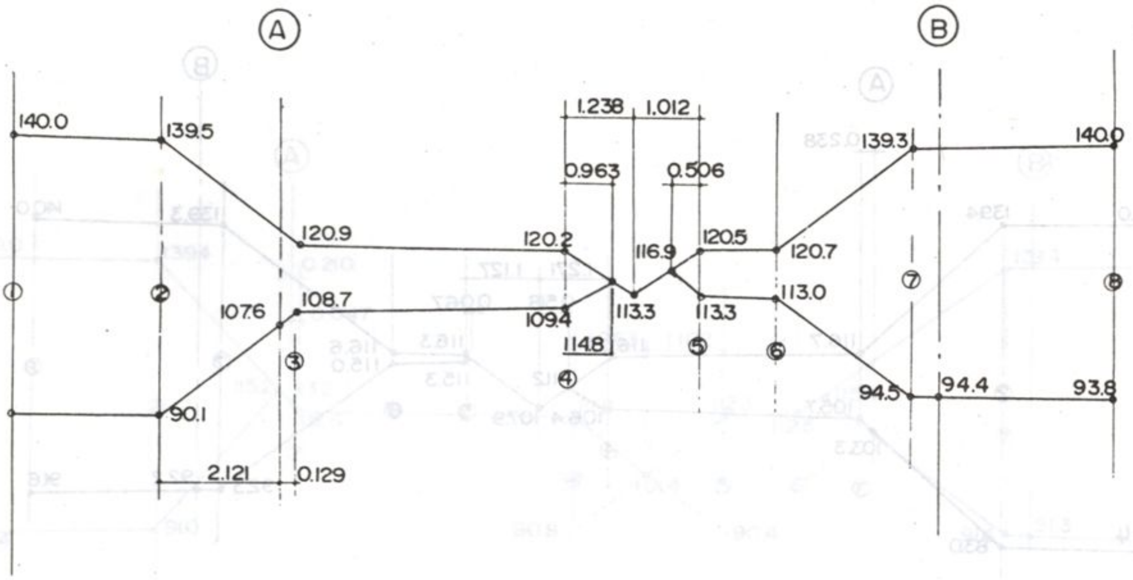
$$Pe = N \cdot Ap \cdot \sigma pe = 12 \times 1184.52 \times 75.1 \times 10^{-3} = 1067\text{t} \approx 1000\text{t}$$

Cable NO. TC 1.3



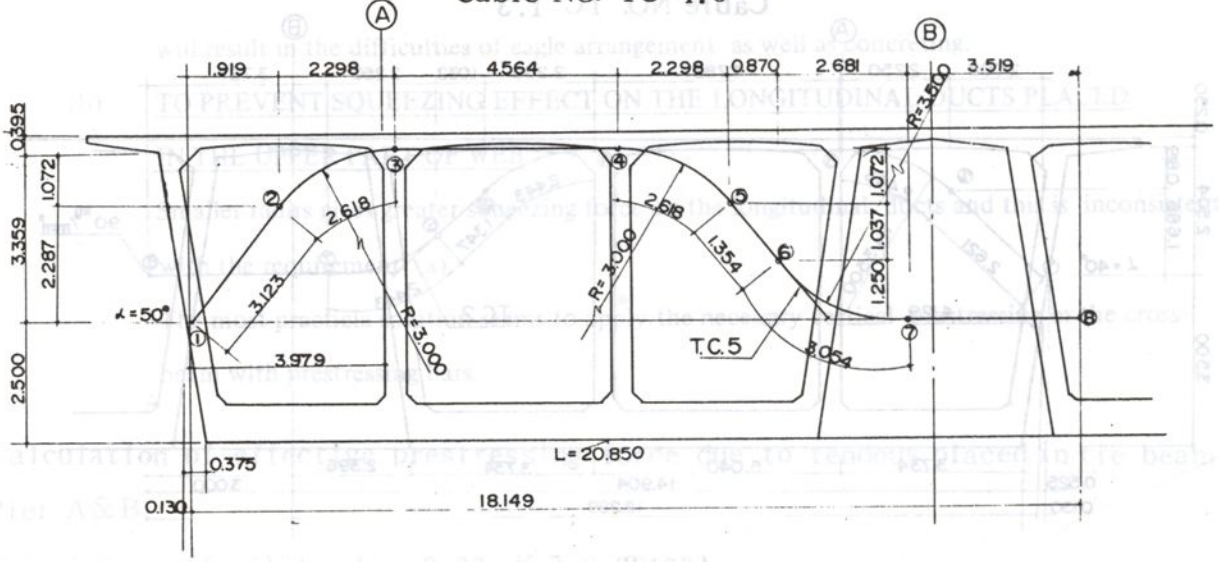
4 — 2 — 5

	①	②	③	④	⑤	⑥	⑦	⑦
ℓ(m)	0	2.621	5.064	9.846	12.289	13.636	16.079	19.806
α(md)	0	0	0.698	0.698	1.396	1.396	2.084	2.094
$K\ell + \mu\alpha$	0	0.0035	0.1462	0.1527	0.2955	0.2973	0.4402	0.4451
σpix	440.0	139.5	120.9	120.2	104.2	104.0	90.1	89.7
ℓ(m)	19.806	17.185	14.742	9.960	7.517	6.170	3.727	0
α(rad)	2.094	2.094	1.396	1.396	0.698	0.698	0	0
$K\ell + \mu\alpha$	0.4451	0.4417	0.2988	0.2924	0.1496	0.1478	0.0050	0
σpix	69.7	90.0	103.8	104.5	120.5	120.8	131.3	140.0



4 — 2 — 6

Cable NO. TC 4.6

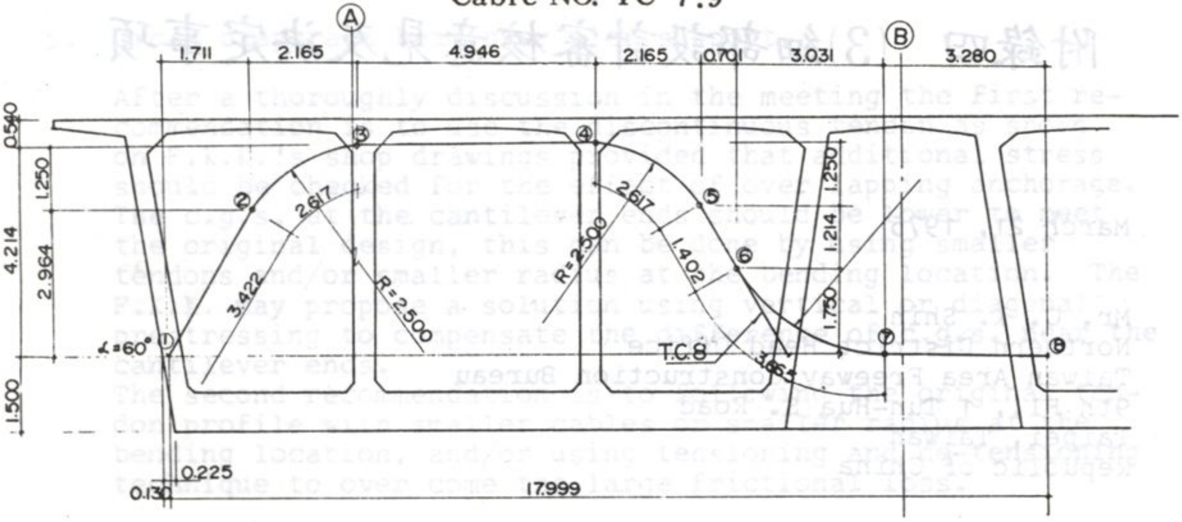


4 − 2 − 7

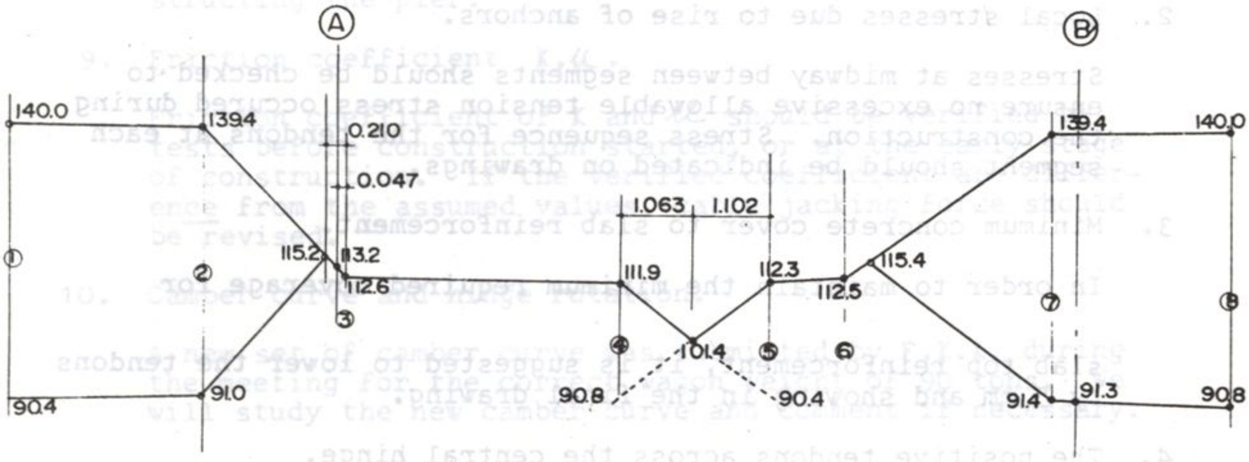
	①	②	③	④	⑤	⑥	⑦	⑧
ℓ(m)	0	3.123	5.741	10.305	12.923	14.277	17.331	20.850
α(md)	0	0	0.873	0.873	1.745	1.745	2.618	2.618
Kℓ + μα	0	0.0042	0.1822	0.1883	0.3662	0.3680	0.5467	0.5513
σpix	140.0	139.4	116.7	116.0	97.1	96.9	81.0	80.7
ℓ(m)	20.850	17.727	15.109	10.545	7.927	6.573	3.519	0
α(rad)	2.618	2.618	1.745	1.745	0.873	0.873	0	0
Kℓ + μα	0.5513	0.5472	0.3691	0.3630	0.1851	0.1833	0.0047	0
σpix	80.7	81.0	96.8	97.4	116.3	116.6	139.3	140.0

4 − 2 − 8

Cable NO. TC 7.9



	①	②	③	④	⑤	⑥	⑦	⑧
ℓ(m)	0	3.422	6.039	10.985	13.602	15.004	18.669	21.949
α(md)	0	0	1.047	1.047	2.094	2.094	3.142	3.142
Kℓ + μα	0	0.0046	0.2174	0.2240	0.4369	0.4388	0.6532	0.6596
σpix	140.0	139.4	112.6	111.9	90.4	90.3	72.9	72.5
ℓ(m)	21.949	18.527	15.910	10.964	8.347	6.945	3.280	0
α(rad)	3.142	3.142	2.094	2.074	1.047	1.047	0	0
Kℓ + μα	0.6576	0.6530	0.4400	0.4334	0.2205	0.2186	0.0044	0
σpix	72.5	72.9	90.2	90.8	112.3	112.5	139.4	140.0



附錄四 (3)細部設計審核意見及決定事項

March 21, 1975

Mr. C. K. Shih
Northern District Head Office
Taiwan Area Freeway Construction Bureau
9th Fl., 1 Tun-Hua S. Road
Taipei, Taiwan
Republic of China

Subject: Yuan Shan Bridge - Post Tension

Dear C.K.:

Refer to your letter dated Jan. 29, 1975, your file number 北圓 64-526-1 (6) regarding the post tension detail drawings, and calculation of the superstructure of the subject project. The telex and written communications between our office and F.K.K. during the period of Feb. 28 to March 17 were attached as our attachment No. 1. For the purpose to clarify the discussion, we set a meeting on March 18, 1975 with the general contractor, Continental Engineering Corp, sub-contractor F.K.K. and with the attention of your staffs. We would like to conclude our comments as follows:

1. Longitudinal tendons spacing in web.

   The F.K.K.'s submission shows that the longitudinal tendons at web area have 5 numbers in a row spaced at 8 cm center to center which will provide minimum workable spacing between tendons. It is suggested to bundle every two tendons together in order to provide enough spacing for concreting work. The overall arrangement should not change the resulting c.g.s.

2. Local stresses due to rise of anchors.

   Stresses at midway between segments should be checked to ensure no excessive allowable tension stress occured during the construction. Stress sequence for the tendons at each segment should be indicated on drawings.

3. Minimum concrete cover to slab reinforcement.

   In order to maintain the minimum required coverage for slab top reinforcement, it is suggested to lower the tendons by 5 mm and showing in the final drawing.

4. The positive tendons across the central hinge.

   The drafting error has been clarified in the meeting.

5. Cross diaphragm beam over pier support.

After a thoroughly discussion in the meeting the first recommendation is to use the discontinuous tendon as shown on F.K.K.'s shop drawings provided that additional stress should be checked for the effect of over lapping anchorage. The c.g.s. at the cantilever ends should be lower to meet the original design, this can be done by using smaller tendons and/or smaller radius at the bending location. The F.K.K. may propose a solution using vertical or diagonal prestressing to compensate the difference of c.g.s. near the cantilever ends.
The second recommendation is to following the original tendon profile with smaller cables or smaller radius at the bending location, and/or using tensioning and de-tensioning technique to over come the large frictional loss.

6. Shrinkage coefficient and total prestress loss.

F.K.K. use average creep of 2.1 and shrinkage coefficient of 0.00013 in their calculations. In the original design the maximum creep is 2.5 with loading in 7 days, the maximum shrinkage coefficient is 0.0005. The effects of these coefficient to the prestress loss were discussed during the meeting. It was believed that the bridge behaviour will not be significantly affected by the additional prestress loss. However, the final concrete stresses on the assumption of greater prestress loss was checked. In order to reduce the steel relexation loss, we suggest to hold the tendon for 10 minutes before anchoring. This will greatly reduce total prestress losses. And attached herewith as calculation number 2, the result gives acceptable concrete stresses.

7. Wagon weight.

F.K.K. will submit the necessary modification according to the correct wagon weight of 90 tons.

8. Vertical Shortening of pier.

It is suggested that the vertical shortening of pier due to dead load of the bridge can be compensated when constructing the pier.

9. Friction coefficient K, μ.

Friction coefficient of K and μ should be verified by tests before construction started, or at the early stage of construction. If the verified coefficient are difference from the assumed values, cable jacking force should be revised.

10. Camber curve and hinge rotation.

A new set of camber curve was submitted by F.K.K. during the meeting for the correct wagon weight of 90 tons. We will study the new camber curve and comment if necessary.

11. Camber at last segment.

The abrupted camber at last segment for the first set of calculation was due to the incorrect wagon weight, it was clarified during the meeting.

Also attached herewith a set of computer out put which shows the stress due to actural tendons arrangement of F.K.K. for your reference.

Should you have further questions and comments, please do not hesitate to write me again. Best regards, I remain,

Sincerely yours,

T. Y. LIN INTERNATIONAL

Ernest S.J. Loh

Ernest S.J. Loh
Vice President

ESJL/ys

Encl. as above

TELEX TO F. K. K. Feb. 28. 1975

1. Tendon spacing (vertical) at web area should be increased from 8 cm to 20 cm min. to provide 1½" min, clear spacing between tendons.

2. Tendon anchored at that section should check kern point to avoid local tension.

3. Top slab coverage 10 cm is not enough, Prefer to use 12.5 cm (4" - 1½" - 6/8" - 5/8") = 1.1"<1.5" min.

4. Positive tendon should be anchored at the cantilever tip. Drawing showing positive tendons across joint is not correct.

5. Tie bean tendon C. G. should follow original drawing. (Since we need shear component to reduce shear.) Also stress condition should be verified.

6. Assumed shrinkage 0.00013 seems too low. Please verify . with actual concrete test value.

7. Wagon self weight 200 T. (original 90 T)

8. Vertical pier deflection due to dead load should be adjusted when casting the segment over the pier.

9. Friction loss due to K. U (L and ∝) should be verified. Since ASSHO value may not satisfy individual individual case.

10. Hinge detail should be verified for new proposed . Since due to 0.0005L will increase rotation demand.

11. Proposed camber at last segment with sbrubted curve will cause additional impact. It should be further study.

4. 3. 75 1330

ATTENTION MR HO CONTIENGIN REPLYING YTLX FEB 28:

AA WE DID NOT GIVE ANY DRAWING OF VERTICAL TENDON DIAMETER OF SHEATH FOR VERTICAL TENDON WILL BE 35MM AND IN ORDER TO GIVE MINIMUM CLEAR SPACING 38MM BETWEEN TENDONS, MINIMUM SPACING OF VERTICAL TENDONS SHOULD BE 35 + 38 = 73MM.

BB TENDON ARRANGEMENTS WERE DETERMINED TO GIVE NECESSARY PRESTRESSING FORCE AND EXCENTRICITY IN CONFORMITY WITH ORIGINAL DESIGN THERFORE THERE IS NO NECESSITY TO CHECK CONCRETE STRESSES.

CC MINIMUM THICKNESS OF TOP SLAB IS 20CM AND DIAMETER OF SHEATH OF LONGITUDI-INAL TENDON IS 60MM THEREFORE 10CM COVER IS MINIMUM IN ORDER TO PLACE SHEATH AND TWO DIRECTIONS REINFORCEMENT AT THIS SECTION.

DD DRAWING NO 34. SHOULD BE REVISED TENDON SHOULD BE CUT AT JOINT SECTION.

EE CHANGING DIRECTION OF TENDON WOULD NOT BE ALLOWED. TENDONS SHOWN IN ORIGINAL DRAWING COULD NOT BE PRESTRESSED TO GIVE NECESSARY PRESTRESSING FORCE AT MIDDLE PARTS. IF VERTICAL COMPONENT OF PRESTRESSING FORCE IS NECESSARY, ADDITIONAL INCLINED PRESTRESSING BARS SHOUL BE PLACED.

FF IN ORDER TO VERIFY LOSS OF PRESTRESS (25000 PSI) GIVEN BY ORIGINAL DESIGN, SHRINKAGE OF CONCRETE WAS ESTIMATED AS 0.00013.

GG 200T WAS WEIGHT INCLUDING CONCRETE THERFORE DEFLECTION DUE TO WEIGHT OF WAGON SHOULD BE RECALCULATED BY MULTIPLYING 90/200 TO PREVIOUS CALCULATED DEFLECTION.

HH ADOPTION OF ASSHO VALUES FOR U AND K WERE FIXED DURING PREVIOUS MEETING (SEPT 25 1974 DR.LO MR LOH)

II NEWLY PROPOSDE DELTA THETA IS UNKNOWN TO US FURTHER WE THINK HINGE DESIGN IS NOT OUR ALLOTED TASK.

FREYSSIKOGEN

ANSWERING TO FEB. 28, TELEX:

AA. The concerned min. verical spacing is at principal tendon which located at web area.

BB. Since anchorage location were changed, the local stress will be changed. A check to determing whether excessive tension produced by new anchorage depends on the check.

CC. Top of slab provided with min. 1½" cover required more than 10 cm for top row of tendons. The bott. of slab required less cover as shown in ASSHO.

DD. Acknowledged.

EE. Vertical componetnt is necessary.

FF. Shrinkage will effect deflection more seriously than strength and stress. For better results a test should be proceed and re-submit.

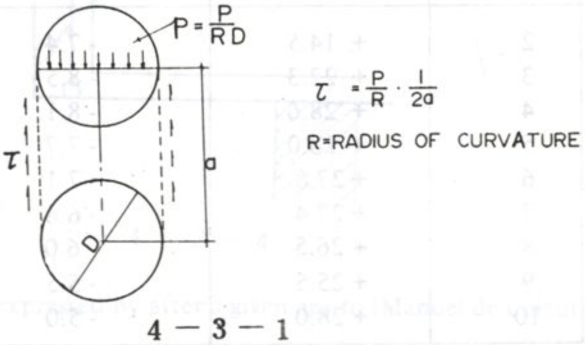
GG. Plesae revise and re-submit

HH. Testing proof for this project is still warrent.

II. You are correct in this responsibility.. We will notify proper channel for further study.

A) Clear spacing between Ducts in Web

Vertical clear spacings of 20mm between ducts are widely used in the region of straight arrangement of ducts, on Japan. The minimum distances between ducts in curved part should be determined considering forces resulting from curvature and change in direction of the tendons to avoid crushing of duct.



4 — 3 — 1

The most unfavourable section is that at the face of pier where the distance between two ducts is 10cm and raduis of curvature of upper ducts is 30m . Assuming 110t of prestressing force, P and τ are given as follows:

$$P = \frac{110 \times 10^3}{3000 \times 6} = 6.1 \text{ Kg/ cm}^2$$

$$\tau = \frac{110 \times 10^3}{3000 \times 20} = 1.8\ \text{Kg/cm}^2$$

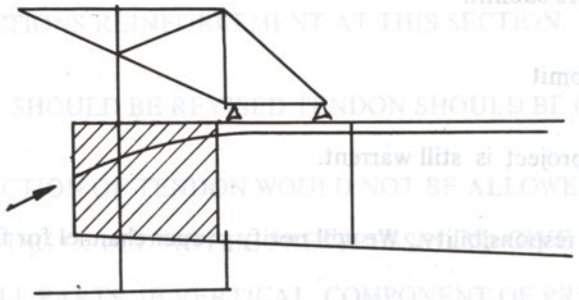
Therefore, sufficient safety is assured for the crushing and shearing of concrete.

B) Local stresses

The check of concrete stresses at the end face of segment where the tendons are anchored, can not be carried out due to unapplicability of normal theories of Strenght of Materials

The diffusion of prestressing force from the anchorages, which progressively affects the whole section, should be taken into account. Beyond a certain distance, assuming to be equal to the depth of the girder, it may be assumed the stress distribution is straight.

Concrete stresses are calculated at the front section of previous segment under the actions of prestressing tendons anchored at newly placed segment, the weight of this segemt and wagon.



4 — 3 — 2

The concrete stress at top face is shown in the following Table.

Concrete Stress at Top Fiber (Kg/cm^2)

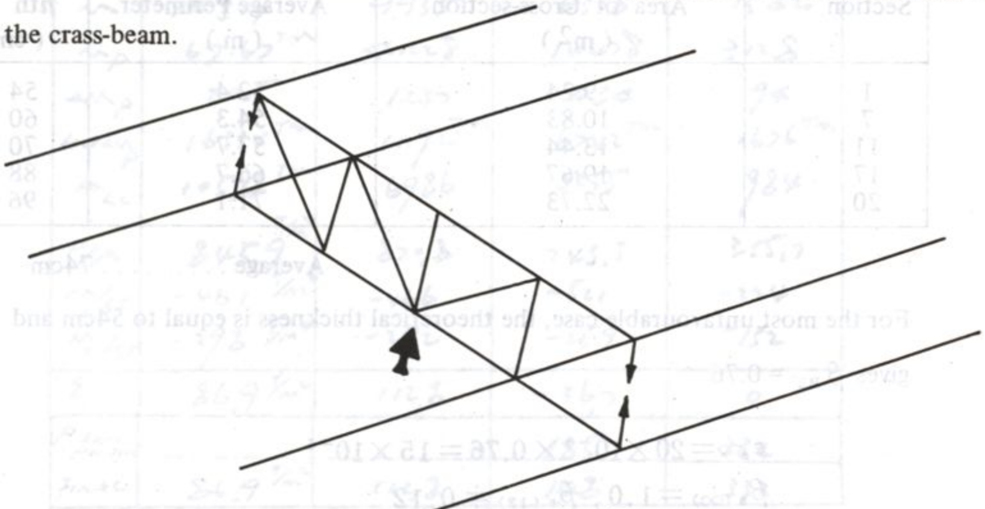
Section	Prestress	Stress due to weight	Total
2	+ 14.5	- 7.4	+ 7.1
3	+ 22.3	- 8.5	+ 13.8
4	+ 28.6	- 8.1	+ 20.5
5	+ 28.0	- 7.7	+ 20.3
6	+ 27.8	- 7.1	+ 20.7
7	+ 27.4	- 6.6	+ 20.8
8	+ 26.5	- 6.0	+ 20.5
9	+ 25.5	- 5.5	+ 20.0
10	+ 28.0	- 5.0	+ 23.0

C) Minimum Cover for Top Reinforcement

1.5″ is equivalent to 37mm in the metric system, but the difference between 37mm and 35mm (the present minimum cover) is not so serious as to be under the necessity of changing all the drawings. In general, protection of the reinforcement depends more the quality of the concrete, its compaction and impermeability, rather than on the distance between the outer face and the bar. On the other hand, thin concrete cover is desirable from crack-control point of view. Anyhow 2mm difference does not play an important role in necessity for durability in this case when 35mm of concrete cover is given to the top reinforcement.

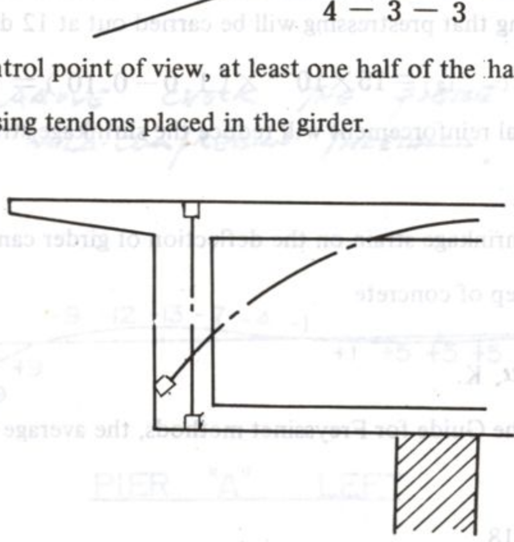
E) Indirect Support of Girder

The system of indirect support of girder shall be analysed using a truss model. Sufficient vertical reinforcement or tendons shoud be placed in the girder to resist the hanging-force from the crass-beam.



4 — 3 — 3

From the crack-control point of view, at least one half of the hanging-force should be resisted by vertical prestressing tendons placed in the girder.



4 — 3 — 4

F) Shrinkage of Concrete

Shrinkage of concrete will be expressed by after a given age to (Manuel de Calcul "Effects structuraux du fluage et des déformation différées" Juillet 1973 Comite Européen du Béton):

$$\varepsilon_s(t, t_o) = \varepsilon_{so\phi}\left[\beta_{s(t)} - B_{s(to)}\right)$$

$$\varepsilon_s\infty = \beta_{s1} - \beta_{s2}$$

$\beta_{s1} = 20 \times 10^{-5}$ (Outdoor structure)

β_{s2} Coefficient depending on theoretical thickness of the concrete element.

$$h_{th} = \text{Ttheoretical thickness} = \lambda \frac{2\,A_c}{\mu}$$

A_c = area of concrete cross-section

π = perimiter of concrete cross-section in contact with atmosphere

For box section, inner chambers do not contact directly with the fresh air, therefore, an average perimiter of cross-section using only the whole outer perimeter and that including the perimeter of inner, chambers, is calculated.

Theoretical thickness h_{th} where $\lambda = 1.5$

Section	Area of Cross-section (m^2)	Average Perimeter (m)	hth (cm)
1	9.24	52.4	54
7	10.83	54.3	60
11	13.44	57.7	70
17	19.67	66.7	88
20	22.73	71.1	96

Average 74cm

For the most unfavourable case, the theoretical thickness is equal to 54cm and this gives $\beta_{B2} = 0.76$

$$\varepsilon_{s\infty} = 20\times10^{-5}\times0.76 = 15\times10^{-5}$$

$$\beta_{s(\infty)} = 1.0,\ \beta_{s(12)} \doteqdot 0.12$$

assuming that prestressing will be carried out at 12 days after concreting

$$\varepsilon_{s(\ ,12)} = 15\times10^{-5}\times(1.0-0.10) = 13.5\times10^{-5}$$

The longitudinal reinforcement will reduce the shrinkage strain and 13.0×10^{-5} seems to be resonable value.

The effect of shrinkage strain on the deflection of girder can be neglected as compared with that due to creep of concrete

H) Coefficients μ. K.

According to the Guide for Freyssinet methods, the average values of μ and K are given as follows:

$\mu = 0.18$

$k = \mu\cdot\beta = 0.18 \times 9/1000 = 0.0016$

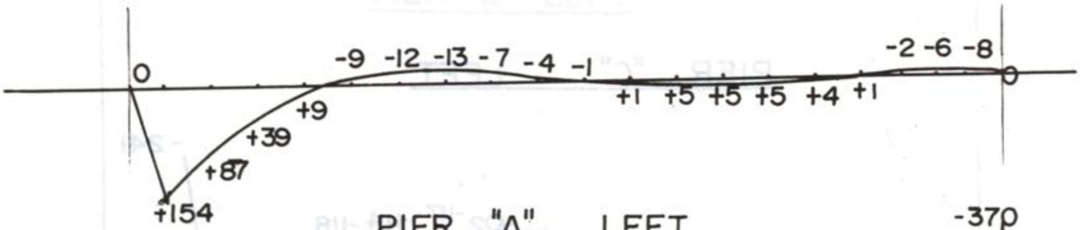
According to AASHO, μ and K are given as 0.30 and 0.0020, respectively therefore the calculated results will give safe values as compared with the real ones, provided that special attension will be paid on the regidily of the ducts, on the distance and on the fixing of their supports, on the care taken in placing the tendons and on the precautions taken during concreting.

It should be possible to justify the values used for μ and K to verify them accurately on site and every provision should be made on site for respecting the values actually obtained.

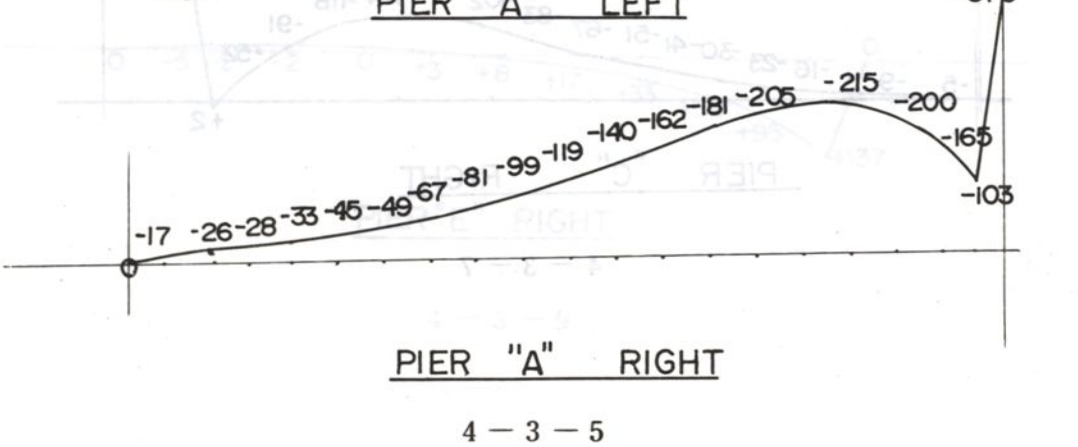
ASSUME FURTHER 3% LOSS DUE TO SHRINKAGE

SECTION	1	5	10	15
A	22.754 m²	18.49	13.47	10,41
S_T	35.43 m³	23.98	12.95	7.47
$P_{orig.}$	19845 T	16600	10197.	3818
P_{NEW}	19249 T	16102	9891	3703
M_D	81,486 TM	51183	20786	4710
M	14319 TM	9955	6308	1582
M_P	67167 TM	41228	14478	3128
ΔM_P	2015	1237	434	94
M_{temp}	16334 TM	11192 TM	6742 TM	1676 TM
$M_{L.L}$	10578 TM	6986	3255	984
P/A	845.9 T/m²	870,8	743.3	355,7
M/S_T	-461 T/m²	-466	-521	-224
$M_{L.L}/S_T$	-298 T/m²	-292	-259	-132
Σ	86,9 T/m²	112,8	-367	0
P. ADDN. POSITIVE			185	332
FINAL	86.9 T/m²	112,8	148	332

FROM ABOVE CHECK THE FIBRE WILL BE WITHIN MILD-COMPRESSION THRESHOLD.

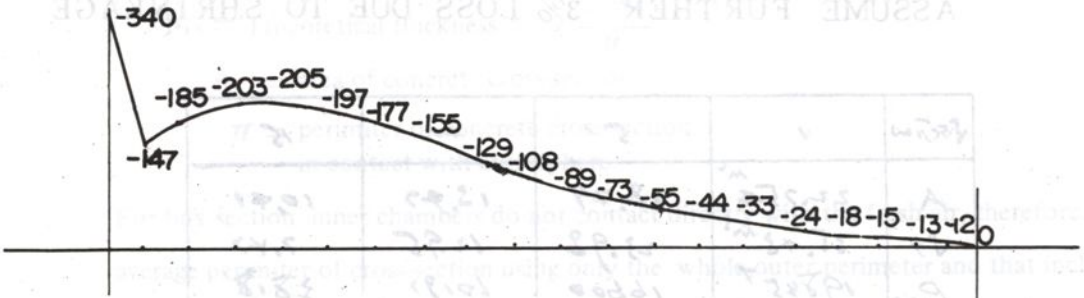


PIER "A" LEFT

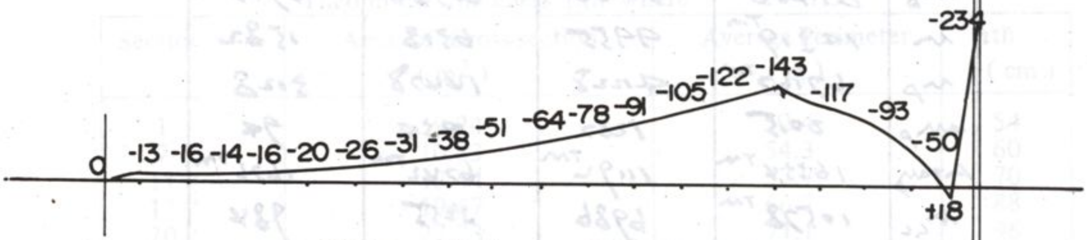


PIER "A" RIGHT

4 — 3 — 5

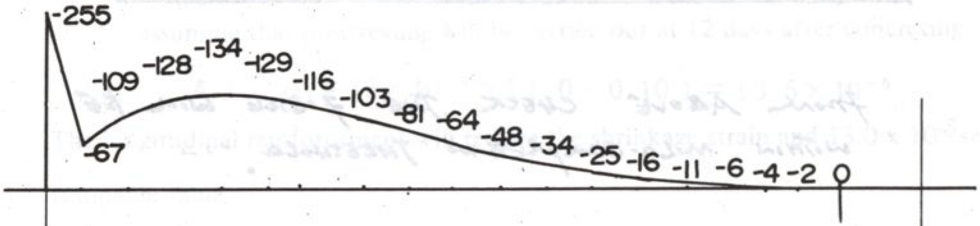


PIER "B" LEFT

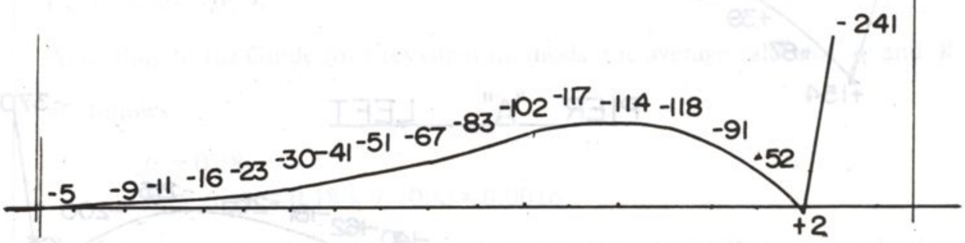


PIER "B" RIGHT

4 — 3 — 6

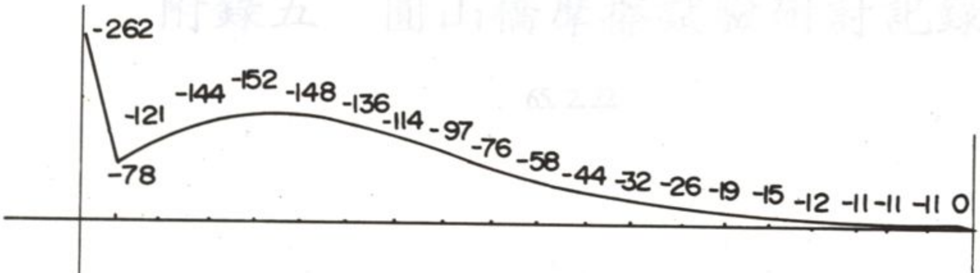


PIER "C" LEFT

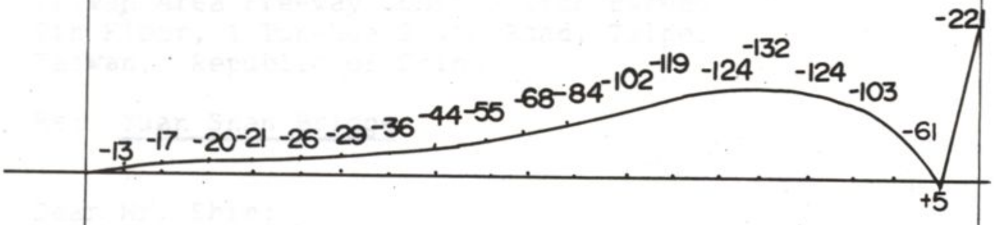


PIER "C" RIGHT

4 — 3 — 7

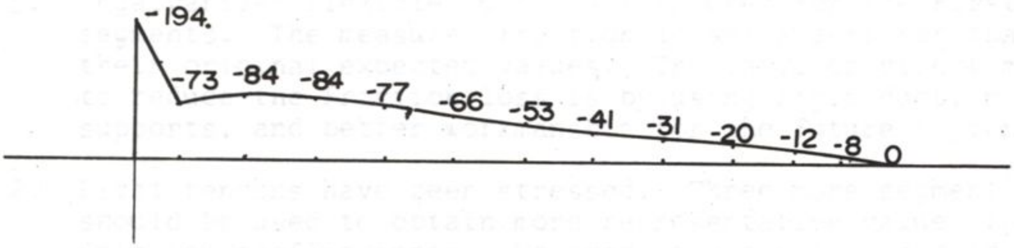


PIER "D" LEFT

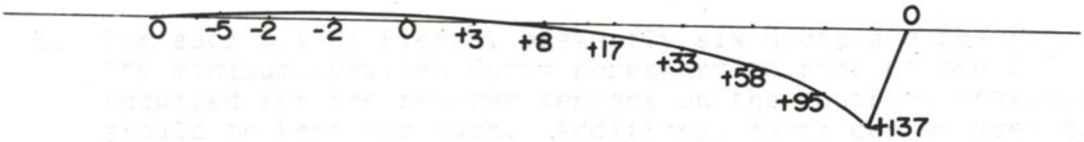


PIER D RIGHT

4 — 3 — 8



PIER "E" LEFT



PIER"E" RIGHT

4 — 3 — 9

附錄五　圓山橋摩擦試驗研討記錄

65. 2. 22.

Mr. C. K. Shih
Northern District Head Office
Taiwan Area Freeway Construction Bureau
9th Floor, 1 Tun-Hua South Road, Taipei
Taiwan, Republic of China

Re: Yuan Shan Bridge

Dear Mr. Shih:

Upon your request we attended a meeting with your engineers, and the general contractor and his prestress sub-contractor For the subject project regarding the friction losses of their prestress system. The following are our comments after the discussions, meetings and field inspections:

1. Ungalvanized flexible duct has been used for the first few segments. The measured friction losses are higher than their original expected values. The basic or direct method to reduce the friction loss is by using rigid duct, more supports, and better workmanship for the future segments.
2. Eight tendons have been stressed. Three more segment pairs should be used to obtain more representative values for friction coeffiecients. However, these values should be reviewed as construction proceeds and revised as necessary.
3. Net section properties should be used to calculate stresses only as unforseen problems are encountered later.
4. Stressing equipment should be calibrated, so that accurate stressing and friction data can be obtained.
5. The maximum stress in the prestress steel should never be greater than 0.8 f_{su} at the anchorage.
6. For each box at Pier C, presently six ducts are reserved. The minimum required ducts necessary to provide the 290 tons required for the reserve tendons on the contract drawings should be kept for such. Additional ducts can be used to compensate for unexpected friction loss if necessary, or

can be terminated after reliable friction coefficients are obtained and verified.

7. The ducts should be clean from all foreign substances both in side and out side.

8. During field inspection heavy rust was found on the rolls of prestressing strand. Additional protective measures should be carried out.

9. The contractor will review and revise his stressing calculations, and will submit for our review in due course.

10. The construction work was found satisfactory in general.

Should you have any further questions or comments, please do not hesitate to write us again.

Sincerely yours,

T. Y. LIN INTERNATIONAL

Kam Lo

Kam Lo
Vice President

KL/ys

cc: T. Y. Lin International
Mr. James Jurkovich

T. Y. Lin, Taiwan, Inc.

Minutes of a Meeting held on Sunday 22 Feb. 1976

1. Mr. Shie outlined the aim of the meeting and asked for attention to be paid in the solution of friction problem.

2. Dr. Inomata suggested an approach to this problem as follows;

   (1) Eight cables were tensioned on the site and the estimated average value was:

   μ =0.30 on the assumption that the ratio k/μ is equal to 0.016. Although the number of data is not sufficient to determine any statistical value, this estimated value seems to be greater than the assumed one.

   (2) The contractor should try to adopt every possible means in order to reduce the friction on the site; increasing the rigidity of sheaths, decreasing the fixing distance of their supports, the special precautions taken during concreting.

   (3) In order to determine the probable values of the coefficient of friction, more data should be gathered. For the time being it is better to continue the construction work more 3 segments in both sides, on the assumption that the coefficients of friction are the same as these estimated on 8 cables which were already tensioned. The recalculated tensioning log will be submitted to the C.B. by the end of Feb.

   (4) After getting the sufficient number of data, 32 cables, a statistical analysis will be carried out and the further tentioning log based on the new values, shall be prepared by the contractor in order to get the approval of the C.B.

   (5) If prestress is calculated on the net concrete section, instead of gross section, the calculated prestress is increased by about 6 - 7%.

   The concrete stresses due to loading after grouting will be decreased about 4 - 5%, when these stresses were calculated on the basis of the transformed section.

   Therefore the actual prestress is 6 - 7% greater than the design value, on the basis of the same prestressing force. If the prestressing force is 6 - 7% less than the design value, the actual prestress is the same as the design value. This fact seems to be an approach to the problem of friction loss. Few percent loss of prestressing force does not affect the necessary prestress.

(6) When the applied prestressing force at the critical section becomes less than the design value, some additional cables shall be placed into the reserved sheathes which were provided for safety. The additional cables shall be anchored at the place of an intermediate cross-beam.

3. The T.Y.Lin International said that that they have many experiences of smaller friction loss, with an appropriate precaution on the site and they have no objections for using the net concrete section and the transformed section in design. The design with the gross section gives always the safe - side results. They pointed out that this problem should have been discussed one year before.

4. Mr. Shie said that all the prestressed concrete structures have been designed on the basis of the gross section and he wanted to hold this principle in order to keep the consistency with the other projects.

5. Mr. Shie concluded as follows:

(1) Primary point is to make all efforts in order to reduce the actual coefficients of friction; increasing the rigidity of sheaths; decreasing the fixing spacing of their supports; special precaution taken during concrete. Tensioning of the reserved cables will be carried out, if necessary.

(2) If the required prestressing force could not be obtained even under the previous conditions, the prestress can be checked on the basis of net section.

Shunji Inomata

Dr. Eng. Shunji INOMATA

Correction Factors for Concrete Stresses Calculated on the Basis of Gross Section

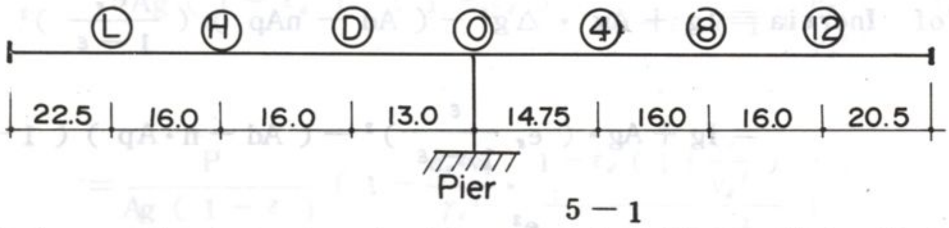
Section	Prestress; (Net Section)	Concrete Stresses due to Loading after Grouting; (Transformed Section)	
		Top	Bottom
L	1.032	0.966	0.996
H	1.052	0.943	0.991
D	1.068	0.921	0.981
0	1.072	0.917	0.980
4	1.066	0.924	0.984
8	1.052	0.943	0.992
12	1.033	0.964	0.996

Note; Diameter of duct = 6.5cm

Ratio of Modulus of Elasticity = 6.15

Cross-sectional area of one tendon = 11.85cm²

Position of section



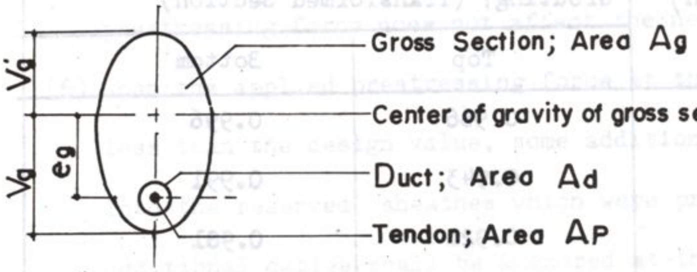
5 — 1

The actual concrete stress in net or transformed section could be obtained by multiplying the concrete stresses calculated on the basis of gross section by the above correction factors.

At critical section o , actual prestress is 7.2% greater than the design value, if the prestressing force is the same as the design. That means; 92.8% of design prestressing force is enough to give the design prestress. Concrete tensile stress due to live loads, at the top fiber is 8% less than that calculated on the gross section

Correction Factors

Concrete stresses in the net concrete section or the transformed section can be obtained by multiplying the concrete stresses calculated on the basis of gross-section by the correction factors.



5 − 2

Area ; $A - Ag - Ad + n \cdot Ap = Ag\ (\ 1 - \varepsilon\ )$

where $\varepsilon = (\ Ad - n \cdot Ap\ )\ /\ Ag$

Displacement of C.G.

$$\Delta g = e_g \cdot \frac{\varepsilon}{1 - \varepsilon}$$

Eccentricity of C.S $\equiv e_g + \Delta g = e_g \dfrac{1}{1 - \varepsilon}$

Moment of Inertia $\equiv Ig + Ag \cdot \Delta g^2 - (\ Ad - nAp\ )\ (\ \dfrac{e_g}{1 - \varepsilon}\ )^2$

$$= Ig + Ag \cdot (\ e_g \frac{\varepsilon}{1 - \varepsilon})^2 - (\ Ad - n \cdot Ap\ )\ (\ 1 - \frac{e_g}{1 - \varepsilon}\ )^2$$

$$= Ig + Ag \frac{e_g^2}{(\ 1 - \varepsilon\ )}\ (\varepsilon^2 - \varepsilon)$$

$$= Ig - Ag \cdot e_g^2\ \frac{\varepsilon}{1 - \varepsilon}$$

$$= \frac{1 - \varepsilon\ (\ 1 + \frac{e_g^2}{\gamma_g^2}\ )}{1 - \varepsilon} . Ig$$

$$\text{where } \gamma_g^2 = \frac{Ig}{Ag}$$

Distance between top fiber and C.G. of transformed section

$$\equiv V' - \Delta g = V' . \frac{1 - \varepsilon\ (\ 1 + \frac{e_g}{V_g^{\prime}}\ )}{1 - \varepsilon}$$

Distance between bottom fiber and C.G. of transformed section

$$\equiv \mathrm{V} + \triangle \mathrm{g} = \mathrm{V}\,\frac{1-\varepsilon\left(1-\frac{e_g}{v_g}\right)}{1-\varepsilon}$$

Section modulus

$$\text{Top fiber} \equiv \mathrm{Wg.t}\cdot\frac{1-\varepsilon\left(1+\frac{e_g{}^2}{\gamma_g{}^2}\right)}{1-\varepsilon\left(1+\frac{e_g}{v_g{}'}\right)}$$

$$\cong \mathrm{Wg}\cdot \mathrm{t}\left[1-\varepsilon\left(\frac{e_g{}^2}{\gamma_g{}^2}-\frac{e_g}{v_g{}'}\right)\right]$$

$$\text{Bottom fiber} \equiv \mathrm{Wg}\cdot \mathrm{b}\,\frac{1-\varepsilon\left(1+\frac{e_g{}^2}{\gamma_g{}^2}\right)}{1-\varepsilon\left(1-\frac{e_g}{v_g}\right)}$$

$$\cong \mathrm{Wg}\cdot \mathrm{b}\left[1-\varepsilon\left(\frac{e_g{}^2}{\gamma_g{}^2}+\frac{e_g}{v}\right)\right]$$

Prestress ; Ap = O , For Net-Section $\varepsilon_d = \frac{Ad}{Ag}$

$$\text{Top fiber} \equiv \frac{P}{Ag\left(1-\varepsilon_d\right)}-\frac{P\cdot e_g}{\left(1-\varepsilon_d\right)}\cdot\frac{1-\varepsilon_d\cdot\left(1+\frac{e_g}{v_g{}'}\right)}{1-\varepsilon_d\cdot\left(1+\frac{e_g{}^2}{\gamma_g{}^2}\right)}\cdot\frac{V_g{}'}{I_g}$$

$$=\frac{P}{Ag\left(1-\varepsilon\right)}\left[1-\frac{e_g\cdot v_g'}{\gamma_g{}^2}\cdot\frac{1-\varepsilon_d\left(1+\frac{e_g}{v_g{}'}\right)}{1-\varepsilon_d\left(1+\frac{e_g{}^2}{\gamma_g{}^2}\right)}\right]$$

$$=\frac{P}{Ag\left(1-\varepsilon\right)}\cdot\left[\left(1-\frac{e_g\cdot v_g'}{\gamma_g{}^2}\right)-\frac{e_g\cdot v_g'}{\gamma_g{}^2}\cdot\frac{\varepsilon_d\left(\frac{e_g{}^2}{\gamma_g{}^2}-\frac{e_g}{v_g'}\right)}{1-\varepsilon_d\left(1+\frac{e_g{}^2}{\gamma_g{}^2}\right)}\right]$$

$$=\frac{P}{Ag\left(1-\varepsilon\right)}\cdot\left(1-\frac{e_g\cdot v_g'}{\gamma_g{}^2}\right)\cdot\left[1+\frac{\varepsilon_d\cdot\frac{e_g{}^2}{\gamma_g{}^2}}{1-\varepsilon_d\left(1+\frac{e_g{}^2}{\gamma_g{}^2}\right)}\right]$$

$$=\frac{P}{Ag\left(1-\varepsilon\right)}\cdot\left(1-\frac{e_g\cdot v_g'}{\gamma_g{}^2}\right)\cdot\frac{1-\varepsilon_d}{1-\varepsilon_d\left(1+\frac{e_g{}^2}{\gamma_g{}^2}\right)}$$

$$\cong \frac{P}{Ag} \left(1 - \frac{e_g \cdot v'_g}{\gamma_g{}^2} \right) \left[1 + \varepsilon_d \left(1 + \frac{e_g{}^2}{\gamma_g{}^2} \right) \right]$$

$$\text{Bottom fiber} \equiv \frac{P}{Ag\,(1 - \varepsilon_d)} + \frac{P \cdot e_g}{(1 - \varepsilon_d)} \cdot \frac{1 - \varepsilon_d \left(1 - \frac{e_g}{v_g} \right)}{1 - \varepsilon_d \left(1 + \frac{e_g{}^2}{\gamma_g{}^2} \right)} \cdot \frac{v_g}{I_g}$$

$$\cong \frac{P}{Ag} \left(1 + \frac{e_g \cdot v_g}{\gamma_g{}^2} \right) \left[1 + \varepsilon_d \left(1 + \frac{e_g{}^2}{\gamma_g{}^2} \right) \right]$$

Conection Factors For Concrete Stresses Calculated On The Basis Of Gross Section

(1) For Prestress & Loading Before Grouting

Top & Bottan ················ $1 + \varepsilon_d \left(1 + \frac{e_g{}^2}{\gamma_g{}^2} \right)$

where $\varepsilon_d = \frac{Ad}{Ag}$

(2) For Loading After Grouting

Top ···························· $1 + \varepsilon \left(\frac{e_g{}^2}{\gamma_g{}^2} - \frac{e_g}{v'_g} \right)$

Bottom ························ $1 + \varepsilon \left(\frac{e_g{}^2}{\gamma_g{}^2} + \frac{e_g}{v_g} \right)$

where $\varepsilon = \frac{Ad - n \cdot Ap}{Ag}$

March 16, 1976

Mr. C. K. Shih
North District Head Office
Taiwan Area Freeway Construction Bureau
9th Floor, 1 Tun Hua South Road
Taipei, Taiwan
Republic of China

Subject: Project 12
Yuan Shan Bridge

Dear Mr. Shih:

Reference is made to letter No. 064 from Continental Engineering Corporation dated March 4, 1976, regarding friction losses of tendons and calculation of prestressing Force based on the net section submitted by FKK for subject project. Our comments are as follows:

1. The correction factors listed in the letter mentioned above seem to apply either when all the cable ducts are taken out or when all the cables are stressed and grouted. However, in the actual construction sequence, the cables are grouted as soon as they are stressed for each segment. In other words, as the construction work progresses, the sections change gradually from net sections to transformed sections as shown in the attached sheet.

2. The average correction factor is near a unit, not 1.072, as claimed by FKK.

3. If a net section and/or transformed section is used for the design, additional Intermediate steps should be considered according to the construction sequence, which means that the actual tendon holes should be considered in accordance with each stage of prestress.

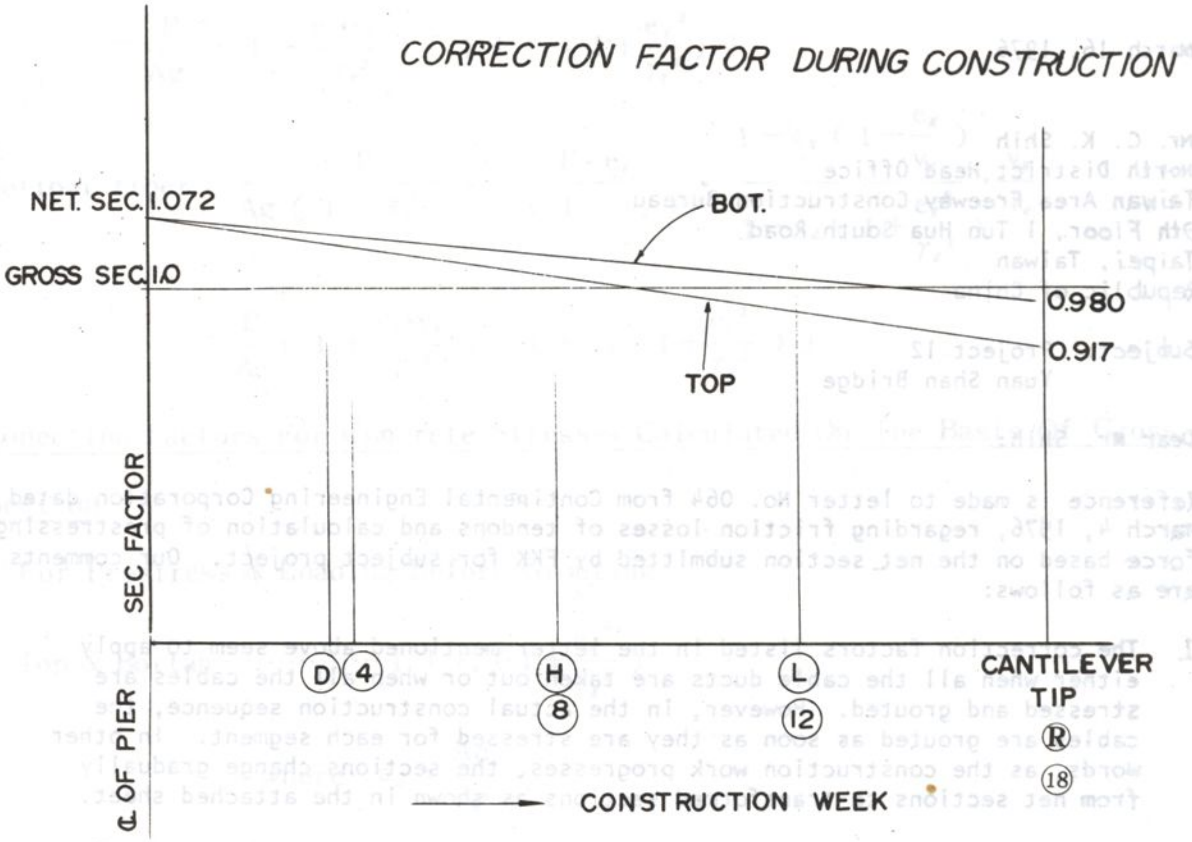
Should you have any questions or comments, please do not hesitate to write us again.

Sincerely yours,

T. Y. LIN INTERNATIONAL

Kam Lo
Vice President

Encl.



5 — 3

THE AVERAGE CORRECTION FACTOR DURING CONSTRUCTION

ARE AVE. (TOP) $= \frac{1.072 + 0.917}{2} = 0.995$

AVE. (BOT.) $= \frac{1.072 + 0.980}{2} = 1.026$

*P/A MAY NOT CONSIDERED IN THIS CORRECTION FACTOR

附錄六　骨礫及混凝土配合設計試驗

一、序　　言：

本材料試驗室應大陸公司 63 工發字第 170 及 198 號函之要求，代辦「高速公路圓山橋之混凝土配合設計」。本試驗之目的有三：

1. 骨材之物理性質。
2. 混凝土之初步配合設計。
3. 各種摻料（ admixtures ）對混凝土強度及乾縮量之影響。

試驗所用之骨礫、水泥及摻料均由大陸公司提供。混凝土配合設計由本室負責。一共試拌 6 盤，分於 10 月 23 日及 10 月 30 日拌合，每盤所加之摻料不同（請參照第三節），各盤均灌製 15 公分直徑 30 公分高之圓柱試體 9 個及 10 公分× 10 公分× 47 公分之乾縮測量試體 2 個。

大陸公司復於 11 月 25 日再要求追加試驗一種新摻料「Mighty」對混凝土之效果，有關此項試驗結果全部記錄於本報告第六節內。

二、材　　料：

1. 水泥：臺灣水泥公司出品之「品牌」第一種波特蘭水泥（ Portland Cement Type Ⅰ ）。
2. 骨礫：粗骨礫及細骨礫均取自台北縣土城鄉「泰中砂石廠」。

2－1　細骨礫篩分析結果：

Sieve	Fraction retained g	Fraction retained %	Cumulative retained %	Cumulative passing %	ASTM Designation C33 Specification requirements, % passing
3 / 8 in.	0	0	0	100	100
No 4	7	2	2	98	95 - 100
No 8	19	5	7	93	80 - 100
No 16	36	10	17	83	50 - 85
No 30	96	27	44	56	25 - 60
No 50	129	36	80	20	10 - 30
No 100	57	16	96	4	2 - 10
pan	16	4	-		
Total	360	100	246		
Fineness Modulus			$\frac{246}{100}=2.46$		

2 － 3　粗骨礫篩分析結果：

Sieve	Fraction retained kg	Fraction retained %	Cumulative retained %	Cumulative passing %	ASTM C33 requirements % passing
1 in.	0	0	0	100	95 - 100
3/4 in.	3.25	19	19	81	—
1/2 in.	9.35	54	73	27	25 - 60
3/8 in.	3.00	17	90	10	—
No. 4	1.30	8	98	2	0 - 10
No. 8	0.40	2	100	0	
No. 16	0	0	100		
No. 30	0	0	100		
No. 50	0	0	100		
No. 100	0	0	100		
Total	17.30	100	780		
Fineness Modulus			$\frac{780}{100}=7.80$		

2 － 3　含泥量試驗結果：

試　樣	含　泥　量（％）
細　骨　礫	1.4

2 － 4　比重及吸水率試驗結果：

試　樣	面乾飽和鬆比重吸	吸水率　％
細骨礫	2.60	1.80
組骨礫	2.61	1.39

2 － 5　粗骨礫單位體積重：

粗骨礫單位體積重＝ 1,600 kg / m^3

3.摻料：

品　　名	製　造　廠　商	台　灣　代　理　商
Tricosal BV Special	Chemische Fabrik Gruenau Gmbh W. Germany	海灣股份 有限公司
Sika Plastiment	Sika Chmical Corporation U.S.A.	大新建設 股份有限公司
Zeecon	Crown Zellerbach U. S. A.	裕昌貿易 股份有限公司
WRDA	W. R. Grace & Co. U. S. A.	天成貿易 股份有限公司

三、配合比例：

編號	摻料用量	每立方公尺混凝土所需材料				水灰比 W/C	砂率 S/A	實測場度 (公分)	測定之單位體積重量 kg / m^3
		水	水泥	砂 (S.S.D)	石子 (S.S.D)				
P	無	181 (183)	470 (476)	723 (732)	1,040 (1,053)	0.385	41 %	12	2,385
T	Tricosal B.V Special，水泥重量之0.2％	157 (159)	470 (475)	723 (731)	1,040 (1,051)	0.334	41 %	12	2,365
S	Sika Plastiment 50 kg 水泥用100 c.c.	165 (165)	470 (471)	723 (724)	1,040 (1,041)	0.351	41 %	16	2,395
C	無	211 (217)	470 (483)	723 (742)	1,040 (1,068)	0.449	41 %	14	2,380
Z	Zeecon,水泥重量之0.2％	199 (202)	470 (478)	723 (736)	1,040 (1,058)	0.423	41 %	12	2,390
W	WRDA，50 kg 水泥用100 c.c.	202 (206)	470 (480)	723 (738)	1,040 (1,062)	0.430	41 %	12	2,385

註：(　)括弧內數據爲根據實際測定之混凝土單位重經換算後之修正值。

四、試驗方法

1. 骨礫之篩析試驗：依照 ASTM C136-67 " Sieve or Screen Analysis of Fine and Coarse Aggregates "之規定方法試驗。

2. 粗骨礫之比重及吸水率試驗：依照 ASTM C127-59 " Specific Gravity and Absorption of Coarse Aggregate "之規定方法試驗。

3. 細骨礫之比重及吸水率試驗：依照 ASTM C128-59 " Specific Gravity and Absorption of Fine Aggregate "之規定方法試驗。

4. 骨礫含泥量試驗：依照 ASTM C117-67 " Materials Finer Than No. 200 Sieve in Mineral Aggregates by Washing "之規定方法試驗。

5. 骨礫之單位體積重試驗：依照 ASTM C29-67 T " Unit Weight of Aggregate "之規定方法試驗。

6. 骨礫之表面水量測定：依照 ASTM C566-67 " Total Moisture Content of Aggregate by Drying "之規定方法試驗。

7. 混凝土之塌度試驗：依照 ASTM C143-66 " Slump of Portland Cement Concrete "之規定方法試驗。

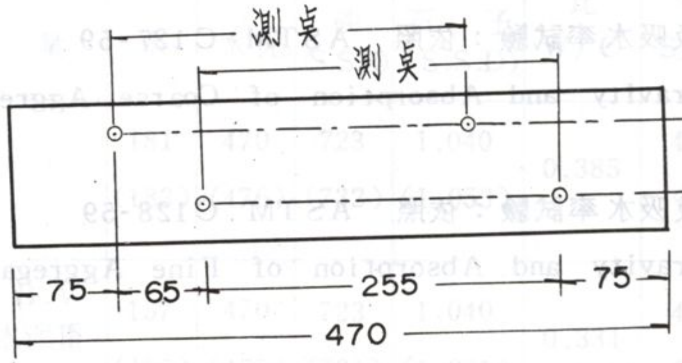
8. 混凝土之單位體積重試驗：依照 ASTM C138-63 " Weight per Cubic Foot, Yield, and Air Content of Concrete "之規定方法試驗。

9. 混凝土抗壓強度試驗：依照 ASTM C39-66 " Compressive Strength of Molded Concrete Cylinders "之規定方法試驗。

10. 混凝土試體之製作：依照 ASTM C192-66 " Concrete Compression and Flexure Test Specimens, Making and Curing in Laboratory "之規定方法製作。

11. 乾縮量之測定試驗：依照 ASTM C341-67 T " Length Change of Drilled or Sawed Specimens of Cement Mortar and Concrete "之規定方法測定。

每一種混凝土澆灌 10 ㎝ × 10 ㎝ × 47 ㎝ 乾縮試體二個（如下圖），每一試體也分二層澆灌，並以電動振動器搗實，然後置於噴霧室中養護 24 小時始拆模，拆模後再置於噴霧室養護二天，然後移出室內空氣中養護，並定期記錄其乾縮變化。



乾縮試体

圖 6 － 1

五、試驗結果：

1.抗壓強度：

(1)無摻料混凝土：拌合日期： 10 月 23 日

塌度＝ 12 ㎝

試體編號	齡期（天）	抗壓強度	
		kg / cm²	psi
P 1	2½	155	2,200
P 2	〃	144	2,050
P 3	〃	153	2,180
平均	〃	151	2,140
P 4	7	250	3,560
P 5	7	230	3,270
P 6	7	252	3,590
平均	7	244	3,470
P 7	28	348	4,950
P 8	28	388	5,520
P 9	28	377	5,350
平均	28	371	5,270

註：英文編號爲拌合編號，請參照第三節。阿拉伯數字爲試體編號。

(2)摻入 Tricosal B. V Special 者，

拌合日期：10 月 23 日；塌度＝12 cm

試體編號	齡期（天）	抗壓強度	
		kg / cm^2	psi
T1	2½	245	3,480
T2	〃	243	3,450
T3	〃	230	3,270
平均	〃	239	3,400
T4	7	339	4,830
T5	〃	337	4,790
T6	〃	343	4,880
平均	〃	340	4,830
T7	28	497	7,707
T8	〃	464	6,600
T9	〃	434	6,170
平均	〃	465	6,610

⑶摻入　Sika Plastiment 者；

拌合日期：10月23日

塌　度＝16 cm

試體編號	齡期（天）	抗壓強度	
		kg / cm²	psi
S1	2½	197	2,810
S2	〃	201	2,860
S3	〃	201	2,860
平均	〃	200	2,840
S4	7	312	4,440
S5	〃	331	4,700
S6	〃	323	4,600
平均	〃	322	4,580
S7	28	438	6,230
S8	〃	457	6,500
S9	〃	466	6,630
平均	〃	454	6,450

⑷純摻料混凝土；

拌合日期： 10 月 30 日

塌　度＝14 *cm*

試　體 編　號	齡　期 （ 天 ）	抗　壓	強　度
		kg / cm²	psi
C1	2½	159	2,250
C2	〃	159	2,250
C3	〃	156	2,220
平　均	〃	158	2,240
C4	7	262	3,730
C5	〃	250	3,500
C6	〃	250	3,560
平　均	〃	254	3,600
C7	28	383	5,440
C8	〃	368	5,230
C9	〃	348	4,950
平　均	〃	366	5,210

(5)摻入 Zeecon 者；

拌合日期： 10月 30 日

塌　度＝ 12 cm

試　體	齡　期	抗　壓　強　度	
編　號	（　天　）	kg / cm²	psi
Z 1	2½	182	2,590
Z 2	〃	186	2,680
Z 3	〃	183	2,600
平　均	〃	184	2,620
Z 4	7	295	4,200
Z 5	〃	295	4,200
Z 6	〃	286	4,060
平　均	〃	292	4,150
Z 7	28	355	5,040
Z 8	〃	380	5,400
Z 9	〃	392	5,570
平　均	〃	376	5,340

⑹摻入WRDA　者；

拌合日期：10月30日

塌　　度＝12 ㎝

試　體	齡　期	抗　壓　強　度	
編　號	（　天　）	kg／cm²	psi
W1	2½	188	2,670
W2	〃	194	2,760
W3	〃	190	2,700
平　均	〃	191	2,710
W4	7	264	3,750
W5	〃	275	3,910
W6	〃	272	3,870
平　均	〃	270	3,840
W7	28	332	4,720
W8	〃	383	5,450
W9	〃	382	5,430
平　均	〃	366	5,200

2.乾縮變化：

乾縮量 (X10-6) 齡期(天) ＼ 試體編號		P	T	S	C	Z	W
噴霧室內	1	0	0	0	0	0	0
	3	8	72	30	33	6	20
試驗室內	5	101	177	161	63	105	101
	7	158	219	215	122	182	159
	14	284	347	390	313	343	324
	21	402	458	460	389	432	414
	28	467	553	526	479	527	522
	42	604	684	840	—	—	—
	43	—	—	—	507	561	674
	57	600	732	864	—	—	—
	62	—	—	—	491	520	539
	69	534	692	754	—	—	—
	70	—	—	—	548	560	605
	84	—	—	—	622	612	636
	85	579	750	884	—	—	—

註：＊乾縮變化量之每一數據，均為該種混凝土兩個試體上所有測定值之平均值（每一試體表面有4個測定值）

六、追加試驗項目：「Mighty」摻料之試驗結果

1.說明：「大陸公司」於 11 月 25 日要求追加新摻料「Mighty」對混凝土之效果，並委派代銷商「台北貿易有限公司」送來樣品。本室遂依照前述試驗方法，使用相同材料，於 12 月 3 日拌合摻入「Mighty」之二種配合比例之混凝土，以求該項摻料對混凝土強度及乾縮之關係。

2.材料：(1)水泥、骨礫之物性，請參照第二節第 12 項。

(2)摻料：

品名	製造廠商	台灣代理商
Mighty	Kao Soapco. Japan	台北貿易有限公司

3.配合比例：

編號	摻料用量	每立方公尺混凝土所需材料				水灰比 W/C	砂率（%） S/C	實測塌度（公分）	測定之單位體積重量 kg/m³
		水	水泥	砂 (S.S.D)	石子 (S.S.D)				
M_A	Mighty 水泥重量之 1%	147 (134)	470 (430)	621 (568)	1,010 (924)	0.312	38	20	2,457
M_B	Mighty 水泥重量之 1%	162 (146)	520 (468)	523 (471)	1,010 (909)	0.311	34	22	2,460

註：（　）括弧內數據爲根據實際測定之混凝土單位重，換算後之修正值。

b.每立方公尺混凝土之水泥用量468公斤者；

拌合日期： 12 月 3 日

塌　度＝22 ㎝

試體	齡期	抗壓強度	
編號	（天）	kg／cm²	psi
M_B-1	2½	352	5,000
-2	〃	380	5,420
-3	〃	385	5,480
平均	〃	372	5,300
M_B-4	7	399	5,680
M_B-5	〃	465	6,610
-6	〃	467	6,640
平均	〃	444	6,310
M_B-7	28	493	7,000
-8	〃	560	7,970
-9	〃	492	6,990
平均	〃	515	7,320

⑶乾縮變化：

乾縮量（X10-6） 試體編號 齡期（天）		M_A	M_B
噴霧室內	1	0	0
	3	72	152
試驗室內	7	218	323
	14	240	355
	21	310	410
	28	280	392
	42	470	503
	57	552	535
	77	600	608
	84	705	710
	97	625	630

註：＊乾縮變化量之每一數據，均爲該種混凝土兩個試體上所有測定值之平均值（每一試體表面有 4 個測定值）。

七、試驗結果之比較：

1. 抗壓強度一覽表（表內每一數據均爲三個試體之平均試驗結果，編號請參考第5頁之第三節及第15頁之第六節第3項。

齡期 項目 編號	抗壓強度（kg/cm²）			水泥用量（kg）*	水灰比 W/C	塌度 Slump（cm）
	60小時	7天	28天			
P	151	244	371	183	0.385	12
C	158	254	366	217	0.449	14
T	239	340	465	159	0.334	12
S	200	322	454	165	0.351	16
Z	184	292	376	202	0.423	12
W	191	270	366	206	0.430	12
M_A	312	414	481	134	0.312	20
M_B	372	444	518	146	0.311	22

註：* 每立方公尺混凝土之水泥用量。

2.乾縮變化一覽表（表內之每一數據，屬編號P，C，T，S，Z，W者爲8個測點結果之平均值，屬M_A，M_B者爲4個測點之平均值。編號請參考第5頁之第三節及第 15 頁之第六節第3項。）

養護	齡期 \ 編號 \ 項目	乾縮變化量（10－6）							
		P	C	T	S	Z	W	M_A	M_B
噴霧室內	1 天	0	0	0	0	0	0	0	0
	3 天	8	33	72	30	6	20	72	152
試驗室內（氣乾）	5 天	101	63	177	161	105	101	—	—
	7 天	158	122	219	215	182	159	218	323
	14 天	284	313	347	390	345	—	240	355
	21 天	402	389	458	460	432	414	310	410
	1 月	467	479	553	526	527	522	280	392
	1½ 月	604	507	684	840	561	674	470	503
	2 月	600	491	732	864	520	539	552	535
	2½ 月	534	548	692	754	560	605	600	608
	3 月	579	622	750	884	612	636	705	710
	3½ 月	—	—	—	—	—	—	625	630

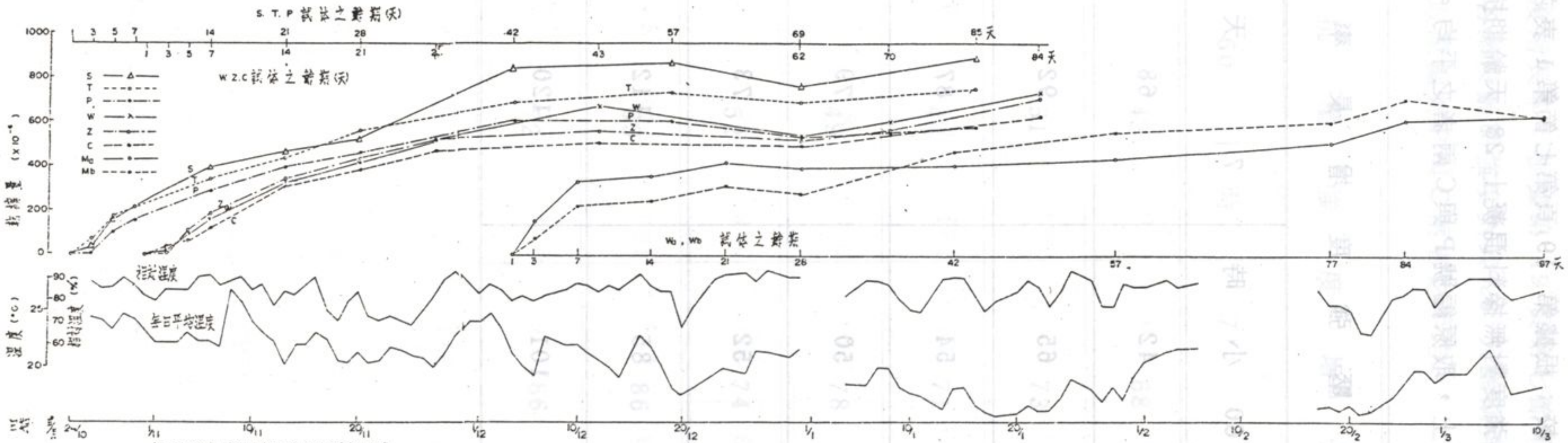
註：1月以 28 天計

3.各齡期之抗壓強度增長率：根據第 19 頁第七節第1表之資料，推算各種配合比例混凝土之各階段齡期強度對其 28 天齡期強度之百分率。編號N者爲無摻料混凝土，取原編號P與C兩者之平均。

齡期 項目 編號	抗壓強度增長率（%）		
	60 小時	7 天	28 天
N	42	68	100
T	51	73	100
S	44	71	100
Z	49	78	100
W	52	74	100
M_A	65	86	100
M_B	72	86	100

4. 各齡期之抗壓強度比較：根據第 19 頁第七節第1表之資料，推算各種配合比例混凝土之各齡期強度對無摻料混凝土 28 天齡期抗壓強度之百分率。編號N者為無摻料混凝土，取原編號P與C兩者之平均。

齡期 項目 編號	抗壓強度增長率（%）		
	60 小時	7 天	28 天
N	42	68	100
T	65	92	126
S	54	87	123
Z	50	79	102
W	52	73	99
M_A	85	112	130
M_B	101	120	140



每日平均溫度相對溼度變化與混凝土乾縮變化曲線

圖 6 — 2

附錄一　混凝土養護期間每日平均氣溫及相對濕度紀錄

1.本試驗室內

月	星	日	平均溫度	相對濕度	月	星	日	平均溫度	相對濕度
份	期	期	°C	%	份	期	期	°C	%
10	六	26	24.4	88	11	六	16	23.0	90
	日	27	24.2	85		日	17	22.4	75
	一	28	23.2	86		一	18	20.5	70
	二	29	24.7	90		二	19	20.3	79
	三	30	24.3	87		三	20	21.2	84
	四	31	23.0	81		四	21	20.3	72
11	五	1	22.0	79		五	22	20.6	70
	六	2	22.0	84		六	23	21.7	72
	日	3	22.0	84		日	24	21.4	70
	一	4	25.0	84		一	25	21.0	68
	二	5	24.2	90		二	26	20.8	74
	三	6	24.2	91		三	27	20.0	81
	四	7	23.8	86		四	28	21.2	88
	五	8	26.9	88		五	29	22.9	92
	六	9	25.6	90		六	30	24.0	87
	日	10	24.0	84	12	日	1	24.0	82
	一	11	23.0	87		一	2	24.7	86
	二	12	22.2	77		二	3	23.2	84
	三	13	20.0	84		三	4	21.1	79
	四	14	20.9	82		四	5	20.7	81
	五	15	21.0	86		五	6	19.2	79

月	星	日	平均溫度	相對濕度	月	星	日	平均溫度	相對濕度
份	期	期	°C	%	份	期	期	°C	%
12	四	5	20.0	81	12	日	29		
	五	6	19.2	79		一	30	20.8	90
	六	7	22.8	81		二	31	21.6	90
	日	8	—	—	1	三	1	—	—
	一	9	21.8	84		四	2	—	—
	二	10	21.9	87		五	3	—	—
	三	11	21.1	86		六	4	18.4	81
	四	12	20.5	83		日	5	—	—
	五	13	19.9	86		一	6	18.3	88
	六	14	18.9	84		二	7	19.9	88
	日	15	—	—		三	8	19.9	87
	一	16	22.7	91		四	9	18.1	80
	二	17	21.8	86		五	10	17.6	75
	三	18	20.1	83		六	11	17.4	74
	四	19	18.1	83		日	12	—	—
	五	20	17.4	67		一	13	16.2	89
	六	21	17.9	76		二	14	18.0	90
	日	22	—	—		三	15	18.1	90
	一	23	19.2	89		四	16	16.5	82
	二	24	19.7	91		五	17	15.9	75
	三	25	—	—		六	18	15.5	80
	四	26	19.3	92		日	19	—	—
	五	27	21.3	88		一	20	15.7	84
	六	28	21.3	93		二	21	16.4	89

月份	星期	日期	平均濕度 °C	相對濕度 %	月份	星期	日期	平均濕度 °C	相對濕度 %
1	三	22	16.0	86	2	六	15	—	—
	四	23	16.0	77		日	16	—	—
	五	24	17.0	84		一	17	16.1	84
	六	25	18.8	93		二	18	16.3	77
	日	26	—	—		三	19	15.9	77
	一	27	17.9	89		四	20	16.4	75
	二	28	16.7	76		五	12	15.6	65
	三	29	18.1	76		六	22	15.8	64
	四	30	17.9	86		日	23	—	—
	五	31	19.0	85		一	24	17.0	80
2	六	1	20.3	85		二	25	18.1	82
	日	2	—	—		三	26	19.4	85
	一	3	21.3	88		四	27	19.5	85
	二	4	21.5	85		五	28	18.5	76
	三	5	21.6	86	3	六	1	19.2	83
	四	6	21.6	87		日	2	—	—
	五	7	—	—		一	3	19.2	89
	六	8	—	—		二	4	20.4	89
	日	9	—	—		三	5	21.3	89
	一	10	—	—		四	6	19.5	83
	二	11	—	—		五	7	17.3	79
	三	12	—	—		六	8	—	—
	四	13	—	—		日	9	—	—
	五	14	—	—		一	10	18.0	83

2.噴霧養護室內

月　　份	10			10		11	12			
星　　期	三	四	五	三	四	五	二	三	四	五
日　　期	23	24	25	30	31	1	3	4	5	6
平均溫度℃	23	23	—	23.9	23.5	21.9	23.3	21.9	21.0	20.2
相對濕度％	98			98			98			

附錄二、乾縮試體上各測點之乾縮值

本試驗所用者爲 10″ Whittmore strain gage （ℓ） 係利用機械連桿，將兩測點間之小變形量傳到測微計（ dial gage ），利用齒輪放大顯示出來，精度可達 $\frac{1}{10000}$ 英寸。此種須預先於試體上埋設測點，詳細請參考第四節 11 之附圖。

每次觀測時，須先測定長度 10″ 之標準桿，讀其差數數 D′ ，然後測量試體上兩測點之距離，讀其差數數 D″ ，則測點間之眞正長度爲 $D = D'' + \ell = 10''+D'' - D'$ ，ℓ 爲應變計身長，如下圖所示。各測點長度與標準桿長度之差值卽爲 $D''' = D'' - D'$ 。

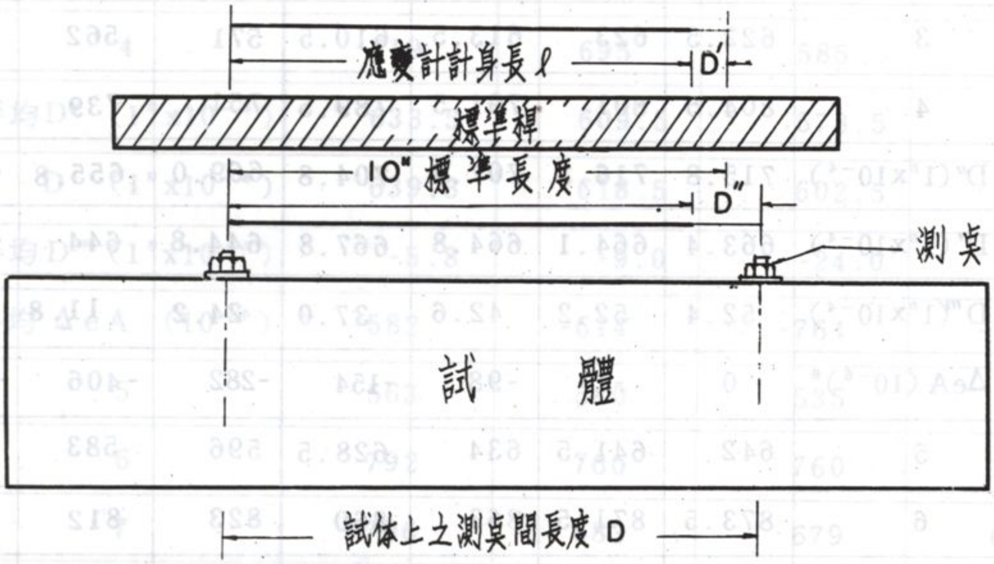


圖 6 － 3

1.無掺料混凝土：拌合日期： 10 月 23 日；塌度＝ 12 *cm*

試體編號	各測點之測量值 D''（$1'' \times 10^{-4}$）							
	養護環境	噴霧室內		試驗室室內				
	測點編號 齡期	1 天	3 天	5 天	7 天	14 天	21 天	28 天
P_A	1	755.5	760.5	751	747.5	715	700	695
	2	680.5	680.5	673.5	671.5	639	622	618
	3	622.5	623	613.5	610.5	571	562	556
	4	804.5	801	791.5	789.5	751	739	737
	平均 $D''(1''x10^{-4})$	715.8	716.3	707.4	704.8	669.0	655.8	651.5
	$D'(1''x10^{-4})$	663.4	664.1	664.8	667.8	644.8	644	643.8
	平均 $D'''(1''x10^{-4})$	52.4	52.2	42.6	37.0	24.2	11.8	7.7
	平均 $\Delta eA\ (10^{-6})$*	0	-2	-98	-154	-282	-406	-447
P_B	5	642	641.5	634	628.5	596	583	577
	6	873.5	871.5	863	860	823	812	803
	7	798	798	789	782.5	747	740	727
	8	668	668	650.5	654	618	602	592
	平均 $D''(1''x10^{-4})$	743.1	742.5	734.1	731.3	696	684.3	674.8
	$D'(1''x10^{-4})$	663.4	664.1	664.8	667.8	644.8	644	643.8
	平均 $D'''(1''x10^{-4})$	79.7	78.4	69.3	63.5	51.2	40	31
	平均 $\Delta eB\ (10^{-6})$*	0	-13	-104	-162	-285	-397	-487
	總平均 Δe	0	8	101	158	284	402	467

註：* Δe 爲 $\frac{\Delta D'''}{10''}$，即各齡期之測量值 D''' 對其 1 天測量值 D''' 之變化量，$\Delta D'''$，除於原測距 10″，即 $\frac{\Delta D'''}{10''}$ ，此爲應變值（Strain）。負值表示縮短。

試體編號		各測點之測量值 D″（1″ × 10^{-4}）			
	養護環境	試驗室室內			
	測點編號 \ 齡期	42天	57天	69天	85天
P$_A$	1	675	652	646	669
	2	600	573	568	553
	3	540	518	515	483
	4	719	695	585	670
	平均D″（1″x10^{-4}）	633.5	609.5	578.5	593.8
	D′（1″x10^{-4}）	639.3	618.5	602.5	597
	平均D‴（1″x10^{-4}）	-5.8	-9.0	-24.0	-3.2
	平均 ΔeA （10^{-6}）	-582	-614	-764	-556
P$_B$	5	563	540	535	522
	6	792	760	760	743
	7	706	683	679	658
	8	573	550	546	535
	平均D″（1″x10^{-4}）	658.5	633.3	630.0	614.5
	D′（1″x10^{-4}）	639.3	618.5	602.5	597
	平均D‴（1″x10^{-4}）	17.2	41.8	27.5	17.5
	平均Δ eB （10^{-6}）	-625	-649	-522	-622
總平均Δe		-604	-600	-534	-579

2.摻入 Sika Plastiment 者：

拌合日期：10 月 23 日

塌　　度：16 cm

試體編號		各測點之測量值 D″(1″x10^{-4})						
	養護環境	噴霧室內		試驗室室內				
	測點編號 ＼ 齡期	1 天	3 天	5 天	7 天	14 天	21 天	22 天
S$_A$	9	712	710	697.5	694	653	646	640
	10	681.5	679	664	660	613	605	600
	11	627.5	623	613.5	609	567	558	550
	12	772	771	758	761	703	708	693
	平均D″(1″x10^{-4})	698.3	695.8	683.3	681	634	629.3	620.8
	D′(1″x10^{-4})	663.4	664.1	664.8	667.8	644.8	644	643.8
	平均D‴(1″x10^{-4})	34.9	31.7	18.5	-13.2	-10.8	-14.7	-23
	平均 Δ e A(10^{-6})	0	-32	-164	-217	-457	-496	-579
S$_B$	13	698	697.5	686	683.5	650	640	638
	14	858	856.5	843.5	840.0	806	792	788
	15	682	678	663.5	662	630	615	610
	16	701.5	699	689.5	687	650	642	636
	平均D″(1″x10^{-4})	734.9	732.8	720.6	718.2	684	672.3	668
	D′(1″x10^{-4})	663.4	664.1	664.8	667.8	644.8	644	643.8
	平均D‴(1″x10^{-4})	71.5	68.7	55.8	50.4	39.2	28.3	24.2
	平均 Δ e A(10^{-6})	0	-28	-157	-211	-323	-432	-473
	總 平 均 Δe	0	30	161	215	390	460	526

試體編號		各測點之測量值 $D''(1''x10^{-4})$			
	養護環境	試驗室室內			
	測點編號 ＼ 齡期	42 天	57 天	69 天	85 天
S_A	9	621	594	588	570
	10	574	555	552	541
	11	533	508	498	478
	12	560	540	543	518
	平均 $D''(1''x10^{-4})$	572	549.3	545.3	526.8
	$D'(1''x10^{-4})$	639.3	618.5	602.5	597
	平均 $D'''(1''x10^{-4})$	-67.3	-69.2	-57.2	-70.2
	平均 ΔeA (10^{-6})	-1022	-1041	-921	-1051
S_B	13	612	592	582	562
	14	760	735	737	717
	15	590	566	558	540
	16	618	592	584	568
	平均 $D''(1''x10^{-4})$	645	621.3	615.3	596.8
	$D'(1''x10^{-4})$	639.3	618.5	602.5	597
	平均 $D'''(1''x10^{-4})$	5.7	2.8	12.8	-0.2
	平均 ΔeB (10^{-6})	-658	-687	-587	-717
	總平均 Δe	-840	-864	-754	-884

2.摻入Tricosal B. V Special 者；

拌合日期： 10 月 23 日

塌　　度＝ 12 *cm*

試體編號		各測點之測量值 D″（1″x10^{-4}）						
	養護環境	噴霧室內		試驗室室內				
	測點編號 ＼ 齡期	1 天	3 天	5 天	7 天	14 天	21 天	28 天
T_A	17	792.5	786.5	773.5	773	744	733	724
	18	908.5	900	889	887.5	857	843	831
	19	811.5	803.5	791	791	749	738	724
	20	978.5	971.5	957.5	956	923	903	892
	平均D″(1″x10^{-4})	872.8	865.4	852.4	851.9	818.3	804.3	792.8
	D′(1″x10^{-4})	663.4	664.1	664.8	667.8	644.8	644	643.8
	平均D‴(1″x10^{-4})	209.4	201.3	187.6	184.1	173.5	160.3	149
	平均 ΔeA (10^{-6})	0	-81	-218	-253	-359	-491	-604
T_B	21	957.5	951.5	937.5	937	895	890	882
	22	758.5	752	765	764.5	728	715	709
	23	720	714.5	703	700	662	649	642
	24	702.5	698.5	684	681	645	637	626
	平均D″(1″x10^{-4})	784.6	779.1	772.4	770.6	732.5	722.8	714.8
	D′(1″x10^{-4})	663.4	664.1	664.8	667.8	644.8	644	643.8
	平均D‴(1″x10^{-4})	121.2	115	107.6	102.8	87.7	78.8	71.0
	平均 ΔeB (10^{-6})	0	-62	-136	-184	-335	-424	-502
	總平均 Δe	0	72	177	219	347	458	553

試體編號		各測點之測量值 D″ (1″x10^{-4})			
	養護環境	試驗室室內			
	測點編號 \ 齡期	42 天	57 天	69 天	85 天
T_A	17	710	688	655	662
	18	819	790	789	770
	19	708	680	681	655
	20	879	854	850	828
	平均 D″ (1″x10^{-4})	779.0	753.0	743.8	728.8
	D′ (1″x10^{-4})	639.3	618.5	602.5	597
	平均 D‴ (1″x10^{-4})	139.7	134.5	141.3	131.8
	平均 ΔeA (10^{-6})	-697	-749	-681	-776
T_B	21	860	836	829	814
	22	690	665	660	643
	23	624	597	592	574
	24	600	575	573	552
	平均 D″ (1″x10^{-4})	693.5	668.3	663.5	645.8
	D′ (1″x10^{-4})	639.3	618.5	602.5	597
	平均 D‴ (1″x10^{-4})	54.2	49.8	51.0	48.8
	平均 ΔeB (10^{-6})	-670	-714	-702	-724
總	平均 Δe	-684	-732	-692	-750

4.無摻料混凝土：

拌合日期： 10 月 30 日

塌　　度＝ 14 cm

試體編號		各測點之測量值 D″ (1″x10^{-4})						
	養護環境	噴霧室內		試驗室室內				
	齡期 測點編號	1 天	3 天	5 天	7 天	14 天	21 天	28 天
C_A	25	870	873	869.5	860	845	836	825
	26	773	775	768	762	739	737	719
	27	591	592	589	578	560	553	551
	28	684.5	687	683.5	675	660	652	642
	平均D″(1″x10^{-4})	729.6	731.8	727.5	718.8	701.0	694.5	684.3
	D′(1″x10^{-4})	645	644.1	649	646.5	644	645.3	645
	平均D‴(1″x10^{-4})	84.6	87.7	78.5	72.3	57.0	49.2	39.3
	平均 Δ eA (10^{-6})	0	31	-61	-123	-276	-354	-453
C_B	29	770	776.5	770	760	737	730	714
	30	715	714.5	710	700	664	661	655
	31	904	905	899	893	877	870	860
	32	756.5	760	757	750	724	716	715
	平均D″(1″x10^{-4})	786.4	789	784.0	775.8	750.5	744.3	736
	D′(1″x10^{-4})	645	644.1	649	646.5	644	645.3	645
	平均D‴(1″x10^{-4})	141.4	144.9	135	129.3	106.5	99	91
	平均 Δ eB (10^{-6})	0	35	-64	-121	-349	-424	-504
	總 平 均 Δe	0	33	63	122	313	389	479

試體編號		各測點之測量值 D″（1″x10⁻⁴）			
	養護環境	試驗室室內			
	齡期 測點編號	43 天	62 天	70 天	84 天
C_A	25	798	783	776	765
	26	690	678	666	655
	27	518	500	483	472
	28	613	597	585	577
	平均 D″（1″x10⁻⁴）	654.8	639.5	627.5	617.3
	D′（1″x10⁻⁴）	618.3	602.5	597.5	594
	平均 D‴（1″x10⁻⁴）	36.5	37.0	30.0	23.3
	平均 ΔeA （10⁻⁶）	-481	-476	-546	-613
C_B	29	685	672	662	651
	30	626	610	600	588
	31	830	818	810	800
	32	685	673	664	654
	平均 D″（1″x10⁻⁴）	706.5	693.3	684.0	673.3
	D′（1″x10⁻⁴）	618.3	602.5	597.5	594
	平均 D‴（1″x10⁻⁴）	88.2	90.8	86.5	78.3
	平均 ΔeB （10⁻⁶）	-532	-506	-549	-631
	總平均 Δe	-507	-491	-548	-622

5. 摻入 Zeecon 者：

拌合日期： 10 月 30 日

塌　　度＝ 12 cm

試		各測點之測量值 D″（1″x10^{-4}）						
體	養護環境	噴霧室內		試驗室室內				
編號	齡期 / 測點編號	1 天	3 天	5 天	7 天	14 天	21 天	28 天
Z_A	33	763	761.5	753	748	728	722	710
	34	665.5	671	665	650	634	622	607
	35	642	640	632	625	605	597	584
	36	770	771	764	759	740	734	722
	平均D″(1″x10^{-4})	710.1	710.9	703.5	695.5	676.8	668.8	655.8
	D′(1″x10^{-4})	645	644.1	649	646.5	644	645.3	645
	平均D‴(1″x10^{-4})	65.1	66.8	54.5	49	32.8	23.5	10.8
	平均 Δ eA(10^{-6})	0	17	-106	-161	-323	-416	-543
Z_B	37	671	671	665	656	640	632	619
	38	783	785.5	779	770	750	745	738
	39	795.5	796.5	789	781	758	750	741
	40	592	594	586	578	560	551	543
	平均D″(1″x10^{-4})	710.4	711.8	704.8	696.3	677	669.5	660.3
	D′(1″x10^{-4})	645	644.1	649	646.5	644	645.3	645
	平均D‴(1″x10^{-4})	65.4	67.7	55.8	49.8	33	-4.2	15.3
	平均 Δ eB(10^{-6})	0	23	-96	-156	-324	-412	-501
	總 平 均 Δe	0	6	105	182	343	432	527

試體編號		各測點之測量值 D″(1″x10^{-4})			
	養護環境	試驗室室內			
	測點編號 \ 齡期	43 天	62 天	70 天	84 天
Z_A	33	673	668	657	651
	34	580	565	557	549
	35	555	543	542	529
	36	694	681	672	664
	平均 D″(1″x10^{-4})	625.5	614.3	607.0	598.3
	D′(1″x10^{-4})	618.3	602.5	597.5	594
	平均 D‴(1″x10^{-4})	7.2	11.8	9.5	4.3
	平均 ΔeA (10^{-6})	-579	-533	-556	-608
Z_B	37	590	577	568	559
	38	706	697	684	676
	39	716	700	692	683
	40	506	495	482	473
	平均 D″(1″x10^{-4})	629.5	617.3	606.5	597.8
	D′(1″x10^{-4})	618.3	602.5	597.5	594
	平均 D‴(1″x10^{-4})	11.2	14.8	9.0	3.8
	平均 ΔeB (10^{-6})	-542	-506	-564	-616
	總平均 Δe	-561	-520	-560	-612

6. 摻入WRDA 者：

拌合日期：10 月 30 日

塌　　度＝ 12 cm

試體編號		各測點之測量值 D″（1″x10^{-4}）						
	養護環境	噴霧室內		試驗室室內				
	測點編號 \ 齡期	1 天	3 天	5 天	7 天	14 天	21 天	28天
W_A	41	599	601.5	593	570	557	551	544
	42	903.5	903	897	881	860	848	840
	43	654	656.5	655	645	630	622	610
	44	775	767	763	759	742	736	725
	平均D″（1″x10^{-4}）	732.9	732	727	713.8	697.3	689.3	679.8
	D′（1″x10^{-4}）	645	644.1	649	646.5	644	645.3	645
	平均D‴（1″x10^{-4}）	87.9	87.9	78.0	67.3	53.3	44	34.8
	平均 Δ e A（10^{-6}）	0	0	-99	-206	-346	-439	-531
W_B	45	775	774	766	760	739	734	722
	46	704	705.5	698	692	675	663	656
	47	855.5	856	849.5	842	815	812	803
	48	799.5	799.5	792	783	765	756	744
	平均D″（1″x10^{-4}）	783.5	783.8	776.4	769.3	748.5	741.3	731.3
	D′（1″x10^{-4}）	645	644.1	649	646.5	644	645.3	645
	平均D‴（1″x10^{-4}）	138.5	139.7	127.4	122.8	104.5	96	86.3
	平均 Δ e B（10^{-6}）	0	12	-111	-157	-340	-425	-522
	總平均 Δ e	0	20	101	159	324	414	522

試體編號		各測點之測量值 D″(1″x10^{-4})			
	養護環境	試驗室室內			
	測點編號 / 齡期	43 天	62 天	70 天	84 天
W_A	41	516	492	479	471
	42	806	795	782	778
	43	580	567	559	552
	44	697	684	676	667
	平均 D″(1″x10^{-4})	649.8	634.5	624	617
	D′(1″x10^{-4})	618.3	602.5	597.5	594
	平均 D‴(1″x10^{-4})	31.5	32.0	26.5	23
	平均 ΔeA (10^{-6})	-564	-555	-614	-649
W_B	45	690	679	668	661
	46	626	612	603	595
	47	764	762	747	742
	48	714	702	693	683
	平均 D″(1″x10^{-4})	698.5	688.8	675.3	670.3
	D′(1″x10^{-4})	618.3	602.5	597.5	594
	平均 D‴(1″x10^{-4})	80.2	86.3	77.8	76.3
	平均 ΔeB (10^{-6})	-583	-522	-607	-622
	總平均Δe	-674	-539	-605	-636

7. 摻入Mighty 者：

拌合日期： 12 月 3 日

塌　　度＝M_A ： 20 *cm*

　　　　＝M_B ： 22 *cm*

試體編號		各測點之測量值 D″（1″x10⁻⁴）					
	養護環境	噴霧室內	試驗室室內				
	齡期 / 測點編號	1 天	3 天	5 天	7 天	14 天	21 天
M_A	49	497	552	………	544	543	530
	50	615	666	………	655	647	636
	51	848	899	………	888	881	874
	52	432	468	………	455	445	440
	平均 D″（1″x10⁻⁴）	598	646.3	………	635.5	629	620
	D′（1″x10⁻⁴）	564	619.5	………	623.3	619	617
	平均 D‴（1″x10⁻⁴）	34	26.8	………	12.2	10	3
	平均 ΔeA（10⁻⁶）	0	-72	………	-218	-240	-310
M_B	53	450	477	………	462	455	453
	54	580	632	………	618	615	602
	55	547	592	………	580	571	564
	56	463	500	………	488	477	468
	平均 D″（1″x10⁻⁴）	510	550.3	………	537	529.5	521.8
	D′（1″x10⁻⁴）	564	619.5	………	623.3	619	617
	平均 D‴（1″x10⁻⁴）	-54	-69.2	………	-86.3	-89.5	-95.2
	平均 ΔeB（10⁻⁶）	0	-152	………	-323	-355	-412

附 錄 七

FINAL REPORT
ON
CONCRETE QUALITY CONTROL

YUAN SHAN BRIDGE

CONTINENTAL ENGINEERING CORPORATION

ACKNOWLEDGEMENT

This report is the result of the combined efforts of the following members.

Directed by : K. F. Ho, Vice President, C.E.C.

Compiled and Reported by : Y. H. Chen, Concrete Advisor

Tested by : P. H. Chang and his Concrete Laboratory Group

Content

1. Scope

1. Description of the Project

Combination Project Numbers 11, 12 and 13 (Station 0 + 33.10 N to Station 1 + 686.3 N) is a bridge structure with two viaducts through the heart of Taipei City. The design was prepared by T. Y. Lin International Structural Engineers, San Francisco, California, U.S.A. and the Taiwan Area Freeway Construction Bureau. The construction consists of three parts (Fig. 1).

(1) Project 12 (Sta. 0 + 856.5 N to Sta. 1 + 527.5 N)

Main Bridge

This structure has 6 traffic lanes over a total length of 671 meters with six spans of prestressed concrete box girders. The arrangements of the spans are 75, 150, 142.5, 142.5, 118 and 43 meters in sequence from west to east.

The superstructures in each bound consist of three cell box girders with haunches at the piers and center hinges at mid-span. The construction of the box girders was carried out by the cantilever method, a progressive construction of cast-in-place segments tied together by post-tensioned tendons which start from each pier and extend to meet the opposite girders at the middle of the span.

The substructures consist of hollow-circular section piers with rigid connections to the superstructures, supported by 40cm x 40cm PC piles of 9 to 42 meters in length, together with 3 meter H-type steel shoes; and by open caissons of 1.5m, 2.5m and 6m in diameters and 11 to 25m in length.

(2) Project 11 (Sta. 0 + 331.0 N to Sta. 0 + 856.5 N) Viaduct

This structure has 6 traffix lanes over a total length of 525.5 m.

(3) Project 13 (Sta. 1 + 527.5 N to Sta. 1 + 686.3 N) Viaduct

This structure has 6 traffic lanes over a total length of 158.8 m.

Both viaduct superstructures are post-tensioned I-girders with partially composite concrete deck on frame piers. The piers consist of concrete columns and caps supported on pile footings. The piles, which are an average of 45 meters in length were reversed circulation cast-in-place concrete piles.

A total volume of 108,000 m^3 of concrete was placed during the three year construction period and an average of 120 m^3 of concrete was produced and placed per day. The maximum daily production was 220 m^3

A well-equipped ELBA-35 cu.m/hr concrete batching plant, a ballast bin, a cement silo etc. were installed and an effective concrete delivery system was planned and established exclusively for this job.

The Prospective, typical plan and longitudinal section of the main bridge are shown as Figures 2 and 3 respectively.

2. Classification of Concrete

(1) Strength Requirement

Concrete was designed in classes according to the required compressive strength at 28 days when mixed and moist cured at $23\pm1.7^{\circ}C$

Table 1

Classification	Class	Age	f'c	fcr	Size of Agg.	Basic Slump
		days	kg/cm^2	kg/cm^2	inch	inch
P1	2	28	350	402	1½"	4
	3				3/4"	4
P2	2	28	240	275	1½"	4
	3				3/4"	4
Bi	2	28	350	402	1½"	4
	3				3/4"	4
B2	2	28	280	322	1½"	4
	3				3/4"	4
B3	2	28	140	161	1½"	4
	3				3/4"	4
Q1	2	3	260	300		4
	3					

Note 1. Coefficient of Variation 15%.

2. Chance of strength being lower than specified: 2 in 10

(2) Special requirements for superstructure of main bridge

A. Strength of Concrete

a. Tensioning shall be commenced only when concrete cylinder strength has reach 260 kg/cm^2 (3700 psi)

b. Ultimate strength of 15 x 30 cm cylinder at 28 day age shall be not less than 350 kg/cm^2 (5000 psi)

c. Tension time is required at 40 hr. minimum after concrete is placed, due to working cycle requirements.

B. Drying Shrinkage of Concrete

According to ASTM C157 with the following modifications:

a. Calculated make three 10 x 10 x 28 cm concrete specimens with 2 cm maximum sized aggregates.

b. Average dry shrinkage after 7 days of moist curing and 21 days of actual drying shall not exceed the specified limit of 0.05%.

II. Concrete Materials

1. Filler Materials of Portland Cement Concrete (Aggregate)

During exploration of aggregate sources in the northern area of Taiwan Ta-Han Chi river natural aggregates were selected. The aggregate was found to be clean, hard, sound, durable and cubical.

Major physical properties are shown as follows:

Table 2

Items	Coarse	Fine
Specific gravity	2.63	2.62
Absorption	1.43%	2.9%
Color Test	Ok	Ok
Los Angeles Abrasion	18.9% (Grade A)	-
Soundness	0.66% (5 cycle)	3.88% (5 cycle)
Flat and Elongate	8.9% (1½"-No. 4)	
Mortar Making	-	106% (28 days)
Under No. 200 Sieve	0.15%	1.35%

Aggregate is classified as metamorphic rock, based on to origin. The majority of fine and coarse grains are quartzite with a little quartz. Some slate particles were also found.

The potential alkali reactivity (ASTM C289-66) test shows that:

$Rc = 39.80 < 70$

$Sc = 50.62 < 35 + \frac{Rc}{2} = 54.9$ (Unit: millimole/1)

The aggregate is not considered potentially alkali reactive.

Aggregates were collected and processed at the riverside and delivered directly to the Yuan Shan Bridge batching plant aggregate storage area.

2. Binder Materials

(1) Cement for Superstructure of Main Bridge

A special agreement was stipulated between the Taiwan Cement Corporation and TAFCB in order to obtain uniform quality cement. The contract specified that the cement must meet ASTM Designation C-150-67 Type 1 with the following limitations:

Bilane surface not more than 3500 cm^2/g

Cube test at 3 days not less than 2600 psi

(2) Cement for Other Structures

Type 1 cement produced by Chutung and Suo Mills of Taiwan Cement Corporation was used for all the structures. All cement was to meet ASTM Designation Type 1

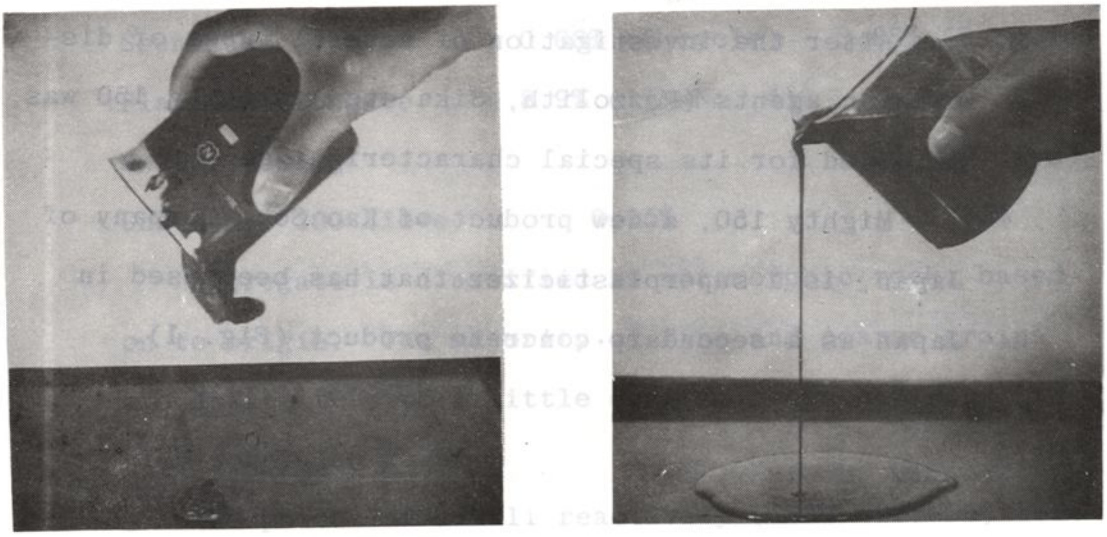
(3) Admixtures

After the investigation of several types of dispersion agents (Pozzolith, Sika etc.), Mighty 150 was selected for its special characteristics.

Mighty 150, a new product of Kao Soup Company of Japan, is a superplasticizer that has been used in Japan as a secondary concrete product (Fig. 1).

Dispersion of Cement
Particles without Dispersants

Dispersion of Cement
Particles with 1% MIGHTY



Flow of Cement
Paste Without Dispersants
W/C = 25 %

Flow of Cement
Paste with 1% MIGHTY
W/C = 25 %

Fig.1

It differs chemically from conventional water reducing admixtures such as lignosulfonates, hydrocarboxylates, and hydro-onlated polymers; is much more effective and is free from chlorides.

Construction Technology Laboratories of the Portland Cement Association, U.S.A. has certified that Mighty 150 meets the requirement for the type A chemical admixture specified in ASTM C494.

3. Water

Taipei City tap water was used as the mixing water for the concrete mix. It is clean and contains no harmful matter (chemical analysis provided as Table 3).

III. Concrete Mix Studies

Prior to the beginning of the concrete mix study, the concrete test program for Combination Project Numbers 11-12-13 was planned and reported to the head office of Continental Engineering Corporation, and a copy was also submitted to the TAFCB Material Testing Laboratory for approval. The report included material investigation, mix design, procedure, test items, laboratory arrangement, test facilities, working schedule, etc. All subsequent laboratory tests were performed in accordance with this program.

1. Strength Development of Steam-cured Concrete Prestressed Precast Members

The compressive strength of these members obtained from laboratory tests should be 4283 psi at 14 hours and 6867 psi at 28 days. These two targets are rather difficult to obtain simultaneously under general conditions. If the 4283 psi strength is retained at an age of 14 hours, the ultimate strength of this mix under normal cure should reach 9000 psi at 28 days. But with the 14 hour steam-cure at its initial stage, the strength of this mix should be down to 9000 x 0.75 = 6570 $<$ 6867 psi (Fig. 2, 3).

Through repeated tests, it was found that the following design mix could barely meet the specified construction requirements:

W/C	Cement	Water	Sand	3/4"-No. 4
0.35	543	190	580	1032

Because the strength of concrete at its later ages is greatly affected by steam curing, it cannot be assured that it could be fortified completely. Mighty 150 (0.5% of CF) was added to this mix in order to assure the strength both at its early and later stages.

Mix Design (Unit: kg/m^3)

W/C	Cement	Admixture	Water	Sand	3/4"-No. 4
0.35	543	2.715	190	580	1032

Strength (psi)

14 hr.	28 days
4964	7100

Curing time and strength relation are shown on Figure 2 and 3.

2. Concrete for Superstructure of Main Bridge

The following series of mixes were performed for investigation purpose:

(1) 450 (475 & 500) kg/m^3 of cement factor without admixture.

  a. Observing the age of concrete, and determing when its strength will reach 260 x 1.15 = 300 kg/cm^2.

  b. Making tests and recording the compressive strength at the age of 28 days.

(2) 500 kg/m^3 cement factor with suitable dosage of superplasticizer; determing when the strength can meet the requirement.

(3) Minimum cement factor with recommended dosage of superplasticizer such that strength at 60 hrs. age can reach 300 kg/cm^2 and strength at 28 days will exceed 350 x 1.15 = 402 kg/cm^2.

(4) Minimum cement factor with ordinary water reducing admixture such that strength at 60 hrs. can reach 300 kg/cm^2 (4250 psi) and strength at 28 days will exceed 402 kg/cm^2 (5725 psi).

Test results are shown in Table 4, 5 and 6 and Figures 4 and 5.

3. Dry Shrinkage Test

Due to lack of test apparatus of the type mentioned in ASTM C157 for measuring dry shrinkage, a JIS type comparator was used.

The size of the concrete specimen was 10 x 10 x 50 cm instead of the 10 x 10 x 28 cm specified. However, the ASTM C157 test sequence and procedure were generally followed.

Six sets of samples for each of the mixes 350, 500, 673, 450, 475, 425 kg/cm^2 of cement were measured at the ages of 1 day, 7 days, 14 days, 21 days and 28 days respectively.

Measurement of dry shrinkage at 28 days is 0.027% - 0.05% (except for Test 3). Please refer to Appendix 1, a series of Dry Shrinkage Test Results.

4. Conventional Concrete Mix

All test mixes included in Project Numbers 11, 12 and 13 were tested according to the concrete test programs. Test results are shown on Tables 4, 5 and 6.

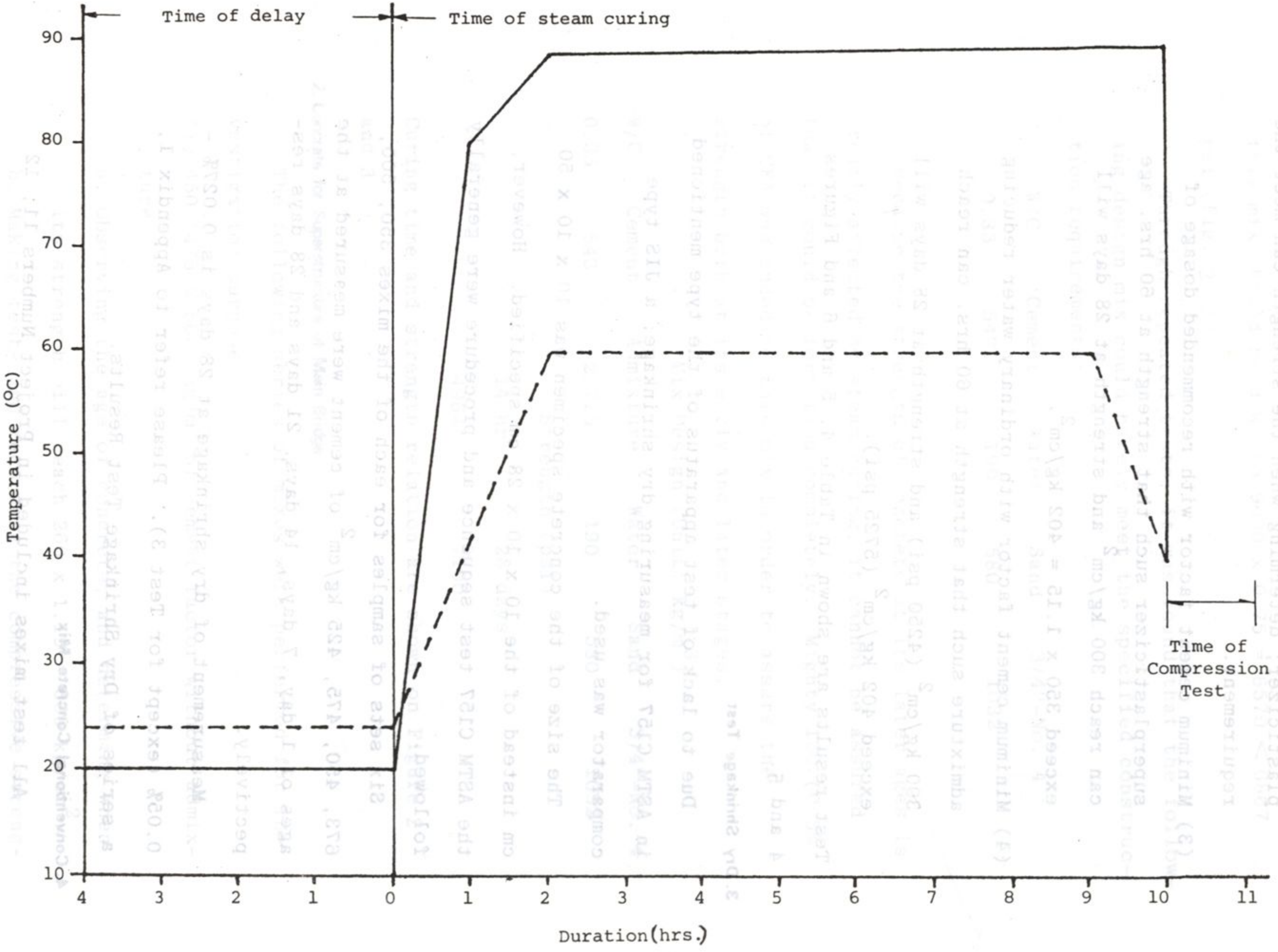

Time of delay
Time of steam curing
Temperature (°C)
90
80
70
60
50
40
30
20
10
4
3
2
1
0
1
2
3
4
5
6
7
8
9
10
11
Time of Compression Test
Duration(hrs.)

Fig. 2

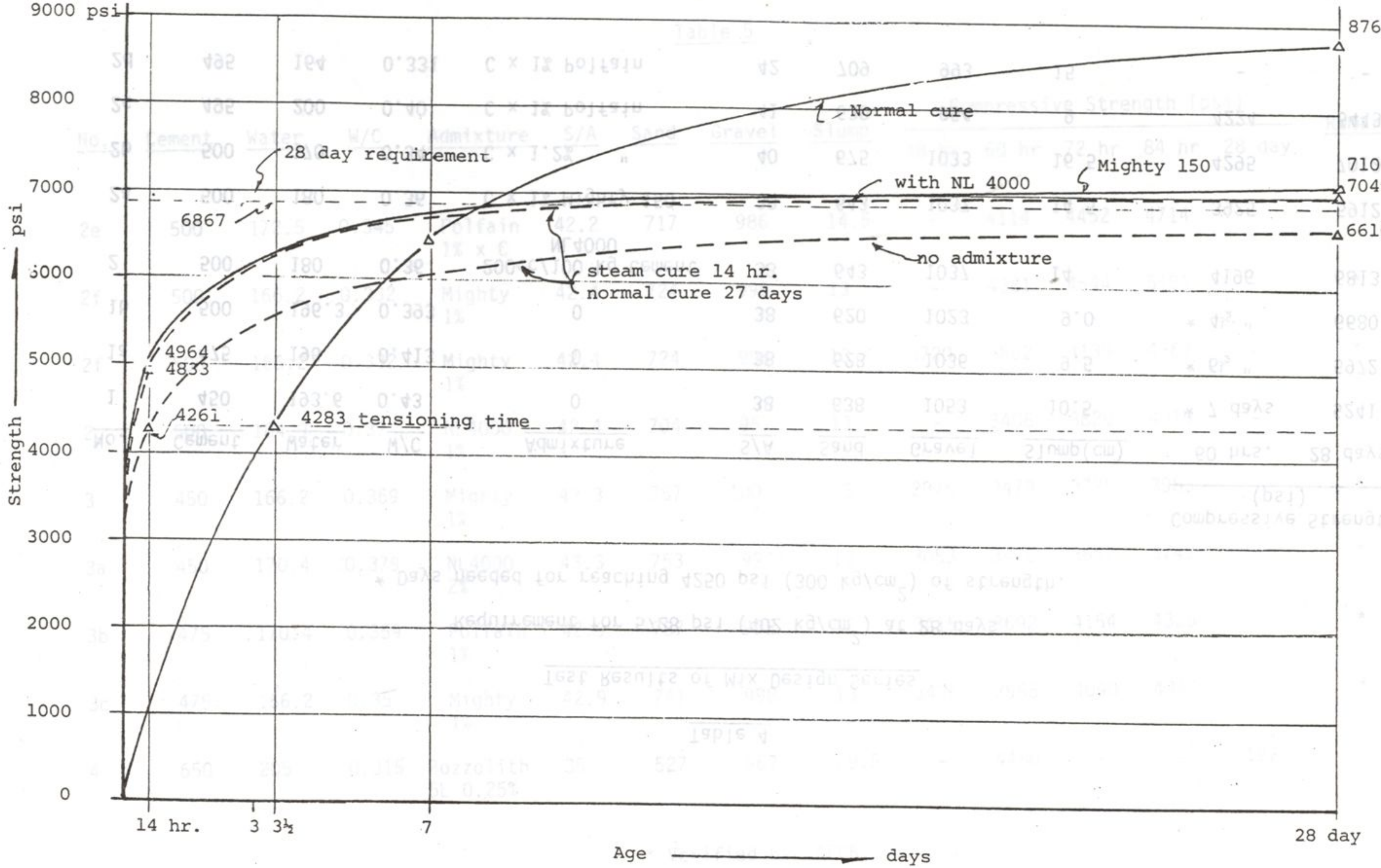

9000 psi
8000
7000
6000
5000
4000
3000
2000
1000
0
Strength ——→ psi
8768
Normal cure
28 day requirement
with NL 4000
Mighty 150
7100
7049
6867
6616
no admixture
steam cure 14 hr.
normal cure 27 days
4964
4833
4261
4283 tensioning time
14 hr.
3
3½
7
28 day
Age ——→ days

Fig. 3

Table 4

Test Results of Mix Design Series

Requirement for 5728 psi (402 kg/cm^2) at 28 days

* Days needed for reaching 4250 psi (300 kg/cm^2) of strength.

									Compressive Strength (psi)	
No.	Cement	Water	W/C	Admixture	S/A	Sand	Gravel	Slump(cm)	60 hrs.	28 days
1	450	193.6	0.43	0	38	638	1053	10.5	* 7 days	5241
1a	475	196	0.413	0	38	628	1036	9.5	* 6½ "	5972
1b	500	196.3	0.393	0	38	620	1023	9.0	* 4½ "	6680
2	500	180	0.36	200cc/100 kg Cement NL4000	38	643	1037	14	4196	6813
2a	500	180	0.36	C x 1% Mighty 150	38	643	1037	14.5	3925	6912
2b	500	170	0.34	C x 1.2% "	40	675	1033	16.5	4295	7012
2c	495	200	0.40	C x 1% Polfain	41	630	954	9	4224	6443
2d	495	164	0.331	C x 1% Polfain	42	709	993	15	-	-

Table 5

No.	Cement	Water	W/C	Admixture	S/A	Sand	Gravel	Slump	Compressive Strength (psi)					Remark
									48 hr	60 hr	72 hr	84 hr	28 day	
2e	500	172.5	0.345	Polfain 1% x C	42.2	717	986	14.5	-	4114	4452	4714	-	*
2f	500	166.2	0.332	Mighty 1%	42.4	724	995	13	-	4381	4544	5183	-	*
2f	500	166.2	0.332	Mighty 1%	42.4	724	995	13	3720	3862	4139	4367	-	*
2g	500	184.9	0.37	NL4000 1%	42.4	704	967	13	-	3408	3820	4019	-	
3	450	166.2	0.369	Mighty 1%	43.3	757	1003	13	2876	3479	3728	3983	-	*
3a	450	170.4	0.379	NL4000 2%	43.3	753	997	13	3053	3472	3657	3742	-	*
3b	475	170.4	0.359	Polfain 1%	42.9	737	992	12	3415	3692	4154	4325	-	*
3c	475	166.2	0.35	Mighty 1%	42.9	741	998	13	3436	3955	4090	4345	-	*
4	650	205	0.315	Pozzolith 5L 0.25%	35	527	967	9.5	-	4480	-	-	7197	

* Verified by TAFCB.

Table 6

DESIGN MIX / SERIES NO.	MAX AGG. SIZE (CM)	SLUMP (CM)	AIR CONTENT (%)	S/A (%)	W/C (%)	UNIT WT. (KG/M^3)								COMPRESSIVE STRENGTH (KG/CM^2)					USED FOR	REMARKS
						WATER	CEMENT	FINE AGG.	COASE AGG.				ADMIX-TURE	14HR.	60HR.	7 DAYS	14 DAYS	28DAYS		
									15MM	25MM	40MM	TOTAL								
420-20-6	20	5	3	37	31	167.3	540	600	827	207		1034	MIGHTY 150Cx05%	334				494	40 x 40 PC PILE	S33
420-20-5	20	5	1	36	32.2	185	575	575	828	207		1035		320				483	40 x 40 PC PILE	S35
350-20-10	20	9.5	1	38	41.3	196	475	628	829	207		1036				316	383	414	PC GIRDER	T69
350-20-12	20	12.5	1	45	44.1	209.3	475	728	720	180		900				289	335	408	DECK SLAB	T.93 by pump
350-20-13	20	13	1	40	40.7	203.4	500	645	783	196		979						423	DECK SLAB, PIER	T.140
350-20-15	20	15	1	38	40.2	206	513	606	900	100		1000				310		416	PIER	T.203
350-20-18	20	18	1	40	37.1	176	475	681	827	207		1034	MIGHTY 150Cx1%		319			473	DECK SLAB	T.264
350-20-18	20	17.5	1	40	39.3	216	549	626	760	190		950			(72hrs) 250	309		435	DIAPHRAGM	T.270
280-40-7.5	40	8	1	38.8	51	178	350	699	446	112	557	1115				223	226	326	FOOTING,CAISSON SEALING	T.31
280-40-10	40	10.5	1	37.4	48.9	183.3	375	661	448	112	559	1119				230		326	" "	T.11
280-40-12	40	12.5	1	45	49.5	197.8	400	769	380	285	285	950				237	312	347	" "	T.102
280-20-12	20	12	1	45.8	49.2	209.3	425	759	728	182		910				268	313	342	" "	T.91 by pump
280-40-15	40	16	1	38.4	49	194	400	660	428	107	536	1071				245	293	345	" "	T.39
280-40-10	40	10.5	2	37	48.9	174	356	654	454	113	567	1134	SIKA(PCR) C x 0.2%			219	278	335	FOUNDATION, PIER	T.188
280-40-12	40	12	2	38.4	48.8	177.8	364	677	440	109	550	1099	SIKA(PCR) C x 0.2%			215	287	336	" "	T.192
240-40-7.5	40	7.5	1	39.6	54.8	178	325	721	445	111	556	1112				203	251	292	FOUNDATION	T.30
240-40-10	40	11	1	38	52.4	183.3	350	679	448	112	556	1120				214		306	"	T.7
240-40-12	40	12.5	1	45	52.5	196.8	375	780	386	289	289	964				206	273	294	"	T.101
240-20-12	20	12.5	1	47	51.6	209	400	790	721	180		901				216	258	299	FOOTING, PIER CAP BEAM	F1 by pump
240-40-18	40	17	2	44	51.4	192.8	375	755	389	97	486	972	SIKA(PCR) C x 0.2%			194		325	REVERSE CIRCULATION PILE	T.6
240-20-12	20	12	2	44	51.7	189.6	367	762	784	196		980	SIKA(PCR) C x0.25%			221	260	298		T.182
210-40-18	40	18	1	45	53.5	214	400	762	377	94	471	942				186		302	CUT-OFF WALL	

Table 6

DESIGN MIX / SERIES NO.	MAX AGG. SIZE (CM)	SLUMP (CM)	AIR CON-TENT (%)	S/A (%)	W/C (%)	UNIT WT. (KG/M^3)								CONPRESSIVE STRENGTH (KG/CM^2)					USED FOR	REMARKS
						WATER	CEMENT	FINE AGG.	COASE AGG.				ADMIX-TURE	14 HR	60 HR	7 DAYS	14DAYS	28DAYS		
									15MM	25MM	40MM	TOTAL								
210-40-12	40	13	1	39.5	56.4	183.3	325	714	442	111	553	1106				198	240	314	SLAB	T.221
210-20-12	20	12	1	44	57.1	198.7	348	781	804	201		1005				198	238	269	WALL	T.95
175-40-15	40	15.5	1	41.2	69.3	194	280	748	432	108	541	1081				130	186	211	"	T.51
140-40-15	40	15	1	42.9	77.6	194	250	790	426	106	532	1064				104	156	177	FOUNDATION SEAL-ING	T.49
140-40-10	40	10.5	1	43.3	73.3	183.3	250	809	429	107	536	1072				109	165	186	" "	T.48
350-15-15	15	15.5	1	42.0	40.0	219	548	655	915			915			(72 hr) 255	318	359	437		T.271
350-20-17	20	17	2	38	39.5	197.3	500	609	804	201		1005	Pozzolith #8 Cx0.25%			339	392	450		T.227
350-20-17	20	17.5	2	38	39.5	197.3	500	609	804	201		1005	SIKA(PCR) C x0.25%			339	399	450		T.223
350-25-10	25	10.5	2	38	36.7	156	425	692	685	457		1042	MIGHTY 150Cx1%		315			445		T.285
350-20-13	20	14	2	41	39.1	176	450	696	811	203		1014	MIGHTY 150Cx1%		304			440		T.286
240-20-17	20	17	1	45	50.3	201	400	777	768	192		960	SIKA(PCR) C x0.25%			249		285	REVERSE CIRCULA-TION PILE	pumping
240-20-13	20	13	1.	47	44.5	178	400	826	754	189		943	MIGHTY 150Cx1%			310		426	CAP-BEAM, DECK SLAB	pumping
240-40-12	40	11	2	38	52.5	169	322	692	457	114	571	1142	SIKA(PCR) C x0.2%			212	255	303	FOOTING	T.193

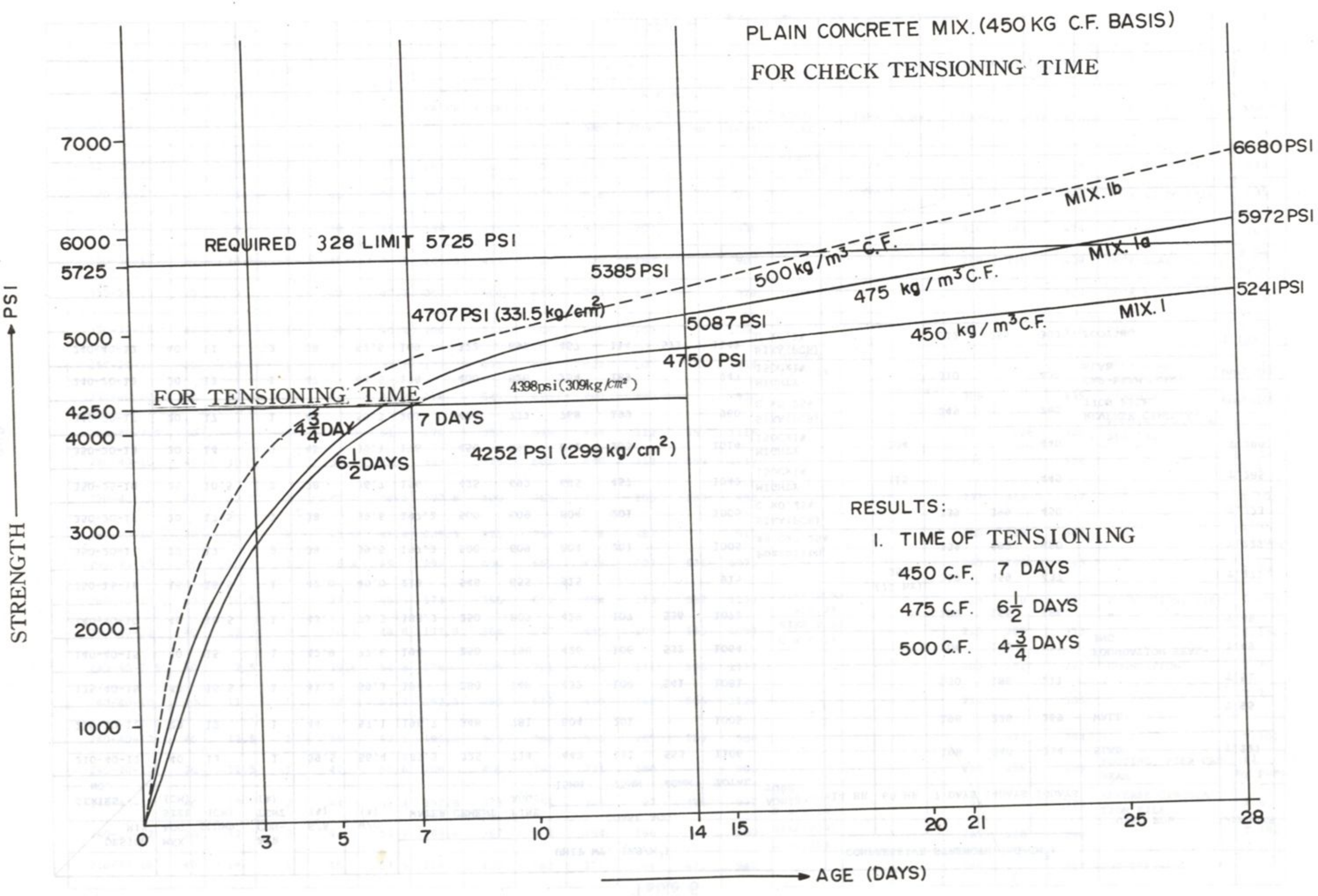

PLAIN CONCRETE MIX. (450 KG C.F. BASIS)
FOR CHECK TENSIONING TIME
7000
6000
5725
5000
4250
4000
3000
2000
1000
STRENGTH ——→ PSI
REQUIRED 328 LIMIT 5725 PSI
6680 PSI
MIX. Ib
5972 PSI
500 kg/m3 C.F.
MIX. Ia
5385 PSI
475 kg/m3 C.F.
5241 PSI
4707 PSI (331.5 kg/cm2)
5087 PSI
MIX. I
450 kg/m3 C.F.
4750 PSI
4398psi (309kg/cm2)
FOR TENSIONING TIME
4 3/4 DAY
7 DAYS
6 1/2 DAYS
4252 PSI (299 kg/cm2)
RESULTS:
I. TIME OF TENSIONING
450 C.F. 7 DAYS
475 C.F. 6 1/2 DAYS
500 C.F. 4 3/4 DAYS
0
3
5
7
10
14
15
20
21
25
28
——→ AGE (DAYS)

Fig. 4

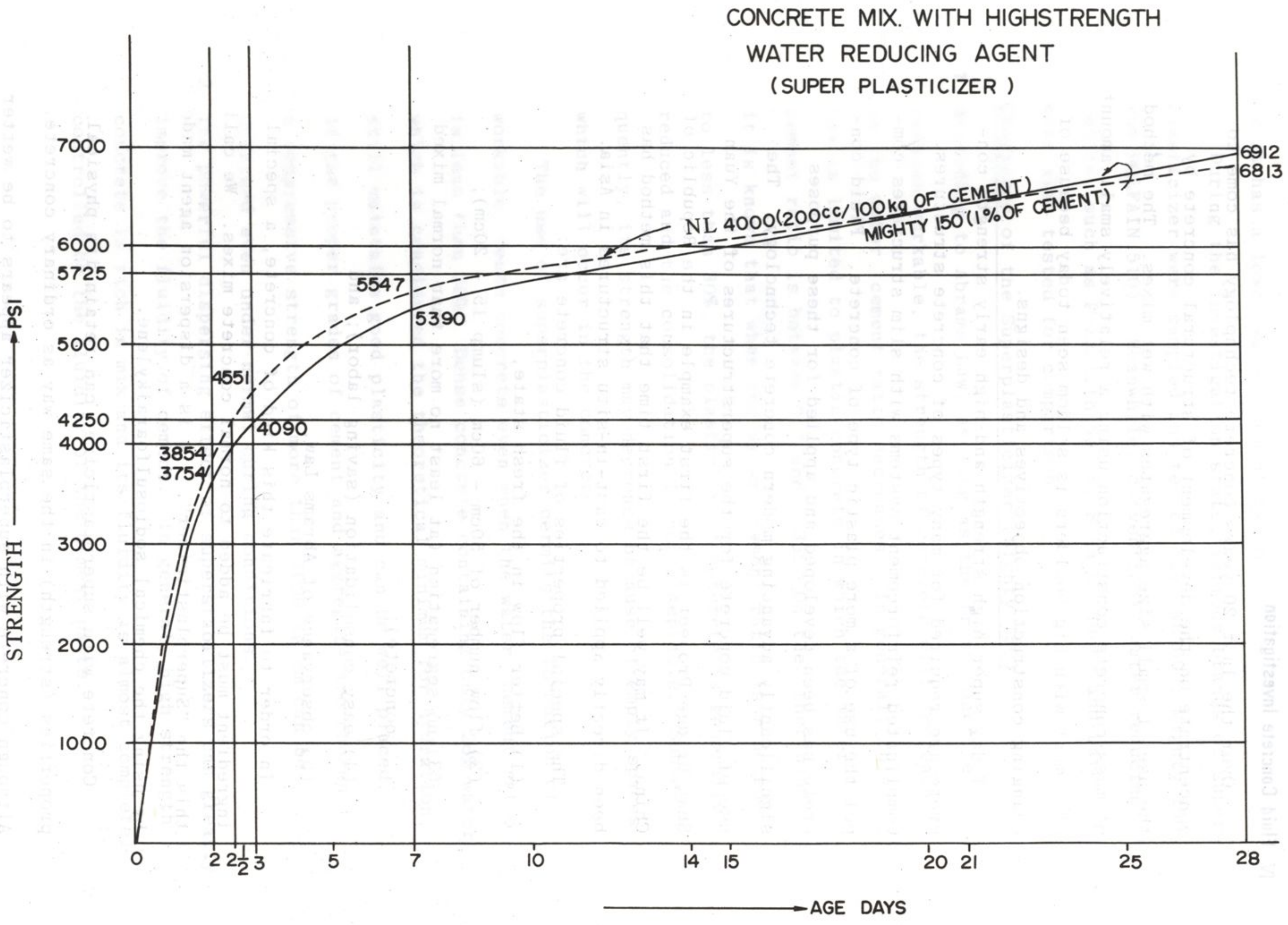

CONCRETE MIX. WITH HIGHSTRENGTH
WATER REDUCING AGENT
(SUPER PLASTICIZER)
STRENGTH ——→ PSI
7000
6000
5725
5000
4250
4000
3000
2000
1000
0
2
$2\frac{1}{2}$
3
5
7
10
14
15
20
21
25
28
——→ AGE DAYS
NL 4000(200cc/100kg OF CEMENT)
MIGHTY 150 (1% OF CEMENT)
3854
3754
4551
4090
5547
5390
6912
6813

Fig. 5

Ⅳ. Fluid Concrete Investigation

Over the last 20 years concrete technology has come to concentrate on the development of structural concrete by the use of small size aggregates with wet mixes. The method of mass concrete construction using a relatively small amount of cement with big boulders is seldom seen today because of changing construction objectives and designs.

Today super-high strength and high early strength concrete are required for many types of concrete structures. Complicated reinforcement systems with slim structures compel the use of a more plastic type of concrete. Fluid concrete has been developed and applied for these purposes significantly advancing modern concrete technology. The use of fluid concrete for the superstructures of the Yuan Shan Bridge Project is the first example in the Republic of China. It may well be the first time that this method has been directly applied to cast-in-situ structures in Asia.

The special properties of fluid concrete are:

(1) Better flow in the fresh state;

(2) flow number of 50cm - 60cm (slump 15 - 20cm);

(3) no segregation (at least no more than normal mixed concrete);

(4) easy consolidation (saving labor); and

(5) observance of Abrams Law.

In order to fabricate this kind of concrete, a special ingredient must be added to normal concrete mixes. We call this the "Superplasticizer". It is a dispersion agent made by using the chemical sodiosulfoaralkylene.

Concrete with superplasticizer can retain its physical properties (strength) in the same way as ordinary concrete. Although concrete with superplasticizer appears to be wetter and to flow more easily, the strength is actually increased as it has a lower water cement ratio.

During the investigation state, various brands of superplasticizer were sampled and tested (eg. NL4000, Mighty 150 and POLFAIN 510). Meanwhile, conventional water reducing agents, such as Pozzolith No. 8, Sika products and Darex were also tested for comparison.

<u>Character of the Superplasticizer (Mighty 150)</u>

1. According to Abrams Law, as long as the fresh concrete remains workable, the strength of concrete will increase as the water cement ratio decreases. In general, this Law is limited to plastic concrete in which the water cement ratio is between 0.7 and 0.35. On the other hand, it is known that when the water cement ratio is reduced to less than 30%, the plasticity of concrete is generally reduced and the consolidation is not acceptable. Subsequently, its strength may decrease because of empty spaces which will occur in the concrete.

   The use of superplasticizer permits the formation of workable, dense concrete even when the water cement ratio is less than 30%. Dense concrete containing only 26% water, which is considered the theoretical minimum for hydration, still maintains good plasticity and can be well placed. If the proper grades of cement and aggregate are chosen, a compressive strength of more than 1000 kg/cm can be obtained even under normal curing conditions.
2. The powerful dispersing effect of superstabilizers can greatly improve the fluidity of cement. In general, high strength concrete is rich in mix and its fluidity is almost completely controlled by the fluidity of the cement paste.

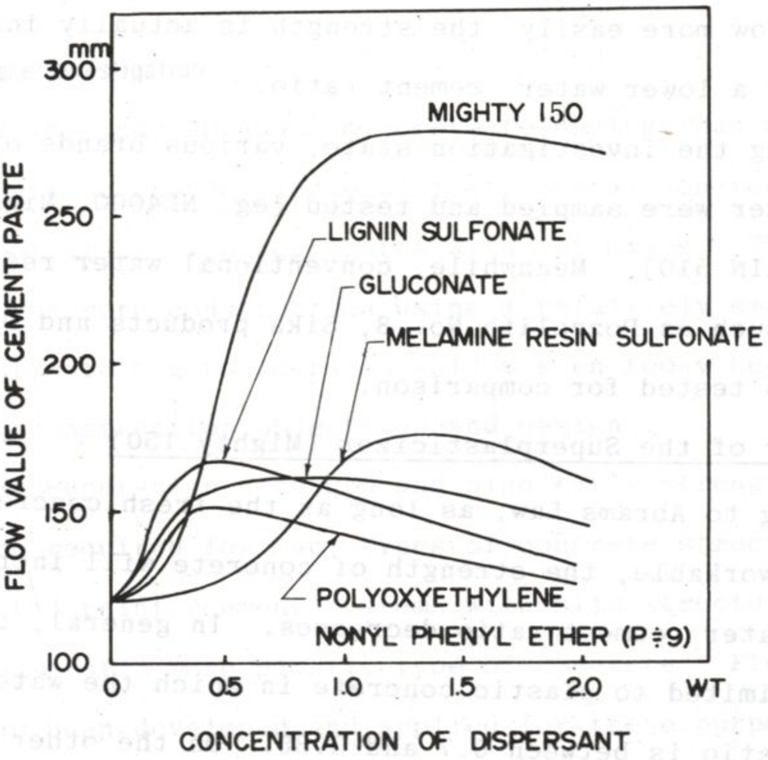


Fig. 6 FLUIDITY OF CEMENT PASTE CONTAINING SEVERAL KINDS OF DISPERSANT

Fig. 6 compares commercial dispersion agents with the superplasticizer Mighty 150. Improvement in the flow values of the cement paste is observed with the use of this additive. For dispersing agents of the lignin sulfonate type, currently the most popular concrete additive, the mix dispersing power appears at a percentage of around 0.3%. Even at higher dosages, the flow value remains unchanged. On the other hand, when Mighty 150 is increased from 0.3% to 1.0%, the paste flow increases remarkable. At amounts higher than 1%, the flow value remains almost constant.

3. The fluidity of fresh concrete mix is remarkably improved, the powerful dispersing effects of superplasticizer Mighty 150.

Table 7

Table 7: Effect of dispersants on properties of concrete

(No water reduced)							
Dispersant	Dosage	W/C	Slump	Air Content	Compressive strength (kg/cm^2)		
		(%)	(cm)	(%)	at 3 days	at 7 days	at 28 days
Plain	0	39	5.5	0.8	180	313	425
Mighty	0.25	"	11.4	1.1	221	377	475
	0.50	"	25.0	0.8	255	432	495
	0.75	"	> 25	0.6	199	302	380
	1.00	"	> 25	0.6	160	270	345
Calcium lignin sulfonate*	0.25	"	10.0	3.9	213	366	467
	0.50	"	17.3	7.1	210	354	448
	0.75	"	21.4	10	125	273	338
	1.00	"	21.3	10	not hardened	not hardened	176

Cement content 440 kg/m^3
S/A = 36.3%
* dried residue from alcohlic fermentation process of sulfite pulp waste.

As shown on Table 7, when superplasticizer is added to concrete, the slump increases with the increase in percentage of superplasticizer when it constitutes more than 0.75% of the mix, the slump of concrete exceeds the measurable limit. In case of lignin sulfonate which is shown for comparison, the slump also increase with the increasing percentage of additive. But it becomes constant at more than 0.75% of dosage.

4. By means of the strong dispersion effect of superplasticizer, the high water reduction necessary to obtain high strength concrete becomes possible (Table 8).

The concrete mix with superplasticizer can maintain good workability at a water cement ratio of as low as 30%. When the conventional dispersion agent is used, it is impossible to maintain workability when the water cement ratio is 30% or lower. The rates of increase in the strength of superplasticizer and lignin sulfonate, compared with the strength of plain concrete, are 80% and 25%,respectively.

Table 8

Table 8: Effect of dispersing agents on properties of concrete.

(Water reduction)

Dispersant	Dosage	W/C (%)	Water reduction rate (%)	Slump (cm)	Air content (%)	Compressive strength (kg/cm^2)		
						at 3 days	at 7 days	at 28 days
Plain	0	39	0	5.7	0.8	192	343	424
Mighty 150 (S/A=36.3)	0.25	35	10	5.7	1.1	241	388	460
	0.50	31	20	4.7	1.3	322	471	574
	0.75	27.3	30	11.2	1.6	472	591	768
	1.00	26.0	33	4.4	1.7	457	550	753
Calcium lignin sulfonate (S/A=34.3)	0.25	35	10	4.6	3.1	263	380	493
	0.50	34	12	7.1	7.2	266	427	519
	0.75	32.9	15	9.1	8.6	not hardened	192	452
	1.00	32.7	16	6.4	10	not hardened	not hardened	345

Cement content 440 kg/m^3

S/A = 36.3% and 34.3%

V. Concrete Production, Delivery and Placing

1. Equipment

All of the concrete equipment is listed below.

(1) Concrete Batching and Mixing Plant

One man control, electronic punch card system (Fig. 10).

Brand	ELBA Compact Plant, EMM 35N III	
Plant Capacity	35 Cu.M/hr.	
No. of Mixer	1 set	
Aggregate	4 compartment (deposit type)	
Cement Silo	100 Ton	2 sets
	200 Ton	1 set

(2) Transit Mixer

Brand	Dram (Local made)
	Engine and Chassis - Hino, Japan
Capacity of Mixer	6 yd^3 - 15 Ton gross weight
Quantities	8 units

(3) Concrete Pump

Brand	Schwing GmbH BP550HDD,	250 HDD
Rated Output	50 Cu.M/hr.	20 Cu.M/hr.
Quantities	2 sets	2 sets

(4) Vibrator

A. Internal Vibrator

(a)	Brand	Wacker (West Germany)	
	Power	Electrical Converter	8 sets
	Vibrator	0.3Y/42	3 sets
		0.4Y/42	10 sets
		1.1Y/42	4 sets
(b)	Brand	Tokuden BF (Japan)	
	Power	Electrical Motor	17 sets
		Gasoline Engine	9 sets

Vibrator	BF-45-6	21 sets
	BF-45-5	11 sets
	BF-45-4	3 sets
	BF-32-6	3 sets

B. Outer Vibrator

Brand	Tokuden FV (Japan)	
Motor	FV-600	18 sets

(5) Belt Conveyer

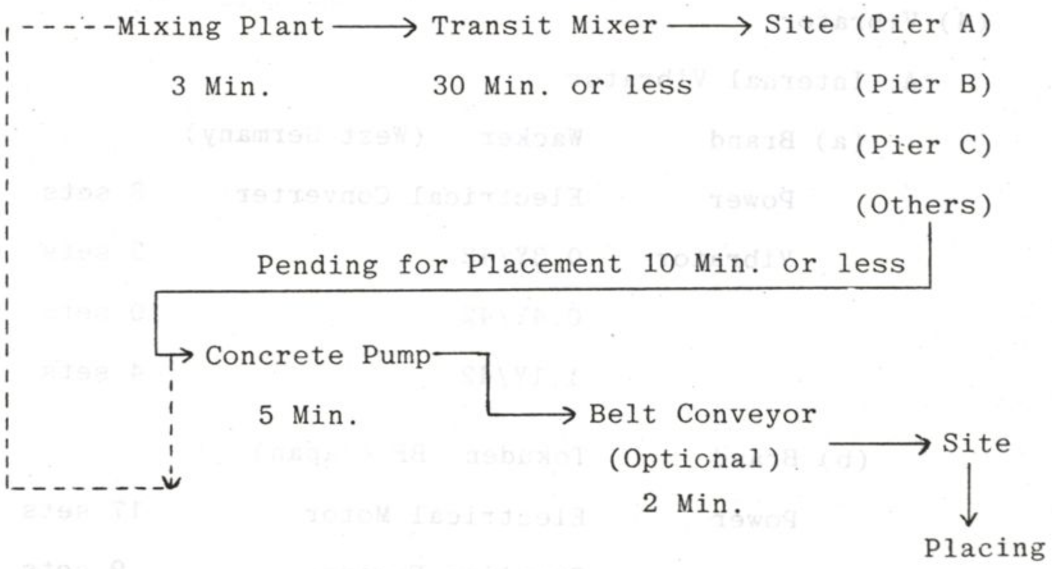
Brand	Yang-Shin Mech. Co. (Local)	
Power	Electrical Motor 1 HP	
Size	40cm x 7m - 9m	
Quantity		20 sets

2. Concrete Production and Delivery

Concrete was produced by the use of field orders. The ticket system was applied to control of quality and quantity (Table 9).

Well-trained technicians operated this plant under the supervision of TAFCB inspectors.

One-round trip typical concrete delivery was approximately 30 minutes as shown below.



Concrete samples were taken at the job site and were cured and tested according to ASTM C31-69. Test results are shown in Appendix 3, Concrete Quality Control Chart.

3. Placing

Properly placed concrete should be free of segregation, motar in intimate contact with the coarse aggregate, the reinforcements and other embedded parts.

The concrete pouring sequence and time control was studied and discussed prior to concrete placing and were strictly supervised. All necessary steps during placing were studied in order to avoid nonconsolidated areas or over vibration.

Fluid concrete was used for the superstructures of the main bridge. There were no significant defects such as bleeding, abnormal setting, dry shrinkage, honey-combs etc. found in these structures.

The slump loss of ready mixed concrete is most likely to be troublesome during hot, dry and/or windy weather, especially when the concrete is hauled for a long distance or transported by means of a long chute or conveyor. Slump loss versus time at different temperatures was measured during placement of mix number 350-20-18. The results are shown as follows:

Mix No.	Target Slump	Time after mixed (Min.)				Temp.
	(cm.)	0	15	30	45	(°C)
350-20-18	18	20 cm	15 cm	10 cm	6 cm	34
	18	18 cm	12 cm	9 cm	6 cm	30
	18	16 cm	10 cm	8 cm	5 cm	24

Therefore, in order to maintain the workability of concrete under adverse weather condition, it is better to place the concrete within 40 minutes after it is mixed. In general, this has achieved reasonable workability during construction.

Work schedules are shown on Tables 10 and 11. Table 10 gives the normal concrete pouring schedule after improvements in the work process and Table 11 is the shortest case.

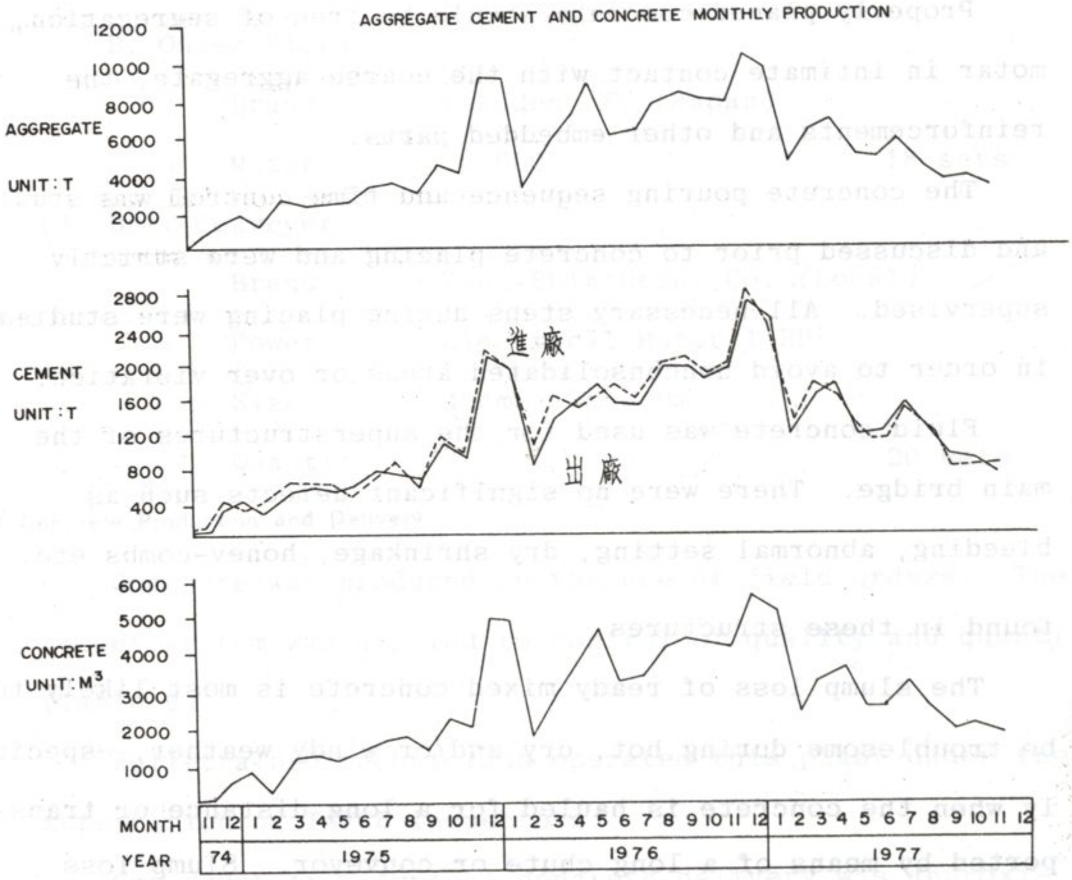


TABLE 10 WORKING CYCLE FOR EACH BOX GIRDER SEGMENT

Working Item \ Day	1			2			3			4			5			6			1		
Hour	12	18	24	12	18	24	12	18	24	12	18	24	12	18	24	12	18	24	12	18	24
Concrete Placing	■	■																	■	■	
Curing			■	■	■	■	■														
Prestressing								■													
Wagon Moving									■												
Surveying										■	■										
Rebar Installation												■	■	■	■						
Inner Forming & Sheathing																■	■	■			

Ⅵ. Field Quality Control and Evaluation

(Refer to Appendix 2, Quality Control Chart.)

Notation:

V: Coefficient of Variation

X: Average Strength of Concrete

fcr: Required Strength of Concrete

fc': Designed Strength of Concrete

Rm: Mean Range

1. Prestressed Precast Pile (2–9– to 2–9–4)

(1) The coefficients of variations (V) were all below 10% some were below 5%, which shows that the field control was very uniform and fairly good. The lower the coefficient, the lower the average strength will be. Therefore, the use of an economical lower cement factor mix will achieve the strength requirement. Although the average strength (X = 451 kg/cm^2) is lower than the required strength (fcr = 483 kg/cm^2), the chance of falling below the lower limit is calculated as 1 in 75, based on the actual coefficients. Compared with the assumed possiblity of 1 in 5, this is actually far safe than required. We can therefore consider a revision in the mix proportions to obtain a more economical mix, emphasizing the value of good quality control. If the coefficients remain the same, the average strength would need to be only 432 kg/cm^2 to assure no more than 1 failure in 5.

(2) There are 8 sets with an average strength lower than fc'. Thus all the test results are considered acceptable.

(3) Rm = 0.0564 x fcr = 27.1 kg/cm^2. The range chart shows highly irregular variations that do not fall within batch test requirements. The reasons for the greater R may be caused by improper cylinder fabrication, curing and/or testing discrepencies. However, during the period of August and September of 1975, the R values were within the required variation limits.

2. Prestressed Girder (2–1–1 to 2–1–6)

(1) The coefficient of variations (V) are all below 10%, which shows very good field control. The average strength ($\overline{X}$ = 443 kg/cm^2) is higher than the required strength (fcr = 402.5 kg/cm^2), but V is lower than the assumed value (15%). The chance of falling below this lower limit and is calculated as 1 in 2000. It is very clear that the mix is not economical and that revision is needed.

(2) Moving average strengths are greater than fc'. In early and later stages, the moving average was uniform but the middle stage varied, which shows inadequate control.

(3) Rm = 0.0564 x fcr = 22.6 kg/cm^2. R values are high and irregular, especially in the 110 - 240 set and the 290 - 320 set. This may be caused by improper cylinder fabrication, curing and/or testing discrepencies.

3. Bridge Deck, Bridge Pier (Pavement) (2–10–1 to 2–10–2) (2–11–1 to 2–11–4)

(1) The coefficient of variations (V) are all below 10%. This indicates that the field operation has good control. The average strength of Mix No. 350-20-13 ($\overline{X}$ = 426.6 kg/cm^2) is just a little greater than fcr but Mix No. 350-20-18 ($\overline{X}$ = 501.8 kg/cm^2) is considerably greater than fcr. The chances of falling below the lower limit are 1 in 714 and 1 in 10,000 respec-

tively, modified mix proportions to meet the requirements of economy were necessary.

(2) The moving average strength varies and is greater than fc' indicating inadequate strength control.

(3) Rm = 0.0564 x fcr = 22.6 kg/cm^2. R values are irregular but still within batch test control requirements of Mix No. 350-20-13 and they are better than Mix No. 350-20-18. Care must be taken in sampling and testing in order to lower the range.

4. Box Girders, Hinge (2–12–1 and 2–13–1)

(1) The coefficient of variations (V) are all below 10% which shows good field control. The average strength of both mixes are very close to fcr. This shows that the mix designs are very good.

(2) The moving average strength shows stability and will be considered as well controlled.

(3) Rm = 0.0564 x fcr = 22.6 kg/cm^2, are within batch test control requirements for both mixes, but not within test control requirements due to carelessness in the later stage of Mix No. 350-20-15.

5. Foundation, Caissons (2–14–1 to 2–14–4)

(1) Most of the coefficients of variation (V) are below 10%, showing fair to good field control.

(2) The average strength ($\overline{X}$ = 346 kg/cm^2) is only a little higher than fcr. This mix is a well designed.

(3) The moving average strength chart indicates that the strength of test sets in early stages are more uniform than that of those in later stages.

(4) Rm = 0.0564 x fcr = 18.1 kg/cm^2. R values are irregular which shows that the mixing or sampling is not so good.

(5) The chance of falling below the lower limit is calculated as 1 in 90. We can therefore consider a revision to the mix proportions decreasing the strength.

6. Foundation, Bridge Pier, Bridge Deck, Railing (2–3–1 to 2–3–5) (2–15–1 to 2–15–2)

(1) The coefficient of variations (V) are near 10%, showing fair to good field control.

(2) The average strengths ($\overline{X}$ = 345, 342 kg/cm^2) are both greater than fcr. Under good variation control, the mix is uneconomical.

(3) The chance of falling below lower limit requirements is calculated as 1 in 2500. A mix proportion revision is necessary to obtain a more economical mix.

(4) Rm = 15.5 kg/cm^2. In some sections, R values are too high. This shows poor test control.

7. Cast-in-place Concrete Pile by "Reversed Circulation"

Drilling Method (2-2-1 to 2-2-6 and 2-16-1 to 2-16-3)

(1) The coefficient of variations (V) are near 10% it shows the field control is fair to good.

(2) The chance of falling below lower limit requirements is calculated as 1 in 84. We can consider a revision to this mix proportion to lower the strength from 314 kg/cm^2 to 265 kg/cm^2 in order to assure a failure limit of 1 in 5.

(3) Rm = 0.0564 x fcr = 15.5 kg/cm^2. In some section R value is too high which shows that the within batch test control is poor. Care should be taken in cylinder fabrication, curing and testing to lower the R value.

8. Miscellaneous (2–4–1, 2–5–1, 2–6–1, 2–7–1, 2–8–1)

No evaluation.

VII. Conclusions

1. Concrete materials were well selected and controlled. Selection and control would have been more effective if the plant had its own finish screening deck in order to keep significantly undersize aggregate within the specified limits.
2. A better working system was established and implemented. Experienced laborers in well-organized teams worked on each section of concrete placement.
3. The coefficients of variation shown in the Q.C. charts are all very close to 10% with most under 10% and only a few slightly exceeding 10%.
4. A new type of admixture (Superplasticizer Mighty 150) was selected after intensive study and comparison. This contributed greatly to consolidating the construction period.
5. A series of dry shrinkage test results are shown in the appendixes. These indicate that the concrete is within the specified limits.
6. Concrete was designed to meet the required strength for tensioning after 60 hrs. However, the strength was tested by a field test cylinder prior to tension of tendons. If the strength (260 kg/cm^2) was reached prior to the designed 60 hrs., then tensioning could be carried out. During the winter season, the strength did not reach the requirement within the 60 hrs. time limit and tensioning was postponed until the design value was reached as determined by the test cylinders. The concrete was designed at the standard temp. of 23°C. The cooler temperature prolonged the time required from placement to specified strength. The relationship between concrete harden-

ing time and temperature is very important. Hydration of concrete is affected by temperature change; it accelerates with higher temperatures and slows colder temperatures even with the addition of a superplasticizer.

"Saul's Formula" is applied here.

$R = At\ (t + 10)$

R = Degree of heat of hydration

At = Age of concrete (days)

t = Temperature of curing

(Example)

If the concrete mix is designed at 23°C, the designed strength can reach 260 kg/cm^2 at 60 hrs. (2½ days). If the temperature falls to 6°C:

$R = 2.5\ (23 + 10) = 82.5$

$At = \frac{82.5}{6+10} = 5.15$ days (123.6 hrs.)

7. Surface crazing was found in some areas of the top layer concrete of the superstructure on the main bridge although it was covered with a dampened cloth during concrete curing. The crazing consisted of many shallow, random cracks in every direction. This crazing may have been cuased by the rapid loss of moisture from the fresh concrete due to wind, excessive finishing or temperature change. Minor crazing of this nature is sometimes unpreventable and does not harm to the quality of the concrete.
8. This is the first project in the Republic of China in which the contractor had his own laboratory and was able to perform his own quality control.

Ⅷ. Appendixes

1. Drying Shrinkage Test Results

2. Concrete Quality Control Chart

3. Temperature Record

4. Cement Test Report

1. Drying Shrinkage Test Results

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Addirive
673	202	580	724 905 181	30	39.35	10	PC

Date: July 31, 1975

Average Strength at 28 days = 502 Kg/cm²

= 7128 psi

Mea-suring Point (MM) / Set No. / Age(Day)		1	2	3	4	5	6
In Water	1	6.60	4.14	9.49	5.82	7.23	10.82
	7	6.75	4.23	9.58	5.89	7.32	10.92
In Air	14	6.56	4.08	9.48	5.78	7.21	10.77
	21	6.48	4.00	9.38	5.68	7.10	10.67
	28	6.43	3.98	9.33	5.65	7.07	10.66
Effective Length		367	364	369	366	367	369
Dry Shrinkage Value at Different Stages	1-7	+0.04087	+0.02472	+0.02439	+0.01912	-0.02452	+0.02710
	1-14	-0.01089	-0.01648	-0.00271	-0.01092	-0.00544	-0.01355
	1-21	-0.03269	-0.03648	-0.02981	-0.03825	-0.03542	-0.04065
	1-28	-0.04632	-0.04395	-0.04336	-0.04644	-0.04359	-0.04336

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.0445033 %

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500	166.2	724	796 995 199	33.2	42.4	12	Mighty C x 1%

Date: Aug. 25, 1975

Average Strength at 28 days = 449.5 Kg/cm²

= 6383 psi

Mea-suring Point (MM) / Set No. / Age(Day)		1	2	3	4	5	6
In Water	1	7.92	6.14	6.23	7.25	8.66	7.45
	7	8.00	6.19	6.28	7.30	8.69	7.58
In Air	14	7.83	6.05	6.16	7.17	8.57	7.37
	21	7.78	6.01	6.11	7.12	8.53	7.32
	28	7.73	5.97	6.07	7.08	8.48	7.28
Effective Length		36.7 cm	36.8 cm	36.6 cm	36.7 cm	36.9 cm	36.7 cm
Dry Shrinkage Value at Different Stages	1-7	+0.02179	+0.01358	+0.01366	+0.01362	+0.00813	+0.03542
	1-14	-0.02452	-0.02445	-0.01912	-0.02179	-0.02439	-0.02179
	1-21	-0.03814	-0.03532	-0.03278	-0.03542	-0.03523	-0.03542
	1-28	-0.05177	-0.04619	-0.04371	-0.04632	-0.04878	-0.04632

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.0471816 %

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500	172.5	717	789 986 197	34.5	42.4	13	POLFAINE C x 1%

Date: Aug. 27, 1975

Average Strength at 28 days = 448 Kg/cm²

= 6362 psi

Mea-suring Point (MM) / Set No. / Age(Day)		1	2	3	4	5	6
In Water	1	8.24	7.57	7.87	8.15	8.21	6.99
	7	8.30	7.61	7.90	8.19	8.25	7.02
In Air	14	8.15	7.47	7.77	8.05	8.11	6.88
	21	8.11	7.43	7.72	8.02	8.07	6.85
	28	8.07	7.39	7.67	7.96	8.03	6.80
Effective Length		36.7cm	36.8 cm	36.8 cm	36.6 cm	36.7 cm	36.6 cm
Dry Shrinkage Value at Different Stages	1-7	+0.01634	+0.01086	+0.00815	+0.01092	+0.01089	+0.00819
	1-14	-0.02452	-0.02717	-0.02717	-0.02732	-0.02724	-0.03005
	1-21	-0.03542	-0.03804	-0.04076	-0.03551	-0.03814	-0.03825
	1-28	-0.04632	-0.04891	-0.05434	-0.05191	-0.04904	-0.05191

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.050405 %

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500	184.9	704	774 967 193	37	42.4	12	NL 4000 C x 1%

Date: Aug. 28, 1975

Average Strength at 28 days = 442 Kg/cm²

= 6276 psi

Mea-suring Point (MM) / Set No. / Age(Day)		1	2	3	4	5	6
In Water	1	7.41	5.76	5.60	7.20	8.21	6.31
	7	7.43	5.77	5.62	7.21	8.23	6.33
In Air	14	7.28	5.62	5.48	7.09	8.08	6.18
	21	7.23	5.57	5.44	7.02	8.02	6.14
	28	7.23	5.57	5.42	7.02	8.02	6.14
Effective Length		36.9 cm	36.6 cm	36.8 cm	36.7 cm	36.9 cm	36.7 cm
Dry Shrinkage Value at Different Stages	1-7	+0.00542	+0.00273	+0.00543	+0.00272	+0.00542	+0.00544
	1-14	-0.03523	-0.03825	-0.03260	-0.02997	-0.03523	-0.03542
	1-21	-0.04847	-0.05191	-0.04347	-0.04904	-0.05149	-0.04632
	1-28	-0.04878	-0.05191	-0.04891	-0.04904	-0.05149	-0.04632

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.0494083 %

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
450	166.2	757	1003 1cm 80% 2cm 20%	36.9	43.3	12	Mighty C x 1%

Date: Aug. 29, 1975

Average Strength at 28 days = 437.5 Kg/cm²
= 6213 psi

Measuring Point (MM) Age(Day)	Set No.	1	2	3	4	5	6
In Water	1	8.24	8.93	8.76	7.47	5.64	5.89
	7	8.26	8.98	8.80	7.51	5.64	5.96
In Air	14	8.12	8.84	8.66	7.36	5.49	5.81
	21	8.08	8.80	8.62	7.34	5.44	5.77
	28	8.07	8.79	8.61	7.33	5.43	5.74
Effective Length		36.7 cm	36.8 cm	36.8 cm	36.6 cm	36.6 cm	36.6 cm
Dry Shrinkage Value at Different Stages	1-7	+0.00544	+0.01358	+0.01086	+0.01092	0	+0.01912
	1-14	-0.03269	-0.02445	-0.02717	-0.03005	-0.04098	-0.02185
	1-21	-0.04359	-0.03532	-0.03804	-0.03551	-0.05464	-0.03279
	1-28	-0.04632	-0.03804	-0.04076	-0.03825	-0.05737	-0.04098

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.04362%

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
475	201.1	655	995 796 199	42.3	40	13	P C

Date: Sept. 24, 1975

Average Strength at 28 days = 394 Kg/cm²
= 5595 psi

Measuring Point (MM) Age(Day)	Set No.	1	2	3	4	5	6
In Water	1	6.92	7.61	6.56			
	7	6.92	7.61	6.56			
In Air	14	6.83	7.51	6.46			
	21	6.76	7.47	6.40			
	28	6.77	7.46	6.45			
Effective Length		36.7 cm	36.7 cm	36.7 cm			
Dry Shrinkage Value at Different Stages	1-7	0.00000	0.00000	0.00000			
	1-14	-0.02452	-0.02724	-0.02724			
	1-21	-0.04359	-0.03814	-0.04359			
	1-28	-0.04087	-0.04087	-0.02997			

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.0372366 %

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
475 Type I Chutung	201.1	655	995 796 199	42.3	40	12.5	P C

Date: Sept. 24, 1975

Average Strength at 28 days = 405 Kg/cm²
= 5751 psi

Measuring Point (MM) Age(Day)	Set No.	1	2	3	4	5	6
In Water	1	6.58	7.17	8.89			
	7	6.57	7.15	8.88			
In Air	14	6.47	7.05	8.78			
	21	6.43	7.01	8.74			
	28	6.42	7.01	8.74			
Effective Length		36.8 cm	36.7 cm	36.7 cm			
Dry Shrinkage Value at Different Stages	1-7	-0.00271	-0.00544	-0.00272			
	1-14	-0.02989	-0.03269	-0.02997			
	1-21	-0.04076	-0.04359	-0.04087			
	1-28	-0.04347	-0.04359	-0.04087			

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.0426433 %

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500	172.2	718	986 789 197	34.4	42.4	16	Mighty C x 1%

Date: Sept. 25, 1975

Average Strength at 28 days = 465 Kg/cm²
= 6603 psi

Measuring Point (MM) Age(Day)	Set No.	1	2	3	4	5	6
In Water	1	7.78	8.79	6.79			
	7	7.78	8.79	6.79			
In Air	14	7.65	8.70	6.65			
	21	7.60	8.66	6.60			
	28	7.60	8.65	6.59			
Effective Length		36.8 cm	36.7 cm	36.7 cm			
Dry Shrinkage Value at Different Stages	1-7	0.00000	0.00000	0.00000			
	1-14	-0.03532	-0.02452	-0.03814			
	1-21	-0.04891	-0.03542	-0.05177			
	1-28	-0.04891	-0.03814	-0.05449			

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.04718 %

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500 Type I Chutung	172.2	718	986 789 197	34.4	42.4	16	Mighty C x 1%

Date: Sept. 25, 1975

Average Strength at 28 days = 480 Kg/cm^2
= 6816 psi

Measuring Point (MM) / Age (Day)	Set No.	1	2	3	4	5	6
In Water	1	7.39	5.94	7.15			
	7	7.40	5.93	7.15			
In Air	14	7.30	5.83	7.09			
	21	7.20	5.79	7.01			
	28	7.20	5.78	7.00			
Effective Length		36.7 cm	36.5 cm	36.8 cm			
Dry Shrinkage Value at Different Stages	1-7	+0.00272	-0.00273	0.00000			
	1-14	-0.02452	-0.03013	-0.01630			
	1-21	-0.05177	-0.04109	-0.03804			
	1-28	-0.05177	-0.04383	-0.04076			

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.0454533 %

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
550	170.9	651	989 {791 198	31.1	40	14	Mighty C x 1%

Date: Oct. 2, 1975

Average Strength at 28 days = 438 Kg/cm^2
= 6220 psi

Measuring Point (MM) / Age (Day)	Set No.	1	2	3	4	5	6
In Water	1	7.10	5.88	6.32	6.59		
	7	7.09	5.89	6.30	6.56		
In Air	14	7.01	5.74	6.19	6.47		
	21	7.00	5.74	6.18	6.45		
	28	6.95	5.75	6.18	6.44		
Effective Length		36.7cm	36.7cm	36.75cm	36.5cm		
Dry Shrinkage Value at Different Stages	1-7	-0.00272	+0.00272	-0.00544	-0.00821		
	1-14	-0.02452	-0.03814	-0.03537	-0.03187		
	1-21	-0.02724	-0.03814	-0.03809	-0.03835		
	1-28	-0.04087	-0.03542	-0.03809	-0.04109		

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.0388675.

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500 Suo	158.2	719	996 1cm 80% 2cm 20%	31.6	42.2	15	Mighty 150 C x 1.5%

Date: Oct. 18, 1975

Average Strength at 28 days = 459.5 Kg/cm^2
= 6525 psi

Measuring Point (MM) / Age (Day)	Set No.	1	2	3	4	5	6
In Water	1	6.21	6.39	6.80	6.31	5.87	7.39
	7	6.20	6.39	6.82	6.33	5.88	7.40
In Air	14	6.17	6.37	6.78	6.27	5.83	7.38
	21	6.10	6.27	6.68	6.23	5.77	7.29
	28	6.05	6.21	6.63	6.16	5.70	7.24
Effective Length		367	366	369	366	367	367
Dry Shrinkage Value at Different Stages	1-7	-0.00272	-0.0000	+0.00540	+0.0055	+0.0027	+0.0027
	1-14	-0.0109	-0.0055	-0.0054	-0.0109	-0.0109	-0.0027
	1-21	-0.02997	-0.03278	-0.03252	-0.02185	-0.02724	-0.02724
	1-28	-0.04359	-0.04918	-0.04607	-0.04098	-0.04632	-0.04087

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.04450

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500 Type I Chutung	172.2	718	986 1cm 80% 2cm 20%	34.4	42.4	9.5	Mighty 150 C x 1%

Date: Oct. 21, 1975

Average Strength at 28 days =
=

Measuring Point (MM) / Age (Day)	Set No.	1	2	3	4	5	6
In Water	1	5.46	6.78	6.83	5.53	7.66	7.22
	7	5.45	6.80	6.84	5.51	7.64	7.20
In Air	14	5.41	6.72	6.77	5.50	7.58	7.15
	21	5.33	6.66	6.72	5.40	7.53	7.07
	28	5.34	6.66	6.73	5.41	7.54	7.08
Effective Length		366	366	368	367	369	368
Dry Shrinkage Value at Different Stages	1-7	-0.00273	+0.00546	+0.00271	-0.00544	-0.00542	-0.00543
	1-14	-0.01366	-0.01641	-0.01630	-0.00817	-0.02168	-0.01902
	1-21	-0.03551	-0.03280	-0.02989	-0.03542	-0.03523	-0.04080
	1-28	-0.03278	-0.03280	-0.02717	-0.03269	-0.03252	-0.03804

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.03267

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500 Type I Chutung	203.4	645	979 1cm 80% 2cm 20%	40.7	40	11.5	P C

Date: Oct. 30, 1975

Average Strength at 28 days = 483 Kg/cm^2

= 6859 psi

Measuring Point (MM) \ Set No. / Age(Day)		1	2	3	4	5	6
In Water	1	4.03	7.78	6.96	6.60	7.62	5.89
	7	4.02	7.77	6.95	6.61	7.51	5.90
In Air	14	3.91	7.64	6.82	6.50	7.51	5.78
	21	3.87	7.62	6.79	6.47	7.48	5.75
	28	3.84	7.60	6.75	6.44	7.44	5.71
Effective Length		366	368	367	369	367.5	368
Dry Shrinkage Value at Different Stages	1-7	-0.00273	-0.00271	-0.00272	+0.00271	-0.00272	+0.00271
	1-14	-0.03278	-0.03804	-0.03814	-0.02710	-0.02993	-0.02989
	1-21	-0.04371	-0.04347	-0.04632	-0.03523	-0.03809	-0.03804
	1-28	-0.05191	-0.04891	-0.05722	-0.04336	-0.04897	-0.04891

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.04988

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500 Type I Chutung	193.2	645	979 1cm 80% 2cm 20%	38.6	40	15	5 L C x 0.25%

Date: Nov. 8, 1975

Average Strength at 28 days = 431 Kg/cm^2

= 6120 psi

Measuring Point (MM) \ Set No. / Age(Day)		1	2	3	4	5	6
In Water	1	7.80	6.74	8.17	8.27	6.37	7.82
	7	7.84	6.78	8.21	8.33	6.43	7.84
In Air	14	7.67	6.63	8.06	8.19	6.27	7.70
	21	7.65	6.60	8.02	8.15	6.22	7.67
	28	7.54	6.49	7.94	8.04	6.14	7.58
Effective Length		369	368	367	368	366	367
Dry Shrinkage Value at Different Stages	1-7	+0.0108	+0.0109	+0.0109	+0.0163	+0.0164	+0.0054
	1-14	-0.0352	-0.0299	-0.0300	-0.0217	-0.0273	-0.0327
	1-21	-0.0407	-0.0380	-0.0409	-0.0326	-.0410	-0.0409
	1-28	-0.0705	-0.0679	-0.0627	-0.0625	-0.0628	-0.0654

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.0653

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500 Modified Type I Chutung	203.4	645	1cm 783 2cm 196	40.7	40	13	-

Date: June 21, 1976

Average Strength at 28 days = 394.5 Kg/cm^2

= 5602 psi

Measuring Point (MM) \ Set No. / Age(Day)		1	2	3	4	5	6
In Water	1	5.96	6.27	6.56	4.66	6.90	
	7	5.93	6.25	6.55	4.63	6.87	
In Air	14	5.86	6.25	6.52	4.64	6.85	
	21	5.86	6.22	6.50	4.59	6.80	
	28	5.795	6.15	6.43	4.54	6.78	
Effective Length		368	366.5	366.5	366	369	
Dry Shrinkage Value at Different Stages	1-7	-0.008152	-0.005457	-0.002729	-0.008196	-0.008130	
	1-14	-0.02717	-0.005457	-0.01091	-0.005465	-0.01355	
	1-21	-0.02717	-0.01364	-0.01637	-0.019126	-0.0270	
	1-28	-0.04484	-0.03274	-0.03547	-0.03279	-0.03252	

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.035672

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500 Modified Type I Chutung	203.4	645	979	40	40.7	13	

Date: July 16, 1976

Average Strength at 28 days =

=

Measuring Point (MM) \ Set No. / Age(Day)		1	2	3	4	5	6
In Water	1	5.19	7.69	6.165	4.87	5.96	6.725
	7	5.185	7.69	6.165	4.85	5.945	6.705
In Air	14	5.115	7.625	6.10	4.795	5.86	6.625
	21	5.105	7.60	6.10	4.78	5.86	6.62
	28	5.10	7.61	6.105	4.78	5.86	6.62
Effective Length		36.6	367	367	363.5	365.5	366
Dry Shrinkage Value at Different Stages	1-7	-0.001366	0	0	-0.005502	-0.004103	-0.005464
	1-14	-0.020492	-0.017711	-0.017711	-0.020632	-0.02736	-0.027322
	1-21	-0.02322	-0.024523	-0.017711	-0.024759	-0.027359	-0.028689
	1-28	-0.02459	-0.021798	-0.016348	-0.024759	-0.027359	-0.028689

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.023923

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
500 Modified Type I Chutung	203.4	645	979	40.7	40	13	Mighty C x 0.7%

Date: July 25, 1976

Average Strength at 28 days =

=

Set No. Measuring Point (MM) Age(Day)		1	2	3	4	5	6
In Water	1	6.315	7.32	7.645	7.07	5.485	5.945
	7	6.305	7.325	7.635	7.075	5.495	5.945
In Air	14	6.30	7.325	7.63	7.07	5.48	5.94
	21	6.29	7.30	7.62	7.055	5.46	5.935
	28	6.29	7.285	7.61	7.035	5.44	5.925
Effective Length		366	367.5	366	366.5	366	366
Dry Shrinkage Value at Different Stages	1-7	-0.002732	+0.001361	-0.002732	+0.001364	+0.002732	+0
	1-14	-0.004098	+0.001361	-0.004098	+0	-0.001366	-0.001366
	1-21	-0.006831	-0.005442	-0.006831	-0.004093	-0.006831	-0.002732
	1-28	-0.006831	-0.009524	-0.009563	-0.009550	-0.012296	-0.005464

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.008871

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
450	176	696	1cm 811 2cm 203	39.1	41	12	Mighty 150 C x 1%

Date: Oct. 28, 1976

Average Strength at 28 days =

=

Set No. Measuring Point (MM) Age(Day)		1	2	3	4	5	6
In Water	1	4.43	7.73	5.02	2.62	5.35	
	7	4.41	7.72	5.00	2.585	5.22	
In Air	14	4.40	7.70	4.95	2.54	5.155	
	21	4.39	7.65	4.92	2.52	5.125	
	28	4.36	7.61	4.90	2.49	5.10	
Effective Length		366	368	365	364.5	365	
Dry Shrinkage Value at Different Stages	1-7	-0.005464	-0.002717	-0.005473	-0.009602	-0.03565	
	1-14	-0.008197	-0.008152	-0.019178	-0.021945	-0.053429	
	1-21	-0.059290	-0.021739	-0.027997	-0.027435	-0.061444	
	1-28	-0.019126	-0.032409	-0.032877	-0.035665	-0.068493	

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.037754

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
425	156	692	1cm 485 2cm 457	36.7	38.0	10	Mighty 150 C x 1%

Date: Oct. 29, 1976

Average Strength at 28 days =

=

Set No. Measuring Point (MM) Age(Day)		1	2	3	4	5	6
In Water	1	7.56	5.94	6.66	7.33	6.96	
	7	7.56	5.94	6.65	7.33	6.955	
In Air	14	7.51	5.38	6.62	7.30	6.925	
	21	7.49	5.88	6.605	7.26	6.91	
	28	7.45	5.85	5.66	7.23	6.86	
Effective Length		366	366	366	366	366	
Dry Shrinkage Value at Different Stages	1-7	0	0	-0.002732	0	-0.001366	
	1-14	-0.013661	-0.016394	-0.009290	-0.08197	-0.009563	
	1-21	-0.019126	-0.016394	-0.015017	-0.019126	-0.013661	
	1-28	-0.030055	-0.02459	-0.027322	-0.027322	-0.027322	

Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.027322

MIX DESIGN

Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg)	W/C %	S/A %	Slump (CM)	Additive
475	176	681	1cm 827 2cm 207	37.1	40	20	Mighty 150 C x 1%

Date: Oct. 29, 1976

Average Strength at 28 days =

=

Set No. Measuring Point (MM) Age(Day)		1	2	3	4	5	6
In Water	1	6.295	7.52	6.51	6.00	6.62	6.415
	7	6.295	7.525	6.46	5.95	6.62	6.61
In Air	14	6.265	7.48	6.40	5.93	6.59	6.58
	21	6.24	7.465	6.39	5.90	6.58	6.52
	28	6.21	7.43	6.35	5.85	6.54	6.47
Effective Length		366	367	366	365	346	346
Dry Shrinkage Value at Different Stages	1-7	0	+0.001362	-0.013661	-0.013691	0	-0.001366
	1-14	-0.008197	-0.010899	-0.030055	-0.01978	-0.008197	-0.009563
	1-21	-0.015027	-0.014986	-0.032787	-0.027397	-0.010929	-0.029563
	1-28	-0.023224	-0.024528	-0.043716	-0.041096	-0.021858	-0.039615

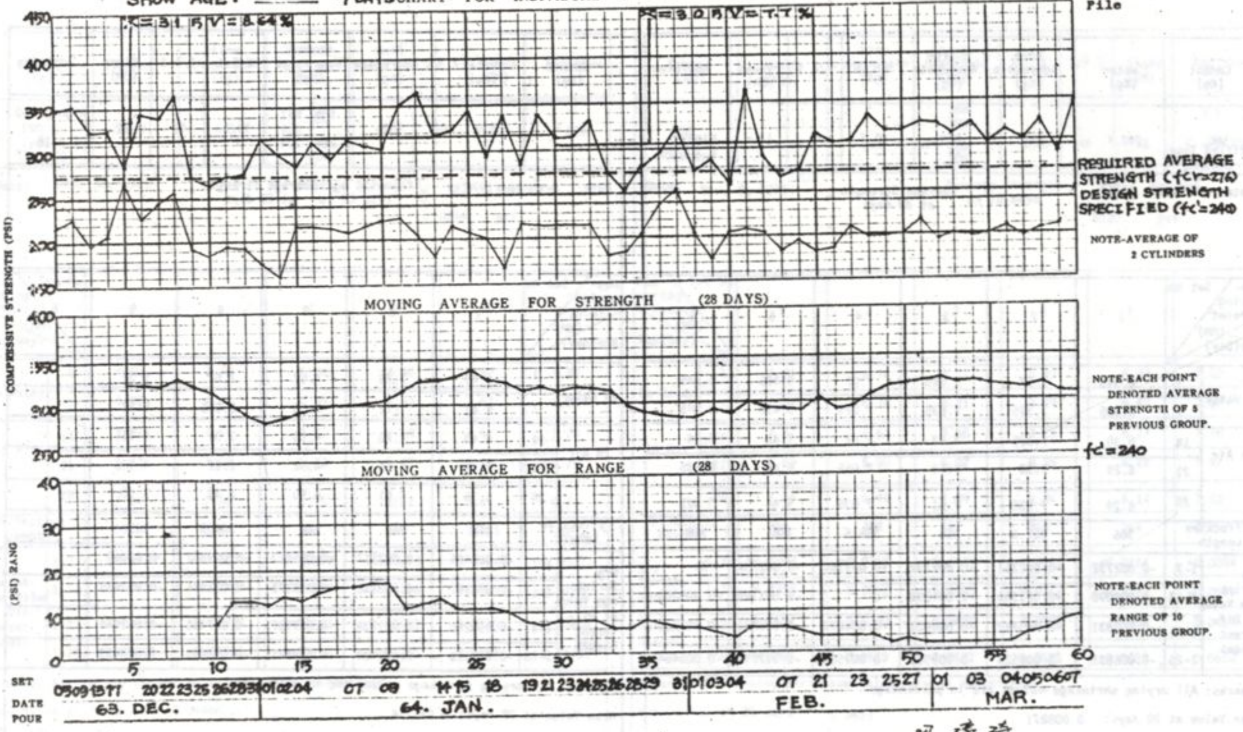
Remarks: All drying shrinkage values are in percentage.

Mean Value at 28 days: 0.032337

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. T.006-240-40-18
FEATURE 反循環樁 Cast-in-Place Concrete Pile

SHOW AGE: ——— 28 DAYS / ——— 7 DAYS CHART FOR INDIVIDUAL STRENGTH



APPROVED BY

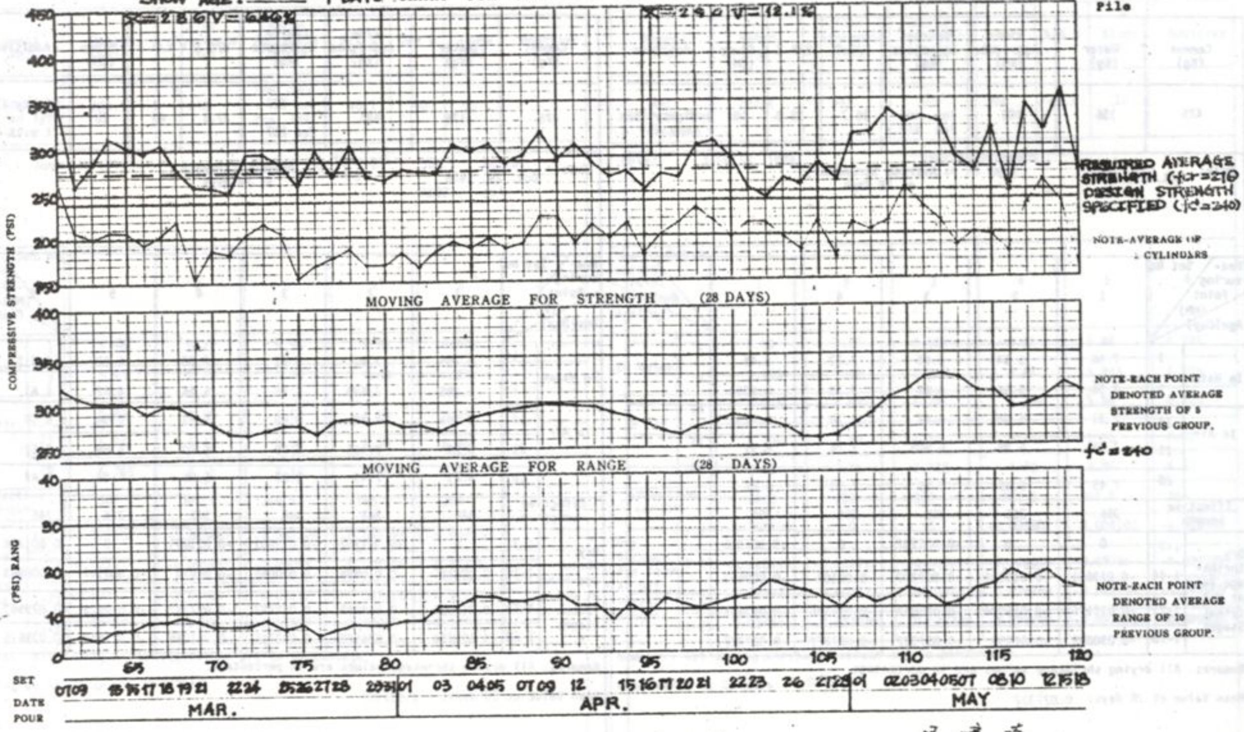
CHECKED BY 叶于平

REPORTED BY 梁瑋琦

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. T.006-240-40-18
FEATURE 反循環樁 Cast-in-Place Concrete Pile

SHOW AGE: ——— 28 DAYS / ——— 7 DAYS CHART FOR INDIVIDUAL STRENGTH



APPROVED BY

CHECKED BY 叶于平

REPORTED BY 梁瑋琦

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. T.006-240-40-18
FEATURE 反循環樁 Cast-in-Place Concrete Pile

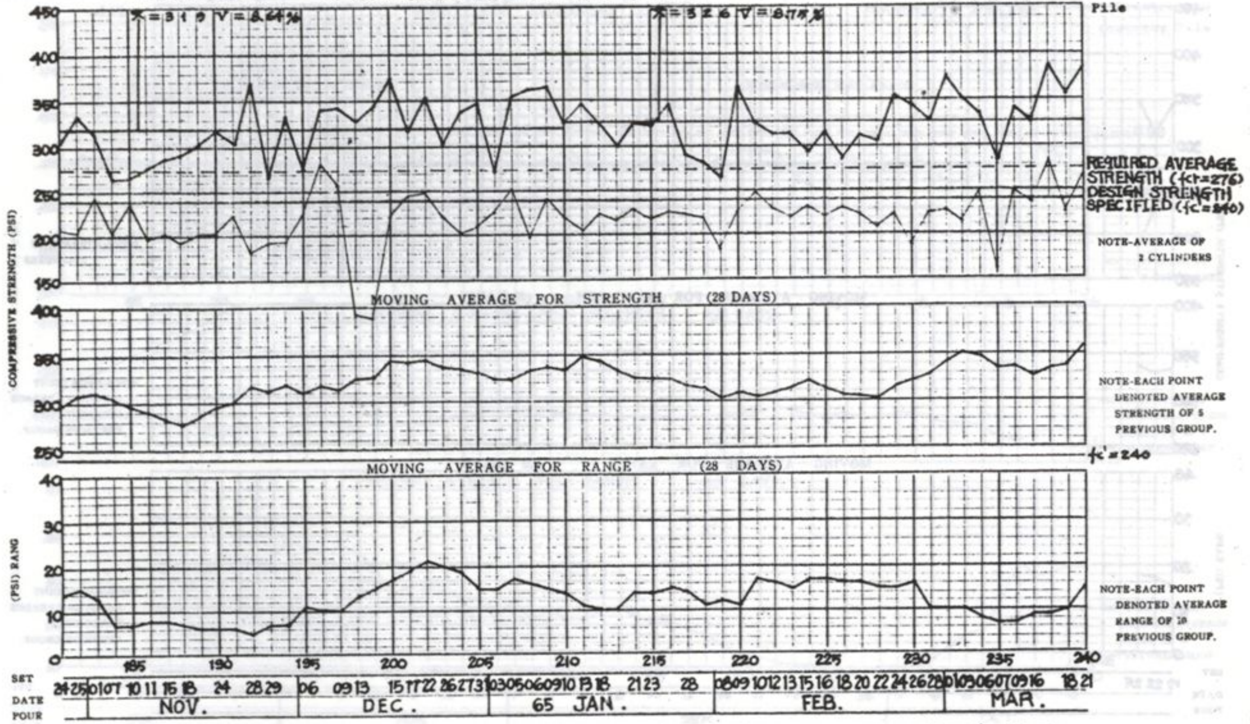
SHOW AGE: —— 28 DAYS —— 7 DAYS CHART FOR INDIVIDUAL STRENGTH

APPROVED BY
CHECKED BY
REPORTED BY

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. T.006-240-40-18
FEATURE 反循環樁 Cast-in-Place Concrete Pile

SHOW AGE: —— 28 DAYS —— 7 DAYS CHART FOR INDIVIDUAL STRENGTH

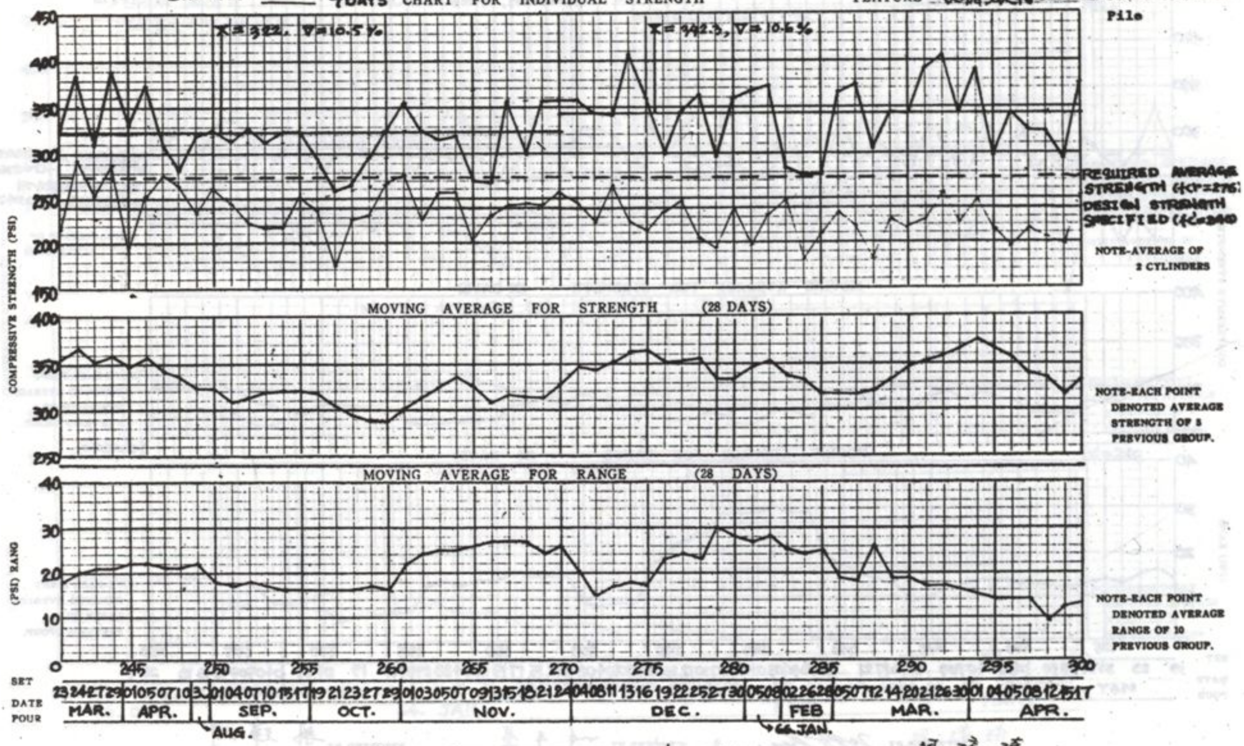


APPROVED BY
CHECKED BY
REPORTED BY

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. T.006-240-40-18
FEATURE [illegible] Cast-in-Place Concrete Pile

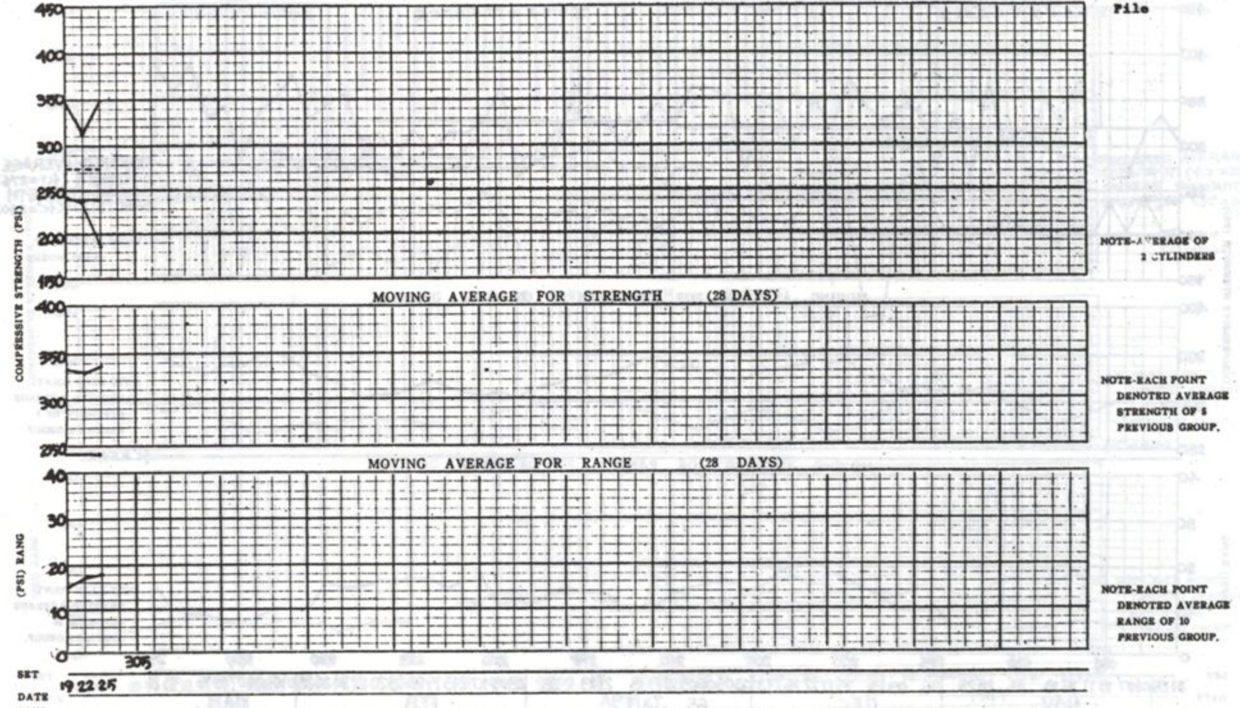
SHOW AGE: —— 28DAYS
—— 7DAYS



APPROVED BY
CHECKED BY
REPORTED BY

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. T006-240-40-18
FEATURE [illegible] Cast-in-Place Concrete Pile



APPROVED BY
CHECKED BY
REPORTED BY

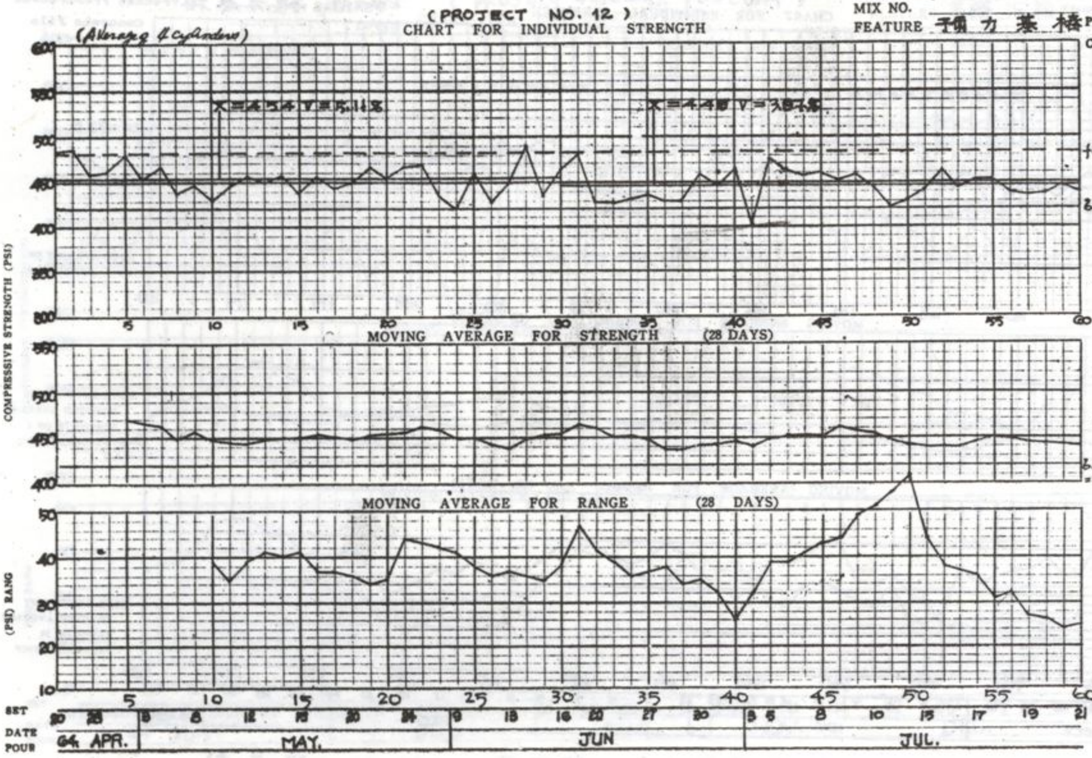
CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 12)

PLANT NO.

MIX NO.

FEATURE 預力基樁 Precast Prestressed Concrete Pile



f'cr=483

f28=420

NOTE-AVERAGE OF 2 CYLINDERS

f28 =420

NOTE-EACH POINT DENOTED AVERAGE STRENGTH OF 5 PREVIOUS GROUP.

NOTE-EACH POINT DENOTED AVERAGE RANGE OF 10 PREVIOUS GROUP.

APPROVED BY

CHECKED BY 張平華

REPORTED BY 沈正松

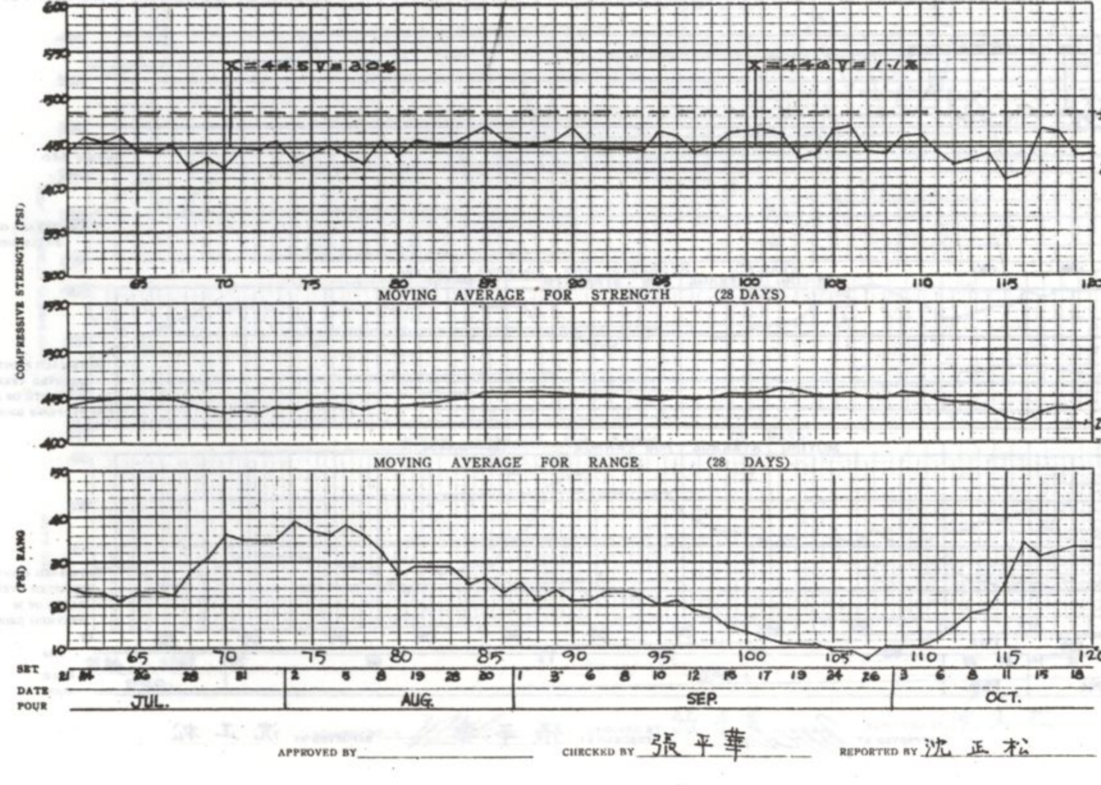
CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 12)

PLANT NO.

MIX NO.

FEATURE 預力基樁 Precast Prestressed Concrete Pile



f'cr=483

f28=420

NOTE-AVERAGE OF 2 CYLINDERS

f28 =420

NOTE-EACH POINT DENOTED AVERAGE STRENGTH OF 5 PREVIOUS GROUP.

NOTE-EACH POINT DENOTED AVERAGE RANGE OF 10 PREVIOUS GROUP.

APPROVED BY

CHECKED BY 張平華

REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 12)

PLANT NO.

MIX NO.

FEATURE 預力基樁 Precast Prestressed Concrete Pile

CHART FOR INDIVIDUAL STRENGTH

NOTE-AVERAGE OF 2 CYLINDERS

NOTE EACH POINT DENOTED AVERAGE STRENGTH OF 5 PREVIOUS GROUP.

NOTE-EACH POINT DENOTED AVERAGE RANGE OF 10 PREVIOUS GROUP.

APPROVED BY YJChen CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

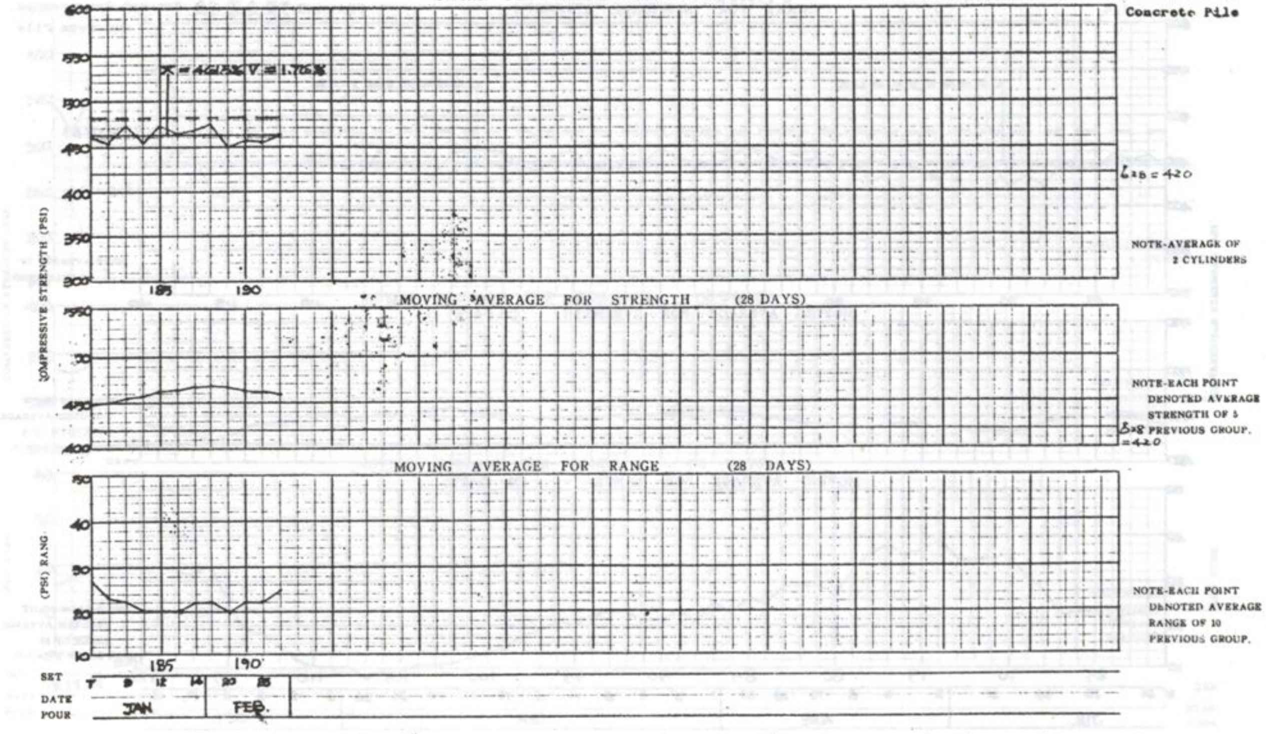
(PROJECT NO. 12)

PLANT NO.

MIX NO.

FEATURE 預力基樁 Precast Prestressed Concrete Pile

CHART FOR INDIVIDUAL STRENGTH



NOTE-AVERAGE OF 2 CYLINDERS

NOTE-EACH POINT DENOTED AVERAGE STRENGTH OF 5 PREVIOUS GROUP.

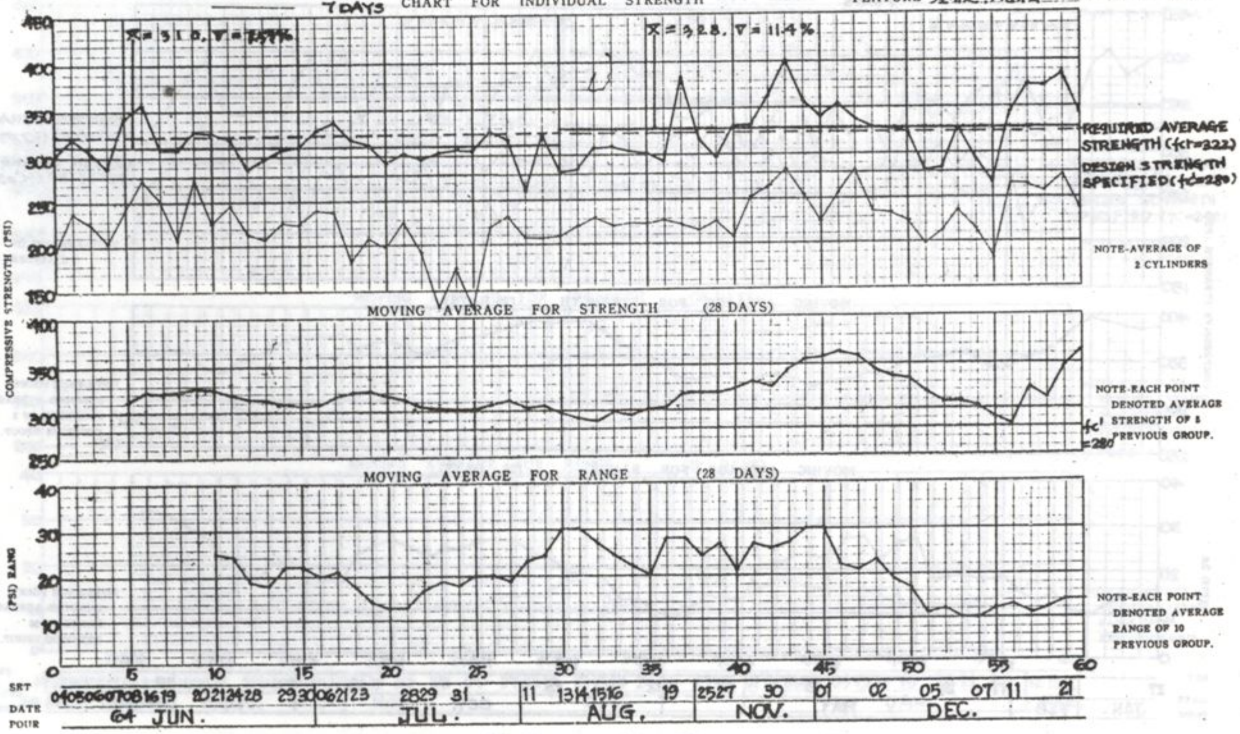
NOTE-EACH POINT DENOTED AVERAGE RANGE OF 10 PREVIOUS GROUP.

APPROVED BY YJChen CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO.
FEATURE 基礎.沉箱 Foundation & Caisson

(PROJECT NO. 12)
CHART FOR INDIVIDUAL STRENGTH

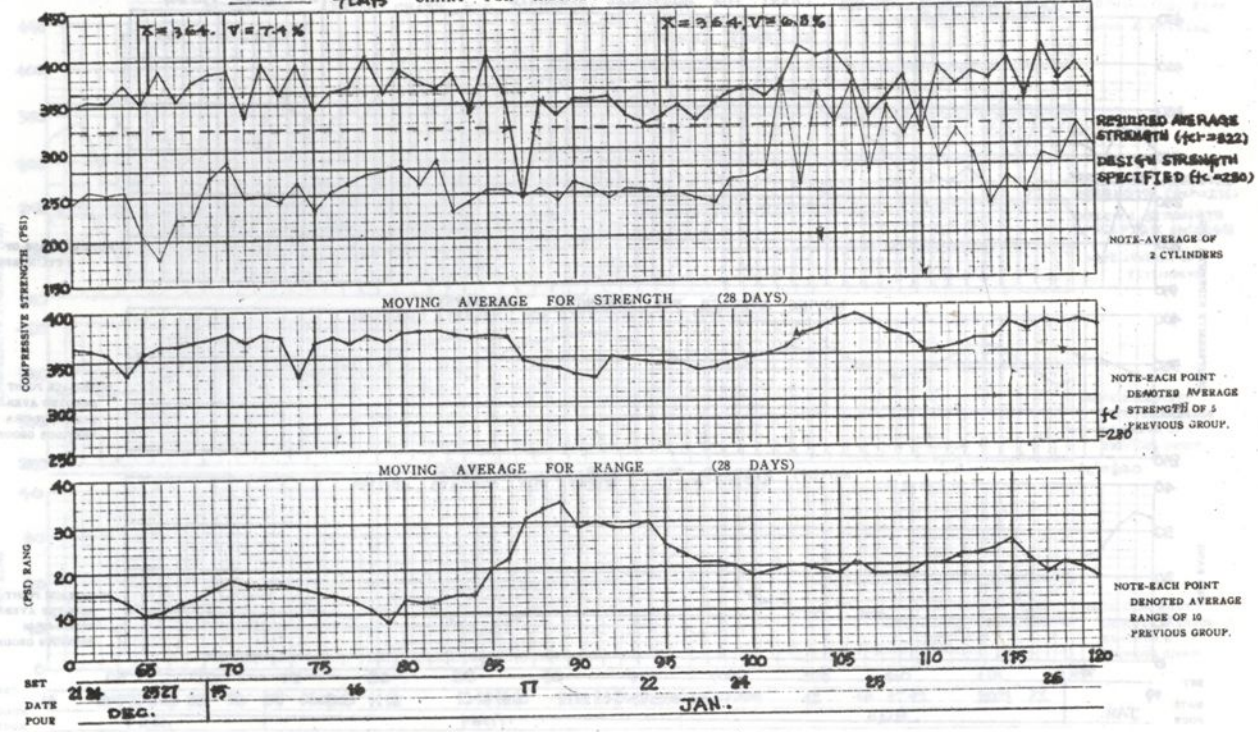


APPROVED BY
CHECKED BY 張平華
REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO.
FEATURE 基礎.沉箱 Foundation & Caisson

(PROJECT NO. 12)
CHART FOR INDIVIDUAL STRENGTH

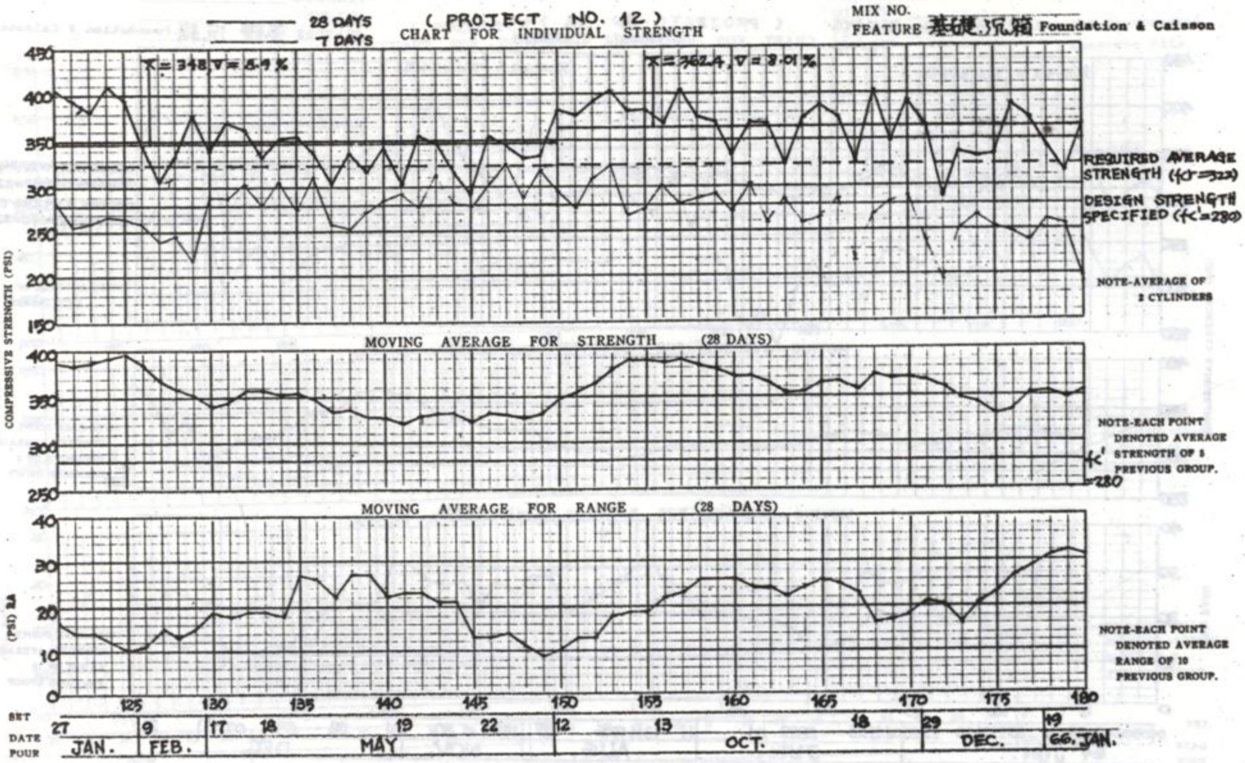


APPROVED BY
CHECKED BY 張平華
REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO.
FEATURE 基礎.沉箱 Foundation & Caisson

(PROJECT NO. 12)

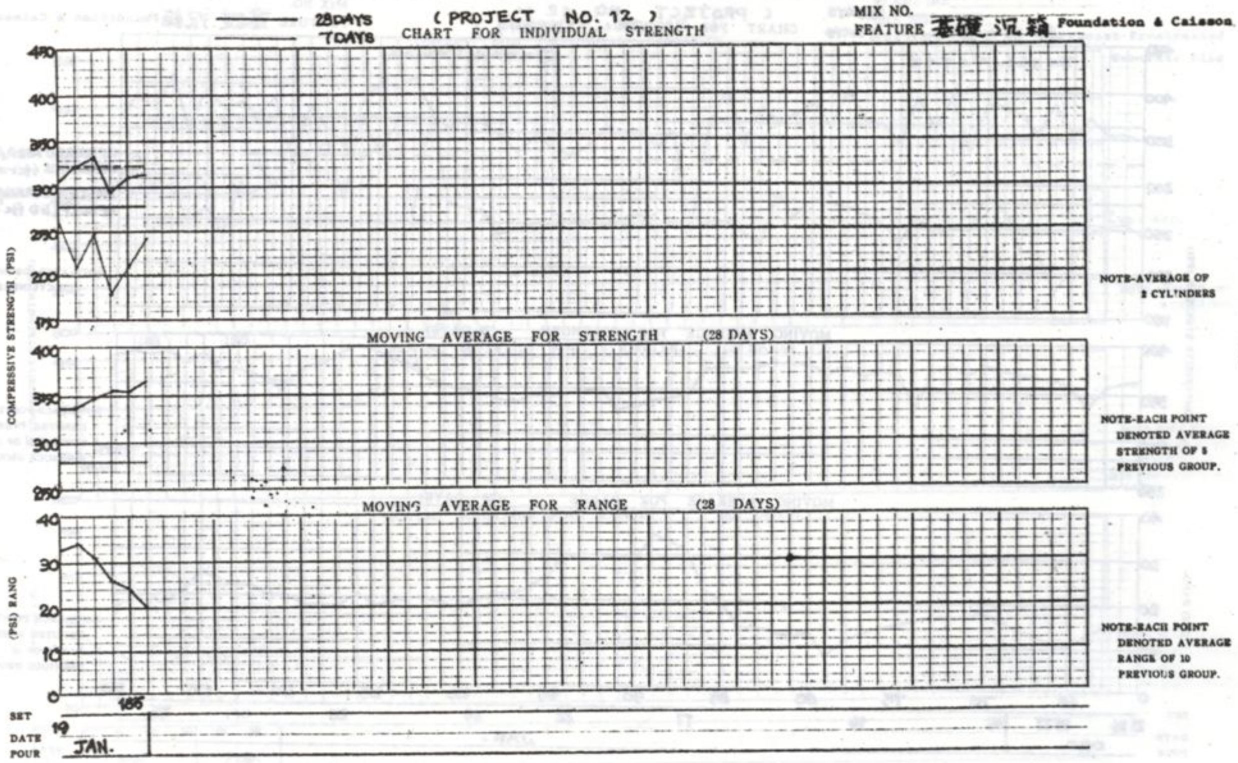


APPROVED BY ______ CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO.
FEATURE 基礎 沉箱 Foundation & Caisson

(PROJECT NO. 12)

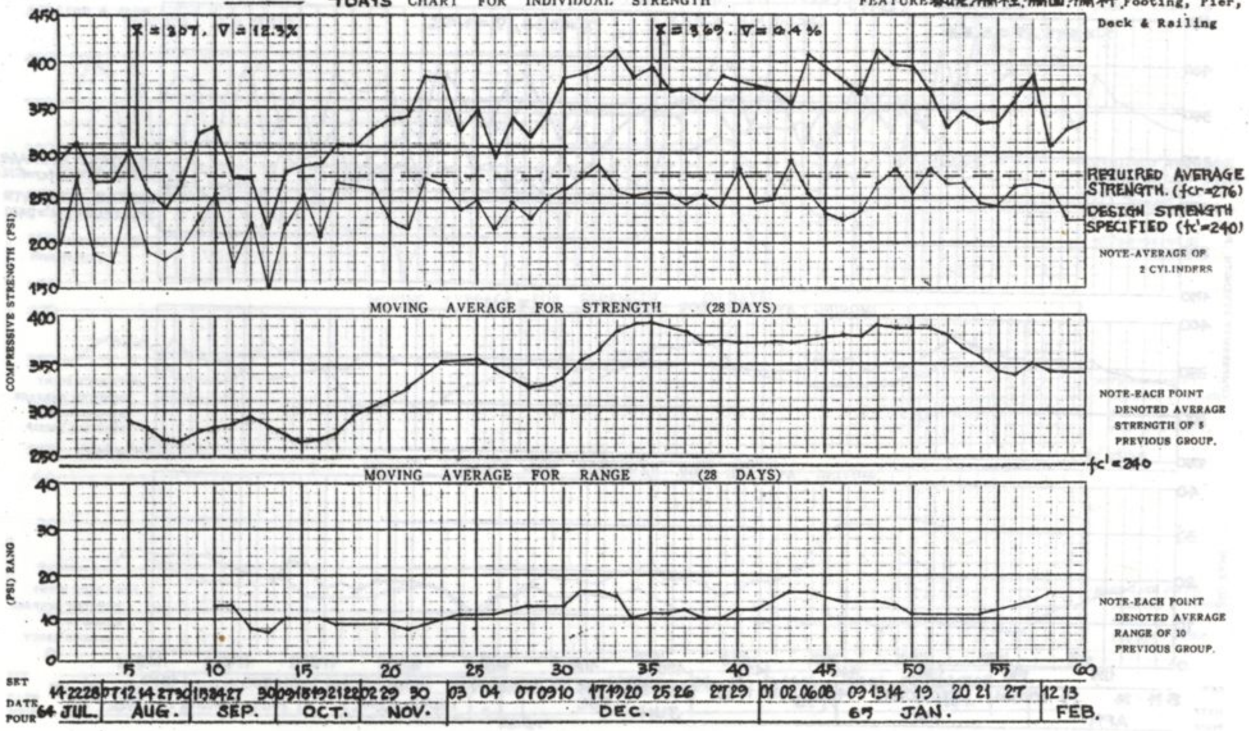


APPROVED BY ______ CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

28DAYS (PROJECT NO. 11)
7DAYS CHART FOR INDIVIDUAL STRENGTH

PLANT NO.
MIX NO. 240-20-12
FEATURE 基礎,橋柱,橋面,欄杆 Footing, Pier, Deck & Railing

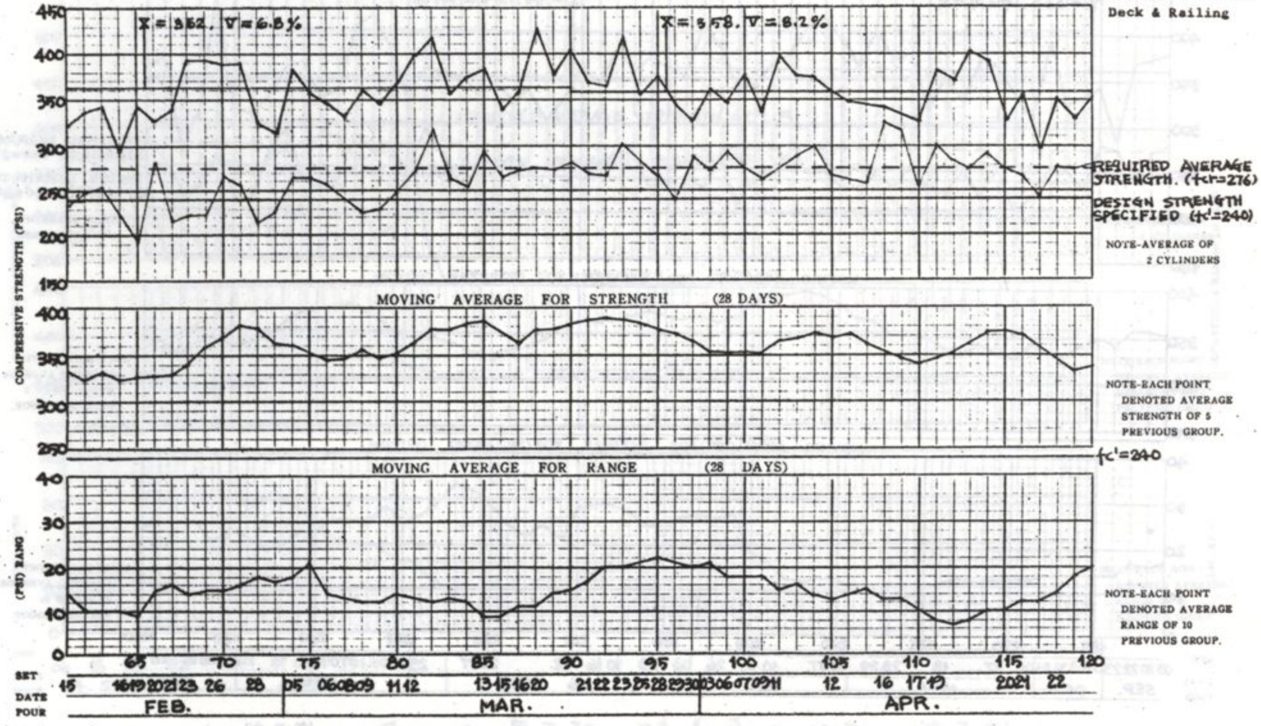


APPROVED BY
CHECKED BY 張平華
REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

28DAYS (PROJECT NO. 11)
7DAYS CHART FOR INDIVIDUAL STRENGTH

PLANT NO.
MIX NO. 240-20-12
FEATURE 基礎,橋柱,橋面,欄杆 Footing, Pier, Deck & Railing

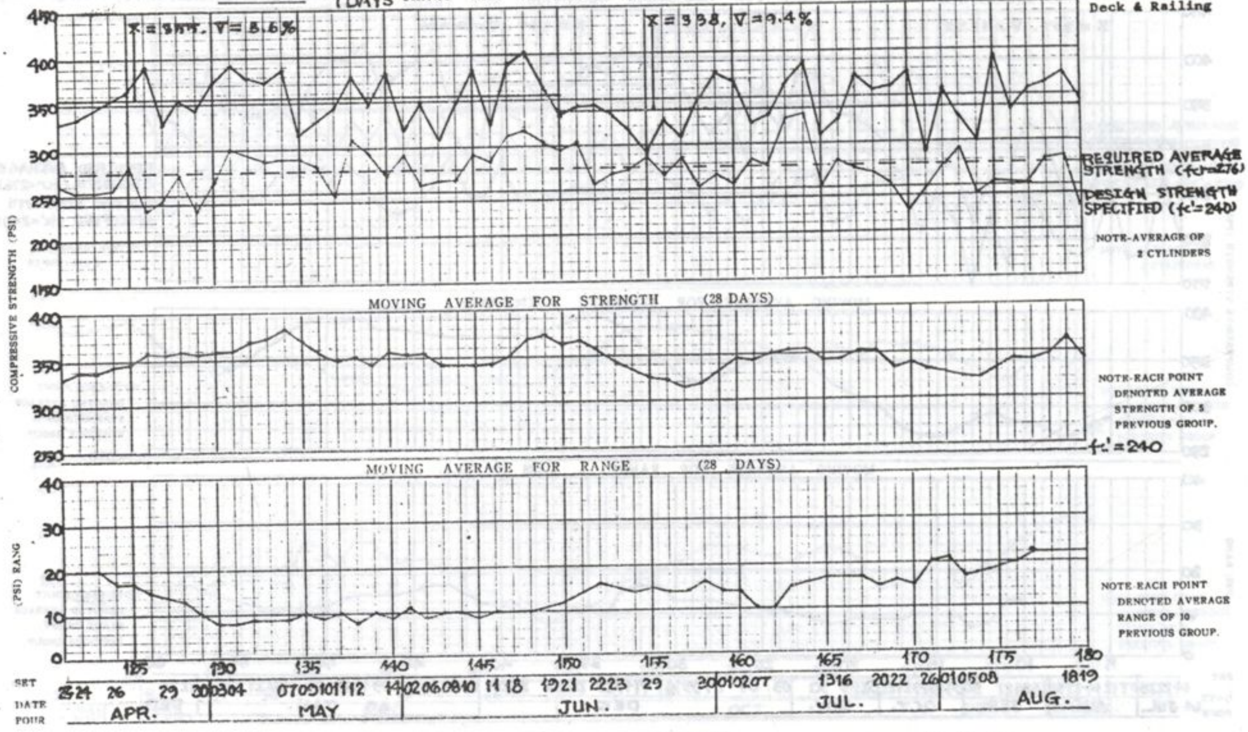


APPROVED BY
CHECKED BY 張平華
REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO. ____________
MIX NO. 240 - 20 - 12
FEATURE 基礎 橋面 橋柱 欄杆 Footing, Pier Deck & Railing

28 DAYS (PROJECT NO. 11)
7 DAYS CHART FOR INDIVIDUAL STRENGTH

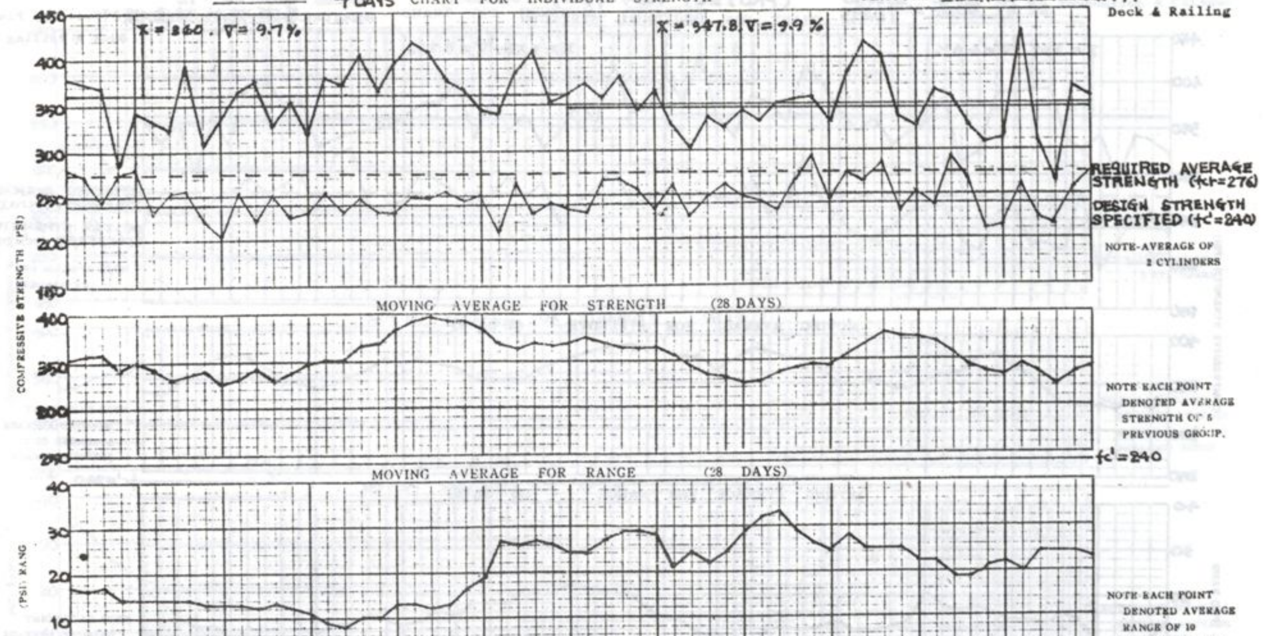


APPROVED BY ____________ CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO. ____________
MIX NO. 240 - 20 - 12
FEATURE 基礎 橋柱 橋面 欄杆 Footing, Pier Deck & Railing

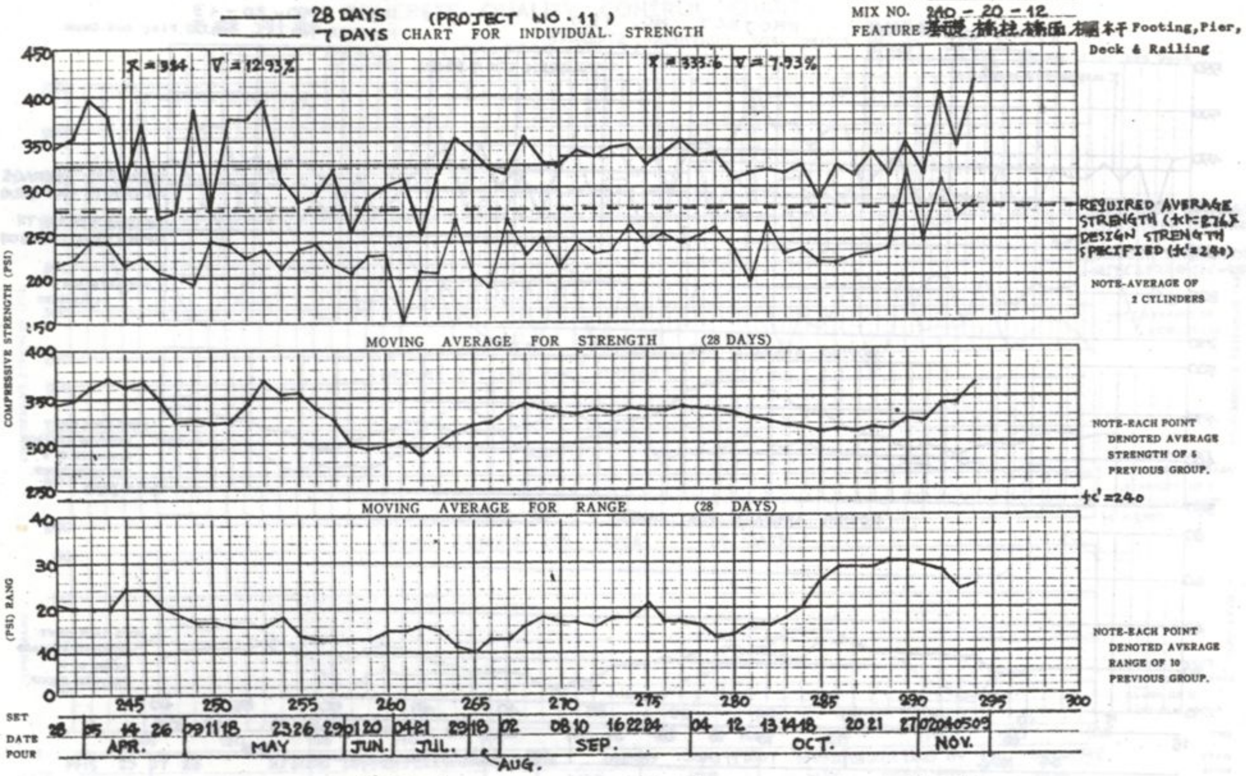
28 DAYS (PROJECT NO. 11)
7 DAYS CHART FOR INDIVIDUAL STRENGTH



APPROVED BY ____________ CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. 240 - 20 - 12
FEATURE 基礎,橋柱,橋面,欄杆 Footing, Pier, Deck & Railing



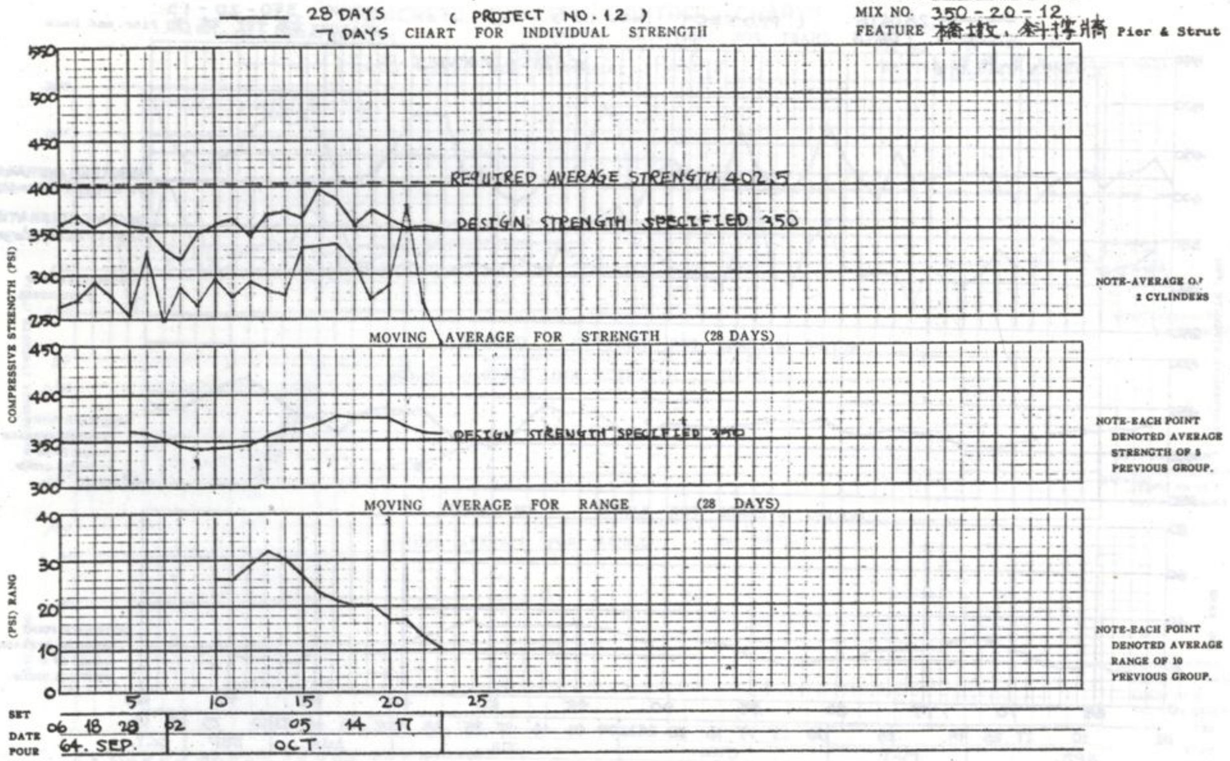
APPROVED BY

CHECKED BY 張平華

REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. 350 - 20 - 12
FEATURE 橋墩、斜撐牆 Pier & Strut



APPROVED BY

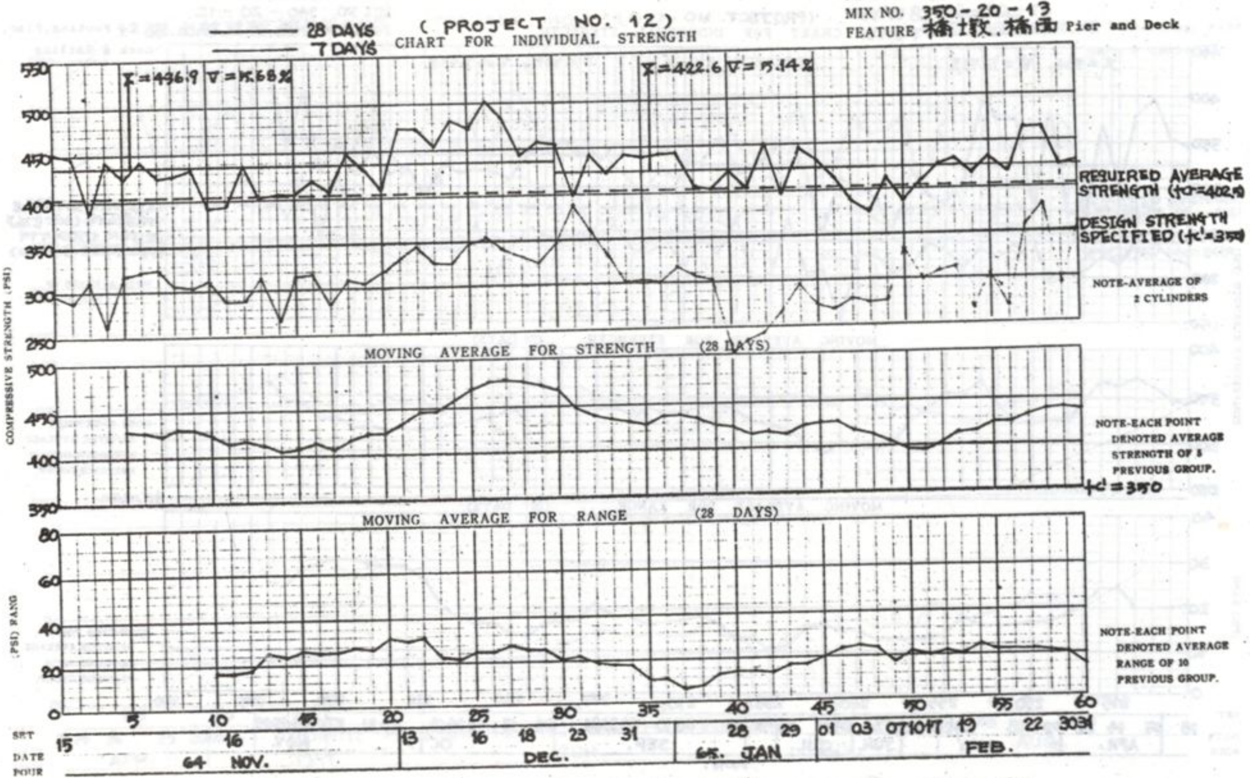
CHECKED BY 張平華

REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 12)

PLANT NO.
MIX NO. 350 - 20 - 13
FEATURE 橋墩. 橋面 Pier and Deck

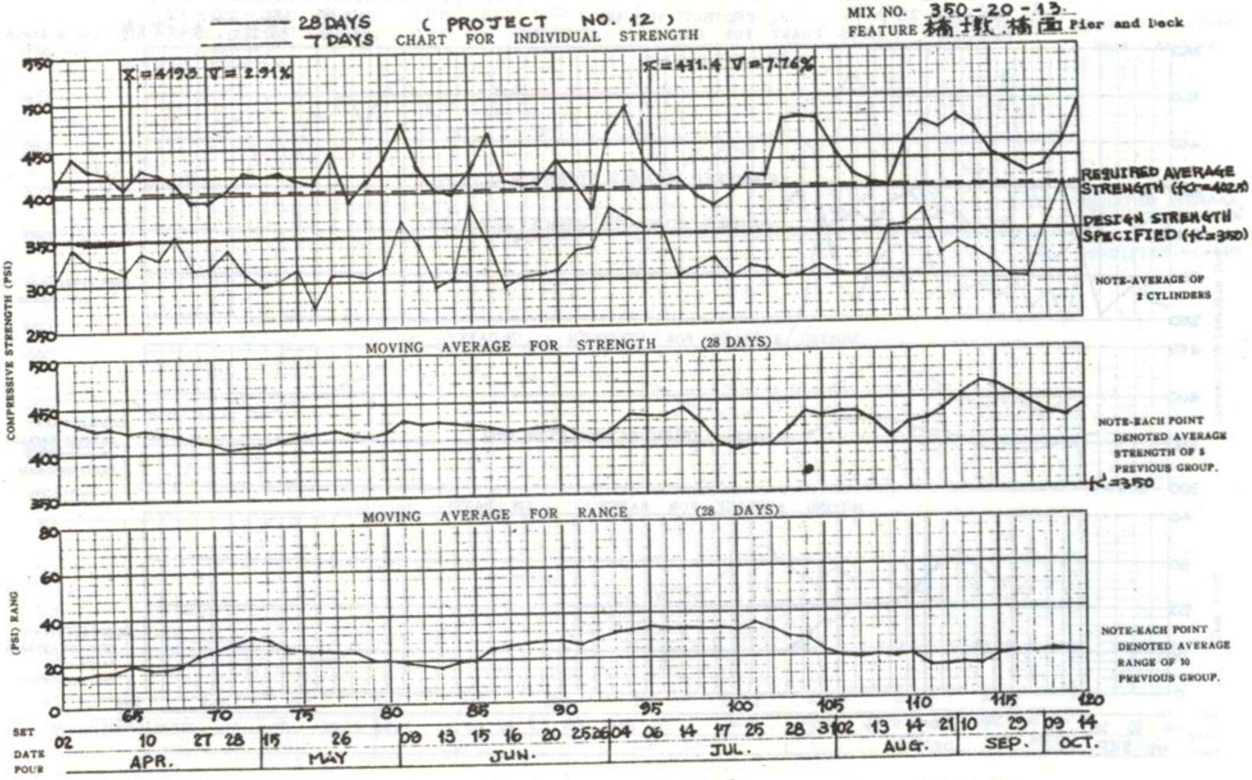


APPROVED BY Y.H.Chen CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 12)

PLANT NO.
MIX NO. 350 - 20 - 13
FEATURE 橋墩. 橋面 Pier and Deck



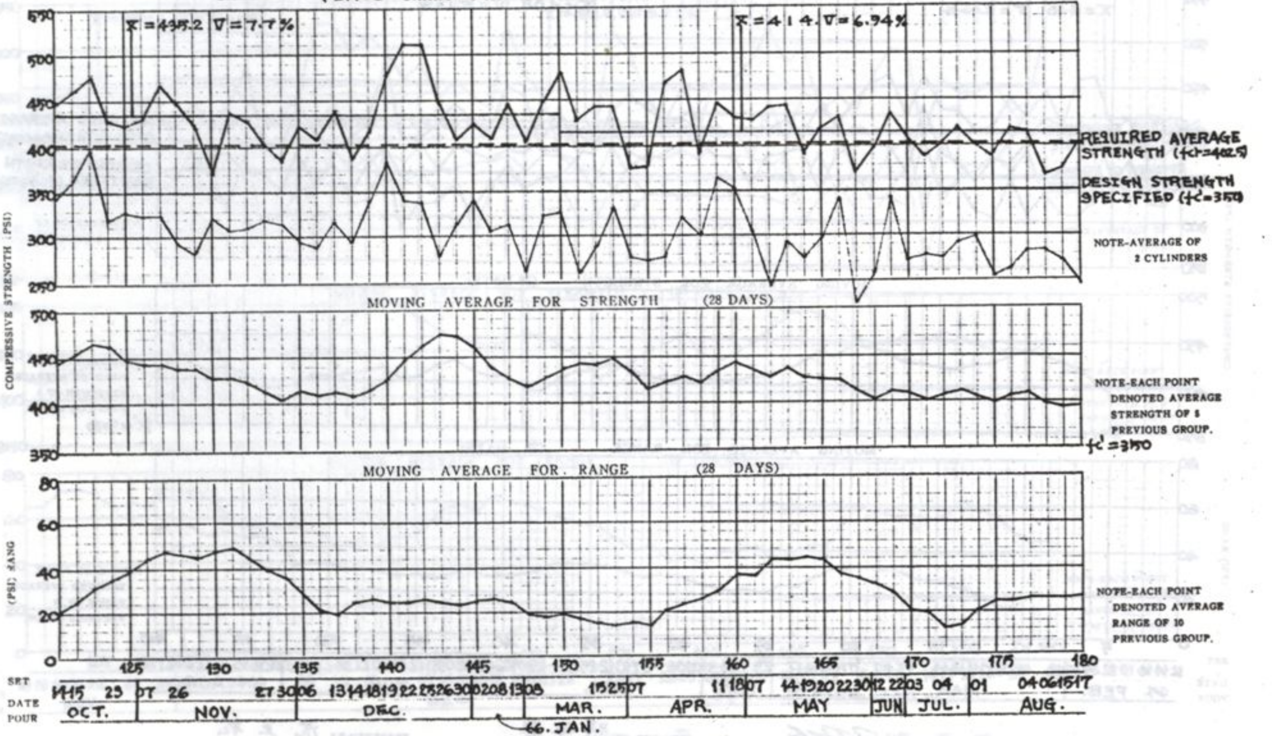
APPROVED BY Y.H.Chen CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 12)

PLANT NO.
MIX NO. 350-20-13
FEATURE 橋墩. 橋面 Pier and Deck

28DAYS
7DAYS CHART FOR INDIVIDUAL STRENGTH



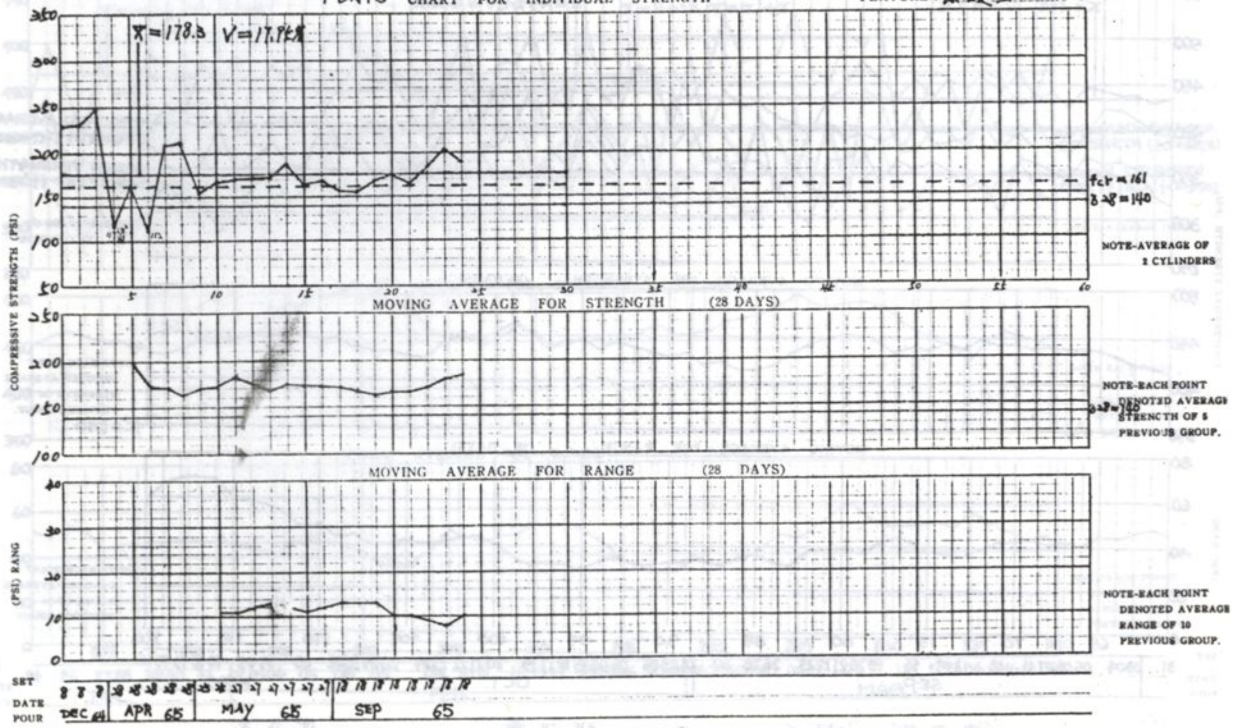
APPROVED BY
CHECKED BY 張平華
REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO 12)

PLANT NO.
MIX NO. 140—40—12
FEATURE 保護壁 Protection Wall

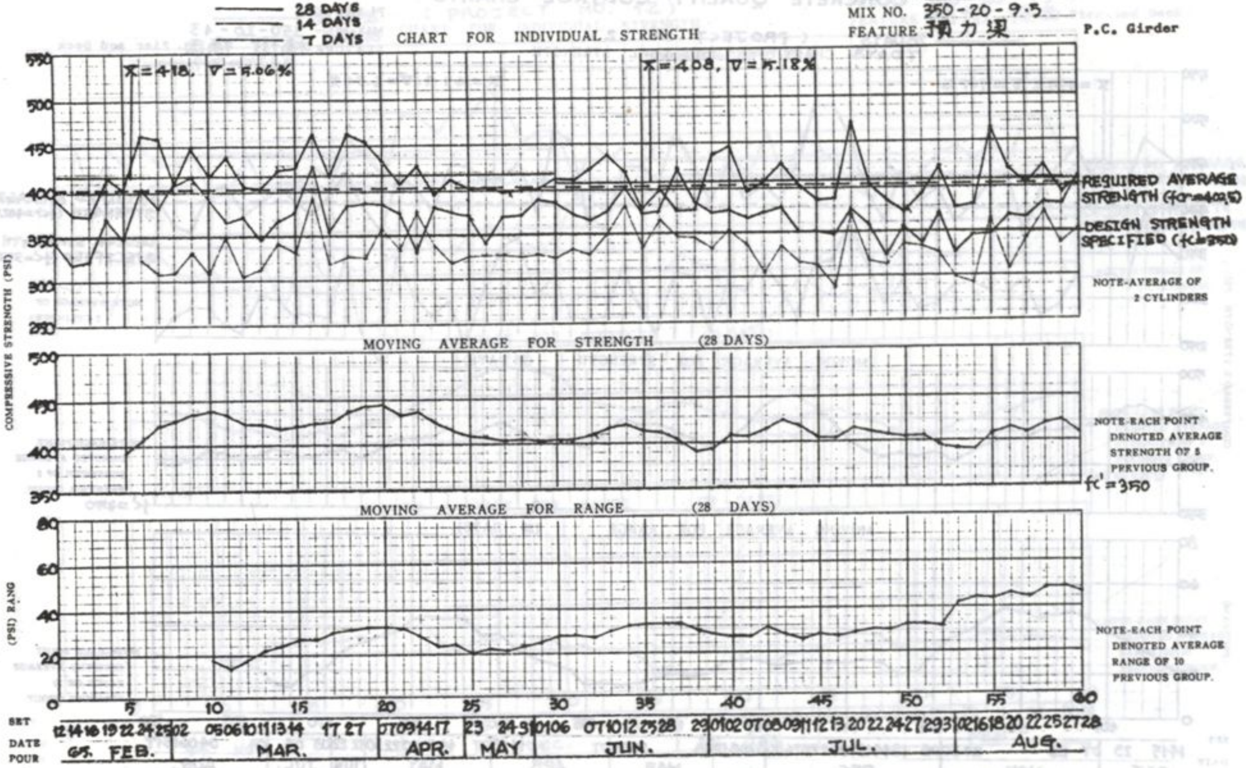
28DAYS
7DAYS CHART FOR INDIVIDUAL STRENGTH



APPROVED BY
CHECKED BY
REPORTED BY 梁瑋琦

CONCRETE QUALITY CONTROL CHARTS

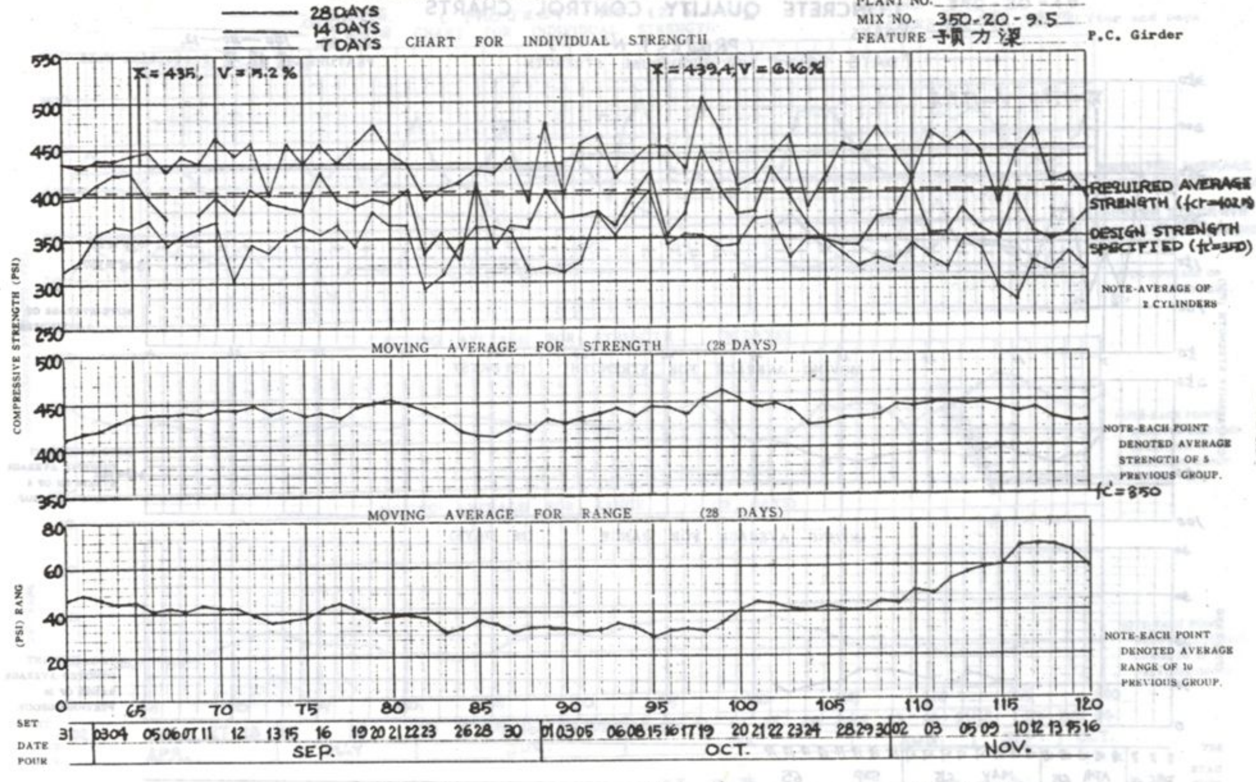
PLANT NO.
MIX NO. 350-20-9.5
FEATURE 預力梁 P.C. Girder



APPROVED BY
CHECKED BY 張平華
REPORTED BY 沈正松

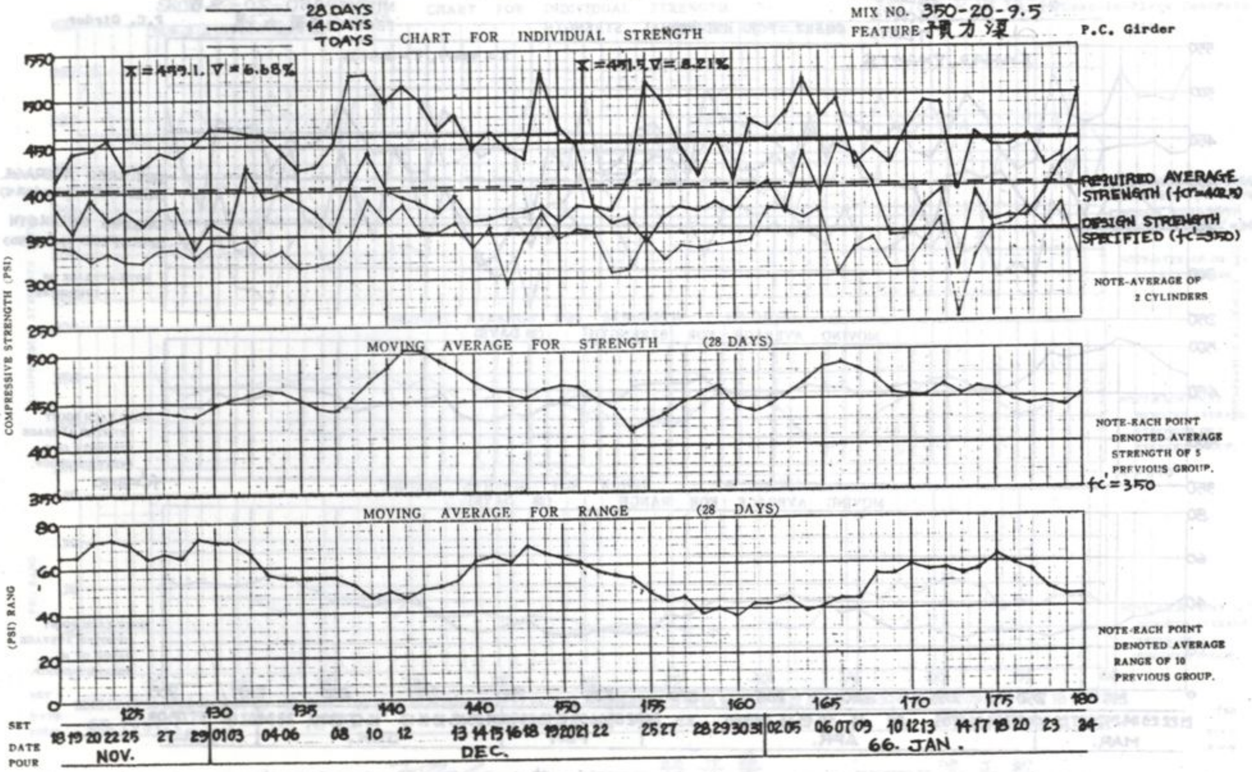
CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. 350-20-9.5
FEATURE 預力梁 P.C. Girder



APPROVED BY
CHECKED BY 張平華
REPORTED BY 沈正松

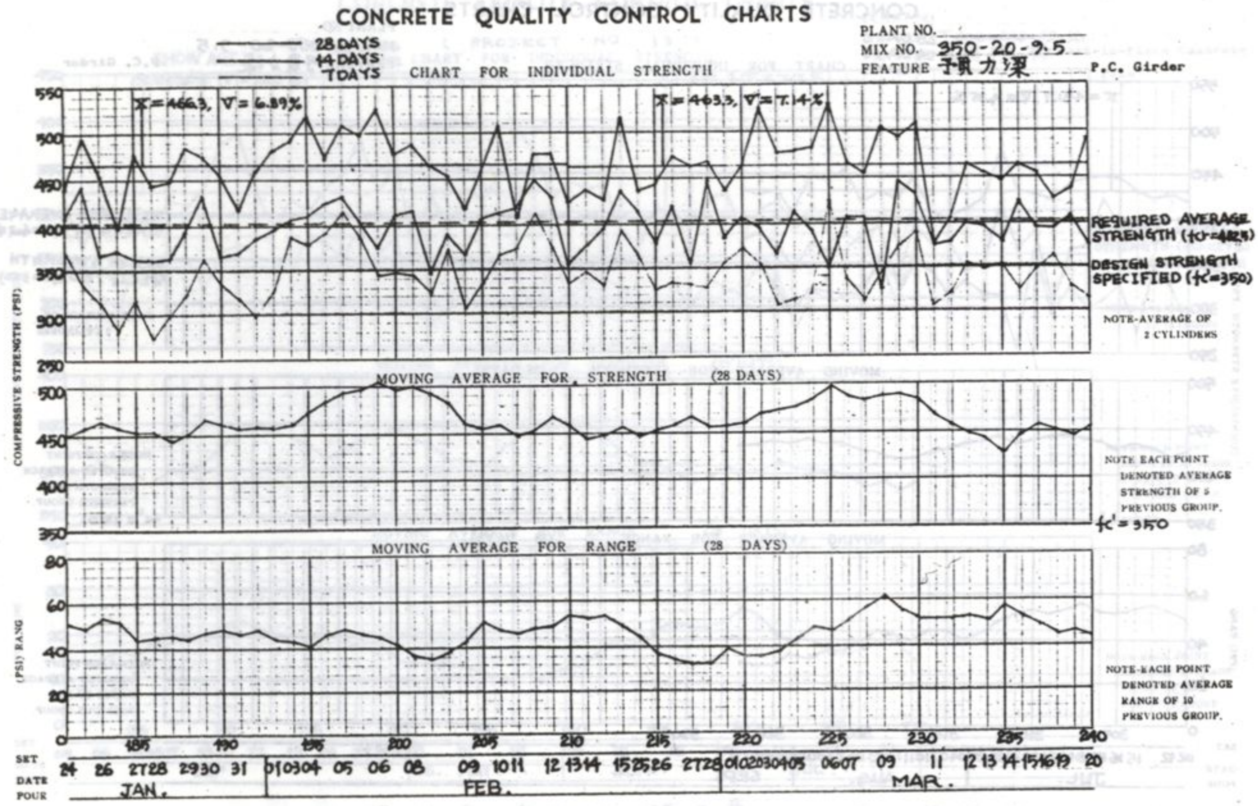
CONCRETE QUALITY CONTROL CHARTS



PLANT NO.
MIX NO. 350-20-9.5
FEATURE 預力梁 P.C. Girder

APPROVED BY CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

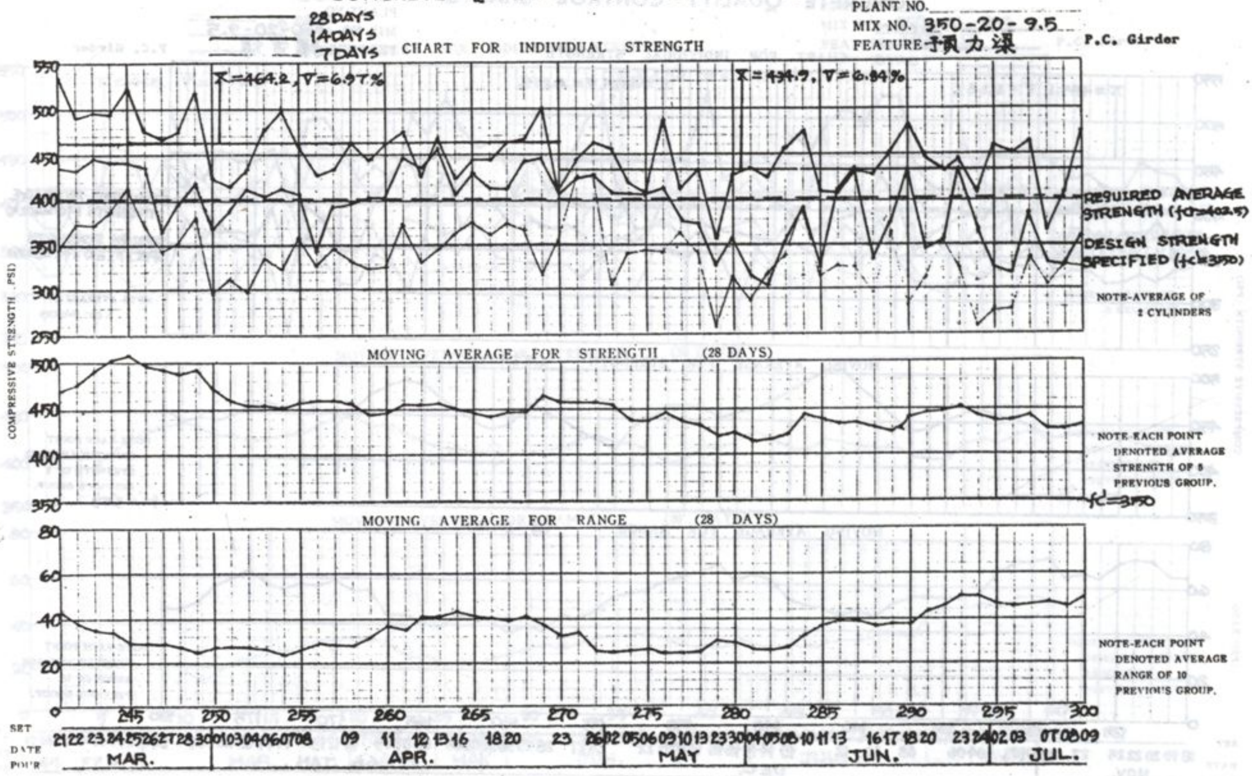


PLANT NO.
MIX NO. 350-20-9.5
FEATURE 預力梁 P.C. Girder

APPROVED BY CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

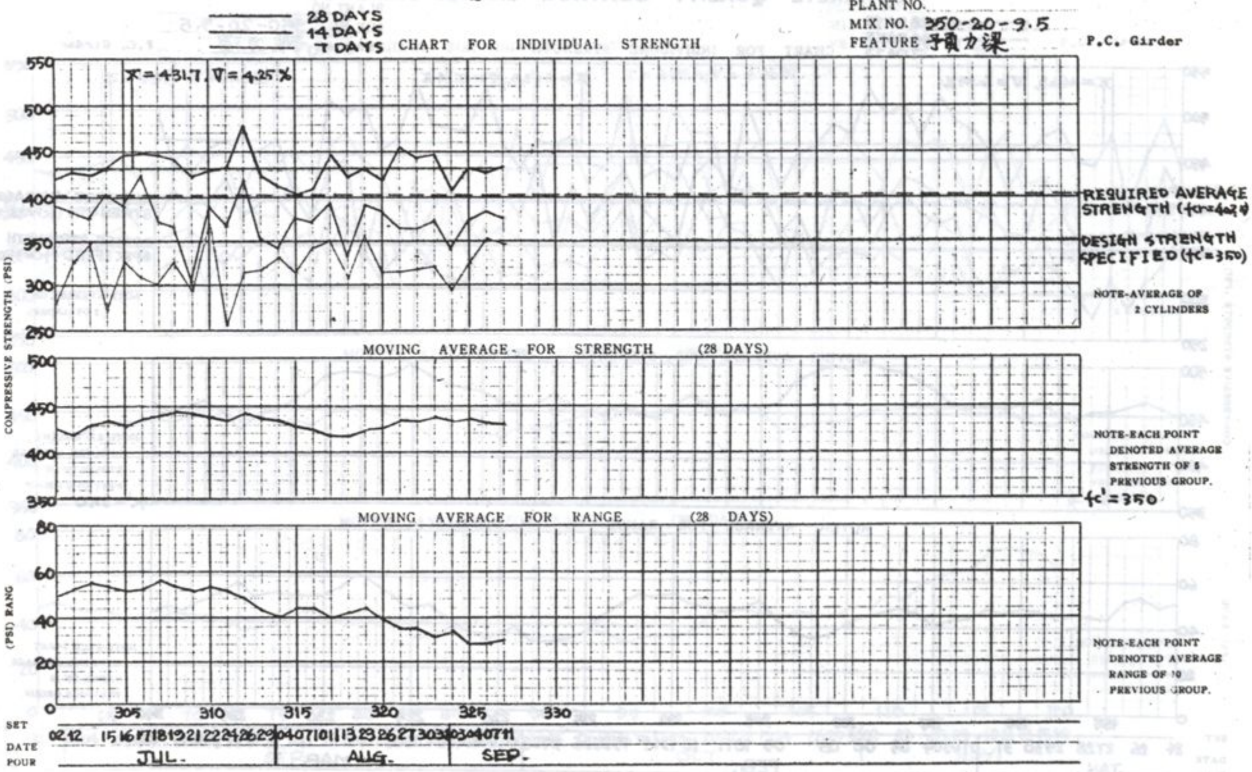
PLANT NO.
MIX NO. 350-20-9.5
FEATURE 預力梁 P.C. Girder



APPROVED BY ______ CHECKED BY 張平華 ______ REPORTED BY 沈正松 ______

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. 350-20-9.5
FEATURE 預力梁 P.C. Girder



APPROVED BY ______ CHECKED BY 張平華 ______ REPORTED BY 沈正松 ______

CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 13)

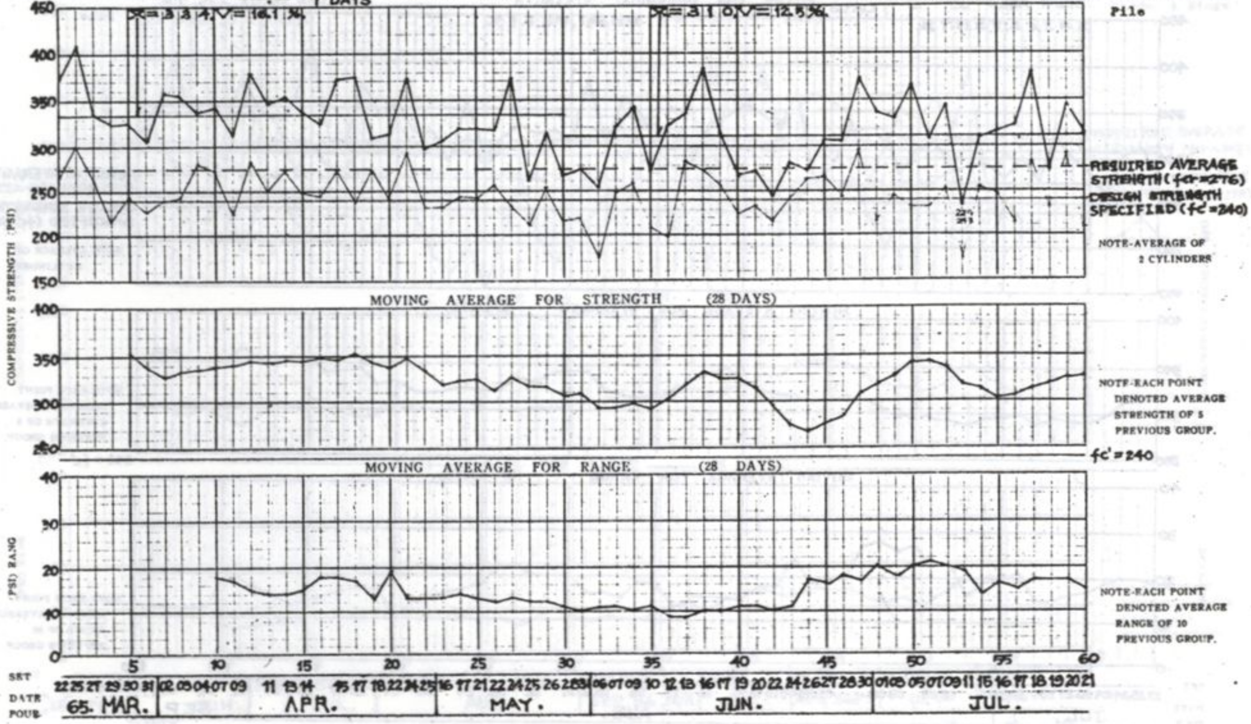
PLANT NO.

MIX NO. T.006—240-40-18

FEATURE 反循環樁 Cast-in-Place Concrete Pile

SHOW AGE: ——— 28 DAYS ——— 7 DAYS

CHART FOR INDIVIDUAL STRENGTH



APPROVED BY

CHECKED BY 張平華

REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 13)

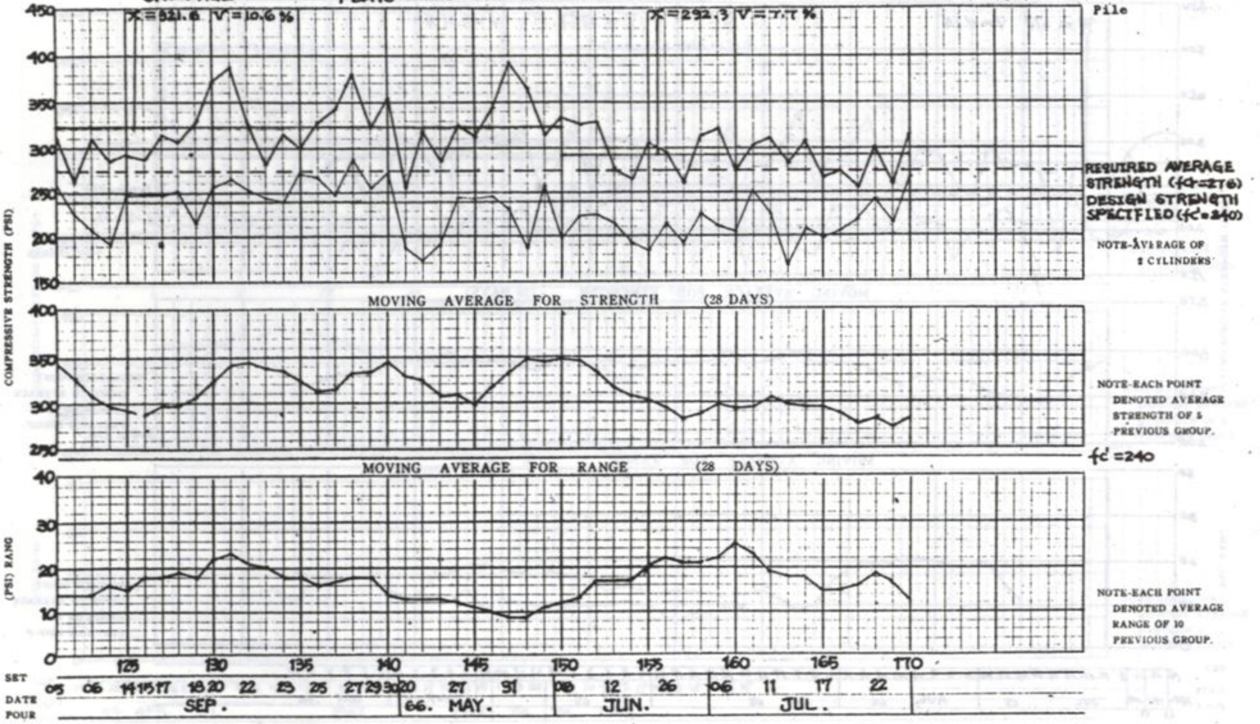
PLANT NO.

MIX NO. T.006—240-40-18

FEATURE 反循環樁 Cast-in-Place Concrete Pile

SHOW AGE: ——— 28 DAYS ——— 7 DAYS

CHART FOR INDIVIDUAL STRENGTH



APPROVED BY

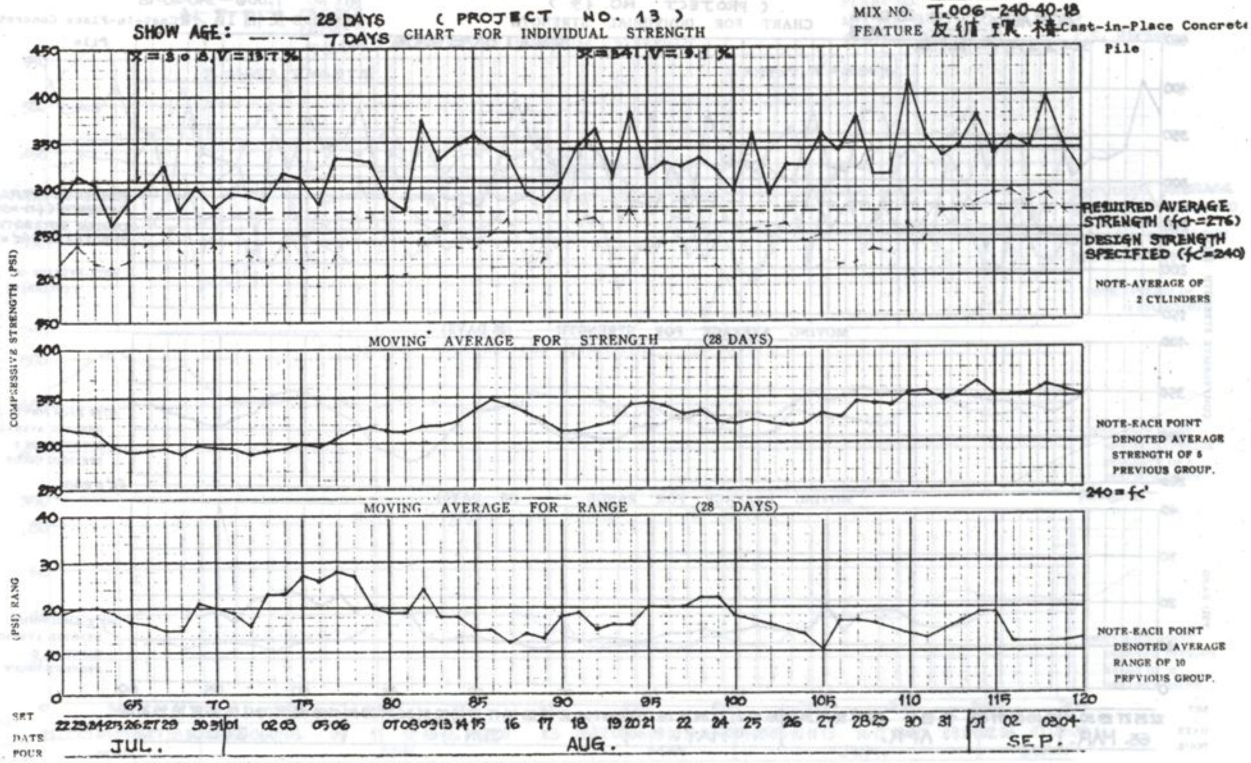
CHECKED BY 張平華

REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

(PROJECT NO. 13)

PLANT NO.
MIX NO. T.006-240-40-18
FEATURE 反循環樁 Cast-in-Place Concrete Pile

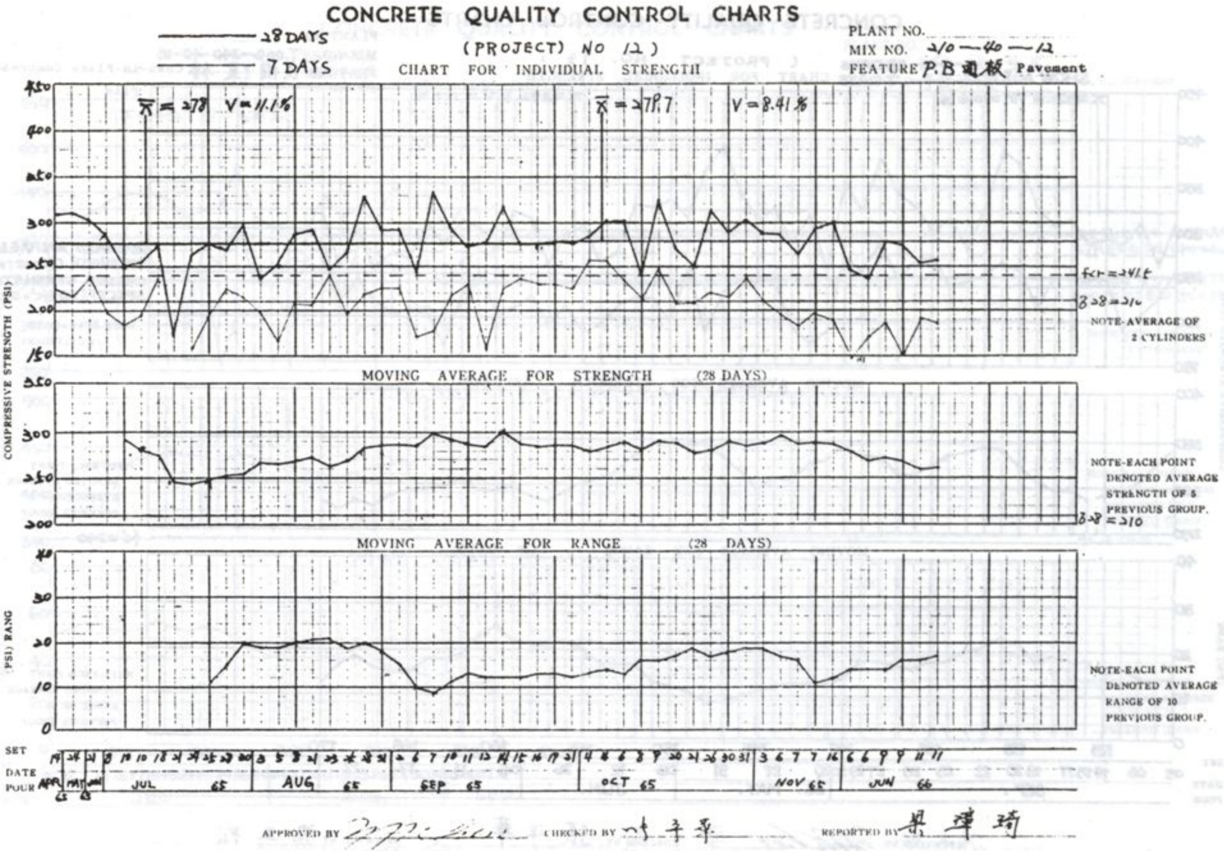


APPROVED BY CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

(PROJECT) NO 12)

PLANT NO.
MIX NO. 210 — 40 — 12
FEATURE P.B 面板 Pavement



APPROVED BY CHECKED BY [illegible] REPORTED BY 吳瑋琦

CONCRETE QUALITY CONTROL CHARTS

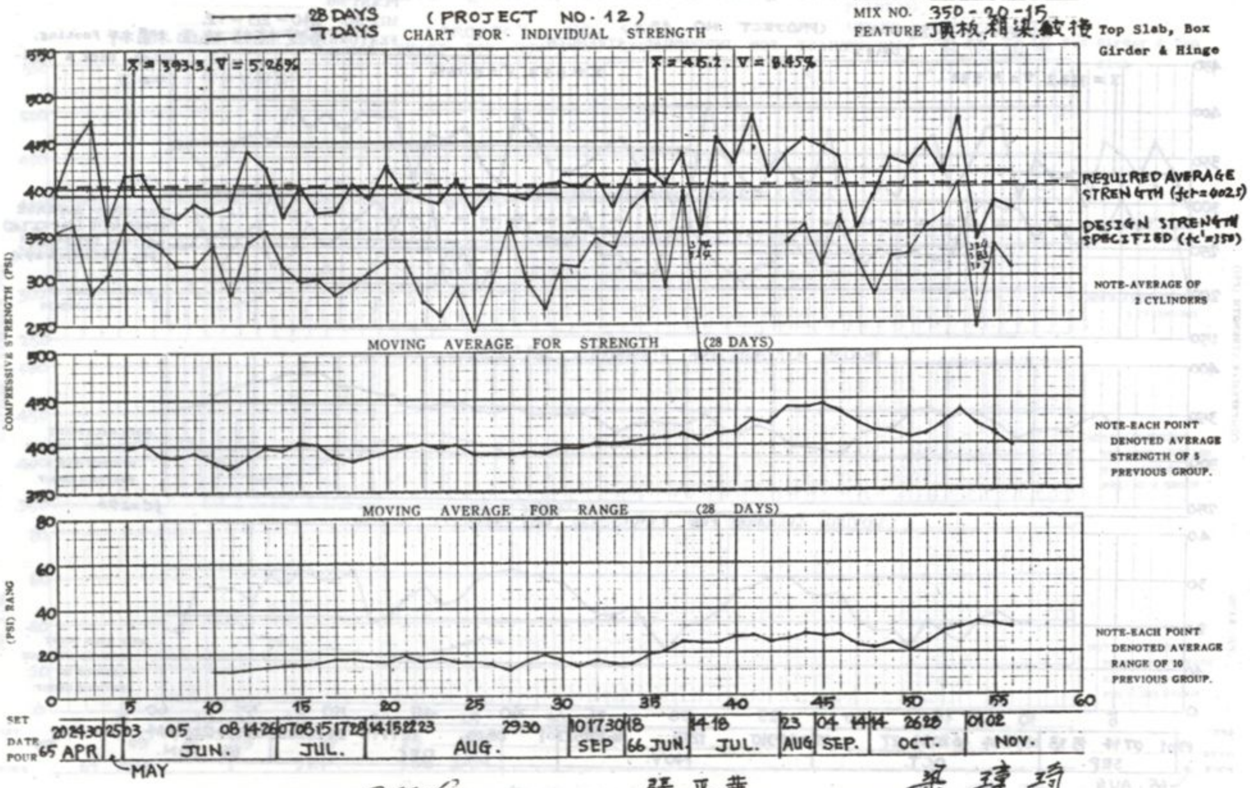
(PROJECT NO. 12)

CHART FOR INDIVIDUAL STRENGTH

PLANT NO.

MIX NO. 350-20-15

FEATURE 頂板,箱梁,鉸接 Top Slab, Box Girder & Hinge



APPROVED BY

CHECKED BY 張平華

REPORTED BY 梁瑋琦

CONCRETE QUALITY CONTROL CHARTS

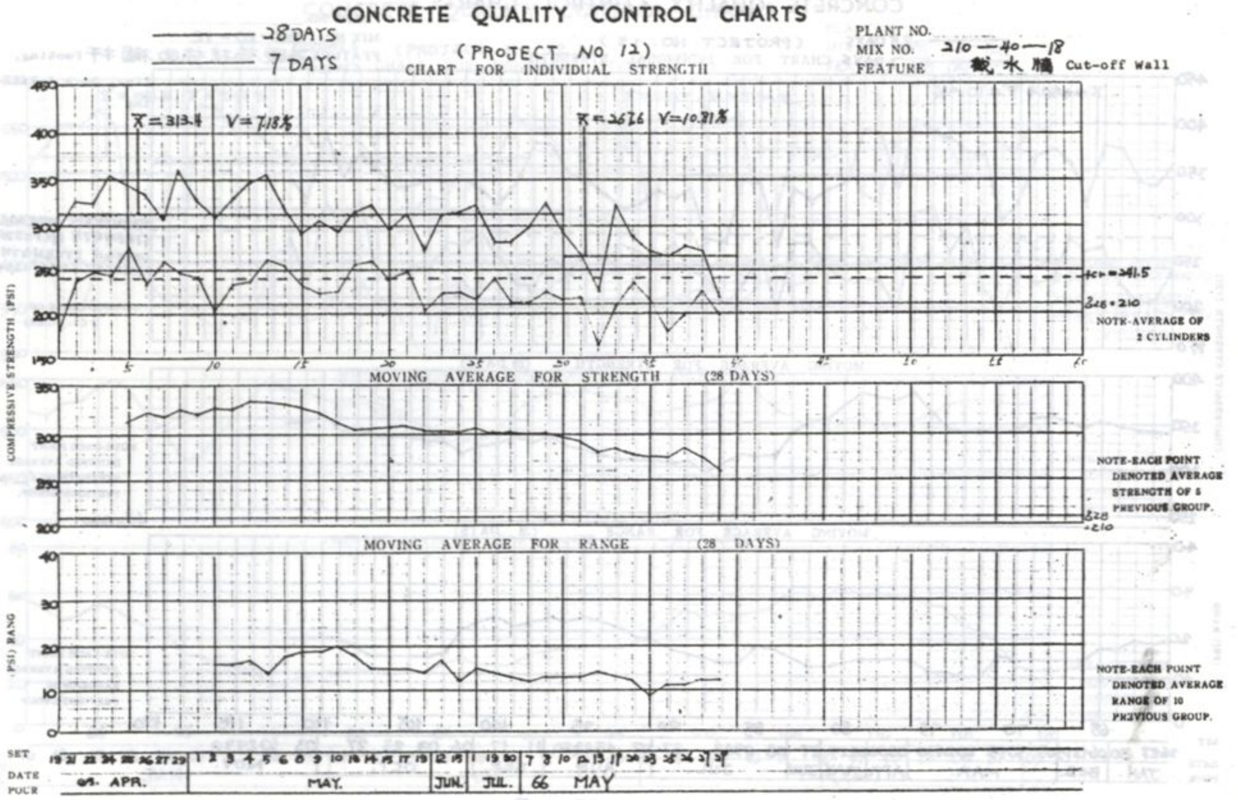
(PROJECT NO 12)

CHART FOR INDIVIDUAL STRENGTH

PLANT NO.

MIX NO. 210—40—18

FEATURE 截水牆 Cut-off Wall



APPROVED BY

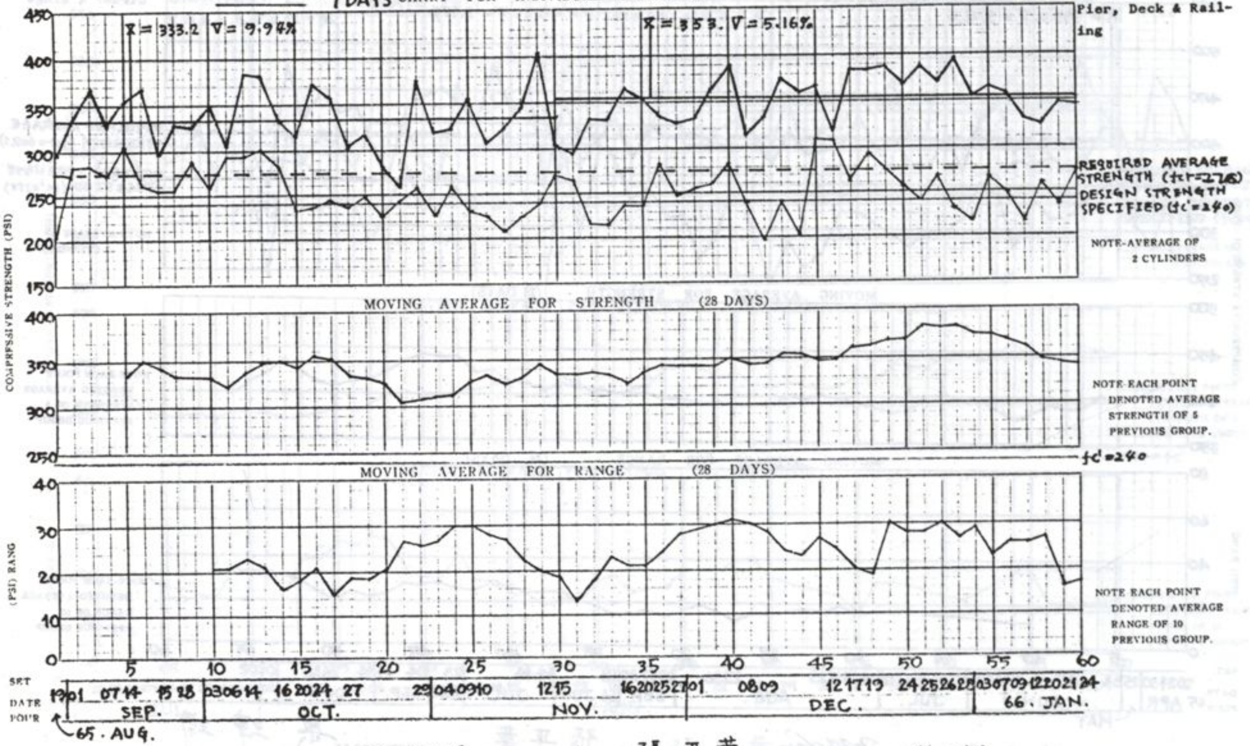
CHECKED BY 張平華

REPORTED BY 梁瑋琦

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. 240 - 20 - 12
FEATURE 基礎.橋柱.橋面.欄杆 Footing, Pier, Deck & Railing

——— 28 DAYS (PROJECT NO. 13)
——— 7 DAYS CHART FOR INDIVIDUAL STRENGTH

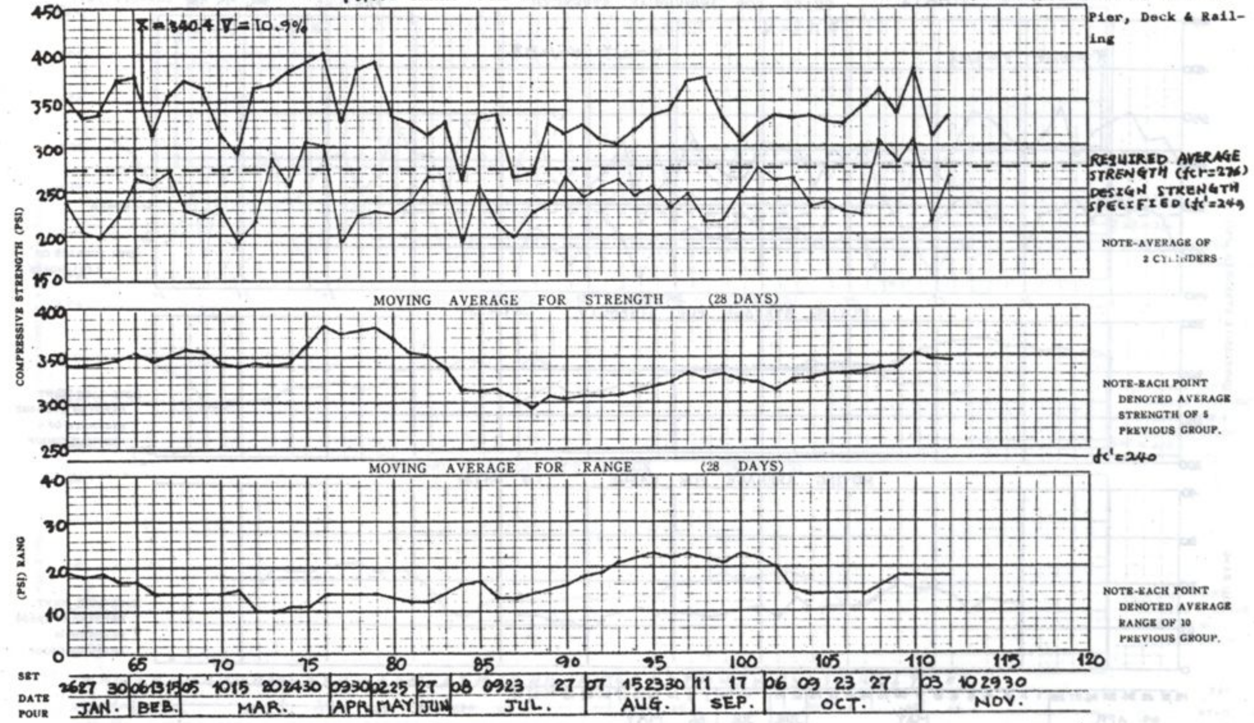


APPROVED BY Y.K. [illegible] CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. 240 - 20 - 12
FEATURE 基礎.橋柱.橋面.欄杆 Footing, Pier, Deck & Railing

——— 28 DAYS (PROJECT NO. 13)
——— 7 DAYS CHART FOR INDIVIDUAL STRENGTH

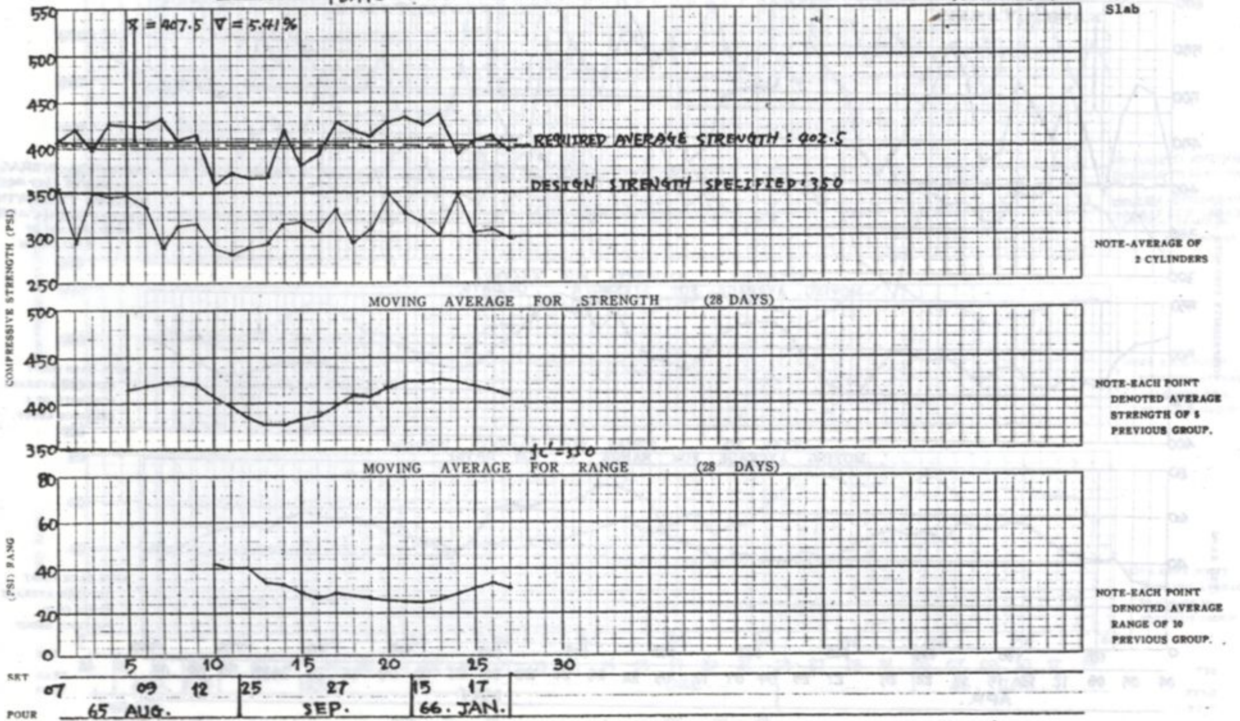


APPROVED BY Y.H. Chen CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

——— 28DAYS (PROJECT NO. 12)
– – – 7DAYS CHART FOR INDIVIDUAL STRENGTH

PLANT NO.
MIX NO. 350-20-17.5
FEATURE 頂板.箱梁 Box Girder & Top Slab

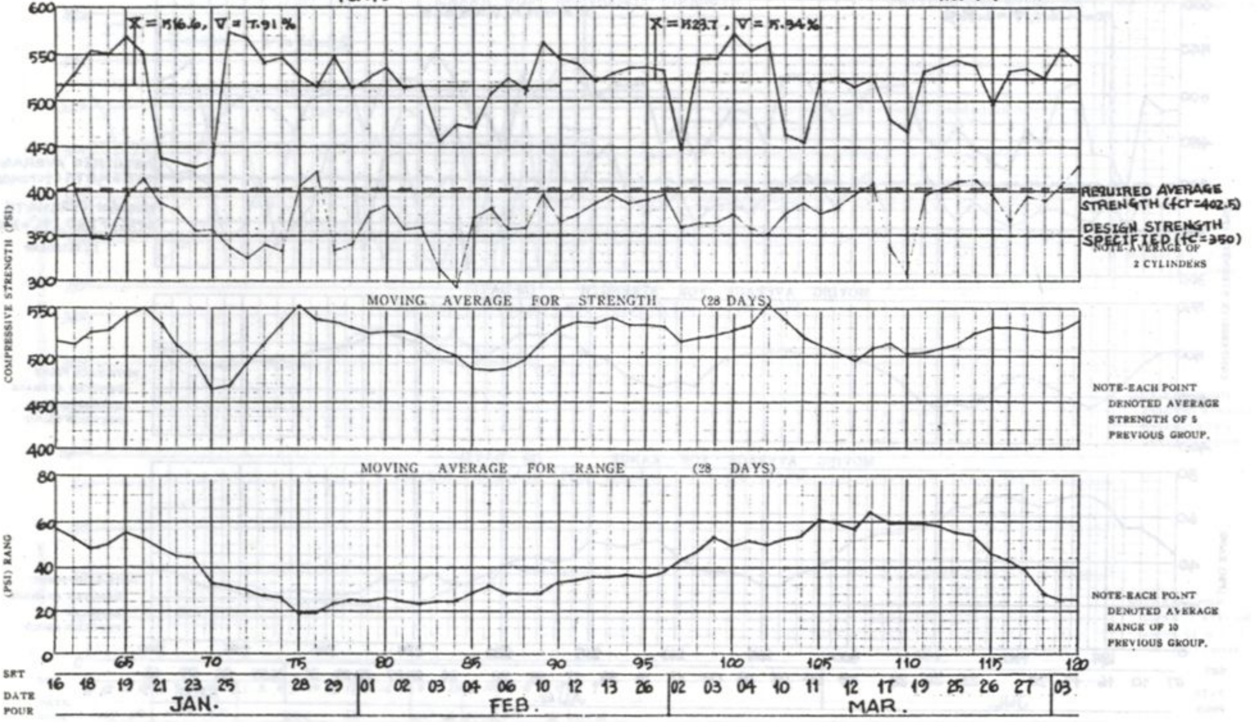


APPROVED BY YHchen CHECKED BY 張平華 REPORTED BY 梁瑋琦

CONCRETE QUALITY CONTROL CHARTS

——— 28DAYS (PROJECT NO. 12)
– – – 7DAYS CHART FOR INDIVIDUAL STRENGTH

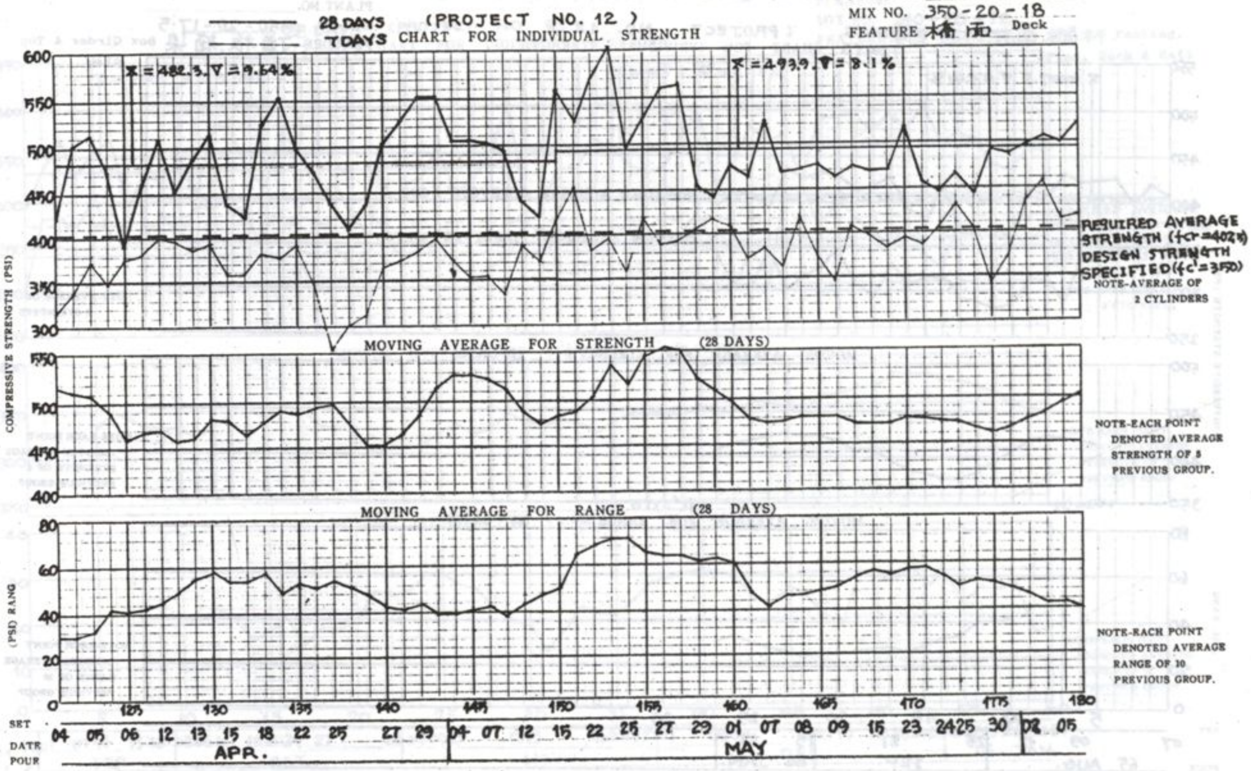
PLANT NO.
MIX NO. 350-20-18
FEATURE 橋面 Deck



APPROVED BY YHChen CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

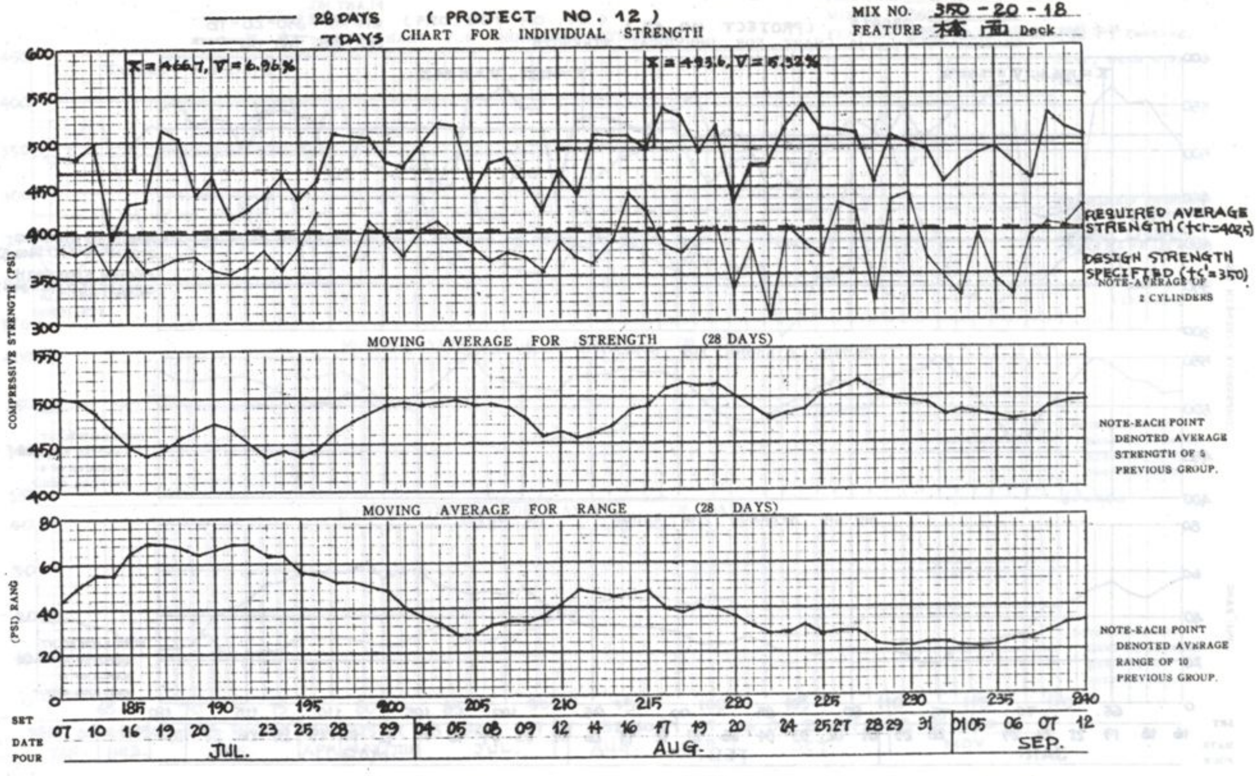
PLANT NO.
MIX NO. 350-20-18
FEATURE 橋面 Deck



APPROVED BY CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

PLANT NO.
MIX NO. 350-20-18
FEATURE 橋面 Deck

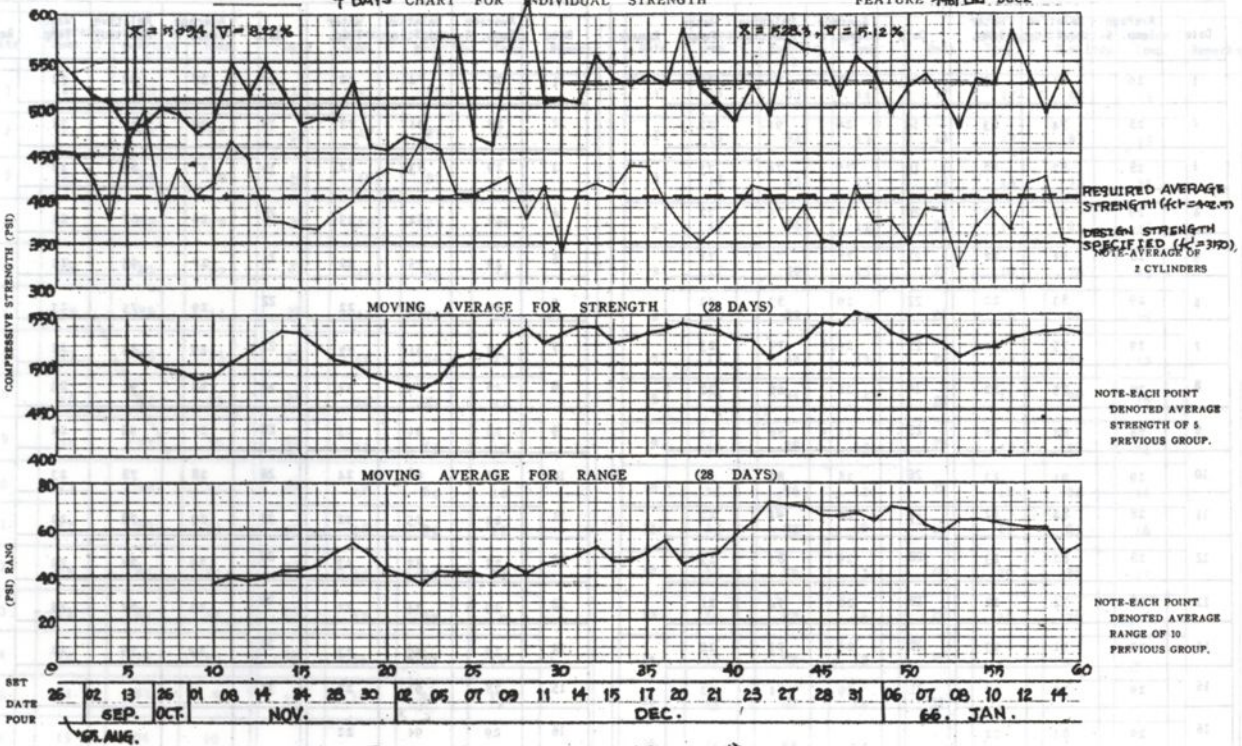


APPROVED BY CHECKED BY 張平華 REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

28 DAYS (PROJECT NO. 12)
7 DAYS CHART FOR INDIVIDUAL STRENGTH

PLANT NO.
MIX NO. 350-20-18
FEATURE 橋面 Deck

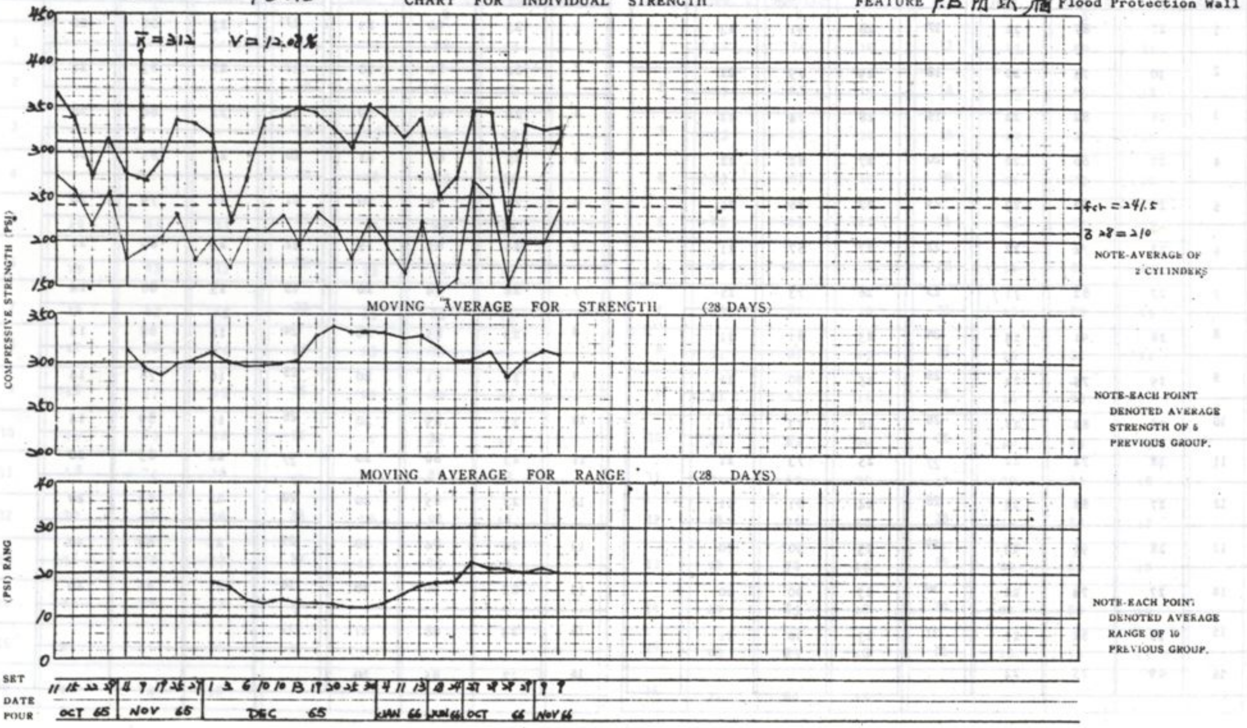


APPROVED BY
CHECKED BY 張 平 華
REPORTED BY 沈正松

CONCRETE QUALITY CONTROL CHARTS

28 DAYS
7 DAYS
(PROJECT NO. 12)
CHART FOR INDIVIDUAL STRENGTH

PLANT NO.
MIX NO. 210-20-12
FEATURE P.B 防洪牆 Flood Protection Wall



APPROVED BY
CHECKED BY 張平華
REPORTED BY 梁瑋琦

Month: Aug. 1975

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	29	91	23	17	28	91	22	
2	28	74	23	18	29	91	22	
3	28	82	22	19	28	74	23	
4	29	75	23	20	29	75	22	
5	28	91	22	21	28	74	21	
6	29	83	22	22	29	83	22	
7	29	75	21	23	29.5	79	23	
8	30	75	22	24	28	86	22	
9	28	82	22	25	27.5	86	22	
10	29	91	23	26	28	82	22	
11	28	82	22	27	28	91	22	
12	28	82	23	28	29	83	23	
13	28	74	22	29	28	74	21	
14	29	91	23	30	29	75	22	
15	29	75	22	31	29	91	22	
16	29	83	22					

Month: Sept. 1975

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remar
1	28	74	22	17	28	82	22	
2	28	91	22	18	30	60	24	
3	29	83	22	19	27	82	22	
4	28	82	23	20	29	67	22	
5	29	75	22	21	27	95	23	
6	29	91	22	22	29	75	23	
7	28	91	22	23	29	83	22	
8	28	74	22	24	28	91	23	
9	28	91	23	25	27	95	22	
10	30	68	24	26	28	78	23	
11	29	75	22	27	29	78	22	
12	28	82	23	28	29	78	22	
13	29	74	22	29	29	67	23	
14	29	75	23	30	29	79	22	
15	28	91	22	31				
16	29	91	22					

Month: Oct. 1975

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	27	87	22	17	28	91	23	
2	30	75	22	18	29	75	22	
3	28	82	22	19	28	74	22	
4	27	86	22	20	27	82	22	
5	27	82	23	21	27	86	21	
6	27	74	22	22	27	87	21	
7	27	82	21	23	26	73	21	
8	28	91	23	24	25	81	21	
9	29	75	22	25	25	90	21	
10	28	82	22	26	26	73	21	
11	28	74	22	27	25	73	21	
12	27	82	22	28	24	81	21	
13	28	91	23	29	23	90	20	
14	27	74	22	30	23	90	20	
15	28	82	22	31	23	90	21	
16	29	75	22					

Month: Nov. 1975

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	23	90	21	17	23	86	20	
2	24	81	22	18	22	83	21	
3	24	90	20	19	22	86	20	
4	25	81	21	20	21	85	20	
5	24	79	20	21	21	79	20	
6	24	85	19	22	22	86	21	
7	24	84	20	23	15	88	14	
8	23	85	20	24	15	88	14	
9	24	81	20	25	16	78	14	
10	23	89	20	26	19	89	14	
11	23	86	20	27	22	85	20	
12	23	85	20	28	22	83	20	
13	24	84	20	29	21	88	20	
14	23	90	20	30	21	88	20	
15	23	88	21	31				
16	22	85	20					

Month: Dec. 1975

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	22	89	20	17	17	86	11	
2	21	87	18	18	17	87	11	
3	21	86	19	19	18	87	11	
4	21	82	19	20	19	79	10	
5	20	88	17	21	18	87	11	
6	19	88	14	22	17	78	10	
7	17	76	13	23	16	79	12	
8	15	83	11	24	16	83	12	
9	15	77	10	25	15	85	13	
10	13	79	11	26	16	84	12	
11	12	80	11	27	18	81	12	
12	14	81	11	28	14	89	12	
13	12	81	11	29	12	82	9	
14	12	78	10	30	16	87	12	
15	11	81	9	31	16	86	13	
16	15	86	10					

Month: Jan. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	16	83	14	17	17	88	15	
2	16	84	13	18	17	87	15	
3	18	83	15	19	18	74	16	
4	16	86	14	20	19	81	16	
5	17	90	15	21	18	78	15	
6	15	82	12	22	16	88	15	
7	15	88	13	23	14	82	12	
8	16	91	14	24	15	90	14	
9	16	88	13	25	17	78	15	
10	15	86	12	26	18	84	16	
11	13	93	10	27	17	83	16	
12	15	88	13	28	20	86	17	
13	15	83	14	29	19	79	16	
14	17	87	13	30	-	-	-	
15	15	91	14	31	-	-	-	
16	16	88	14					

Month: Feb. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	-	-	-	17	22	88	14	
2	-	-	-	18	21	81	17	
3	-	-	-	19	15	86	13	
4	-	-	-	20	16	83	14	
5	19	85	17	21	16	80	14	
6	19	85	17	22	15	88	13	
7	16	83	15	23	15	82	13	
8	17	78	14	24	15	88	14	
9	18	77	15	25	15	82	14	
10	19	83	16	26	15	88	13	
11	18	74	16	27	20	89	17	
12	20	85	16	28	19	92	18	
13	21	77	17	29	15	83	13	
14	22	79	17	30				
15	21	81	18	31				
16	22	74	18					

Month: Mar. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	12	74	10	17	23	90	21	
2	13	67	10	18	20	85	18	
3	13	67	10	19	19	87	17	
4	13	67	10	20	21	79	20	
5	15	69	12	21	16	83	13	
6	16	83	15	22	16	83	14	
7	19	81	16	23	15	87	15	
8	21	67	19	24	15	81	14	
9	21	82	19	25	16	86	17	
10	21	82	20	26	20	84	19	
11	21	82	20	27	20	86	18	
12	18	76	15	28	19	80	18	
13	18	78	16	29	20	84	18	
14	19	85	18	30	21	80	19	
15	20	85	18	31	18	73	15	
16	22	86	19					

Month: Apr. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	17	80	15	17	24	88	22	
2	18	92	17	18	23	87	22	
3	20	84	18	19	23	90	21	
4	19	84	18	20	23	87	21	
5	16	76	14	21	27	89	24	
6	19	87	18	22	27	89	24	
7	19	89	16	23	19	89	18	
8	23	89	21	24	19	89	18	
9	23	83	21	25	23	91	23	
10	23	88	21	26	25	91	23	
11	23	85	23	27	25	93	24	
12	24	87	21	28	27	89	23	
13	24	87	21	29	27	83	24	
14	22	85	18	30	26	81	22	
15	21	86	19	31				
16	23	90	21					

Month: May 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remar
1	27	78	23	17	27	86	24	
2	26	84	23	18	27	81	23	
3	26	88	23	19	28	80	23	
4	24	84	21	20	27	85	23	
5	18	86	16	21	28	79	24	
6	23	78	19	22	28	81	24	
7	24	85	20	23	28	81	24	
8	24	85	21	24	29	89	25	
9	26	78	23	25	28	87	23	
10	-	-	-	26	29	84	23	
11	-	-	-	27	29	79	23	
12	25	89	23	28	27	83	22	
13	24	84	21	29	28	82	22	
14	26	71	23	30	29	79	23	
15	26	91	21	31	29	86	22	
16	26	53	23					

Month: June 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	29	83	22	17	27	85	24	
2	28	78	22	18	29	84	23	
3	29	86	23	19	29	80	22	
4	29	84	22	20	28	86	22	
5	28	85	23	21	30	79	22	
6	28	75	23	22	30	81	23	
7	28	81	23	23	29	90	23	
8	28	76	21	24	30	91	24	
9	29	76	22	25	29	86	23	
10	29	83	23	26	28	76	25	
11	27	86	22	27	28	78	25	
12	28	86	21	28	27	80	26	
13	28	80	22	29	30	79	25	
14	29	83	23	30	29	77	24	
15	28	78	22	31				
16	27	76	24					

Month: July 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	29	87	25	17	31	78	26	
2	30	87	26	18	30	83	25	
3	30	78	25	19	32	85	26	
4	31	80	25	20	31	87	26	
5	29	79	24	21	31	84	27	
6	29	86	24	22	32	93	28	
7	30	79	23	23	30	89	28	
8	29	84	24	24	31	78	26	
9	29	86	23	25	31	79	25	
10	28	81	25	26	31	85	25	
11	28	76	25	27	31	89	26	
12	29	77	28	28	31	89	26	
13	30	76	28	29	30	83	25	
14	30	80	28	30	30	87	25	
15	30	79	26	31	30	89	25	
16	31	80	25					

Month: Aug. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	30	84	27	17	31	93	26	
2	30	87	26	18	31	87	26	
3	30	87	26	19	31	89	26	
4	29	84	25	20	30	81	25	
5	30	85	25	21	31	89	26	
6	29	80	25	22	33	91	27	
7	28	91	26	23	32	94	26	
8	29.5	93	25	24	29	94	27	
9	28	86	26	25	32	94	26	
10	30	86	25	26	32	95	27	
11	30	86	25	27	30	94	26	
12	29	86	25	28	30	89	26	
13	29	89	24	29	30	84	25	
14	30	85	23	30	30	81	24	
15	29	87	25	31	30	83	24	
16	31	93	26					

Month: Sept. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	30	77	26	17	26	84	22	
2	30	79	25	18	26	89	22	
3	30	87	25	19	29	74	23	
4	30	84	25	20	29	76	23	
5	31	75	25	21	28	79	23	
6	30	77	25	22	26	80	24	
7	30	79	26	23	25	83	23	
8	29	79	24	24	25	83	22	
9	28	78	24	25	26	85	23	
10	27	79	24	26	26	77	22	
11	27	84	24	27	24	86	21	
12	27	84	24	28	26	80	21	
13	27	84	24	29	26	77	21	
14	26	75	23	30	26	85	21	
15	27	70	23	31				
16	27	80	23					

Month: Nov. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	25	81	23	17	20	85	19	
2	27	80	23	18	20	84	19	
3	26	80	23	19	21	89	21	
4	24	87	23	20	21	88	21	
5	24	86	23	21	17	68	15	
6	25	86	23	22	18	69	16	
7	27	80	23	23	19	74	18	
8	24	87	22	24	18	68	15	
9	25	81	22	25	18	70	16	
10	24	88	22	26	20	73	21	
11	23	82	23	27	20	70	19	
12	24	75	23	28	21	75	20	
13	24	77	22	29	22	73	19	
14	22	65	20	30	22	74	19	
15	22	73	20	31				
16	24	74	19					

Month: Oct. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	26	82	24	17	25	80	23	
2	25	73	24	18	27	85	24	
3	25	83	23	19	27	85	24	
4	24	77	23	20	27	87	23	
5	26	82	23	21	26	88	24	
6	25	81	23	22	27	80	24	
7	26	83	24	23	27	82	23	
8	23	79	23	24	27	80	23	
9	23	80	23	25	25	81	23	
10	25	78	22	26	25	90	23	
11	25	84	23	27	25	86	23	
12	24	79	21	28	24	83	23	
13	25	80	22	29	25	88	23	
14	24	80	22	30	25	87	22	
15	25	84	22	31	24	82	22	
16	25	86	23					

Month: Dec. 1976

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	21	82	19	17	22	73	13	
2	21	86	18	18	23	72	14	
3	19	86	18	19	23	78	14	
4	20	83	18	20	23	79	14	
5	20	87	18	21	22	76	13	
6	19	79	18	22	17	69	14	
7	19	83	18	23	22	82	15	
8	19	83	18	24	20	82	14	
9	15	75	14	25	19	83	15	
10	13	69	13	26	14	72	14	
11	16	72	14	27	14	82	14	
12	17	82	14	28	13	87	13	
13	17	79	14	29	14	81	13	
14	17	68	14	30	14	90	15	
15	24	76	17	31	15	84	14	
16	23	70	19					

Month: Jan. 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remar
1	14	83	14	17	20	80	17	
2	16	73	15	18	18	93	17	
3	16	92	16	19	18	93	18	
4	15	86	15	20	19	83	17	
5	14	81	14	21	18	71	16	
6	15	87	15	22	17	76	16	
7	18	90	16	23	17	62	15	
8	19	87	17	24	18	73	16	
9	20	81	17	25	17	72	16	
10	16	85	17	26	18	72	15	
11	16	86	15	27	16	78	16	
12	17	87	16	28	15	75	17	
13	14	86	15	29	15	88	14	
14	14	80	14	30	12	79	13	
15	15	82	13	31	14	78	13	
16	17	72	15		16.4	81.1	15.5	

Month: Feb. 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	15	76	14	17				
2	13	75	13	18				
3	14	78	13	19				
4	12	87	11	20				
5	12	86	12	21				
6	13	81	14	22				
7	16	73	15	23				
8				24				
9				25				
10				26				
11				27				
12				28				
13				29				
14				30				
15				31				
16								

Month: Mar. 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	23	80	20	17	23	85	21	
2	24	78	20	18	25	75	23	
3	15	69	16	19	21	70	20	
4	14	57	14	20	19	80	19	
5	15	67	16	21	22	75	18	
6	15	78	14	22	23	75	19	
7	16	70	16	23	22	80	20	
8	21	59	17	24	16	76	16	
9	22	75	17	25	17	67	15	
10	20	82	18	26	20	77	16	
11	22	82	19	27	23	79	19	
12	20	79	18	28	26	72	21	
13	21	90	19	29	24	76	21	
14	23	72	19	30	24	81	22	
15	23	82	20	31	18	74	18	
16	24	75	21		20.7	75.4	18.5	

Month: Apr. 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	17	85	17	17	26	80	24	
2	21	72	18	18	19	71	18	
3	21	79	19	19	23	68	20	
4	22	83	19	20	21	73	21	
5	26	73	22	21	22	82	21	
6	26	75	22	22	26	77	21	
7	26	79	22	23	26	75	22	
8	26	77	23	24	27	76	24	
9	24	80	22	25	27	74	25	
10	23	78	21	26	28	70	25	
11	24	74	21	27	29	70	26	
12	25	78	23	28	25	73	23	
13	26	78	24	29	23	82	22	
14	25	81	23	30	26	86	23	
15	26	70	24	31				
16	25	81	23		24.4	76.7	21.9	

Month: May 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	27	76	23	17	24	89	22	
2	28	75	24	18	22	90	22	
3	29	76	26	19	25	87	23	
4	29	69	25	20	28	80	25	
5	29	77	25	21	27	83	23	
6	28	73	26	22	26	86	21	
7	28	76	26	23	26	75	22	
8	28	78	26	24	25	69	22	
9	28	78	26	25	27	72	23	
10	28	77	26	26	28	73	24	
11	28	73	25	27	28	75	24	
12	28	80	26	28	27	81	24	
13	28	72	25	29	27	81	23	
14	29	67	26	30	27	79	23	
15	24	77	23	31	27	81	24	
16	20	84	20		26.9	77.8	24	

Month: June 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	29	82	25	17	28	76	26	
2	25	87	25	18	30	77	26	
3	26	87	24	19	28	81	26	
4	27	86	25	20	28	81	26	
5	28	82	25	21	28	84	26	
6	27	90	25	22	29	84	27	
7	26	90	25	23	29	82	27	
8	27	86	25	24	28	84	26	
9	29	80	26	25	28	88	26	
10	29	73	26	26	27	88	26	
11	29	74	26	27	28	81	27	
12	29	75	27	28	29	80	27	
13	29	74	27	29	29	84	28	
14	30	79	26	30	29	84	28	
15	29	81	27	31				
16	29	80	26		28.2	82	26.1	

Month: July 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	29	80	27	17	31	67	27	
2	29	80	27	18	32	65	28	
3	30	78	27	19	31	64	27	
4	30	82	27	20	32	64	28	
5	31	74	27	21	32	65	28	
6	30	70	27	22	31	67	27	
7	30	75	27	23	30	66	27	
8	31	71	28	24	30	67	26	
9	31	76	28	25	30	67	27	
10	31	71	27	26	28	89	26	
11	31	75	28	27	28	87	26	
12	30	69	28	28	28	83	26	
13	29	74	28	29	29	83	27	
14	29	74	27	30	29	80	26	
15	31	75	27	31	28	80	26	
16	31	67	28		30.1	73.7	27.1	

Month: Aug. 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	28	83	27	17	31	76	28	
2	28	85	27	18	31	72	27	
3	29	85	27	19	29	88	29	
4	30	81	27	20	29	87	27	
5	30	79	27	21	28	86	27	
6	30	83	27	22	28	88	26	
7	30	82	27	23	29	88	27	
8	31	76	28	24	28	85	28	
9	31	77	28	25	29	84	27	
10	31	72	28	26	29	83	27	
11	31	73	28	27	29	78	27	
12	32	80	29	28	30	80	27	
13	31	79	28	29	30	69	27	
14	32	73	28	30	29	75	27	
15	31	73	28	31	30	72	27	
16	31	74	28		29.8	79.5	27.4	

Month: Sep. 1977

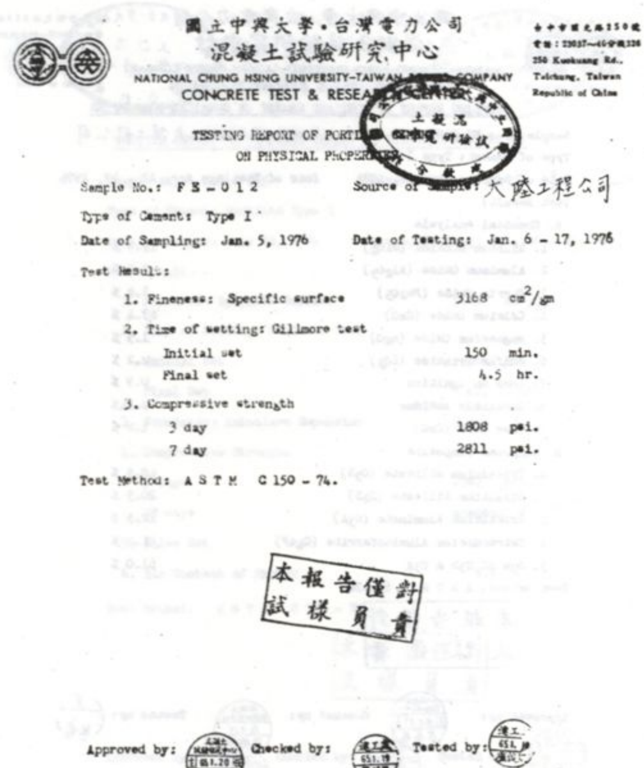
Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remark
1	32	67	27	17	29	76	27	
2	31	69	27	18	30	69	27	
3	31	70	28	19	30	75	27	
4	32	65	28	20	28	86	26	
5	32	64	28	21	27	85	25	
6	32	66	28	22	26	83	25	
7	32	67	28	23	25	85	24	
8	31	70	27	24	28	79	26	
9	29	78	27	25	27	88	24	
10	30	70	27	26	27	86	26	
11	31	67	28	27	27	84	25	
12	30	73	27	28	27	84	27	
13	30	73	27	29	27	86	27	
14	29	83	27	30	29	78	27	
15	30	75	28	31	29.3	76.1	26.7	
16	29	83	27					

Month: Oct. 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	28	79	26	17	26	76	25	
2	27	86	26	18	25	72	25	
3	28	86	27	19	25	69	24	
4	29	75	27	20	25	68	24	
5	28	82	26	21	25	67	23	
6	27	83	26	22	25	69	23	
7	27	83	26	23	27	73	25	
8	28	84	26	24	27	72	24	
9	26	82	24	25	27	72	25	
10	23	65	23	26	27	73	25	
11	23	67	24	27	28	64	25	
12	24	68	21	28	28	68	25	
13	24	70	22	29	28	60	25	
14	25	73	23	30	28	65	25	
15	26	78	24	31	29	68	26	
16	25	78	24		26.4	73.4	24.6	

Month: Nov. 1977

Date	Average Temp. C	Relative Humitidy %	Water Temp.	Date	Average Temp. C	Relative Humitidy %	Water Temp.	Remarks
1	27	79	24	17	20	82	19	
2	24	67	20	18	20	77	19	
3	23	65	20	19	20	79	19	
4	24	81	22	20	20	72	18	
5	26	78	24	21	20	81	18	
6	26	84	25	22	19	64	16	
7	27	79	24	23	19	62	16	
8	25	73	24	24	21	71	19	
9	22	73	21	25	22	74	20	
10	23	71	21	26	22	75	20	
11	22	68	21	27	20	78	19	
12	23	60	20	28	18	77	17	
13	23	68	21	29	17	65	16	
14	23	73	20	30	17	71	16	
15	21	86	19	31	21.8	73.9	19.9	
16	21	84	19					



國立中興大學・台灣電力公司
混凝土試驗研究中心
NATIONAL CHUNG HSING UNIVERSITY-TAIWAN POWER COMPANY
CONCRETE TEST & RESEARCH CENTER

台中市國光路250號
電話：23037-40分機326
250 Kuokuang Rd.,
Taichung, Taiwan
Republic of China

TESTING REPORT OF PORTLAND CEMENT
ON PHYSICAL PROPERTIES

Sample No.: F E - 0 1 2　　Source of Sample: 大陸工程公司

Type of Cement: Type I

Date of Sampling: Jan. 5, 1976　　Date of Testing: Jan. 6 - 17, 1976

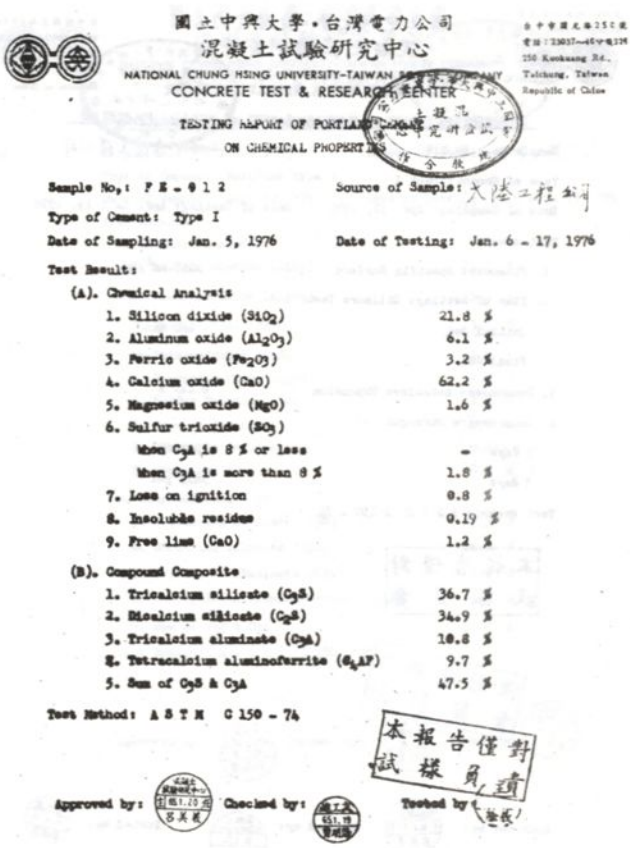
Test Result:

1. Fineness: Specific surface	3168	cm^2/gm
2. Time of setting: Gillmore test		
Initial set	150	min.
Final set	4.5	hr.
3. Compressive strength		
3 day	1808	psi.
7 day	2811	psi.

Test Method: A S T M　C 150 - 74.

本報告僅對
試樣負責

Approved by:　　Checked by:　　Tested by:



國立中興大學・台灣電力公司
混凝土試驗研究中心
NATIONAL CHUNG HSING UNIVERSITY-TAIWAN POWER COMPANY
CONCRETE TEST & RESEARCH CENTER

台中市國光路250號
電話：23037-40分機326
250 Kuokuang Rd.,
Taichung, Taiwan
Republic of China

TESTING REPORT OF PORTLAND CEMENT
ON CHEMICAL PROPERTIES

Sample No,: F E - 0 1 2　　Source of Sample: 大陸工程公司

Type of Cement: Type I

Date of Sampling: Jan. 5, 1976　　Date of Testing: Jan. 6 - 17, 1976

Test Result:

(A). Chemical Analysis

1. Silicon dixide (SiO_2)	21.8 %
2. Aluminum oxide (Al_2O_3)	6.1 %
3. Ferric oxide (Fe_2O_3)	3.2 %
4. Calcium oxide (CaO)	62.2 %
5. Magnesium oxide (MgO)	1.6 %
6. Sulfur trioxide (SO_3)	
When C_3A is 8 % or less	-
When C_3A is more than 8 %	1.8 %
7. Loss on ignition	0.8 %
8. Insolubke residue	0.19 %
9. Free lime (CaO)	1.2 %

(B). Compound Composite

1. Tricalcium siliate (C_3S)	36.7 %
2. Dicalcium siliate (C_2S)	34.9 %
3. Tricalcium aluminate (C_3A)	10.8 %
4. Tetracalcium aluminoferrite (C_4AF)	9.7 %
5. Sum of C_3S & C_3A	47.5 %

Test Method: A S T M　C 150 - 74

本報告僅對
試樣負責

Approved by:　　Checked by:　　Tested by:

國立中興大學・台灣電力公司
混凝土試驗研究中心
NATIONAL CHUNG HSING UNIVERSITY-TAIWAN POWER COMPANY
CONCRETE TEST & RESEARCH CENTER

台中市國光路250號
電話：23037-40分機326
250 Kuokuang Rd.,
Taichung, Taiwan
Republic of China

TESTING REPORT OF PORTLAND CEMENT
ON PHYSICAL PROPERTIES

Sample No.: F E - 0 1 8　　Source of Sample: 大陸工程公司

Type of Cement: Type I

Date of Sampling: Mar. 22, 1976　　Date of Testing: Mar. 22 - 29, 1976

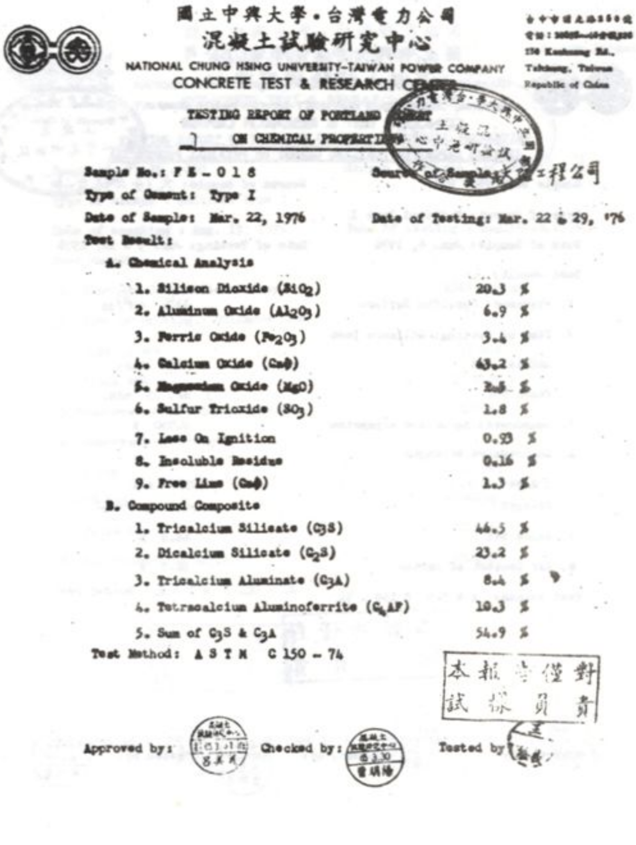
Test Result:

1. Fineness: Specific Surface	3088	cm^2/gm
2. Time of Setting: Gillmore Test		
Initial Set	160	min.
Final Set	5	hr.
3. Soundness: Autoclave Expansion	0.133	%
4. Compressive Strength		
3 days	2330	psi.
7 days	3409	psi.

Test Method: A S T M　C 150 - 74

本報告僅對
試樣負責

Approved by:　　Checked by:　　Tested by:



國立中興大學・台灣電力公司
混凝土試驗研究中心
NATIONAL CHUNG HSING UNIVERSITY-TAIWAN POWER COMPANY
CONCRETE TEST & RESEARCH CENTER

台中市國光路250號
電話：23037-40分機326
250 Kuokuang Rd.,
Taichung, Taiwan
Republic of China

TESTING REPORT OF PORTLAND CEMENT
ON CHEMICAL PROPERTIES

Sample No.: F E - 0 1 8　　Source of Sample: 大陸工程公司

Type of Cement: Type I

Date of Sample: Mar. 22, 1976　　Date of Testing: Mar. 22 - 29, '76

Test Result:

A. Chemical Analysis

1. Silicon Dioxide (SiO_2)	20.3 %
2. Aluminum Oxide (Al_2O_3)	6.9 %
3. Ferric Oxide (Fe_2O_3)	3.4 %
4. Calcium Oxide (CaO)	63.2 %
5. Magnesium Oxide (MgO)	2.5 %
6. Sulfur Trioxide (SO_3)	1.8 %
7. Loss On Ignition	0.93 %
8. Insoluble Residue	0.16 %
9. Free Lime (CaO)	1.3 %

B. Compound Composite

1. Tricalcium Siliate (C_3S)	46.5 %
2. Dicalcium Silicate (C_2S)	23.2 %
3. Tricalcium Aluminate (C_3A)	8.4 %
4. Tetracalcium Aluminoferrite (C_4AF)	10.3 %
5. Sum of C_3S & C_3A	54.9 %

Test Method: A S T M　C 150 - 74

本報告僅對
試樣負責

Approved by:　　Checked by:　　Tested by: