AN IMMERSED VEHICULAR TUNNEL
UNDER THE
RIO PANUCO, MEXICO

R. Y. Young

A TECHNICAL REPORT
in the
Faculty of Engineering

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ABSTRACT

The dissertation depicts the project planning and design development of the subaqueous vehicular tunnel under the Rio Panuco. Various schemes for tunnel construction were investigated for economy in cost and time while assuring structural safety.

Two categories of tunnel cross section, 2 methods of tunnel fabrication and construction, 7 methods of applied waterproofing and 5 types of tunnel foundations were analysed in order to obtain the optimal combination. The final selection was based on cost-benefit relationships as the optimal solution does not necessarily mean the optimal in cost.

Throughout the dissertation, each aspect of immersed tunnel construction was first introduced by a treatise and the newly-acquired understanding was then used to plan and design the tunnel under consideration.

The method of immersed tunnel construction proposed for the present tunnel is unique in that the precast tunnel units are to be fabricated on a launchway, launched, transported by partially submerged gantry cranes, sunk and joined to form a continuous "pipe". It is believed that up to 20% saving was obtained by the use of the proposed method instead of the conventional methods of tunnel construction.

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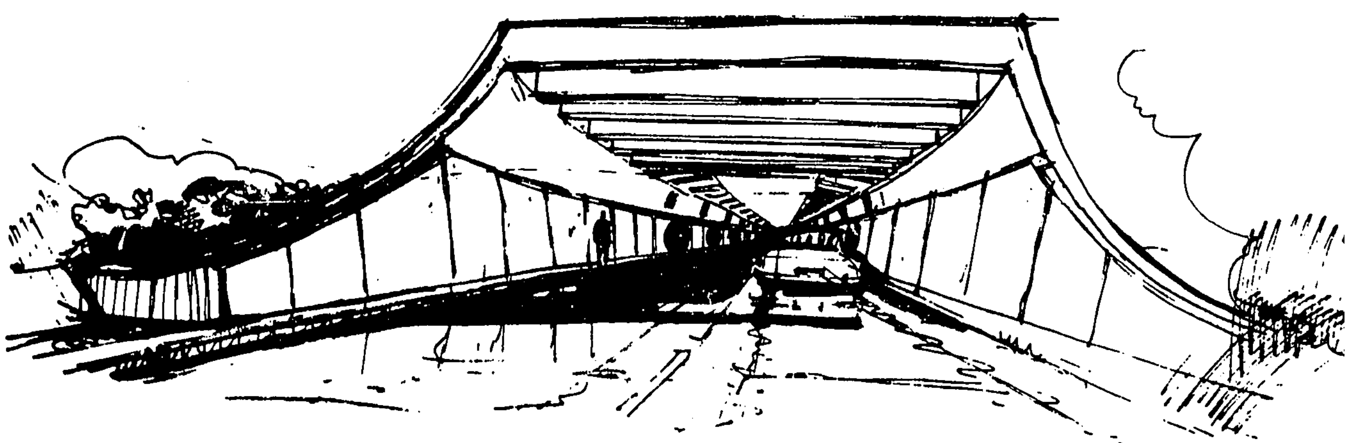
R.Y.YOUNG



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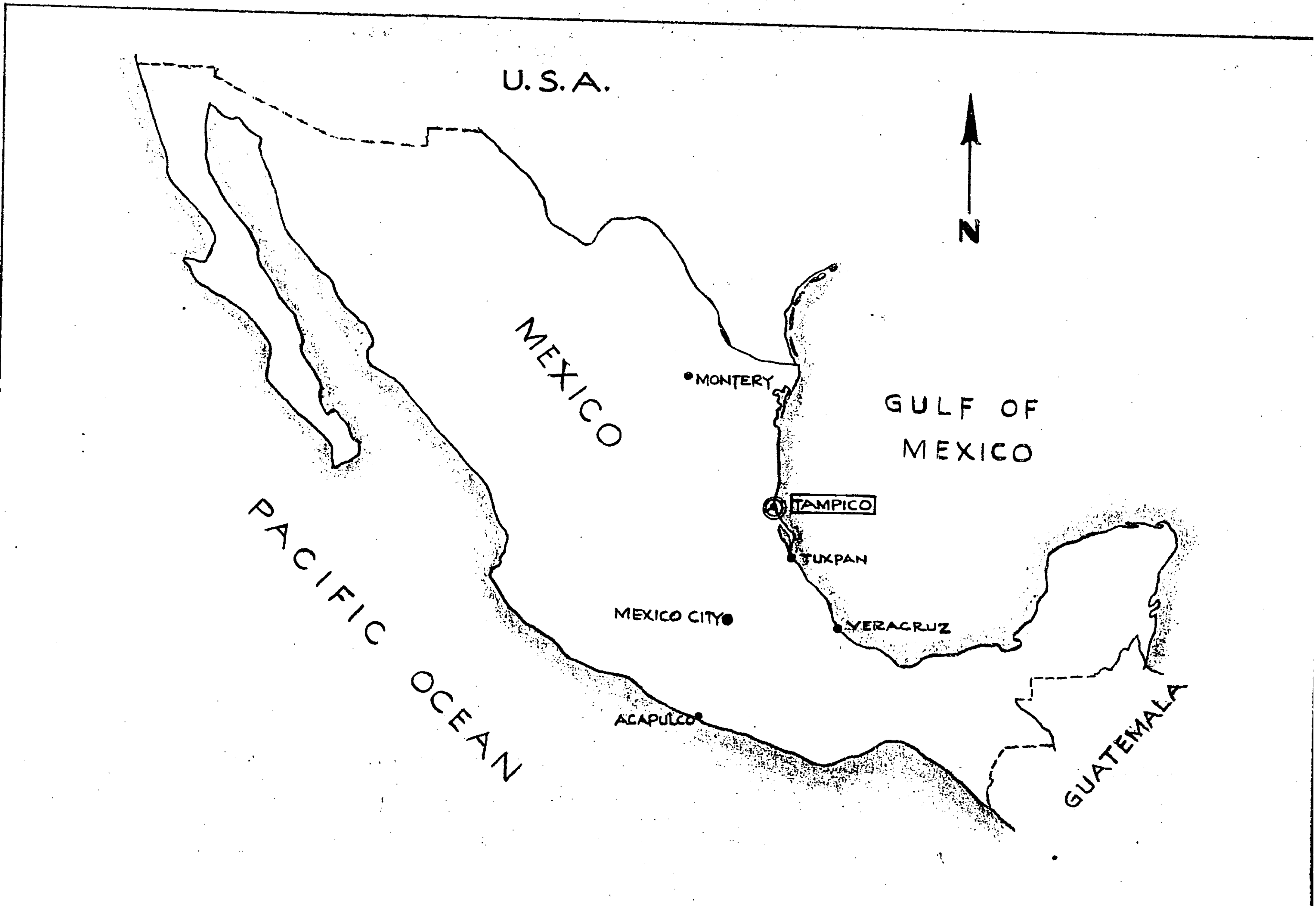
U.S.A.
N
MEXICO
MONTERY
GULF OF
MEXICO
TAMPICO
TUXPAN
MEXICO CITY
VERACRUZ
ACAPULCO
PACIFIC OCEAN
GUATEMALA

ACKNOWLEDGEMENT

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Montreal, Canada
June, 1970

R. Y. Y.

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SYNOPSIS

Of the many underwater vehicular tunnels built in the past few decades, be they for the purpose of traffic or conveyance, a large number have been constructed using the method of immersed-tube.

While there are numerous publications available concerning the construction and design of underwater crossings, many of them are only resumes written for general knowledge. Not many of these papers expound the complete planning and design of the structure but merely describe the project as a whole or describe a particular feature of the project. Most of the few technical books available in English on the subject of tunnelling lack comprehensiveness and the necessary information for underwater tunnel design and construction. The dissertation presented herewith intends, through the complete planning, design and proposed construction methods of a particular tunnel under extremely adverse physical conditions partly to bridge the gap that often occurs in ordinary technical papers on this subject. It must be emphasized, however, that this dissertation is dealing with only a preliminary project, not a final work ready for construction.

To date, many of the subaqueous concrete tunnels built using the immersed-tube method have been constructed by fabricating the tunnel sections in a dry dock on the shore. The dry dock is then flooded and opened and the various tunnel sections are then towed to the site, aligned and sunk to the desired elevations. Some sort of foundation was first prepared or prepared after the sections were in place. This procedure so far has been a common practice and is considered to be one of the conventional methods of subaqueous concrete tunnel construction.

In many concrete underwater crossings, the total cost of fabrication, casting, placing and founding can often be approximately divided into 40-50% in the fabrication of the structural cross section and 50-60% in the casting, placing, sinking and foundation preparation. For a specified set of internal tunnel dimensions, the savings incurred by obtaining the optimal cross section in terms of least materials is often around 5% over that of any cross section designed to meet the specified internal requirements. It seems that substantial economy can be realized by devising better casting, placing, sinking and founding techniques alone. The following dissertation will depict a method which differs markedly from the conventional methods of immersed tunnel construction as well as describing the evolution of the tunnel cross section which considers aspects of architecture, mechanical layout, public safety and structural engineering in arriving at the most preferable solution viewed from the standpoint of utility and cost-benefit relationships. It is believed that a total saving of over 20% of the costs for construction by conventional methods could be achieved by the methods proposed for the design and construction of this tunnel.

Throughout the dissertation, a general introduction stating where possible the reasons and descriptions for "why" and "how" precedes each section in every chapter. In the body of the dissertation, as many reasons and explanations as possible are given for the more important points raised in the hope of answering any queries that a reader unfamiliar with the subject of subaqueous tunnel construction may have. This dissertation contains some very important information on the subject and may be considered to have some value to engineers and designers who are not intimately involved in immersed tunnel construction.

The 13 chapters in the main body constitutes a complete dissertation on the topic treated. However, 4 appendices are included at the end for those readers who wish to go deeper into the aspects of more detailed preliminary planning and design.

CHAPTER 1

INTRODUCTION

General - The demand for subaqueous tunnels arose because of the rapid development of traffic in cities situated along navigable watercourses and the tremendous increase in the pace of urban life. Subaqueous railway and public utility tunnels were the first representatives of this group of traffic carriers, but were soon followed by subaqueous vehicular tunnels. To cope with these new developments, a wide variety of construction methods and materials has been devised and the quality of many construction materials has been improved upon, partly in response to the new problems encountered.

Among the many construction techniques, two principal methods of underwater tunnel construction were developed. These were the shield-driven and the immersed-tube methods. A large number of vehicular tunnels have been built by both these methods with the choice between the two primarily determined by site conditions and particular circumstances. In the case of the Baltimore Tunnel in the U.S.A., a cost saving of up to 30% was attained by the use of the immersed-tube instead of the shield-driven method of construction.

The construction of submerged tunnels by the sinking of prefabricated sections into an underwater trench is well known. Although the first immersed steel shell tunnel was built some 80 years ago, the first concrete tunnel built by such a method dates back to a little over 40 years ago. During the following 20 years, however, only four more were built by this method. Now close to thirty major vehicular tunnels have been built by sinking prefabricated sections. Almost half of these immersed tunnels have been built in the U.S.A. using circular tunnel cross sections while the remaining half have mostly been constructed in Europe, using rectangular cross sections. Canada, now having two rectangular highway tunnels, came into the era of subaqueous tunnel construction in 1959 with the inauguration of the Deas Island Tunnel in Vancouver.

Tampico Tunnel - The tunnel referred to in this report will be Mexico's first major underwater vehicular tunnel project. It will be a two-lane rectangular tunnel to be constructed using the immersed-tube method with the prefabricated concrete sections built on a launching pad which will also be the approach ramp.

1.1 History

To provide continuity of the coastal highway between Tampico, Tamaulipas, and Tuxpan, Veracruz, at its intersection with the Panuco River which the highway system has to cross, the Highways Division, Department of Public Works of Mexico instituted studies to establish the most suitable type of crossing structure which would avoid interference with shipping activities within the Port of Tampico.

A permanent crossing between Tampico and Mata Redonda would also facilitate access to the largely undeveloped area bordering the south bank of the river which is potentially suitable for residential, industrial and commercial activities and, in particular, for an extensive national as well as local port development. Thus, the crossing structure would enhance and stimulate the rapidly expanding economy of the City of Tampico, its suburbs and port.

Comparisons were to be made of the relative technical and economic merits of a long-span, high level bridge and a subaqueous tunnel. In this regard, it was deemed important that special considerations be given to the effect on vehicular traffic of the heavy rains and strong winds which prevail during the hurricane season from July to October of each year.

A four-lane, long-span, high level suspension bridge was first studied by the Mexican Government in 1967. However, little information is available concerning this structure. It is known only that the bridge would, at the present price level, cost in the vicinity of 13 million dollars.

In December, 1967, Per Hall Associates Ltd. of Montreal, Canada, in association with Ing. Arturo M. Morales Dominguez of Mexico City, was requested by the Highways Division, Department of Public Works of Mexico, to report upon the technical feasibility and comparative costs of construction for a two-lane and a four-lane subaqueous tunnel crossing of the Rio Panuco.

An interim feasibility report was submitted to the

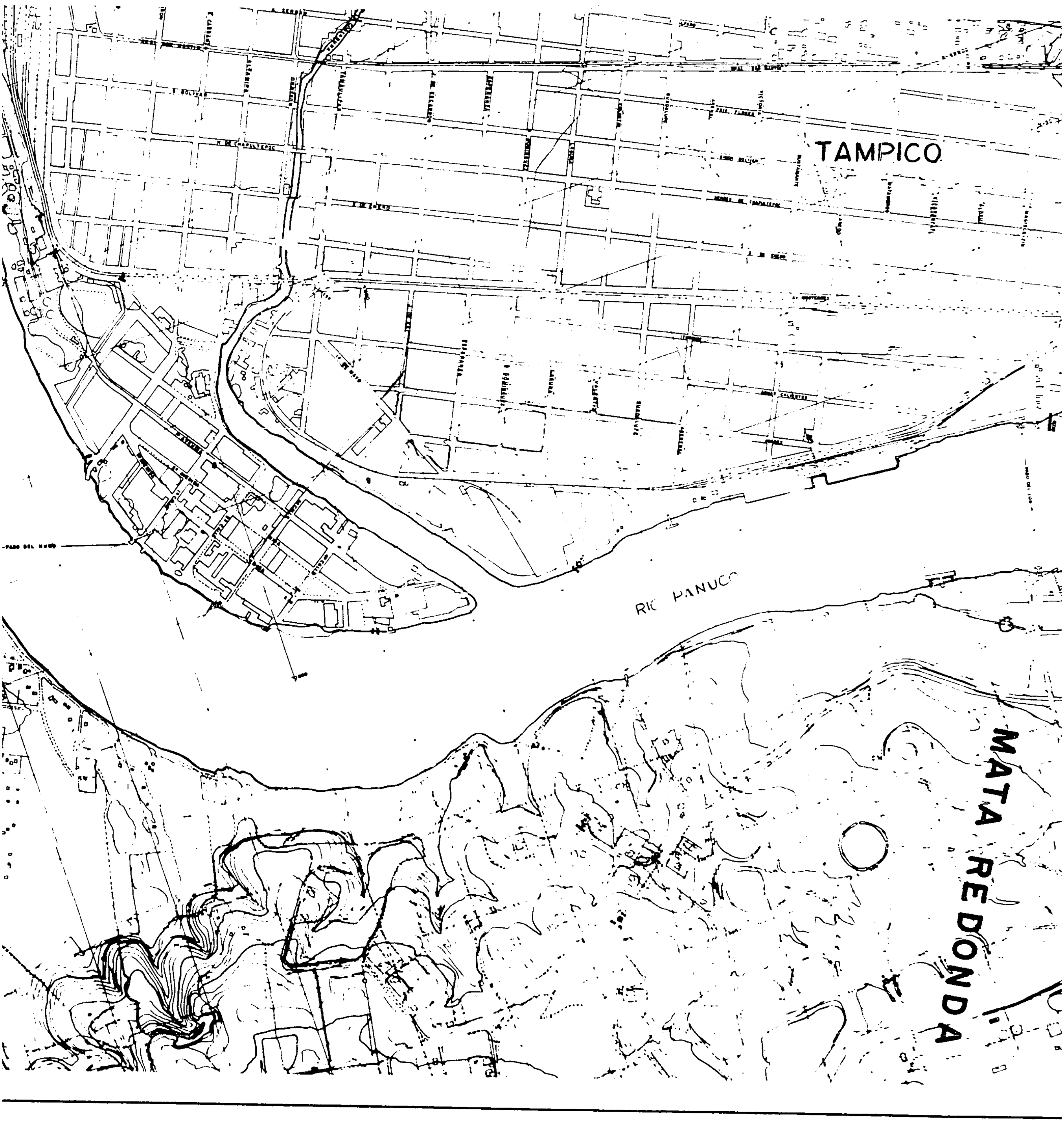
Department of Public Works in June, 1968. The report indicated that a four-lane tunnel could be built between Tampico and Mata Redonda at an estimated cost of 16.8 million dollars or a two-lane tunnel at a cost of approximately 12.1 million dollars, based on 1969 price level.

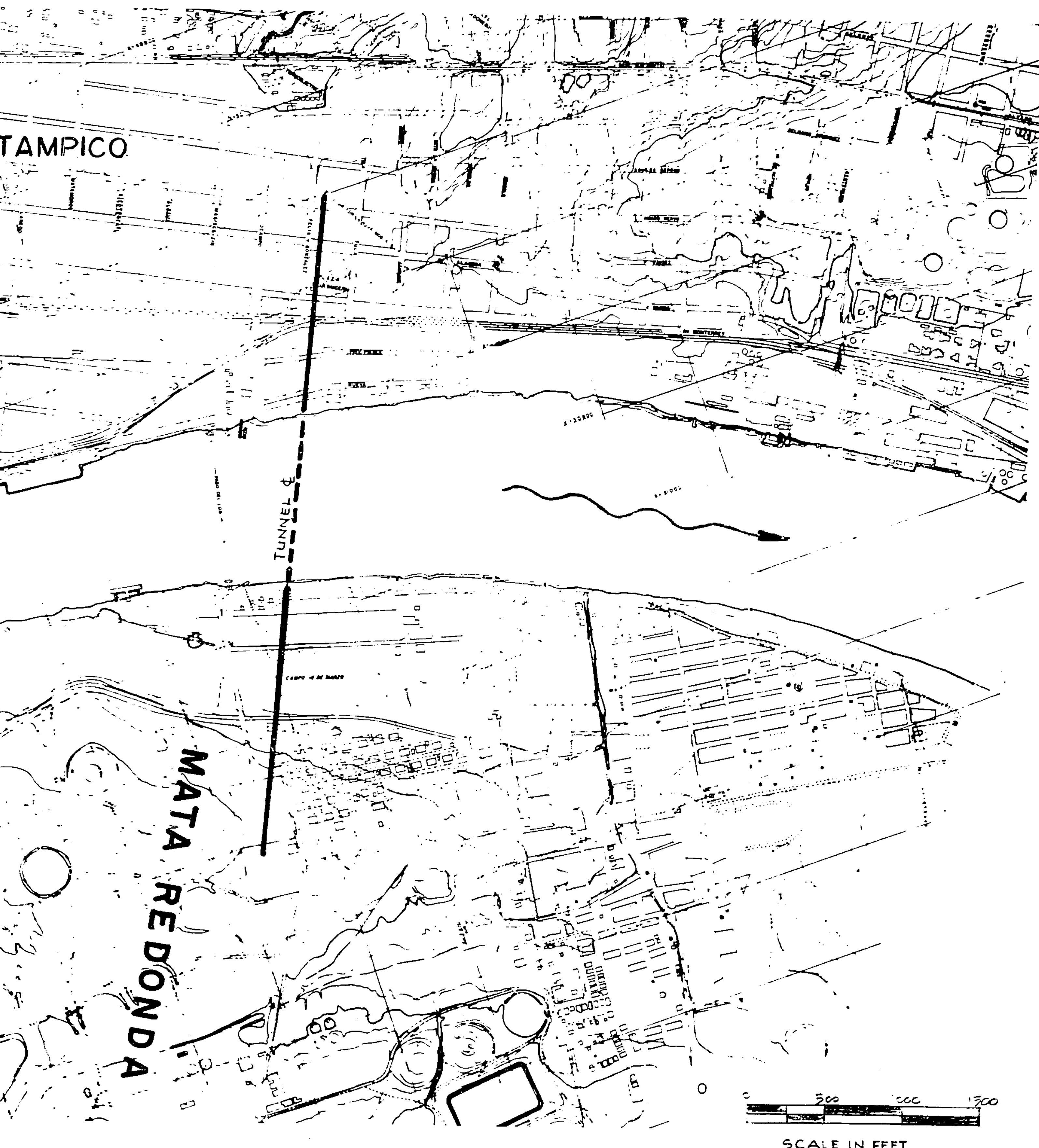
Following a review of the feasibility report submitted, the Highways Division, in November, 1968, authorized the two engineering firms to proceed with a more detailed study relating to project planning and design development for a two-lane tunnel with the alignment as shown in Fig. 1.1.

1.2 Scope

The crossing runs under the Rio Panuco between Tampico, Tamaulipas and Mata Redonda, Veracruz. The scope of the design development included in this dissertation extends only to the tunnel itself and its approaches on either side of the river as shown in Fig. 1.2. However, the complete government project would extend to connect to the major highway system going to Tuxpan and Veracruz.

In selecting the alignment for the tunnel crossing, traffic requirements, river hydraulics and future navigation to the inner port of Tampico were to be duly considered. This report will be developed as required by the government, for an alignment previously chosen for a high level bridge so as to benefit from available information on soil conditions. Also the topography here makes it possible to provide for flood protion of the tunnel.

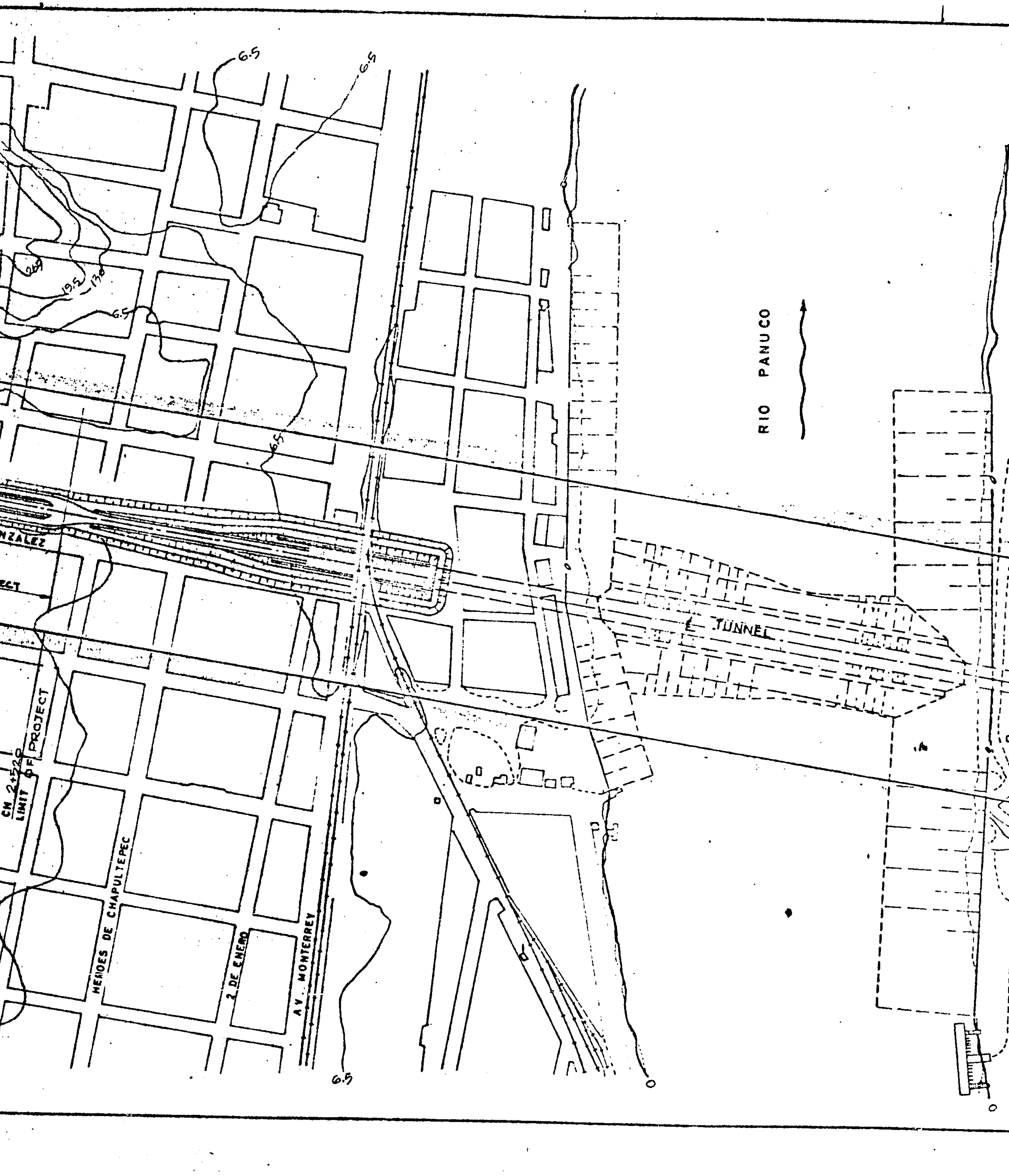

TAMPICO
RIO PANUCO
MATA REDONDA

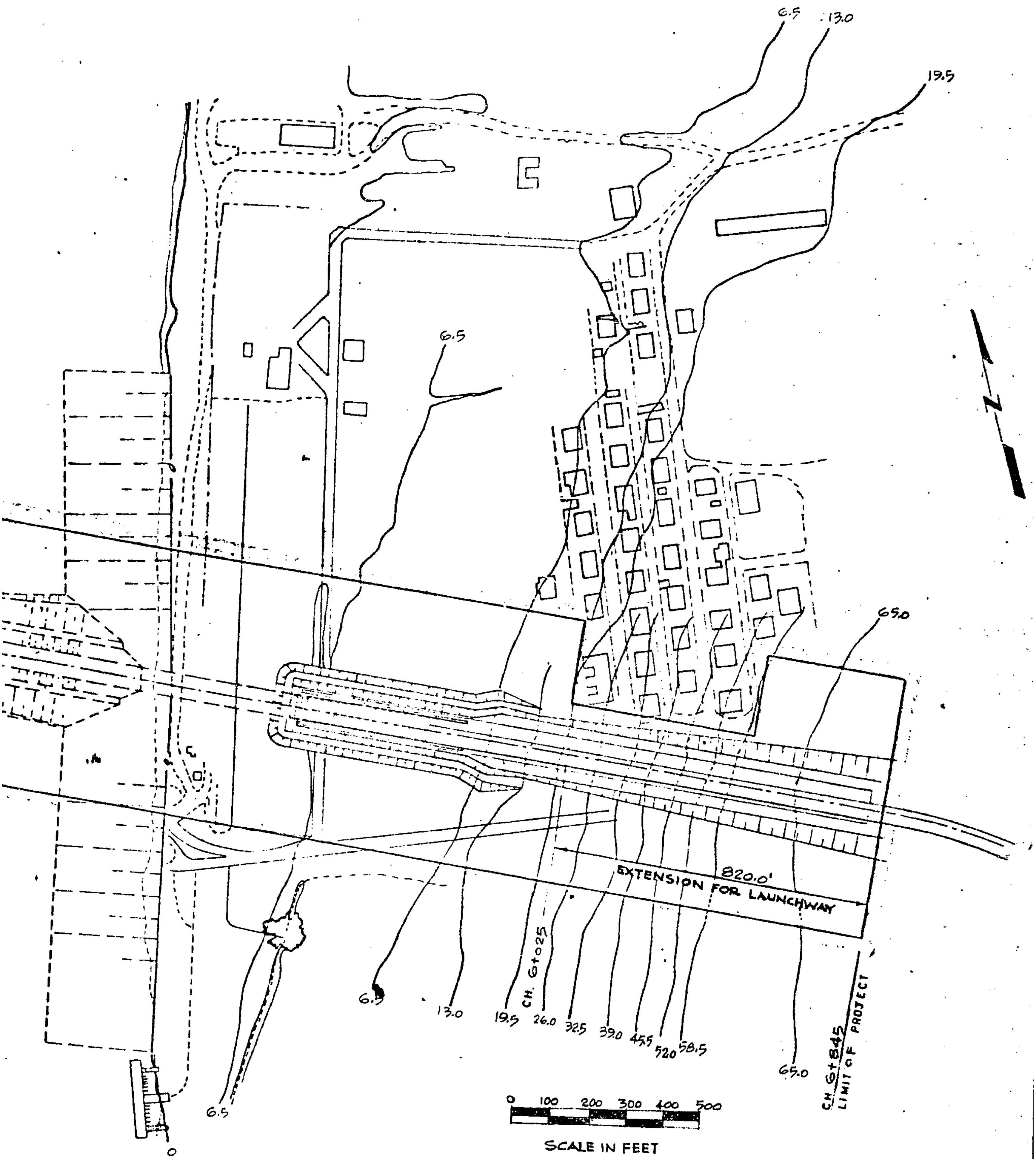


TOPOGRAPHY OF TUNNEL
CROSSING SITE

Fig. 1.1


TULA
FELIX GONZALEZ
NOT IN THE PROJECT
OCAMPO
MAGISCATZIN
DOMINGUEZ
BELISARIO
PRIV. FLORES
CH 2+520
LIMIT OF PROJECT
HEROES DE CHAPULTEPEC
32.5
19.5
13.0
6.5


RIO PANUCO
TUNNEL
CH 2+520
LIMIT OF PROJECT
HEROES DE CHAPULTEPEC
2 DE ENERO
AV. MONTERREY
6.5
13.0
19.5



GENERAL ARRANGEMENT

FINISHED STRUCTURES

Fig. 1.2

CHAPTER 2

PHYSICAL SITE CONDITIONS

General - The most important phase of preliminary work in tunnel construction is the careful exploration of the physical site conditions. The geological and hydrological environments decisively affect both the loads acting on the tunnel and the choice of the preferable construction method to be employed. The general location of the tunnel is governed by existing traffic or economic interests, while the exact location is controlled by the physical conditions prevailing in the area. The more carefully and accurately the geological and hydrological conditions of the proposed location and its environment are explored, the more rapidly and economically can the tunnel be constructed.

Tampico Tunnel - As the accurate determination of the physical conditions of the region concerned influence profoundly the actual construction of the tunnel, these relevant conditions will be analysed in some detail in order to arrive at a set of reasonable and safe parameters for the design and construction of the Tampico Tunnel.

2.1 Climate

The City of Tampico, located on the northern bank of the Rio Panuco, some 7.5 miles from the Gulf of Mexico, has a semitropical climate.

Temperatures exhibit a wide range of variation both seasonal and from year to year. The extreme daily temperatures observed during the period 1921-1967 range between a maximum of 107°F and 33.8° F, the average being 82.5° F and 68.5° F in summer and winter respectively. The annual average is 76° F.

Rainfall in the Rio Panuco basin is extremely variable. Throughout the 1921-1967 period, the annual precipitation at Tampico averaged about 39.5", varying between a maximum of 76.5" and a minimum of 18.2". For the month of September 1955, a total rainfall of 36.3" was recorded. Records of storms in the Rio Panuco basin show precipitation rates as high as 16.2" per day.

Fig. 2.1 presents an analysis of the frequency of cyclones in the Gulf of Mexico Zone which has varied from 0 to 9 per year during the 1921-1968 period. Cyclones and heavy rains occur annually during the months of July through October.

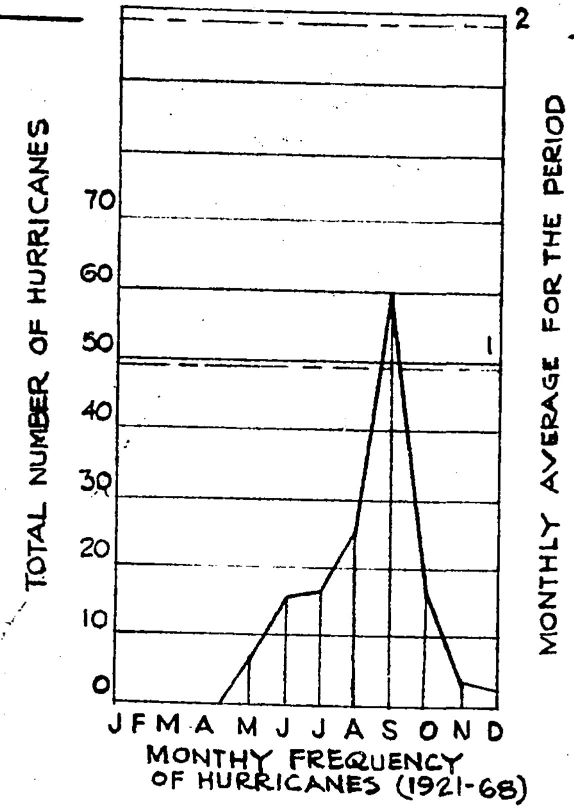
Wind velocities during the passage of cyclones may ordinarily reach 60 to 90 mph. A gust velocity of about 140 mph was recorded during hurricane "Hilda" in 1955.

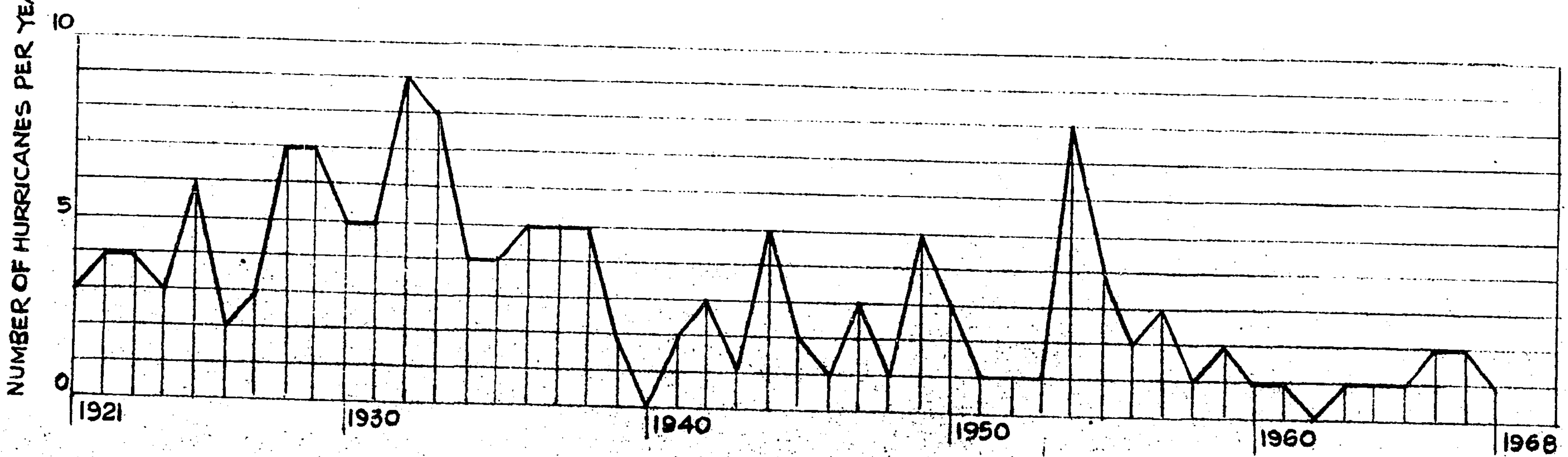
2.2 Shore Topography

The natural topography on both sides of the Rio Panuco in the proximity of the crossing is shown in Fig. 1.1. Low-

FREQUENCY OF HURRICANES

Fig. 2.1





ANNUAL FREQUENCY OF HURRICANES IN THE GULF OF MEXICO ZONES lying lands along the shores rise gently from El. 0.00' to El. +6.5' some 1,000' and 2,300' inland on the Mata Redonda and Tampico sides respectively, thence rising more rapidly to form small hills about 80' high. In general, low-lying lands bordering a navigable river actively used by ocean shipping such as the Rio Panuco, favour a tunnel crossing structure.

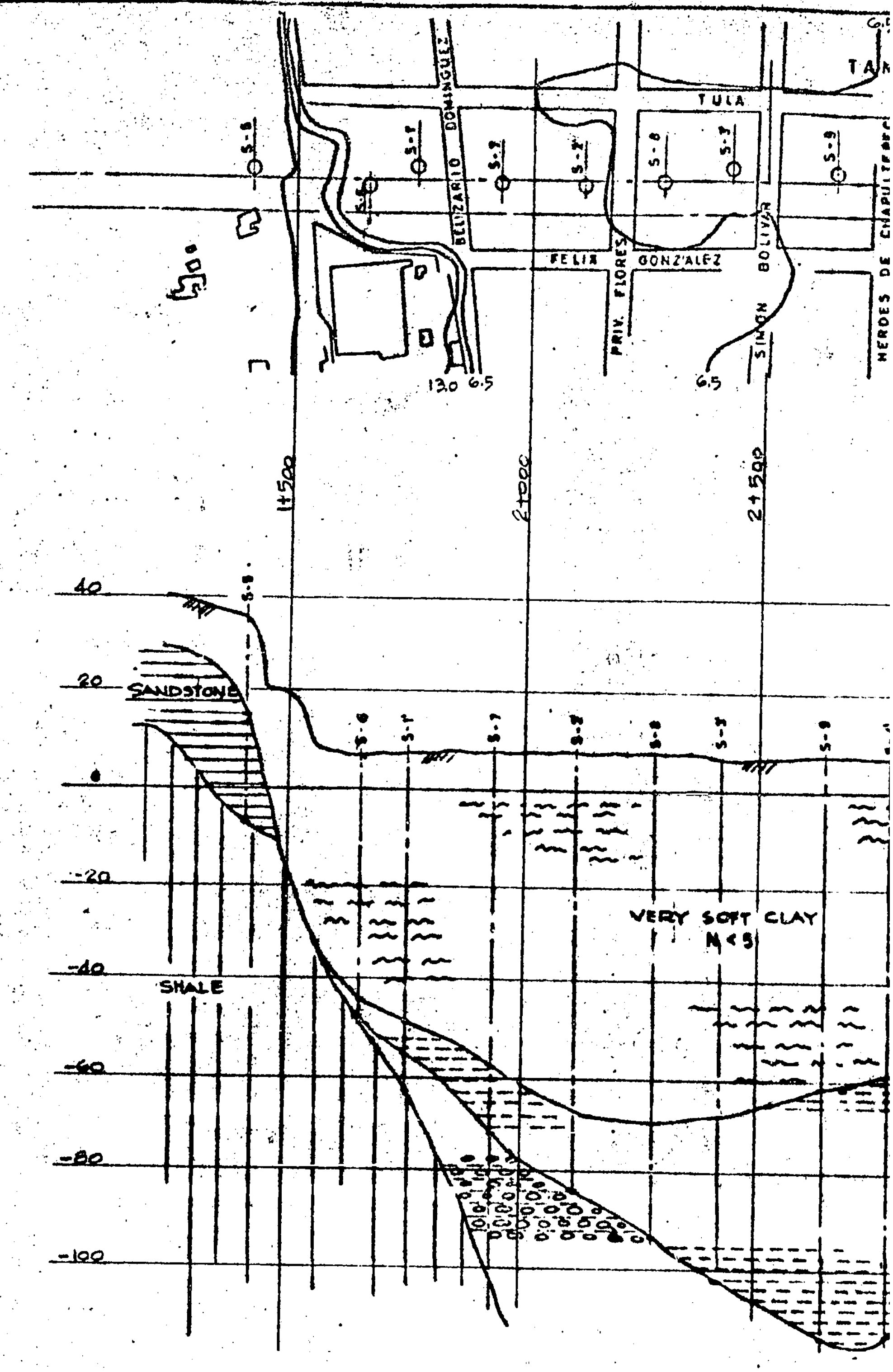
2.3 Soil Characteristics

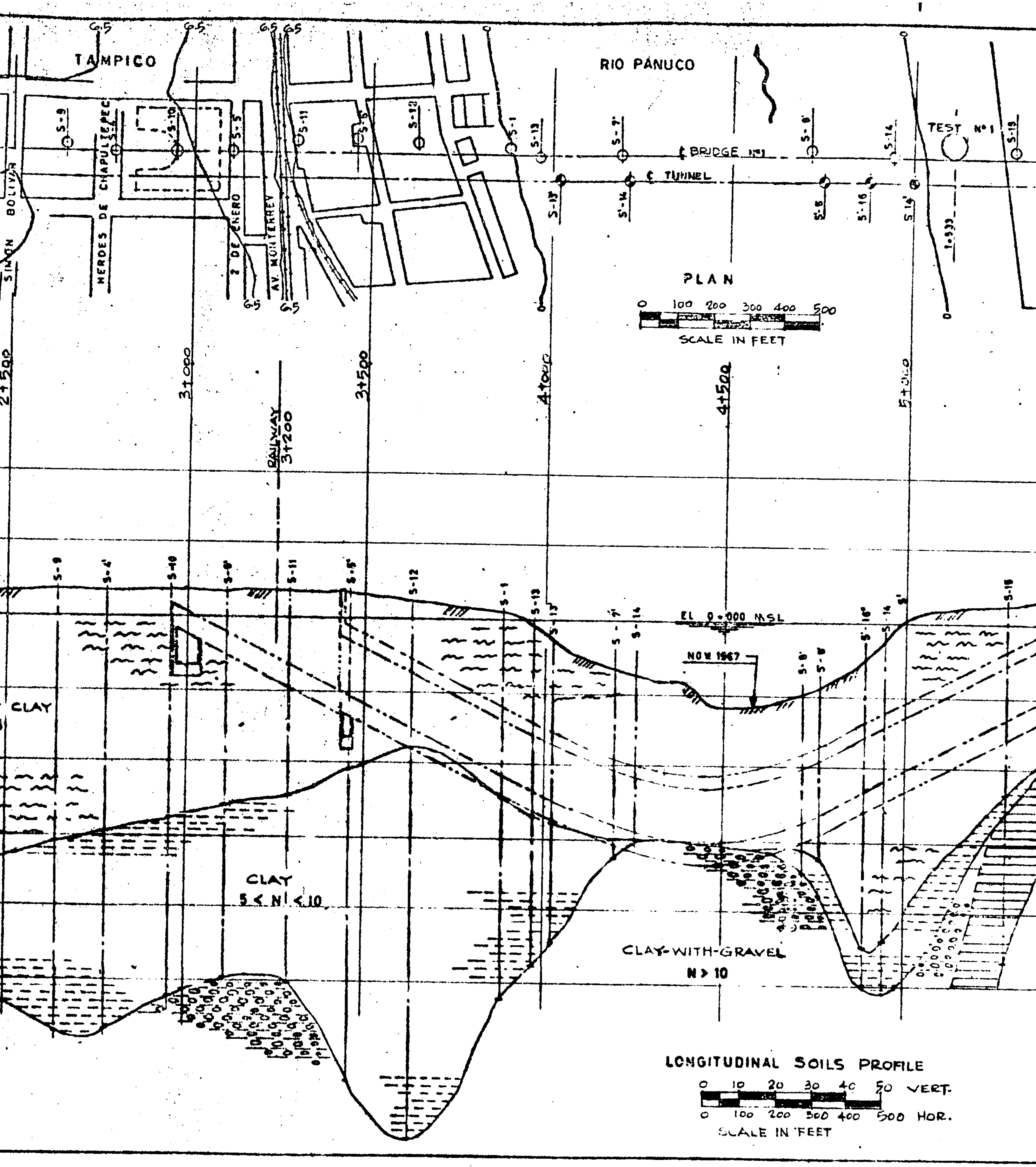
Fig. 2.2, showing the locations of borings, penetration tests, two pumping tests and soil classification profiles, was prepared from data furnished by the Department of Public Works. Although most of the borings were made on the alignment for the bridge study, they are assumed to be equally applicable to preliminary studies for the proposed tunnel alignment which is parallel but 30' upstream.

The soil classification profile indicates little uniformity in thickness in the strata of the various soils encountered. In general, the soils comprise alluvial deposits of clays of different thicknesses, overlying a base of sandstone, hardened clay and weathered shales. The clays, in descending order below the ground surface, may be classified as very soft clay, clay and clay-with-gravel.

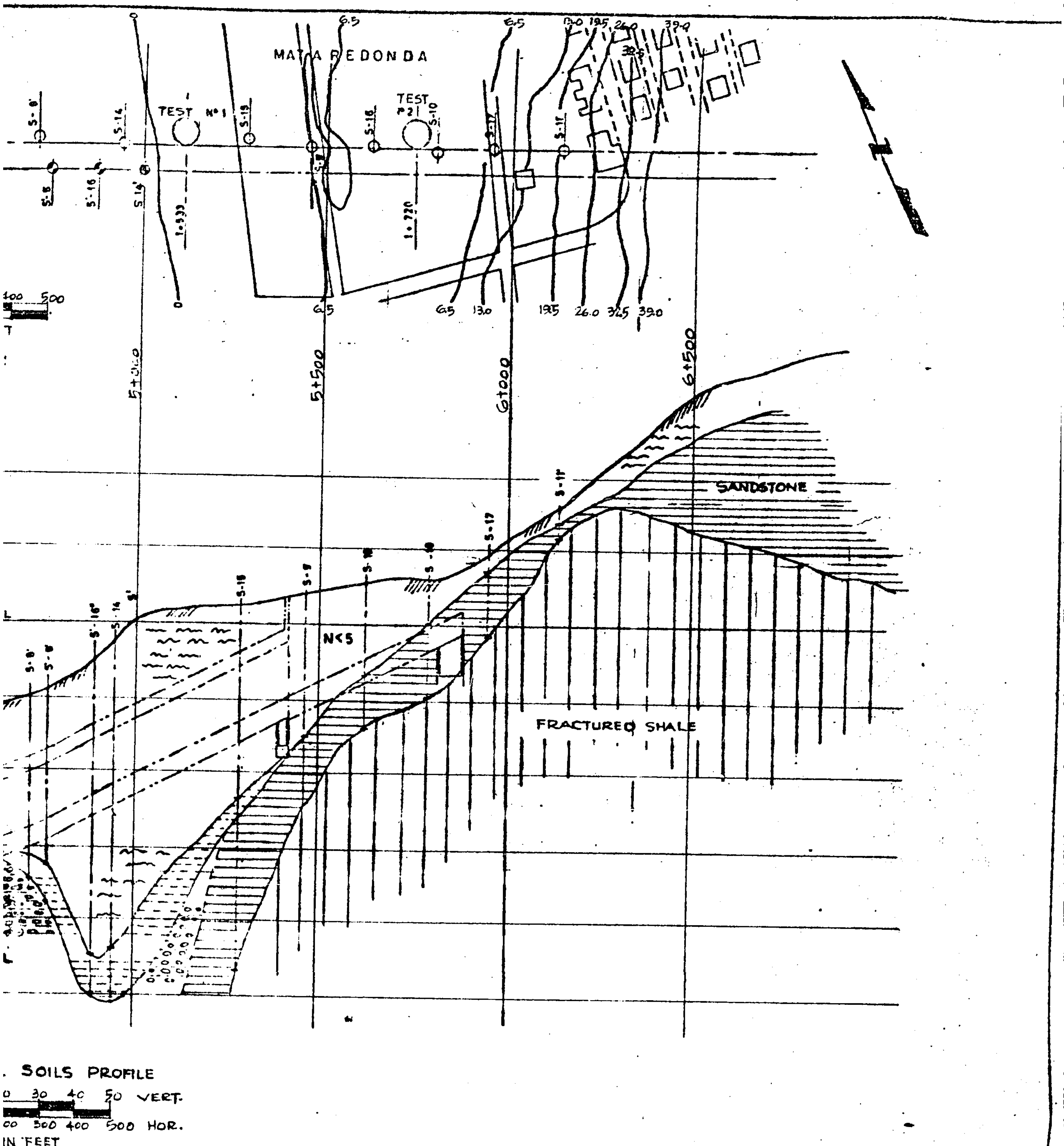
The hills on both sides of the river consist mainly of hardened clay and weathered shales, overlain by sandstone with a relatively shallow layer of soft clay at the surface.

The Department of Public Works estimated that the coef-


TULA
BELIZARIO DOMINGUEZ
FELIX GONZ'ALEZ
PRIV. FLORES
SIMON BOLIVAR
HEROES DE CHAPULTEPEC
13.0
6.5
6.5
1+500
2+000
2+500
40
20
0
-20
-40
-60
-80
-100
SANDSTONE
SHALE
VERY SOFT CLAY
N<5



TAMPICO
RIO PANUCO
PLAN
0 100 200 300 400 500
SCALE IN FEET
BRIDGE Nº1
TUNNEL
TEST Nº1
HEROES DE CHAPULTEPEC
SIMON BOLIVAR
2 DE ENERO
AV. MONTERREY
2+500
3+000
RAILWAY 3+200
3+500
4+000
4+500
5+000
EL 0+000 MSL
NOV 1967
CLAY
CLAY 5 < N < 10
CLAY-WITH-GRAVEL N > 10
LONGITUDINAL SOILS PROFILE
0 10 20 30 40 50 VERT.
0 100 200 300 400 500 HOR.
SCALE IN FEET



BORINGS AND SOIL
CLASSIFICATION PROFILE

Fig. 2.2

ficient of permeability of the water bearing layers at Test No. 1 was in the order of 2×10^{-5} fps. In Test No. 2, artesian flow was encountered between 50' and 60' below ground level with a pressure of 7 psi, equivalent to a 16.5' column of water. The permeability of the sandstone within the artesian layer was estimated to be in the order of 1.0×10^{-4} fps.

2.4 Seismic Considerations

Very little information is available on earthquake-resistant design of structures buried in the ground. No soil tests have been performed at Tampico for the purpose of earthquake analysis.

The Tampico region lies outside the major seismic zones of Mexico and is thus subjected to only moderate shocks resulting from activities in zones of the circum-Pacific belt and the Caribbean loop. North American building codes indicate that Tampico lies within a No. 3 seismic zone subject to shocks of Richter Magnitude 6 to 7.

Earthquake damages could be caused by faulting or shaking. There are no recorded large earthquake epicentres at the tunnel crossing site and the dislocation of the tube by fault movements is not probable. Intense shaking of the ground results in inertia forces and ground distortions. More tests are required to determine the ground shearing deformations.

Another danger from earthquake is that the loose fine sands or silt adjacent to the tunnel may liquefy when subjected to oscillatory stresses and strains. The increase in density of this surrounding fluid could endanger the stability as well as overstressing the outer walls of the tunnel. However, sandy clays or clays with coarse sand or gravel, as found under the Tampico Tunnel, seldom become quick. Local liquefaction may result where spots of clay occurs, but this condition does not endanger the tunnel as a whole. Dynamic soil tests will be required to verify this presumption.

2.5 River Hydraulics

The Rio Panuco exhibits a wide irregularity of flow. During major hurricanes and cyclones, the river overflows to a great extent into low-lying floodplains with the discharge at Tampico largely affected by the locations and capacities of existing lagoons, swamps and other retention areas in the lower basin. The discharge is also affected by the difference in tidal levels between Tampico and the river mouth.

Discharge measurements at Tampico are practically nonexistent, but records for the period 1955-1967 have been obtained for several upstream gauging stations. Although, because of the peculiar flood regime of the river, the data from upstream stations are not readily applicable to the precise determination of hydraulic conditions at Tampico as they relate to the design flood, erosion, siltation, salinity intrusion and other such factors, they have been useful in estimating values of sufficient accuracy to satisfy the requirements of the present study.

2.5.1 Flood Discharge

The maximum annual flood discharges at Tampico for the years 1955-1967 have been estimated on the basis of records available at two upstream locations.

Estimated values of peak flood discharge, corresponding river flow velocities, and river stages are summarized in Fig. 2.3. Estimated values of the maximum discharges at Tampico for the 13-year period have varied from a low of 70,000 cfs in 1967 to a maximum of 600,000 cfs in 1955. The mean annual flood is of the order of 210,000 cfs. These values are subject to modification by more extensive and precise flood-routing studies.

Fig. 2.4 shows the frequency curves of maximum daily river discharges for November-June, the 8 month period contemplated for subaqueous construction operations. Fig. 2.5 gives the duration curve for the river discharges which are likely to occur during the same months.

2.5.2 Water Levels

Fig. 2.3b shows the computed water levels at Tampico, referred to Mean Sea Level of El. 0.00', for the annual peak floods for the period 1955-1967. Fig. 2.6 shows the maximum and minimum water levels likely to occur during the months between consecutive hurricane seasons.

The following tabulation gives water levels at the proposed tunnel site governing design and construction features:

Max. recorded level (1955 flood)	El. +11.50'
Average flood level (computed)	El. + 3.60'

RIO PANUCO

ANNUAL MAXIMUM FLOOD AT TAMPICO

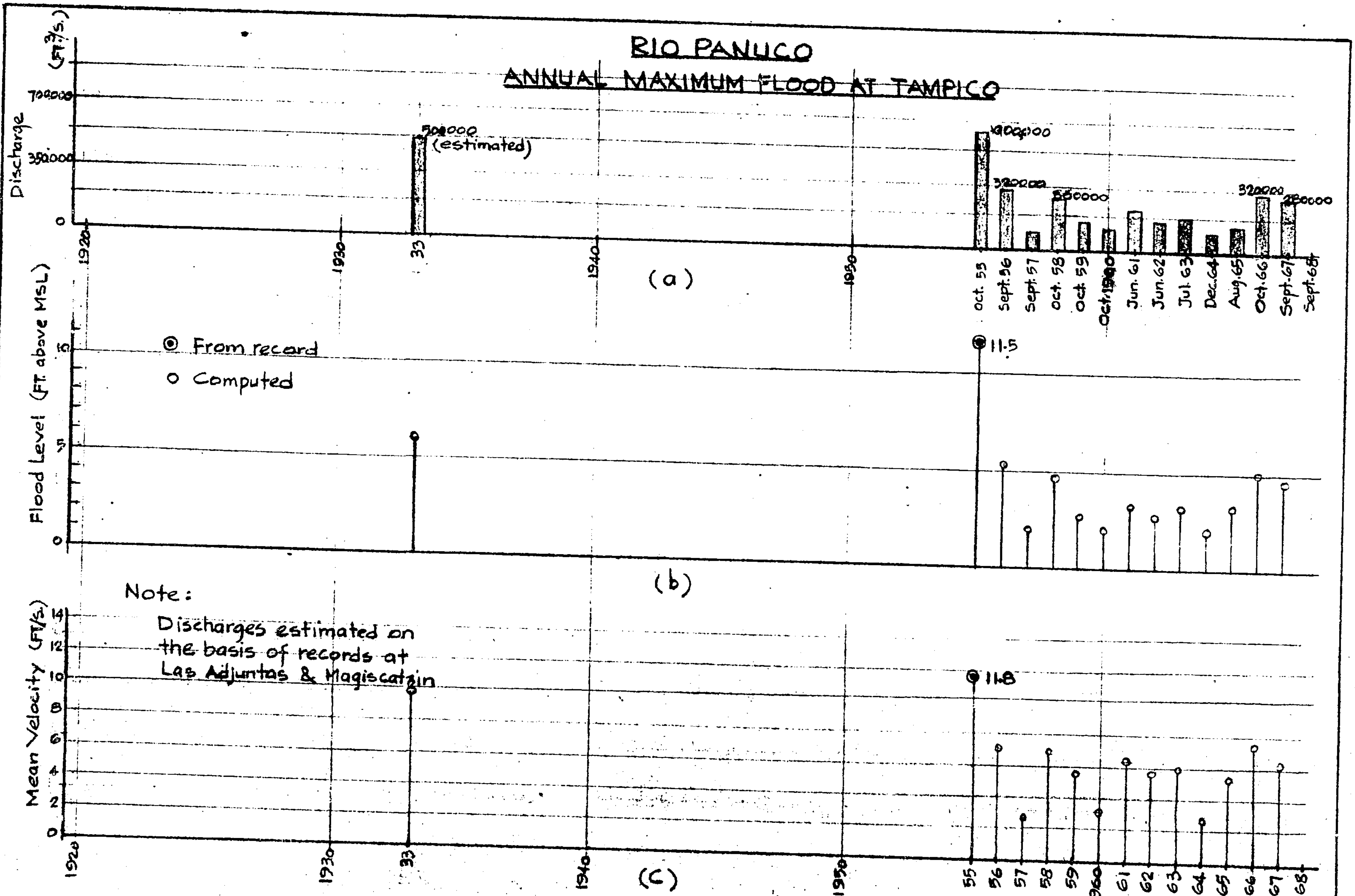
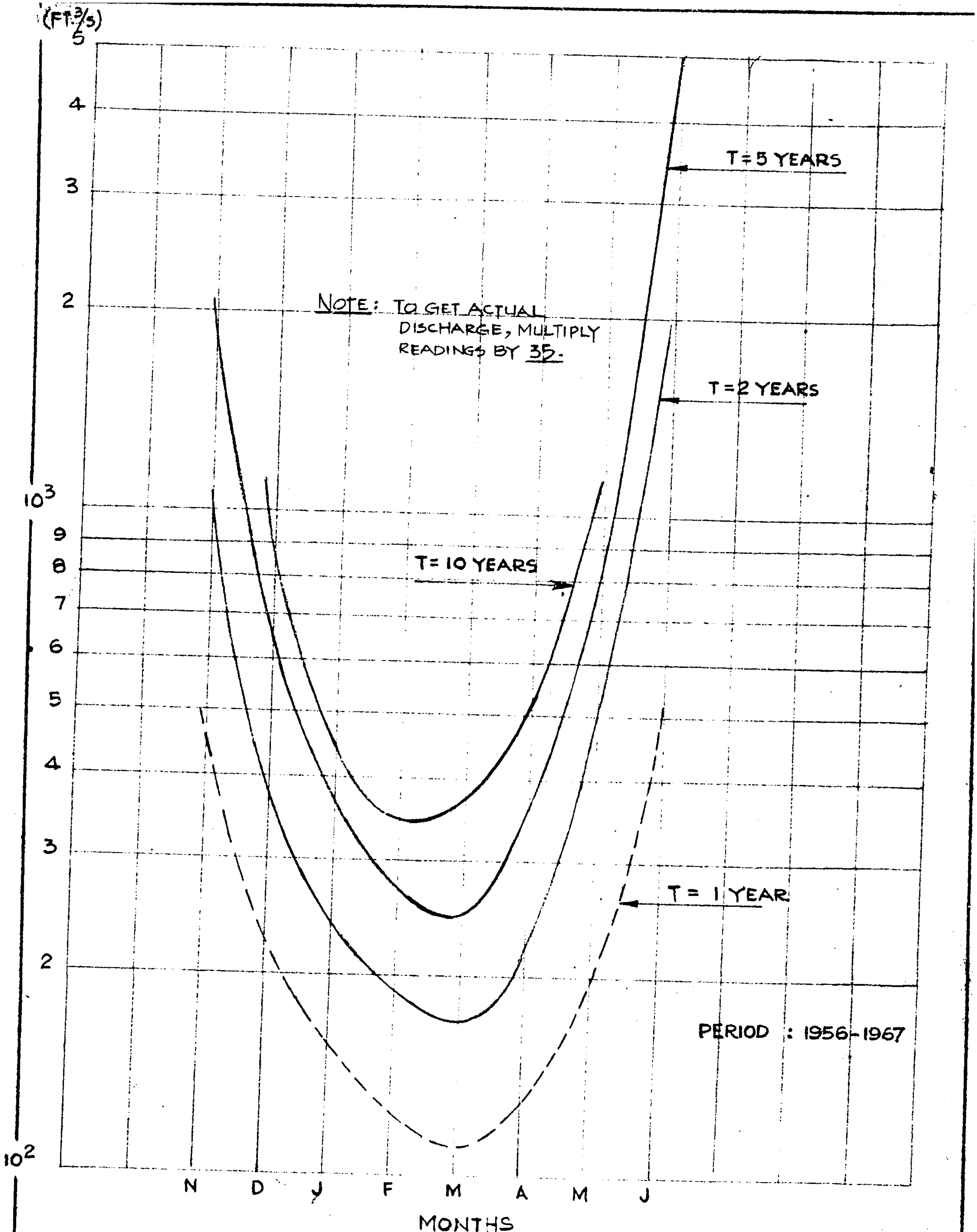
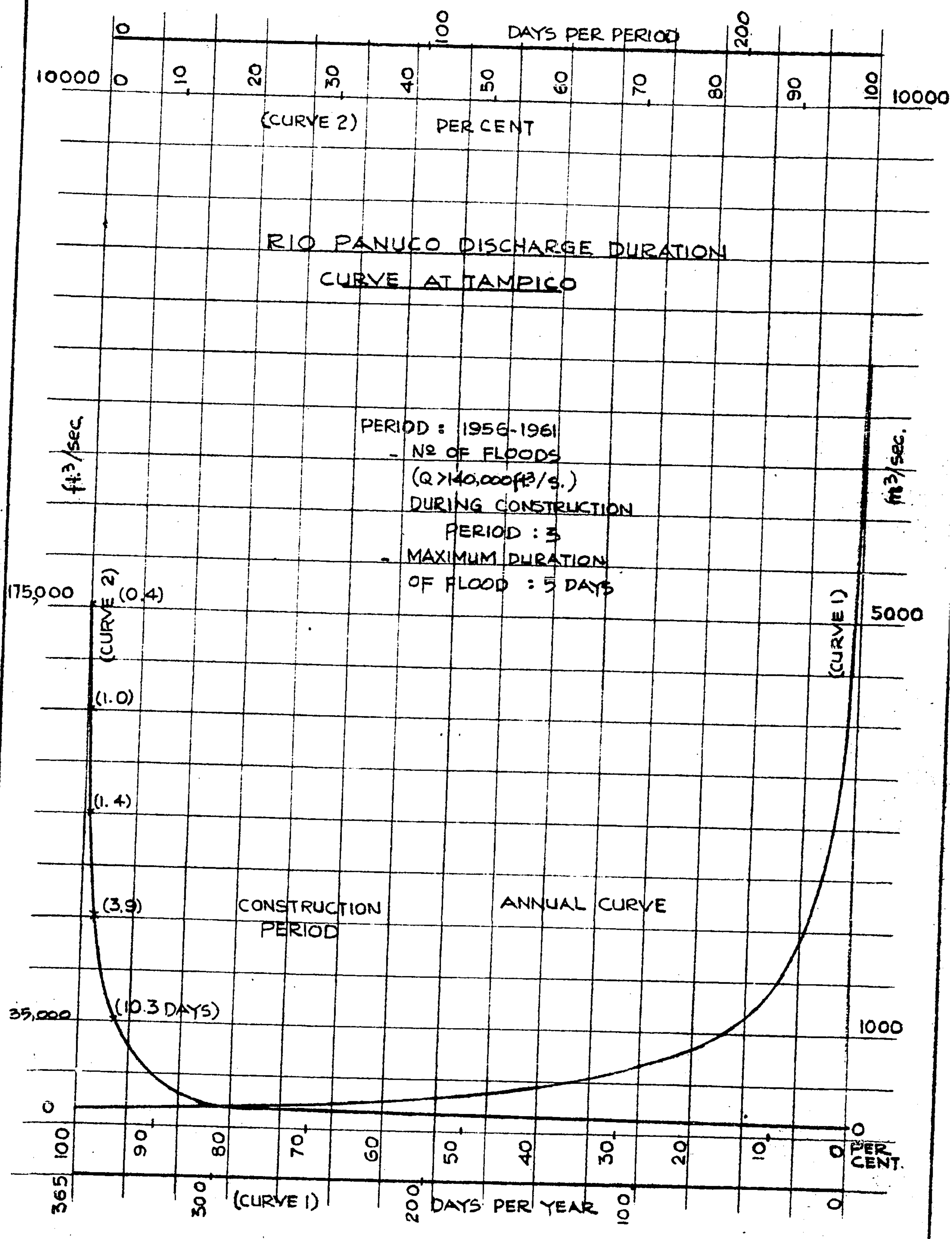


Fig. 2.3



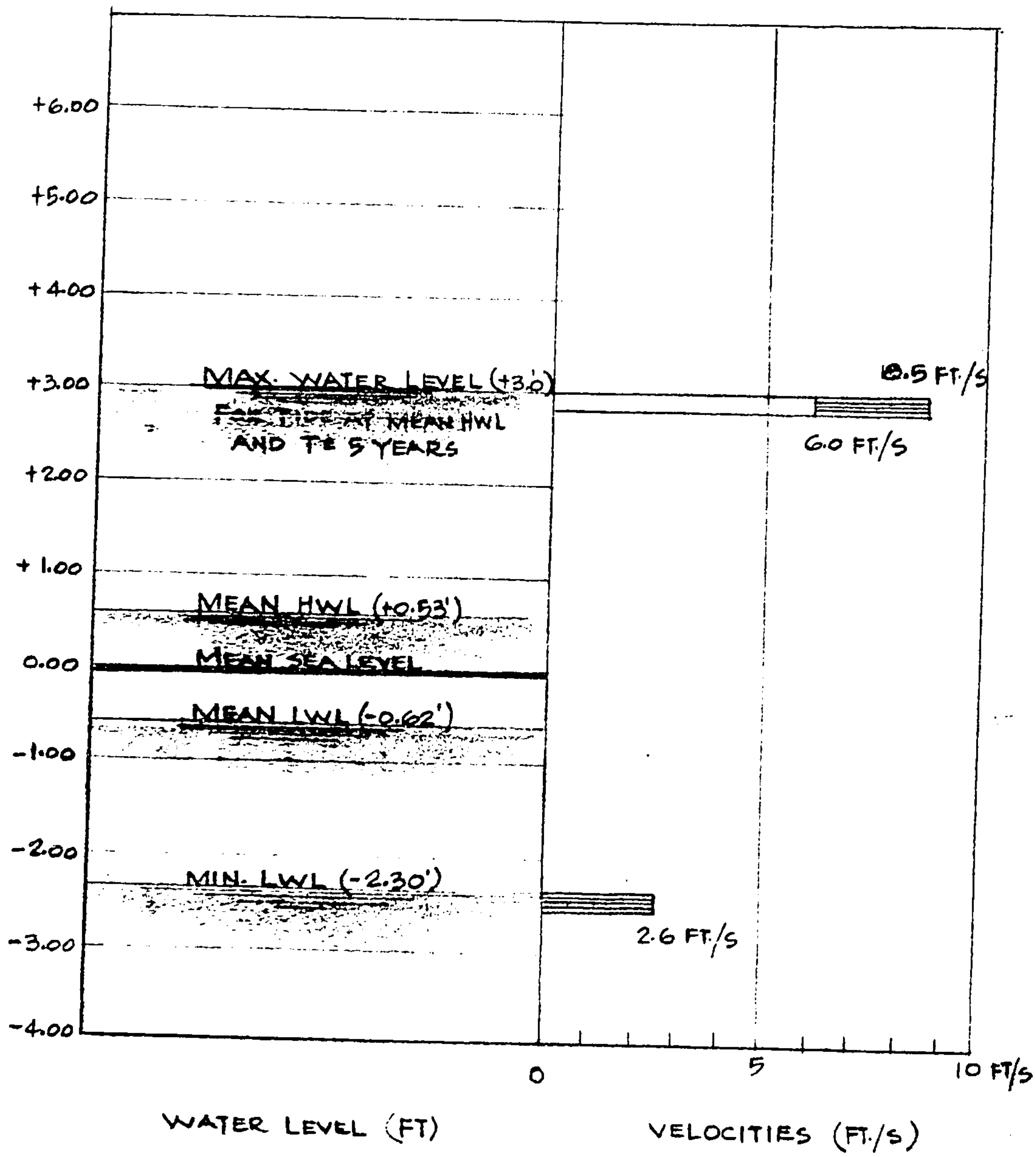
RIO PANUCO FREQUENCY OF MAXIMUM DAILY DISCHARGES DURING CONSTRUCTION

Fig. 2.4



RIO PANUCO DISCHARGE DURATION CURVE AT TAMPICO
DAYS PER PERIOD
0
100
200
10000
0
10
20
30
40
50
60
70
80
90
100
10000
(CURVE 2)
PER CENT
PERIOD : 1956-1961
- Nº OF FLOODS
(Q>140,000ft³/s.)
DURING CONSTRUCTION
PERIOD : 3
- MAXIMUM DURATION
OF FLOOD : 5 DAYS
ft³/sec.
m³/sec.
175,000
(CURVE 2)
(0.4)
(1.0)
(1.4)
(3.9)
(10.3 DAYS)
35,000
5000
(CURVE 1)
1000
CONSTRUCTION PERIOD
ANNUAL CURVE
0
100
90
80
70
60
50
40
30
20
10
0
PER CENT.
365
300
(CURVE 1)
200
DAYS PER YEAR
100
0

Fig. 2.5



MAXIMUM AND MINIMUM WATER LEVELS AND VELOCITIES FOR CONSTRUCTION MONTHS NOV. TO JUNE

Fig. 2.6

Max. level for November-June	El. +3.12'
Min. level	El. -2.30'

2.5.3 River-flow Velocities

River-flow velocities corresponding to the estimated maximum annual flood discharges are given in Fig. 2.3c. The computed river flow velocity at the maximum flood discharge recorded is of the order of 13 fps.

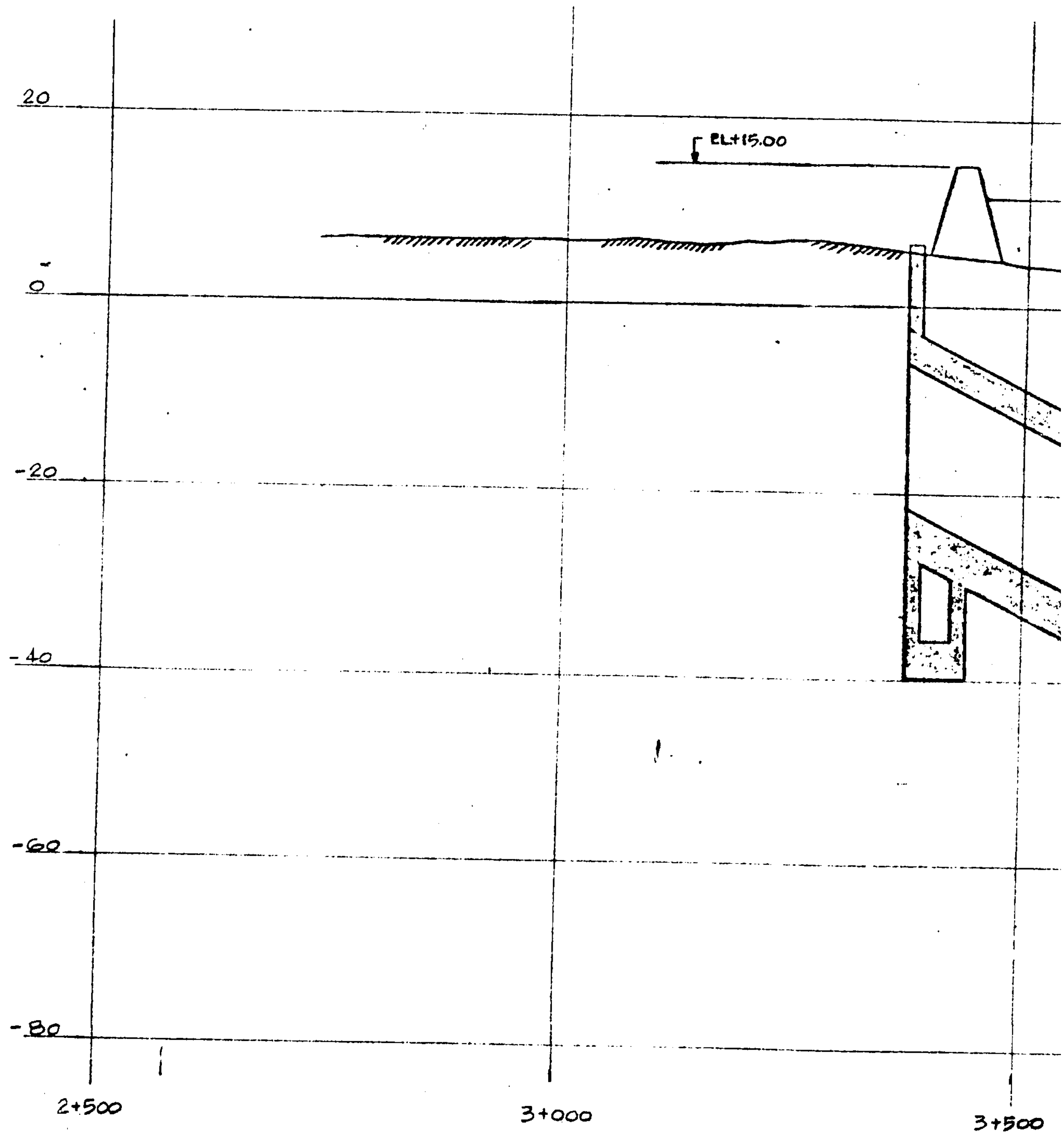
The maximum and minimum current velocities that may be expected during the hurricane-free period are also shown in Fig. 2.6 as 8.5 fps and 2.6 fps respectively. The minimum velocity corresponds to the average tidal current in the Panuco estuary.

2.5.4 Erosion and Sedimentation

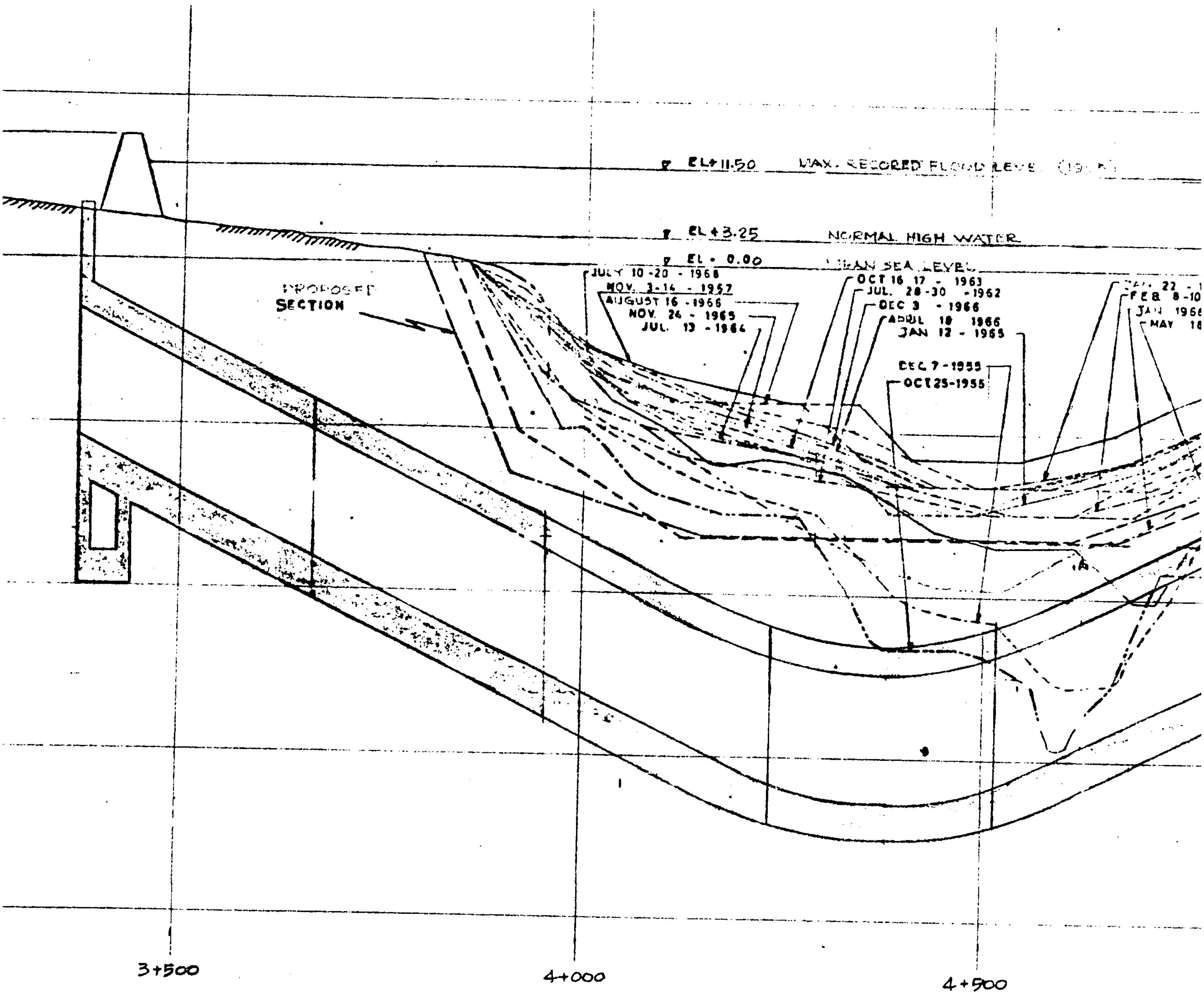
The bottom of the river at the proposed alignment of the tunnel undergoes almost continuous change, as shown in Fig. 2.7, from scour to silt deposits, depending upon the prevailing hydraulic regime.

This phenomenon is influenced not only by variations in the river discharge and velocities - the extremes in scouring naturally occurring during floods generated by cyclones - but also by modifications to the river regime, resulting from the removal or addition of material or structures within the river channel.

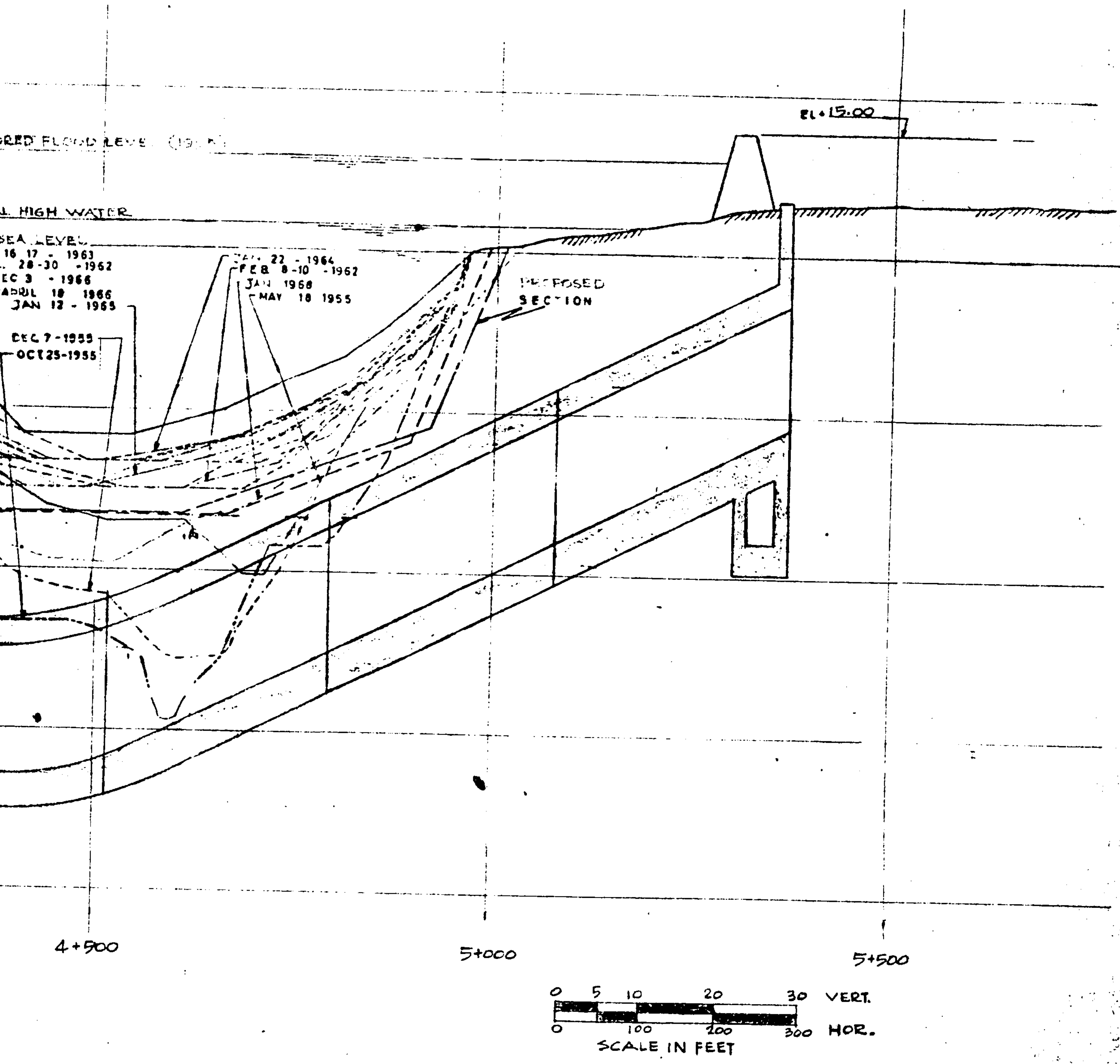
Reliable data concerning the causes and effects of



20
EL+15.00
0
-20
-40
-60
-80
2+500
3+000
3+500



EL+11.50
EL+3.25
NORMAL HIGH WATER
EL - 0.00
PROPOSED
SECTION
JULY 10-20 - 1968
NOV. 3-14 - 1957
AUGUST 16 - 1966
NOV. 26 - 1965
JUL. 13 - 1964
OCT 16 17 - 1963
JUL. 28-30 - 1962
DEC 3 - 1966
APRIL 18 1966
JAN 12 - 1965
DEC 7 - 1955
OCT 25-1955
3+500
4+000
4+500



RIVER BOTTOM PROFILES

Fig. 2.7

past erosion are essential in developing an adequate design for the protection of the tunnel foundation and adjacent river banks. Furthermore, the knowledge of possible siltation rates, based on past records, is of importance for construction planning, as the tunnel elements will be placed in a dredged trench. To this end, additional data is required for the final design.

Scour protection is designed for the flood discharge of 700,000 cfs, corresponding to the 1955 upstream flood level of El. +11.5' and the minimum tidal level of E. -2.8' at the mouth of the river.

2.5.5 Tides and Winds

Tides - The type of tide at Tampico is usually diurnal, although two daily high-waters and low-waters may also occur during certain periods of the year.

The tidal range is relatively low. The maximum range observed for the period 1952-1960 was 5.3'. The characteristic tidal levels for the Tampico Harbour are given in Fig. 2.6.

Winds - For a tunnel project, the direction and velocity of winds blowing on the portals are of particular significance as such winds have to be considered in the design of the tunnel ventilation system.

The available wind data recorded at the Tampico airport during the years 1959-1961, are summarized in Table 2.1 below.

Table 2.1

WIND FREQUENCY
(in percent)

Wind Dir^n	<3 MPH	3-15 MPH	15-30 MPH	30 MPH	Total
S		32.0	7.8	0	39.8
N		11.2	0.2	0	11.4
E		12.8	5.5	0.5	18.8
W		7.7	0.1	0	7.8
	22.2				22.2
					100.0

2.5.6 Salinity Intrusion

Knowledge of salinity intrusion is essential to the planning of placement of the tunnel elements as the salinity affects the density of water and thereby the buoyancy forces on the elements while being lowered into place. It is not so much the overall salinity of the water, but its variation along the depth and in time that is of importance.

In the absence of complete field data on salinity measurements, a preliminary assessment of salinity intrusion was made on the assumption that the estuarine circulation was of stratified type. Existing empirical data and theory pertaining to this type of circulation were applied to the Rio Panuco estuary. The results of this study indicated that the extent of influence of salinity at the tunnel crossing site is dependent upon the river discharge and the tidal level at the river mouth. These influences become negligible for a river discharge of 25,000 cfs when the sea is below the mean low water level.

2.6 River Navigation

Tampico is one of the important deep-water ports of the Republic of Mexico with substantial volumes of inward and outward water-borne traffic by ships of both national and foreign origin. The larger ships are of the following three types as indicated in Table 2.2

Table 2.2

LARGER SHIPS USING TAMPICO HARBOUR

Type	Typical Dimensions		
	Length	Beam	Loaded Draught
Merchant Ships	500'	65'	29.5'
Dry-bulk (ore) Carriers	583'	72'	30.5'
Tankers	640'	82'	30.7'

The first two types usually berth at the wharves located upstream from the proposed tunnel crossing. Tankers normally berth at the wharf which is downstream from the projected crossing although, on rare occasions, they may berth at the other wharves.

Due to serious silting in the lower Rio Panuco, the governing depths available to shipping in the river channel and at the berths are subject to considerable change despite extensive dredging operations carried on almost continuously. limitations to navigation due to depth become greater the farther upstream a ship is required to travel. This is particularly true upstreamfrom the proposed tunnel site.

Available data indicate that the controlling depth of the navigation channel upstream of the crossing site is 27.2', thus restricting the draught of vessels at the tunnel crossing to something less than 26'. This means that incoming vessels must lighten before proceeding to these berths and outgoing vessels must seriously restrict loading.

The water depth over the sand bar at the entrance to the river varies as a result of formation of shallows from shifting sand. The depths have varied between 34' and 26'. A further discussion is presented in Appendix A.

A review of the July 1968 sounding plans prepared by the Marine Works Division, Department of Navy, generally confirms the difficulties experienced at present in maintaining the several reaches of the navigation channel at depths suitable to economic shipping operations.

Another factor affecting the navigation channel is the river currents. Information indicates that currents of 18 knots were reached in some sections of the river in 1955 and a velocity of 7 knots was observed during the rainy season of 1967. These high velocities cause serious scour in the upper section of the harbour, thus changing the depth and width of the shipping channel.

CHAPTER 3

GOVERNING PARAMETERS

The tunnel is to be located in the proximity of the present ferry crossing between Tampico and Mata Redonda, approximately 2.5 miles downstream from the city centre and 5 miles upstream from the mouth of the river. The proposed centreline, as shown in Fig. 1.1, is located parallel to and about 30' upstream from that adopted for the bridge studies carried out by the Mexican Government. The change was made primarily because boreholes obtained later further upstream adjacent to the previous bridge centreline showed more favourable foundation conditions on the Mata Redonda shore for the economical fabrication and launching of the subaqueous tunnel elements.

Although the present traffic across the river, served by ferries and motor launches, is extremely low, the construction of the new highway between Mexico City and Tampico, extended to connect with Tuxpan and Veracruz, together with probable residential, industrial and port developments in the Mata Redonda region, will undoubtedly result in increasing traffic demands. However, since development on the Mata Redonda side at the present is sparse, and since by far the greater part of the traffic carried by the tunnel will be local interurban traffic commuting between Tampico and Mata Redonda, it seems reasonable to assume that the initial demand would be satisfied by a two-lane tunnel. Once good access via the tunnel has been provided, relatively rapid development of the Mata Redonda region can be expected. As most developments require considerable time and assuming that the expected growth can be achieved in a relatively short period of some 10 years, it is more economical, because of the present high opportunity cost of capital, to add a further two-lane tunnel if then necessitated by bigger traffic volumes.

Due to a deficiency of recorded data on the hydraulic characteristics and regime of lower Rio Panuco, it has been necessary to assume a set of design criteria at the tunnel site. As decisions regarding the control of the river have not been reached by the various government authorities at the present time, the lower Rio Panuco is to be deemed to remain in its natural, uncontrolled state. Thus, the values for maximum flood discharge, the conveyance capacity of the river channel, current velocities, and effects of the river regime resulting from the tunnel installation, will be determined for the maximum flood discharge on record of 600,000 cfs which occurred in 1955.

In the absence of a master plan for development of the Port of Tampico, the depth of the navigable channel has been set in consultations between the different governmental authorities concerned at 40', corresponding to the available depth across the ocean bar which limits the draught of navigation in the Port of Tampico to 36'. This restriction will provide a clearance of 4' between the bottom of the ship and the river bed. The 40' depth may be reduced should extensive port development studies so indicate. It seemed reasonable that this depth would be acceptable for hydraulic purposes provided it could be demonstrated that the cross section of the river at the tunnel crossing would have a conveyance capacity at least equal to the maximum design discharge and provided adequate precautions would be taken to ensure protection from scour of the tunnel foundations and the adjoining river banks.

The general arrangement and design of the scour protection must be based on existing records of maximum river flow, taking into account the extensive areas of lagoons, swamps and lowlands which serve as surge reservoirs.

For the design of the roadway, a roadway width of 24' for two lanes, a safety curb width of 2', a vertical clearance of 14'-9", a maximum grade of 5% and a design speed of 45 mph were the minimum requirements stipulated by the Department of Public Works. Further explanations for such specifications will be given in Chapter 4.

CHAPTER 4

TUNNEL GEOMETRICS

General - The selection of the general tunnel alignment, as mentioned previously, is governed primarily by the transprotation or economic interests necessitating the construction of the tunnel and is therefore a function of the purpose which the road is intended to serve. The exact location, however, is controlled by the particular site conditions prevailing in the area.

Normally, the alignment of urban subaqueous vehicular tunnels depends on traffic and town-planning considerations. The necessity for coordinating the location of their entrances with the street network and with the town picture, and thus adjusting the line to town-planning may sometimes result in a tunnel line somewhat inferior to the one chosen in the best interest of the tunnel itself. To accommodate the complex requirements necessary for the overall development of the region, the tunnel line must be selected accordingly.

Tampico Tunnel - Due to the inadequacy of information regarding local town planning, certain freedom will be taken in selecting the tunnel alignment. On the Tampico side, the town is already in existence and the tunnel line will have to blend into the existing street and highway system. On the Mata Redonda side, however, the concerned region is largely undeveloped, and as information on town-planning in this area is unavailable, it must be assumed that future developments will have to emanate from the building of the tunnel structure. Therefore, tunnel alignment will be selected without regard for future town-planning of the south shore, but only in the best interest of the tunnel itself.

The proposed tunnel line has been developed to satify, besides the design discharge capacity, a navigation channel 330' wide by 40' deep with the understanding that the depth may be reduced depending on the outcome of technical and economic studies relating to flood control and port planning, particularly for the upper harbour.

The tunnel alignment has been developed for maximum at-grade roadway connections to the existing street systems in Tampico and Mata Redonda under uncontrolled flood flow conditions. It has been so located that grade separation may be readily installed in the future, when deemed economically feasible.

4.1 Horizontal Alignment

General - The tracing of the route proper should adhere to a straight line if at all possible as it not only shortens the tunnel length and results in better driver visibility, but also simplifies construction procedures and sometimes, allows for a better ventilation system.

Tampico Tunnel - The longitudinal centreline of the tunnel and approaches extends on a straight alignment between Tampico and Mata Redonda, crossing the river at a slightly oblique angle as shown in Fig. 1.1. In Tampico, the centreline runs from a point on the street railway line to the east of the thermoelectric generating station, thence southerly between and approximately parallel to the existing north-south street system. It intersects the northern shore line of the river about 500' downstream from the Tampico ferry terminal, and the southern shore about 330' downstream from the Mata Redonda ferry terminal. In Mata Redonda the centreline continues in a southerly direction passing some 165' westerly from the old Pemex hospital building.

4.2 Vertical Alignment

General - With respect to the longitudinal profile, obviously it is most economical to lay the tunnel as shallow as possible, but with due consideration for future improvements to harbours. It is common practice that a submerged tunnel should have a certain thickness of top backfill, usually rock, as protection from damage in case ships founder on the tunnel. Normally, if such danger does not occur, it would be sufficient to leave the top of the tunnel which is usually protected with a layer of non-structural concrete, approximately 2.5' below, in order to allow latitude for dredges carrying out any eventual deepening of the shipping lane.

The cost per unit length of a submerged tunnel might be higher than a tunnel cast-in-place, provided the difficulties in making a dry site for the latter are not too great. Still more economical than a cast-in-place tunnel is an open ramp construction. For these reasons, it is usually economical to shorten both the submerged portion and the cast-in-place portion of the tunnel as much as possible if soil conditions permit building in situ and if the surrounding topography allows having portals closer to the river banks. At this stage, it must be borne in mind that the shorter the tunnel, the more simpler and more economical the ventilation system.

The typical profile of a prefabricated subaqueous tunnel resembles that of a trough, as the central sag is flanked by inclined sections with a suitable grade governed by traffic, tunnel safety and economic requirements. Normally, grades over 3% have a significant effect on the operating speeds of trucks, although the effect is somewhat less marked with a tunnel than a bridge because the truck can employ the downgrade to acclerate above its normal cruising speed, thus providing some momentum to assist in negotiating the upgrade. Grades as high as 7% will have little effect on North American passenger cars, but trucks will be reduced to a crawl, and depending on the percentage of trucks, separate climbing lanes will generally be provided. In the case of tunnels, a 5% grade is considered the optimum compromise provided that it also allows for suitable entrance and exit locations.

Tampico Tunnel - As indicated previously, any decrease in sag of the tunnel profile will be directly reflected in reduced quantities of both dry and subaqueous excavation and of construction materials. This, in turn, will substantially reduce costs and, importantly, lessen the time required for subaqueous construction operations in the hurricane-free season from November to June.

The profile of the submerged section of the tunnel primarily is governed by either (a) the required depth of the navigation channel serving the upper harbour area, or (b) the cross sectional area of the river channel necessary to accommodate the maximum design discharge.

If the tunnel profile were raised by 6.5', for example, the discharge capacity would be reduced to less than 90% of the peak flood discharge. This decrease in capacity may, of course, be remedied under a system of flood control by diverting a relatively small portion of the peak flood to a retention basin from a point above the tunnel crossing.

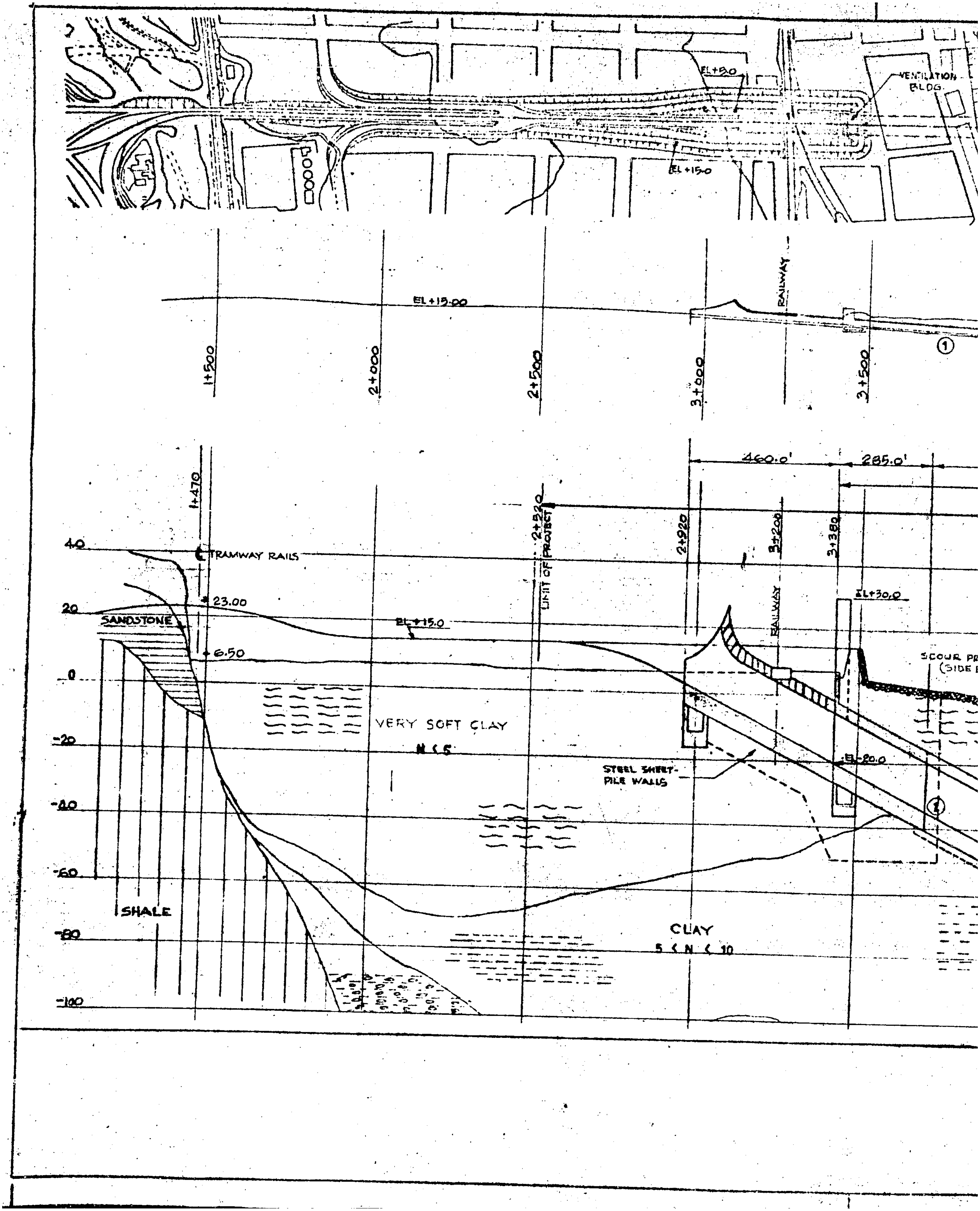
In the subaqueous portion of a tunnel crossing, the soil conditions do not have as great an effect on the profile of an immersed tunnel as they would have on that of a bored tunnel. In the case of the immersed tunnel, as long as it is designed to withstand the differential settlements which will inevitably occur, the profile can be varied at will to satisfy the conditions of navigation and flood discharge.

As the designing of the tunnel elements for differential settlements can easily be achieved, the present profile is chosen primarily for the fulfilment of navigational and hydraulic requirements. To satisfy these demands, the sag of the vertical tunnel alignment corresponds closely to the dip of the river cross section with the bottom of the tunnel at El. -70' at its lowest point. From the central sag curve, straight portions inclined at 5% gradient extend north and south until approximately 400' inland, stopping at the tunnel portal. Beyond the portals, approach ramps continue the 5% grade for some 460' on both sides of the river. This configuration is believed to be the optimum solution in satisfying the stipulated requirements as having the portals farther inland would lenghthen the closed portion of the tunnel, and having them closer shoreward would place them too near the river, obstructing the river flow during flood periods, causing considerable scouring and possibly inundating the tunnel.

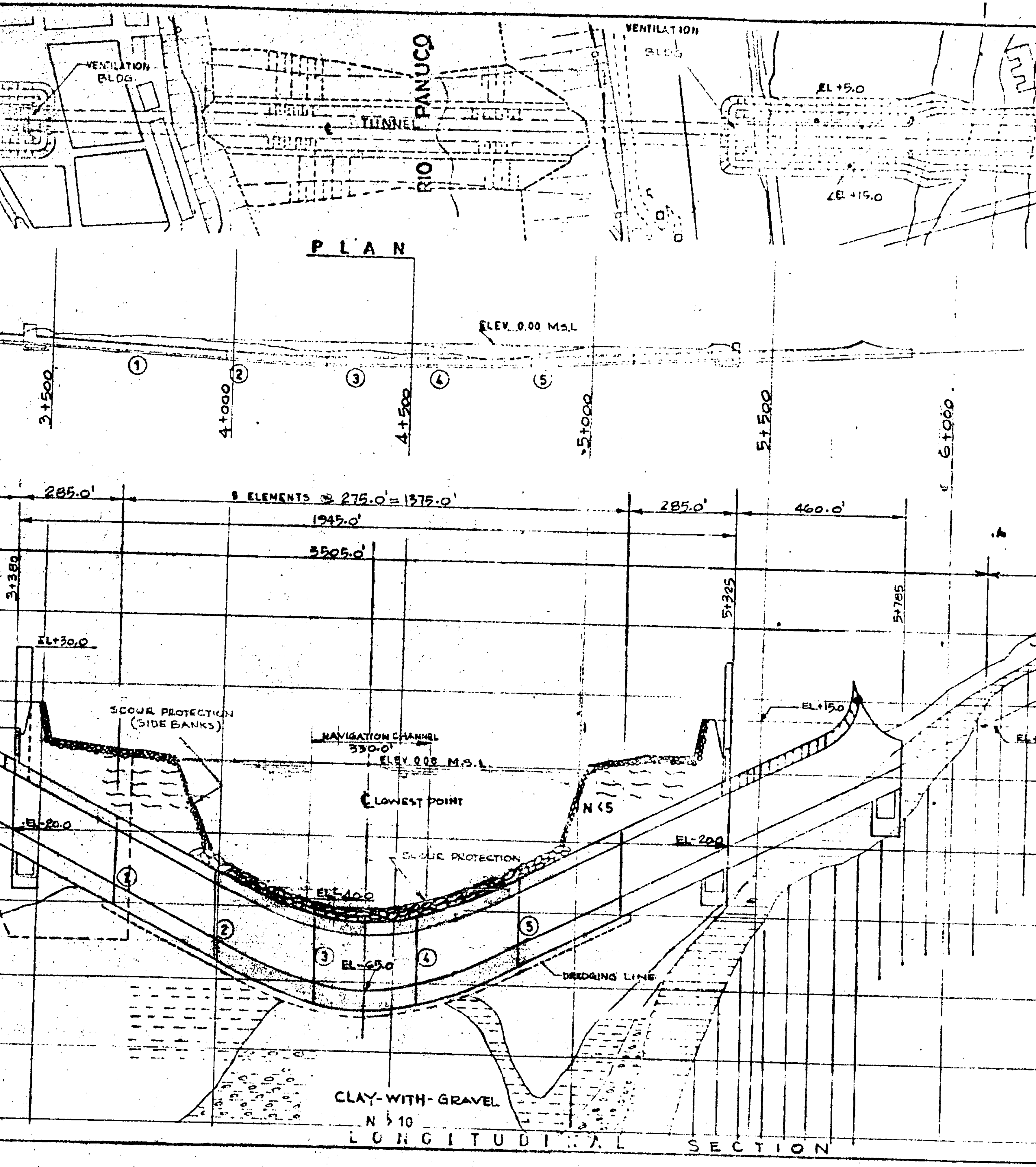
With its present entrance and exit tunnel locations, the tunnel, having the roof of the portals at El. -2.5', would be inundated unless flood protection walls surround the portals. As shown in Fig. 4.1, concrete retaining walls with the top at El. +6.5' enclose the portals to retain the backfill and to serve as an extra safety measure against minor flooding when the water level is below El. +6.5'. The earth dykes with the top at El. +15.0' surrounding the walls will offer primary protection against natural floods including the worst recorded.

4.3 Roadway Configuation

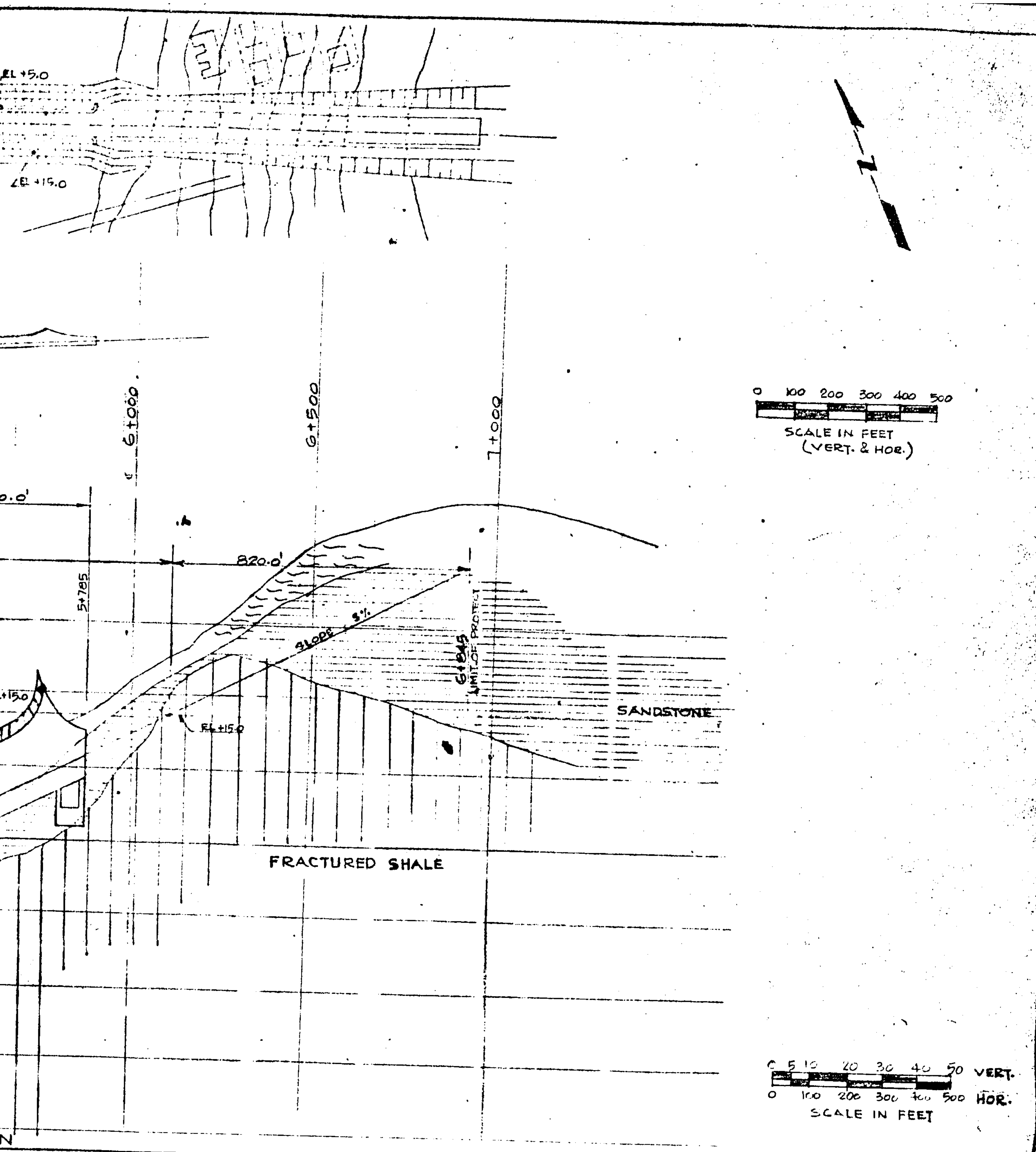
As mentioned in Chapter 3, the minimum roadway requirements were stipulated by the Department of Public Works. The rectangular cross section to be used complies with these requirements with added features for extra safety, comfort and economy.



EL+20.0
VENTILATION BLDG.
EL+15.0
EL+15.00
RAILWAY
1
1+500
2+000
2+500
3+000
3+500
460.0'
285.0'
1+470
2+520
LIMIT OF PROJECT
2+920
3+200
3+380
40
20
0
-20
-40
-60
-80
-100
TRAMWAY RAILS
23.00
6.50
SANDSTONE
EL+15.0
RAILWAY
EL+30.0
SCOUR PR
(SIDE
VERY SOFT CLAY
N < 5
STEEL SHEET-PILE WALLS
EL-20.0
2
SHALE
CLAY
5 < N < 10



VENTILATION BLDG.
VENTILATION BLDG.
RIO PANUCO
TUNNEL
EL +5.0
EL +15.0
PLAN
ELEV. 0.00 M.S.L.
3+500
4+000
4+500
5+000
5+500
6+000
285.0'
5 ELEMENTS @ 275.0' = 1375.0'
285.0'
460.0'
1945.0'
3505.0'
3+380
5+325
5+785
EL+30.0
SCOUR PROTECTION (SIDE BANKS)
NAVIGATION CHANNEL 330.0'
ELEV 0.00 M.S.L.
LOWEST POINT
N < 5
EL+15.0
EL-20.0
EL-20.0
SCOUR PROTECTION
EL-40.0
EL-65.0
DREDGING LINE
CLAY-WITH-GRAVEL
N > 10
LONGITUDINAL SECTION



PLAN AND LONGITUDINAL SECTIONS

Fig. 4.1

Vertical Clearance - The specified vertical clearance of 14'-9" above the crown of the tunnel roadway will accommodate normal commercial traffic since this will constitute a major portion of the traffic using the tunnel facility. The ceiling, however, will be made 10" higher to provide clear space for the installation of lighting fixtures, traffic signals, television cameras, etc.

Roadway Width - For an urban arterial, a roadway width of 24' is sufficient. However, such a cross section would normally have additional parallel traffic lanes, parking lanes, or shoulder. Within the tunnel section, such provision would be costly. In this restricted section, it is more desirable that in case of emergency, the traffic behind be able to manoeuvre past a stopped vehicle. This is feasible with a median strip. Thus including the median strip, a width from curb to curb of 29'-6" is provided. A 2' wide safety curb is also installed to serve as walkways to the exits in time of emergency.

Median Treatment - Some means of inducing the separation of two-way traffic is desirable in order to maintain both safety and reasonable speed. It being impracticable to install a positive median barrier which would preclude an emergency passing manoeuvre, it is proposed to install a 5'-6" wide median strip of "jiggle bars" (relieved concrete ribs) in the roadway surface between the two normal traffic lanes of 12'wide.

The proposed median is of a nature that would discourage driving on it and thus increase the separation of opposing traffic streams, reduce driver tension and provide a degree of latitude for driver error or mechanical malfunction. It would, however, make an emergency passing manoeuvre feasible at a crawl speed. This type of construction also has the advantage of a distinctly contrasting appearance to smooth pavement even when covered with dirt or dust.

As an additional precautionary measure, 1' of the running lanes adjacent to the jiggle bars might be paved with a "rumble strip" texture which emits an unusual sound as a warning when a vehicle is running on it.

Emergency Exits - In the event of fire or other remote emergency, provision is made for the cross section chosen, for leaving the tunnel, on either side of the roadway, by means of the 2' safety curbs to stairways spaced 275' apart and thence by way of the pedestrian tubes in the upper half of the tunnel walls to exits in the ventilation buildings located at the tunnel portals. To minimize the psychological effect resulting from confinement in a narrow passageway, portholes like those on a ship will be installed 20' apart. This treatment will break the monotony of the passageway and the interior of the tunnel. As there are no pedestrian walkways in the traffic tube, these emergency passageways will also serve as access to various locations throughout the length of the tunnel.

Design Speed - Nominal design speed has little influence on roadway geometrics. The horizontal alignment is straight. The vertical alignment has a sag curve under the river. Although, for the purpose of illustration, the sag curve has been developed in accordance with AASHO recommendations, in this case, the 'headlight distance' criteria do not apply because the tunnel is lit.

Crest vertical curves on the approaches are shown to conform with minimum stopping sight distances, but these could easily be lengthened. Thus, although the nominal design speed is 45 mph, neither the horizontal nor vertical alignment in the tunnel is restrictive. The sensation of driving in a two-lane tunnel, fairly close to opposing traffic and quite close to the tunnel wall will tend to reduce speeds. The fact that the tunnel is a part of an urban street network with drivers attuned to driving at grade-street speeds will also tend to encourage the same level of speed in the tunnel. For an urban grade-street system, 45 mph is a reasonable and appropriate design speed.

CHAPTER 5

TUNNEL PROPER

General - Up to the time of the building of the submerged tube vehicular tunnel under the River Maas in Rotterdam between 1938 and 1941, all tunnels of this type, except one, were designed with a circular or octagonal cross section. One of the reasons for the choice of these circular sections was that they were well suited to the placing of granular fill under the tunnel. The notable exception to circular or octagonal immersed tunnels is the Harlem Tunnel in New York which is rectangular and is supported by tremie concrete.

Where steel enjoys a competitve position, as it does in the United States, trench type tunnels have usually been constructed as concrete-lined steel shell sections. A list of major trench type highway tunnels is presented in Table 5.1. It can be seen that rectangular tunnels have found wide acceptance in Europe while circular steel shell tunnels are popular in the U.S.A. and U.K. where steel is one of the more economical construction materials. It is interesting to note that both of Canada's immersed tunnels are of rectangular cross section, taking after the Maas Tunnel in Rotterdam.

The steel tunnel section lends itself very well to construction on slipways. It also has the advantages of possibly better waterproofing, of continuous foundation support given by the backfill, thus, less sensitivity to differential settle-

Table 5.1

LIST OF TRENCH TYPE HIGHWAY TUNNELS

Country	Year Opened	Name & Location	Closed Length	Number of Lanes
		RECTANGULAR TUNNELS		
Germany	under constr.	Elbe, Hamburg	3,450'±	6
Holland	1969	Heinenoord, Rotterdam	2,014'	6
Denmark	1969	Limfjord, Aalborg	1,670'	4
Belgium	1969	Schelde, Antwerp	2,250'	6 & 2 rlwy. tracks
Sweden	1968	Tingstad, Gothenburg	2,085'	6
Holland	1968	IJ, Amsterdam	3,410'	4
Holland	1967	Benelux, Rotterdam	2,610'	4
Canada	1967	Lafontaine, Montreal	4,560'	6
Holland	1966	Coen, Amsterdam	1,927'	4
France	1964	du Vieux-Port, Marseille	1,957' 1,914'	2 2
Germany	1961	Kiel Canal, Rendsburg	1,968'	4
Canada	1959	Deas Island, Vancouver	2,160'	4
Cuba	1958	Bay of Havana	2,624'	4
Holland	1941	Maas, Rotterdam	3,528'	4

Table 5.1 - cont'd

Country	Year Opened	Name & Location	Closed Length	Number of Lanes
		CIRCULAR OF ORTHOGONAL TUNNELS		
U.S.A.	under constr.	Mobile River, Alabama	4,000'±	4
Argentina	under constr.	Parana-Sta. Fe, Parana	8,690'	2
Hong Kong	under constr.	Cross Harbour, Hong Kong	5,300'±	2
U.S.A.	1963	Chesapeake Bay, Va.	5,200'	2 tubes of 2 each
U.S.A.	1962	Oakland-Alameda Calif. (Webster St.)	3,340'	2
U.S.A.	1928	Oakland-Alameda Calif. (Posy)	3,510'	2
U.S.A.	1957	Baltimore Harbour, Md.	7,660'	2 tubes of 2 each
U.S.A.	1957	Hampton Roads, Va.	6,863'	2
U.S.A.	1953	Baytown, Texas	3,350'	2
U.S.A.	1952	Elizabeth River Va.	3,350'	2
U.S.A.	1950	Passedena, Texas	2,940'	2
U.S.A.	1941	Bankhead, Alabama	3,109'	2
USA-Canada	1930	Detroit-Windsor Detroit River	5,140'	2

ment, and of better structural qualities. However, the diameter of a circular cross section depends on the width of the roadway. Consequently, the height of the cross section will usually be more than required for the vertical clearance of the roadway. For a given depth of water and thickness of the rock protection on top of the tunnel, this necessitates a lower roadway elevation than that required for a rectangular cross section and results in longer and more expensive approaches. In the case of more than two lanes, these drawbacks are more pronounced. Whether the lanes are contained in one tube or divided into two separate tubes, it is obvious that the depth and width of the trench and the approaches will be considerably in excess of that required for one rectangular cross section.

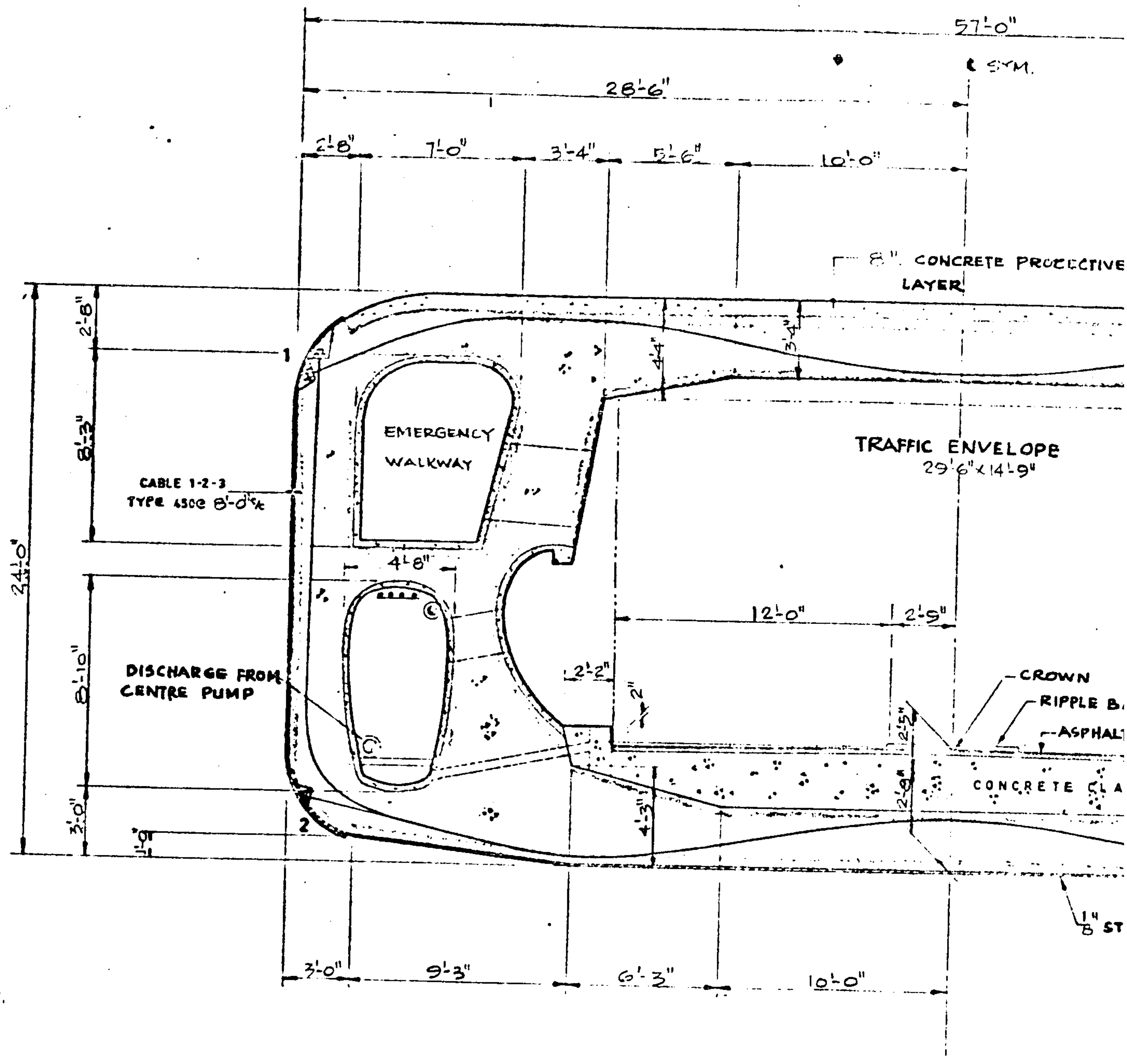
Tampico Tunnel - Because of its functional aspects, coupled with the fact that steel would have to be imported if a steel shell cross section were to be used, a rectangular concrete cross section has been adopted for the tunnel at Tampico. Concrete is manufactured locally and is readily available. For a steel shell section, a large quantity of steel is required for a project of this size. Highly qualified welding personnel would be necessary to ensure watertightness at the joints as leakage is extremely difficult to locate in a steel shell tunnel. This difficulty arises when water, seeping through the defective welds slowly leaks into the tunnel interior via the possible hairline shrinkage or tensile cracks of the concrete tunnel walls. From observation within the tunnel, there is no sure way of tracing the leakage back to the faulty weld since the seepage could have come through any of the welded joints. For these reasons, steel shell sections necessitate the use of automatic welding processes as much as possible and the testing of all seams, a procedure which could be unduly expensive if faulty welding persists. However, once the watertightness of the seams is ensured, the steel shell would offer the most absolute waterproofing system.

Unless financing or other special considerations favour the use of a steel shell tunnel, a rectangular concrete tunnel is preferable. The structural design of tunnels differs from that of ordinary structures in that tunnels must have enough mass to prevent uplift by buoyant forces once in place and yet be light enough to float while being towed to the site should such a sinking procedure be adopted. Almost as much concrete is needed in both concrete and steel shell sections in order to meet the stability requirements. The weight and cost of materials are in direct proportion to the free tube cross section. Therefore, it would seem that concrete sections are more economical than steel sections.

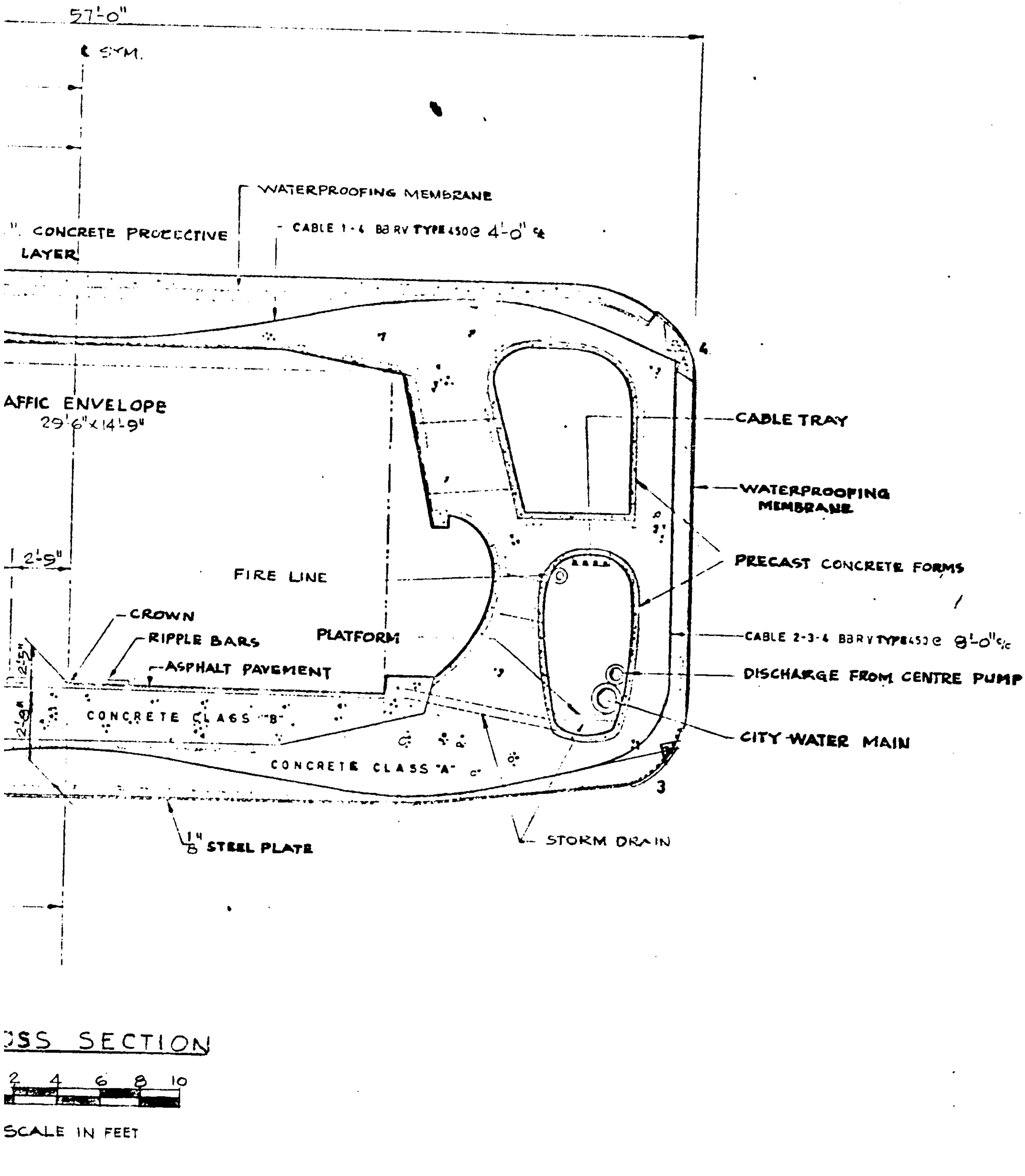
Depending on the water depth, ground permeability and width of tunnel, it is normally more economical to build as much of the tunnel structure as possible in the dry behind a cofferdam if the site conditions permit. Such consideration is especially important for the Tampico Tunnel because the hurricane-free time available for subaqueous construction is relatively short. On the Mata Redonda side, the temporary cofferdam is built as close to the river bank as practicable to allow a maximum of cast-in-place construction thus shortening the length of base slab required for the launchway by temporarily using the bottom slab of the in-situ structures as part of the launching pad. On the Tampico side, however, it is preferable to bring in the submerged sections as far inland as practicable because the poor soil conditions render cast-in-place construction expensive. The maximum distance inland which can be tolerated is governed by the depth of water available since the elements have to be brought in submerged by the portal cranes. The permanent protective dykes on both sides have been moved towards the river banks as much as possible without obstructing the river flow during flood periods. The arrangement of tunnel elements shown in Fig. 4.1 considers all these aspects.

Bearing the above discussion in mind, the Tampico Tunnel proper, i.e., the closed portion of the tunnel, has been conceived to have concrete units of rectangular cross section, 24' high by 57' wide, as illustrated in Fig. 5.1. Of these, 5 elements, each 275' long, will be precast and post-tensioned on a launchway on the Mata Redonda shore and moved to positions as indicated in Fig. 4.1 all in accordance with the construction procedure described in Chapter 11. The other two units, each 285' long and positioned to flank the 5 elements, will be constructed in place within dewatered cofferdams.

The tunnel portals will be located at the junctions between the outer ends of the 285' units and the adjoining approach structures.



CROSS SECTION



CROSS SECTION OF SUBMERGED ELEMENTS

Fig. 5.1

5.1 Cross Section

Most tunnel cross sections evolve from two basic forms; namely, the circular and the rectangular. The former is favourable to stress considerations, the latter, economical in dredging and backfilling when the tunnel contains many traffic lanes.

The major advantage of the rectangular cross section over the circular cross section is the fact that shallower tunnels of equal horizontal capacity can be constructed. This reduces the amount of dredging necessary and permits more gentle gradients in the tunnel profile. If a suction dredge were used, it would normally be less expensive to dredge a wider but shallower trench than a narrower and deeper trench because the efficiency of the suction dredge is greatly reduced by depth, hence increase in the unit cost. If the depth were too great, the dredge may have to be modified or dredging may have to resort to a clam shell dredger, which becomes costly to operate.

For a stipulated tunnel requirement for internal dimensional clearances, the inner shape of the cross section is practically defined and the freedom to choose a section to satisfy these requirements is actually limited. Several cross sections satisfying the minimum requirements were studied for the present tunnel to enable relative cost comparisons to be made.

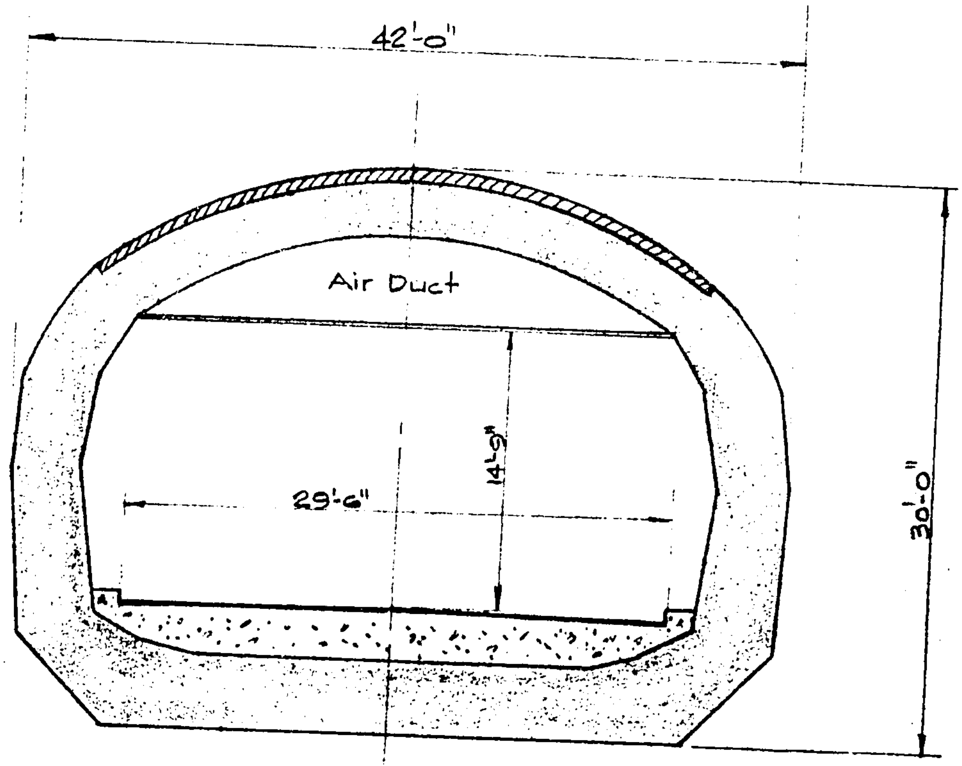
The savings in cost incurred by obtaining the cross section with least materials was found to be of small magnitude - around 5% over a random cross section designed to meet the specified internal dimensional requirements. This percentage represents 2-4% of the total cost of the underwater crossing. Thus the true area of economization lies not in the meticulous choice of the cross section, but in the regions of construction methods pertaining to element fabrication, foundations and placing operations.

Several cross sections providing more features than the minimum required were also studied. For the purpose of comparison, the best solutions from these two categories of cross sections are presented below in detail in order to best select the more suitable cross section for the present tunnel. These two types of cross sections are shown in Fig. 5.2.

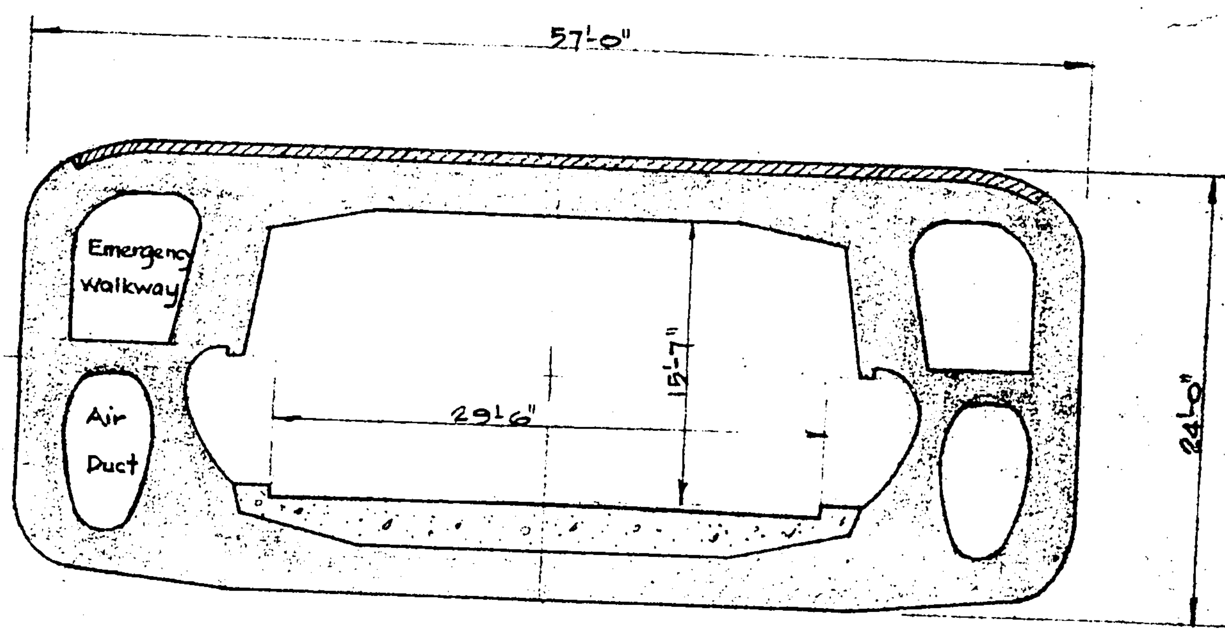
5.1.1 Type I Cross Section

The cross section marked "Type I" provides the specified space for traffic, utilities and ventilation with the optimum use of construction materials. The overall dimensions of this section are 30.0' high by 42.0' wide. The stress distibution throughout the cross section is favourable as tensile stresses could largely be eliminated by providing bottom prestressed tie rods such that the resultant compression line would remain within the concrete. Top tie rods are not provided lest they may be damaged by fire as they would be exposed. Due to the compression, the section would tend to be watertight of its own accord. A watertight membrane on the outside of the tunnel provides extra safety against leakage.

A suspended ceiling under the vaulted roof would serve



TYPE I
(a)



TYPE II
(b)

TUNNEL CROSS SECTIONS

Fig. 5.2

very useful purposes as lighting fixtures and traffic lights could be recessed into this false ceiling. For acoustical reasons, the side walls are slightly out of plumb. This angle of slope avoids standing sound waves, which inevitably occur in tunnels with vertical side walls, by reflecting them towards the ceiling. Treating the ceiling with sound absorbing, acoustical plaster would reduce the noise level substantially.

As the only space available for ventilation is between the concrete roof and suspended ceiling, as shown in Fig. 5.2a, a longitudinal ventilation system as will be explained in Chapter 10, will have to be contemplated because the space is not large enough for transverse systems in which the duct must be divided by a partition wall. In the longitudinal system, the air is blown along the duct, from the intakes to the exhausts. However, since two-way traffic exists in the same traffic tube, this ventilation system would not be as effective as it should be. The larger air duct available results in moderate air velocity, therefore, all the fans can be installed in one ventilation building at one end of the tunnel. Had the air duct area been small, the velocity would have become excessively high, and to keep the air speed to a reasonable limit, the fans would have to be installed in two ventilation buildings in order to provide the same capacity.

This cross section meets only the bare minimum in order to conserve space and material. For this reason, there are no emergency exits in the tunnel. The tunnel users would not be able to fully escape the effects of fires and other emergencies.

Another disadvantage of this section is that the city water main, telephone and power cables and various drainage basins would have to be buried under the roadway ballast due to the lack of space. To add extra lines in the future would involve the breaking up of the pavement, thus additional costs. Installing the service lines above the false ceiling would reduce the area of the ventilation duct and thus increase the air velocity and hence requires larger fan capacity.

5.1.2 Type II Cross Section

The cross section marked "Type II" provides, in addition to the minimum space requirements, four separate ducts. The two upper ones serve as emergency escape passages or pedestrian walkways if deemed desirable, and the two lower ones as tubes for ventilation, drainage and service conduits. The introduction of a rectangular cross section with a flat top and rigid walls results in reducing the effective overall height of the structure and its protective covering to 24'. This reduced height of 6' will result in dredging economies. The overall width of this section is 57'. This increased width is not only essential for the provision of near fixed-end supports for the top and bottom slabs, but also necessary for the accommodation of the four separate longitudinal tubes.

By virtue of its shape, this section is subjected mainly to bending. If regular reinforced concrete were chosen as the construction material, hairline cracks would develop due to the large stresses inherent in a section in bending. To achieve additional watertightness and to avoid excessive regular reinforcing steel, this section would be constructed of prestressed concrete. A watertight membrane on the outside of the tunnel would then provide extra safety against leakage.

The concave lower half of the inner tunnel walls in the form of an ear as shown in Fig. 5.2b is intended to be continuously illuminated by providing a row of fluorescent lamps at the top of the ear. The architectural treatment would enhance not only the interior appearance of the tunnel, but being highlighted, also benefit driver comfort by producing a feeling of spaciousness. Such a feeling would induce driving closer to the safety curb thus functioning as further means of separating opposing traffic to maintain optimum safety and traffic speed. Because of the shape of the walls at their lower portions, sound waves emanating from the vehicle level would tend to be partly confined by the concavity of the walls with the remainder being reflected to the ceiling. These portions of the walls must be lined with sound absorbent material to reduce the noise level which is concentrated at about the vehicle level. Further lining of the ceiling would greatly improve the acoustics in the tunnel.

As carbon monoxide and other contaminants originate from the lower level, ventilation ducts located at this level would permit a more flexible and effective ventilating system. A large portion of these harmful gases will be discharged by suction to the exhaust air tube before they permeate the entire tunnel. Noting that the ventilation tube area, instead of having one large duct as in the Type I cross section, is divided into two ducts, air velocity would be high. To maintain it to an acceptable limit, the required fans will have to be installed in two ventilation shafts, one at each portal of the tunnel. The Type I cross section requires only one shaft.

5.1.3 Summary of Comparisons Between Cross Section

Table 5.2 below generalizes the above discussions on the two types of tunnel cross sections presented. As can be seen from the Table, the cost of the Type I cross section, which is the most economical section satisfying the minimum requirements, is 10% lower than the cost of the Type II cross section, which offers more in terms of functional and comfort aspects. From consideration of cost-benefit relationships, it was deemed worthwhile to select the Type II section with better architectural feature, mechanical layout, public safety and structural engineering.

Inspection of the Table indicates that the Type II cross section is superior than the Type I cross section in all respects except cost. However, as many features are added for such a relatively small cost (approximately 3% of the total cost of the tunnel project), the Type II cross section has been adopted for the Tampico Tunnel.

5.2 Structural Design

In many immersed tunnels, the structural design of the

Table 5.2

SUMMARY OF COMPARISONS

Items	Type I	Type II	Remarks
Road clearances	29'-6" x 14'-9"	29'-6" x 14'-9"	Equal
Long. roadway alignment	Deeper descent	Shallower descent	II superior
Facilities for secondary traffic	None	Limited facilities for ped. & emergencies	II superior
Provisions for utilities	Adequate for foreseeable demand	Space available for unforeseen future demand	II superior
Ventilation system	Long. system in tube with opposing traffic	Transverse sys. not dependent on direction of traffic	II superior
Structural considerations	Requires post-tensioning	Requires post-tensioning	Equal
Watertightness	Concrete in compression	Concrete in compression	Equal
Aesthetics	All fixtures recessed in suspended ceiling.	All fixtures mounted on ceiling, but lower walls concave & port holes on upper walls	II superior
Estimated Costs:			
Structural fabrication	$1,950,000	$2,455,000	
Dredging trench	645,000	470,000	
Backfill & scour protection	546,000	546,000	
Total:	$3,141,000	$3,471,000	I superior

Difference = 10% based on the immersed portion (1,380') of the tunnel.

various critical cross sections along the tunnel alignment does not present major difficulties. Although the tunnel is subjected to a complexity of loading conditions, it can be analysed as a normal rigid frame using the usual method of moment distribution, column analogy, finite difference, finite element, or any applicable structural analysis. Existing electronic computer programmes would be very useful, especially when a multitude of loading conditions has to be considered.

From the moment of launching to the time they rest in their final position and are covered by backfill, the tunnel elements go through a number of stages of different loading and stress conditions. For the final design, all these loading conditions, including the case of a sunk ship on top of the tunnel must be checked in detail.

From experience with other tunnels, the major problems to be overcome in a relatively narrow tunnel are the floatation and stability requirements. With these satisfied, the cross section proportioned will, with slight modifications, ordinarily meet the demand for structural safety. A choice of either reinforced concrete or prestressed concrete may have to be made in order to bring the stresses within the allowable limits. In wider multi-lane tunnels, however, the flooded condition could governed the design of the cross section of the tunnel as in the case of the Deas Island and Lafontaine Tunnels in Canada. Two possible modes of tunnel flooding are conceivable. One is flooding through communication with the outside water, thus reducing the volume for buoyancy to that occupied by the solid concrete; the other is flooding through inundation of the tunnel interior by water which is not under the same head as the outside surrounding water. The latter could become critical because of the longer spans for the tunnel slabs and the reduced net hydrostatic pressure acting on the walls causing less "opposing" moments and thrust forces in the slabs.

No attempt has been made to analyse in profound detail the effects on the structure brought about by loadings incurred during the various construction stages. It suffices to analyse the structure subjected to the severest loading conditions which could occur in order to obtain reasonable quantities of construction materials such that a sensible cost estimate may be made. Appendix B displays an engineer's approach to the preliminary design of the tunnel, bearing in mind that the sole purpose is to achieve a cost estimate, after ascertaining that the tunnel structure would be stable against uplift caused by hydrostatic pressure and that the stresses would not exceed the maximum allowable.

The major component of a tunnel structure is the portion between the portals. As this closed portion has to resist the multitude of loading conditions, sufficient effort has been made to analyse the cross section to ensure satisfactory performance.

As may be seen from Fig. 5.1, the typical cross section of the tunnel structure adopted is what may be called a flat rectangular Vierendeel frame, with braced sidewalls, supported on an elastic foundation. The sidewalls provide fairly rigid supports for the top and bottom slabs of the tunnel. The support and span bending moments in the slabs of this cross section are fairly equal in magnitude; for example, due to uniform loading, both moments are approximately $wl^2/16$.

The design of the typical cross section has been based on the most unfavourable combination of the following loading systems:

- Dead load (element completed).
- Hydrostatic pressure at normal high water, El. +3.25'
- Hydrostatic pressure at max. flood level, El. +11.50'
- Soil pressure from granular backfill on tunnel sides
- Earthquake loadings
- Flooded-tunnel condition

with a factor of safety against uplift of 1.05 when ballasted and 1.20 after the installation of the rock protective layer.

To conform with the method of construction, handling, transporting and placing of elements, the cross section has also been checked and found to be satisfactory for the following conditions:

- Element ready for transport-- dead load of structure, end bulkheads, access shafts, and part of roadway ballast sufficient to overcome buoyancy and provide a net submerged weight on the supporting portal cranes of 75 tons in salt water and 350 tons in fresh water.
- Transport to the site-- the submerged element to be transpoted by 2 portal cranes moving along runways con-

toured to the foundation profile; element to be suspended at 4 points approximately 56' from the ends.

The prestressing force is provided by cables of high-tensile strength steel, with a maximum prestressing force of 560 tons. The cables are spaced at 4'-0" apart in the slabs and 8'-0" apart in the sidewalls.

The 285' long cast-in-place tunnel section at the Mata Redonda side will be prestressed in situ. The section at the Tampico shore will have conventional reinforcement, with concrete thicknesses increased approximately 20-25% as it is impracticable to prestress the sections due to the steel pile walls which act as outer forms for the section.

CHAPTER 6

TUNNEL APPROACH STRUCTURES

<u>General</u> - The prime functions of the approaches to a tunnel are to provide fast and safe transition from the normal highway level to the level in the closed portion of the tunnel, to offer protection against inundation of the tunnel by ground water or general flooding of the surrounding terrain and to permit the proper transition by means of sun screens, from the bright sunlight of the open road to the lower intensity of lighting in the interior of the tunnel.

At both ends of any submerged tunnel, buildings are usually erected either precast or in-situ. These may be the portal buildings, which normally house the ventilation plant, if any, the substation for power supply, control room, etc. Where the structure is not the portal to the tunnel, a section of cast-in-place tunnel is usually connected to the open approach section. The length of in-situ tunnel, if any, is of course, governed by a comparison between the cost of the submerged tunnel and that of the tunnel in-situ.

The most economical form of ramp can be constructed as an open earth ramp in soil where the water penetration is small, such as clay, unfissured shale and rock. Slight seepage, in such cases, can be dealt with by ordinary drainage, the water being collected in a basin under the portal building and pumped away. Such basins are required, in any case, to collect storm water from the ramps. In many cases, however, it is not possible to build such an open drained ramp due to poor physical site conditions and this results in a much more costly approach structure. If a concrete trough had to be adopted for the ramp, a considerable weight of concrete or other ballast would be necessary to prevent the ramp from floating. However, in many cases, it has been possible to use soil as the necessary ballast to stabilize the concrete structures; for example, by extending the base slab beyond the sidewalls so that more soil will rest on the structure.

Tampico Tunnel - The present tunnel has two concrete approach structures each 460' long, with open grilled tops, abutting each tunnel portal. Each ramp is to be constructed in place in the form of a U-shaped trough of decreasing depth to suit the rising 5% roadway grade, roofed over with concrete louvres and with special architectural contours at the open ends to emphasize aesthetic qualities. These features can be seen in Fig. 6.1 and Fig. 6.2. Open drained ramps constructed in soil as approaches are not practicable due to the poor soil on the Tampico shore and the artesian flow on the Mata Redonda shore which cause considerable water penetration.

The inside opening of the approach structure, for architectural purposes, has been made 40' wide - a few feet wider than that of the tunnel tube. The tops of the walls are to be at El. +6.5', the approximate level of natural ground. The natural ground or the backfill in the vicinity of the approaches is to be graded to 1.5' below the top of the walls as a safeguard water from the ramps. In many cases, however, it is not possible to build such an open drained ramp due to poor physical site conditions and this results in a much more costly approach structure. If a concrete trough had to be adopted for the ramp, a considerable weight of concrete or other ballast would be necessary to prevent the ramp from floating. However, in many cases, it has been possible to use soil as the necessary ballast to stabilize the concrete structures; for example, by extending the base slab beyond the sidewalls so that more soil will rest on the structure.

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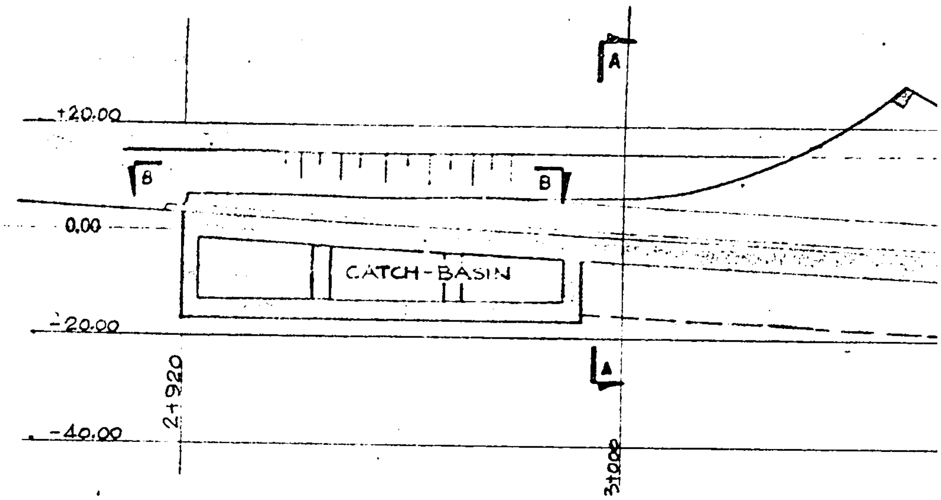
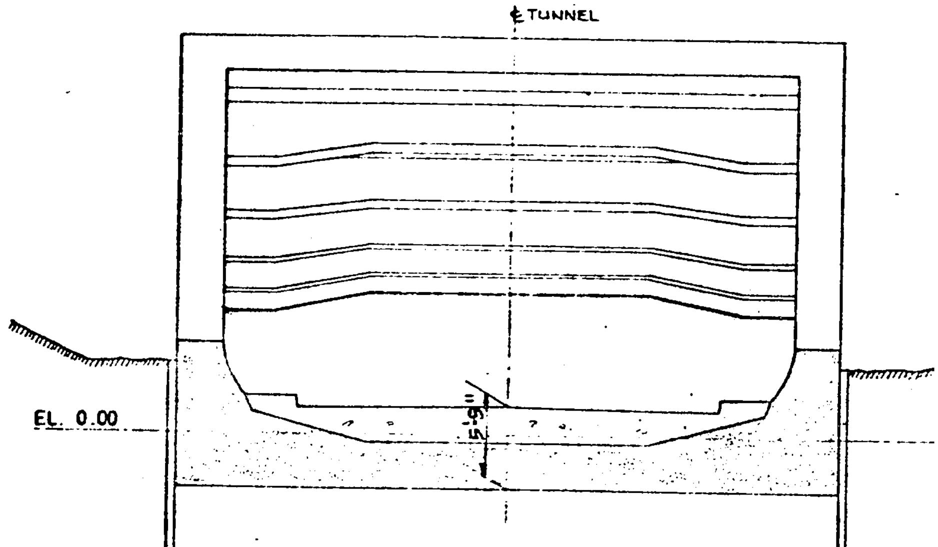
6.1 Tampico Approach

On the left bank of the river, the crown of the roadway slopes upwards at 5% grade from E. -20.0' at the junction with the tunnel section to El. +3.25' at the upper end of the approach structure. It continues on fill for about 500' partly at 5% grade then flattens off to El. +15.0'.

From a review of the soils profile, it can be seen that the quality of soil is extremely poor on this side of the river with $N < 5$. To perform the necessary excavation and backfill in order to build the approach structure would be costly as the excavation slope must be very gentle to achieve stability. The dewatering system which must be installed in order to build the structure would be extremely expensive, if not impossible. To avoid such difficulties, the entire length of the approach structure will be constructed between two steel sheet-pile walls which serve as a retaining structure during the excavation work between these walls; as an impedence to water seepage thus assisting the dewatering task; and as outer formwork for the walls of the approach structures.

As the approach structure will be entirely cast-in-place, it is reasonable to assume that the cross section of the approach structure will be monolithic and could be designed as a frame with rigid joints.





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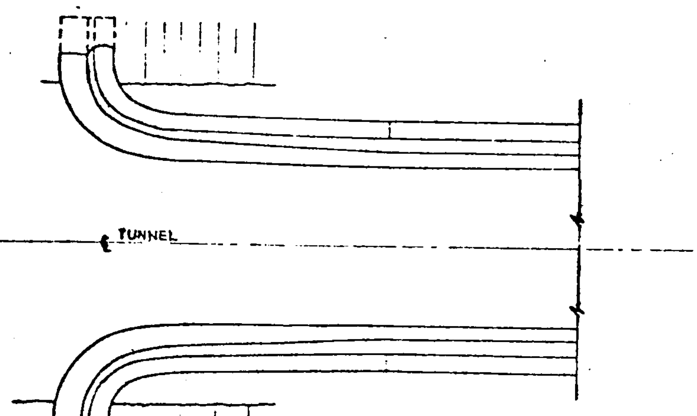
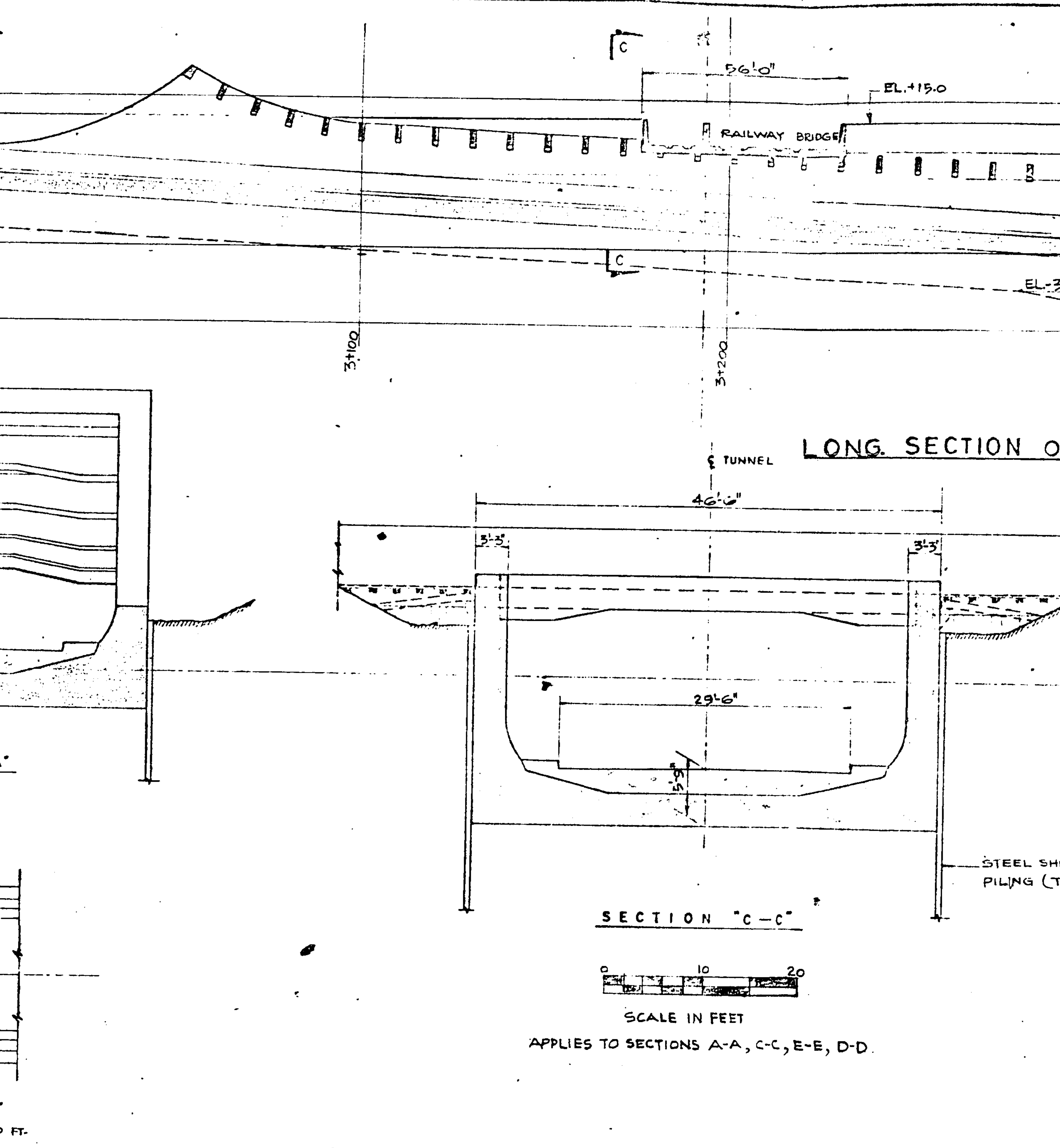
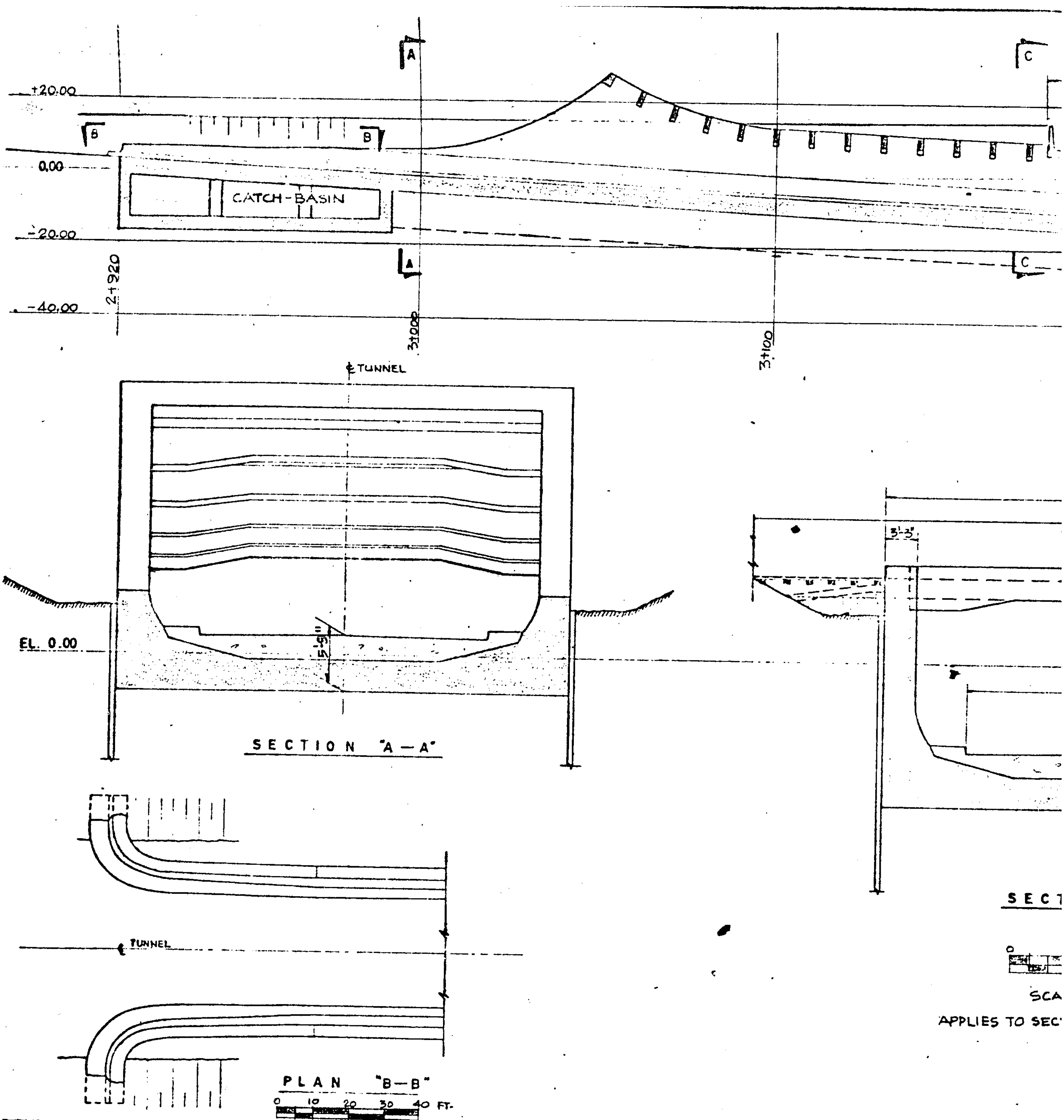


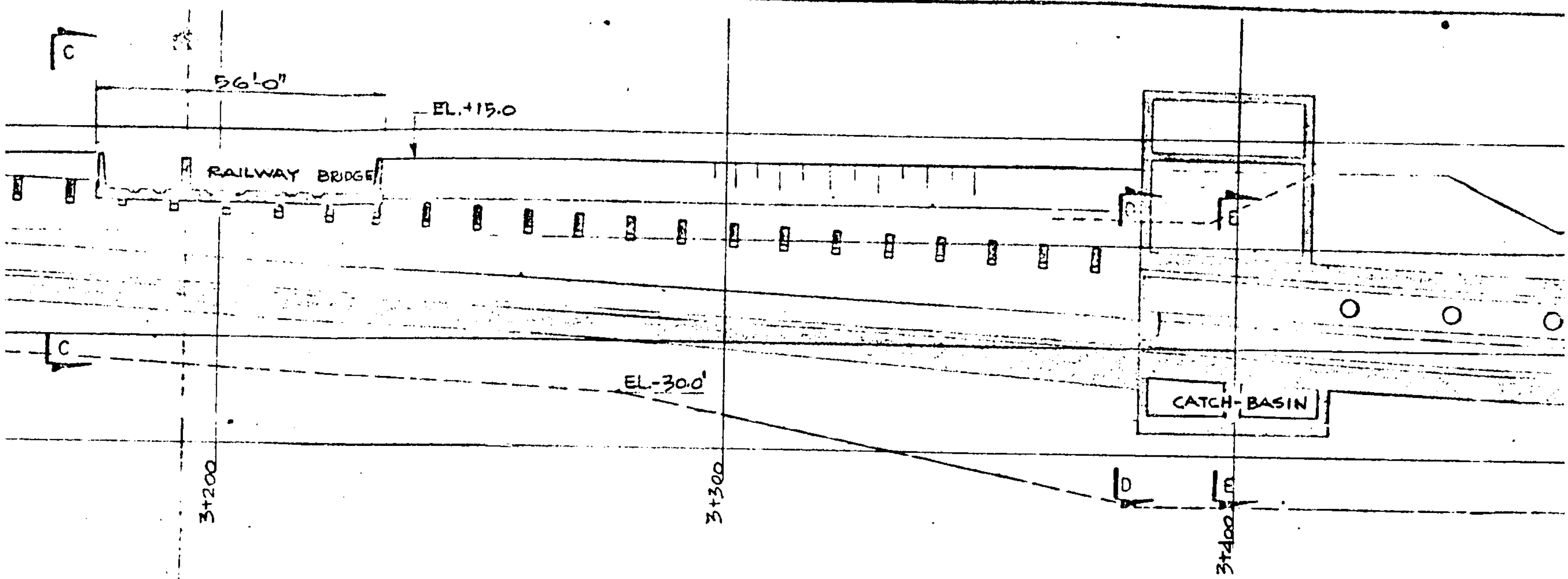
PLAN "B — B"

0 10 20 30 40 FT.

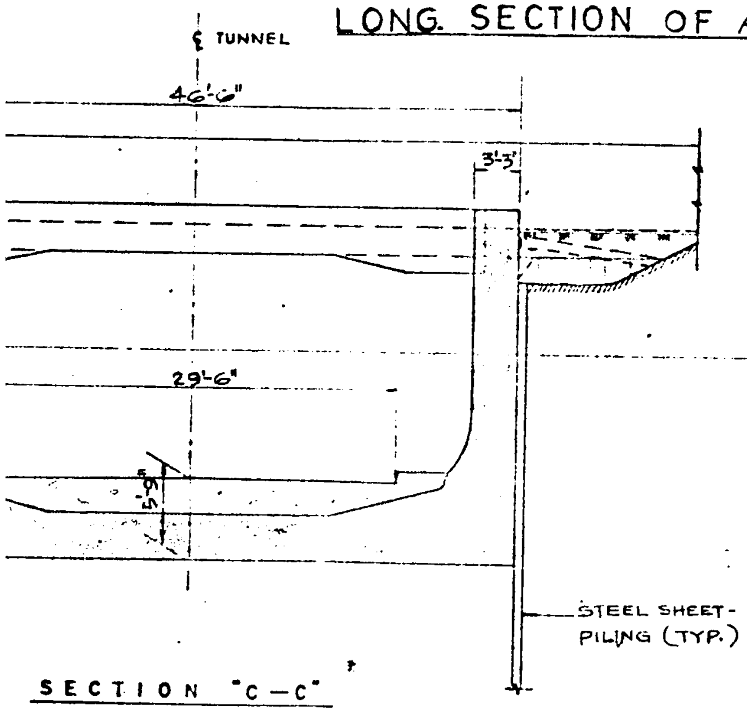


C
56'-0"
EL.+15.0
RAILWAY BRIDGE
C
EL.-3
3+100
3+200
℄ TUNNEL
LONG. SECTION O
46'-6"
3'-3"
3'-3"
29'-6"
5'-9"
STEEL SH
PILING (T
SECTION "C-C"
0
10
20
SCALE IN FEET
APPLIES TO SECTIONS A-A, C-C, E-E, D-D.
FT.

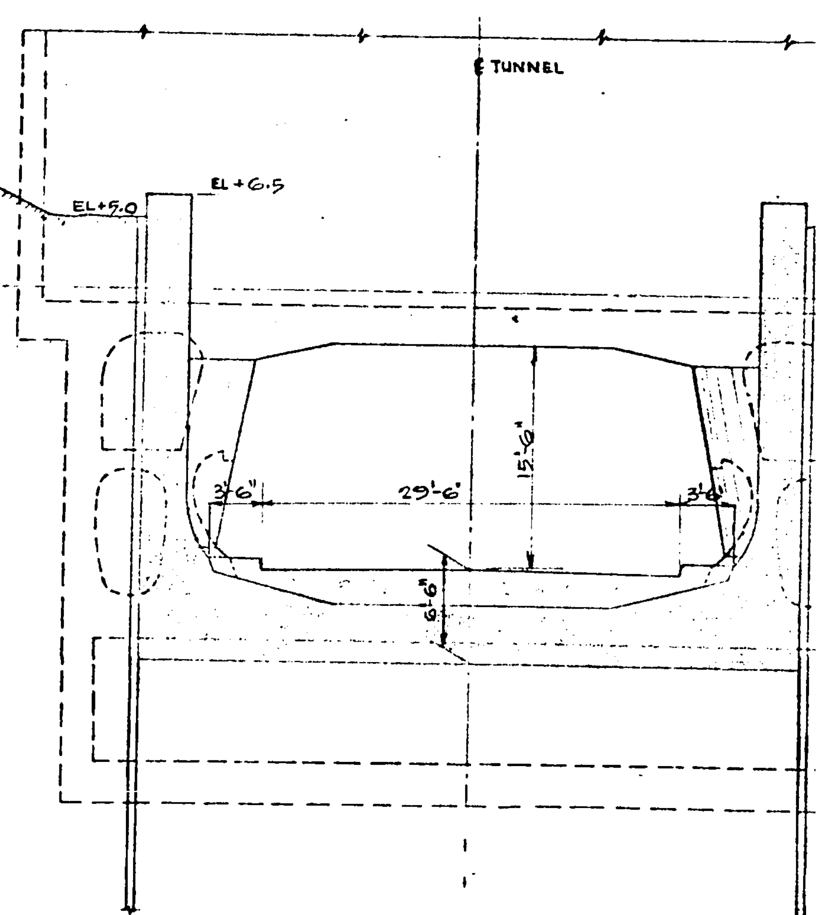

A
C
+20.00
B
B
0.00
CATCH-BASIN
-20.00
A
C
2+920
-40.00
3+000
3+100
℄ TUNNEL
3'-3"
EL. 0.00
5'-5"
SECTION "A—A"
SECT
SCA
APPLIES TO SEC
℄ TUNNEL
PLAN "B—B"
0 10 20 30 40 FT.



LONG SECTION OF APPROACH



SECTION "C-C"

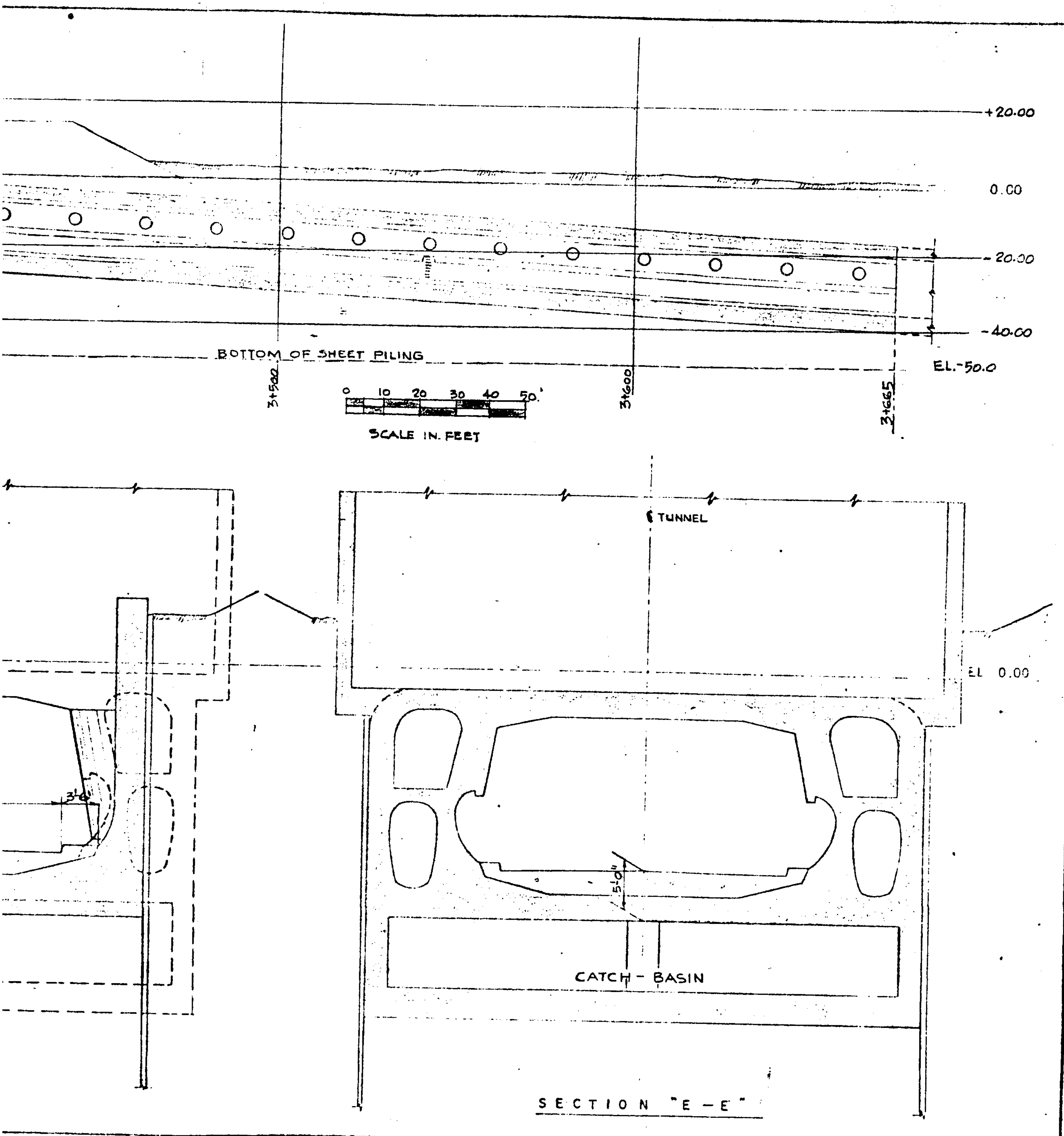


SECTION "D-D"

0 10 20

SCALE IN FEET

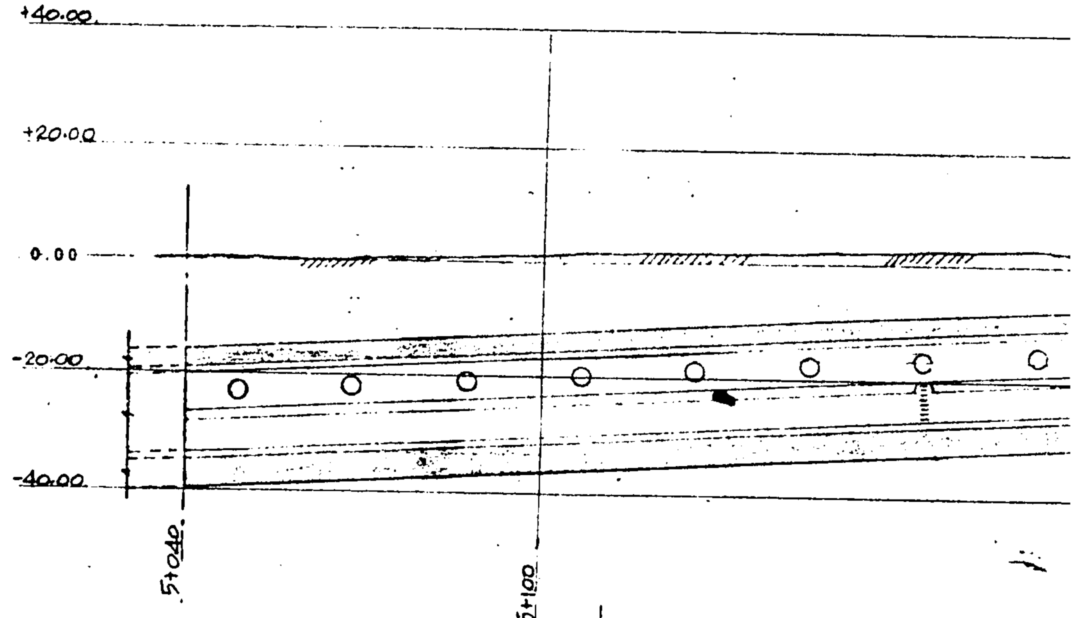
ES TO SECTIONS A-A, C-C, E-E, D-D.

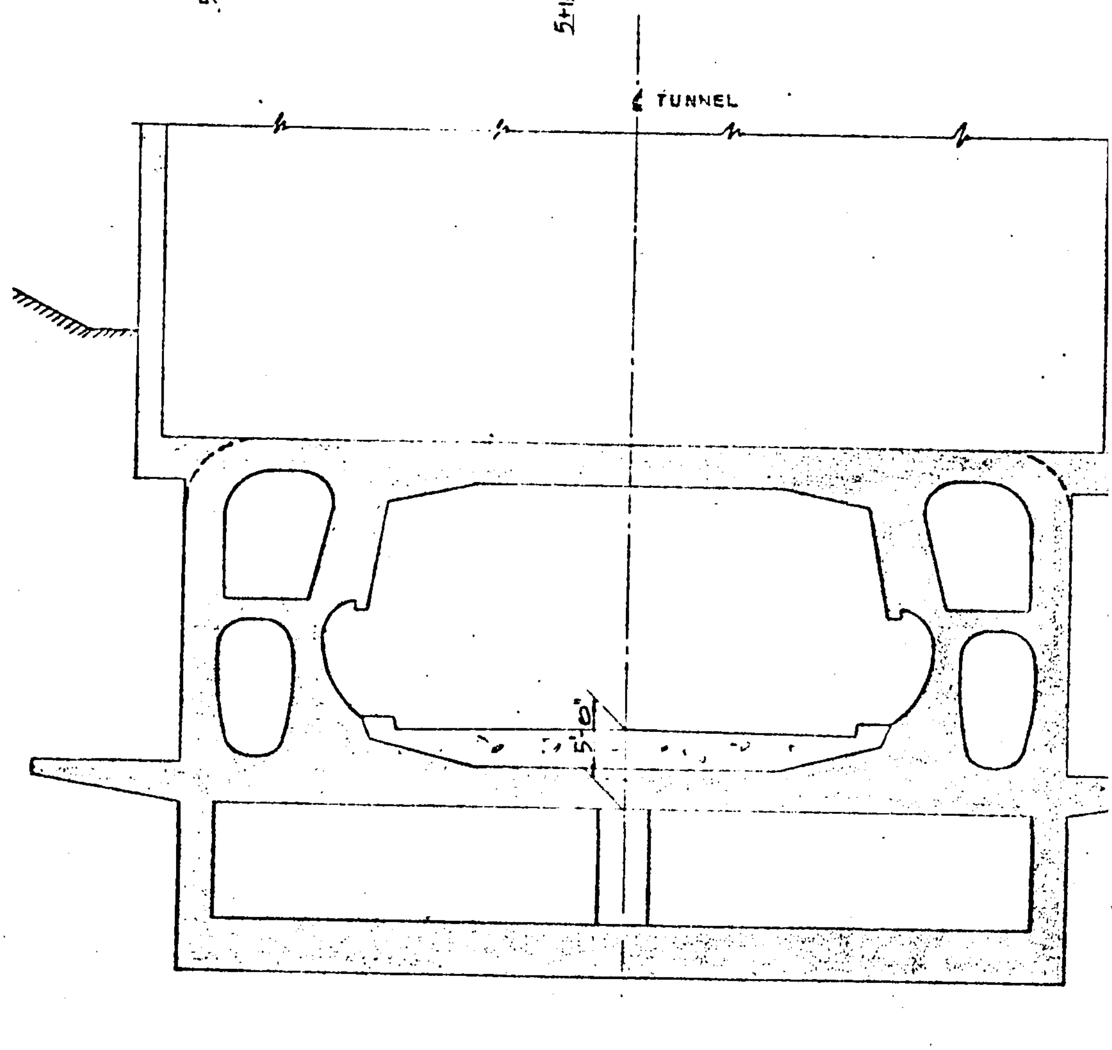


TAMPICO APPROACH

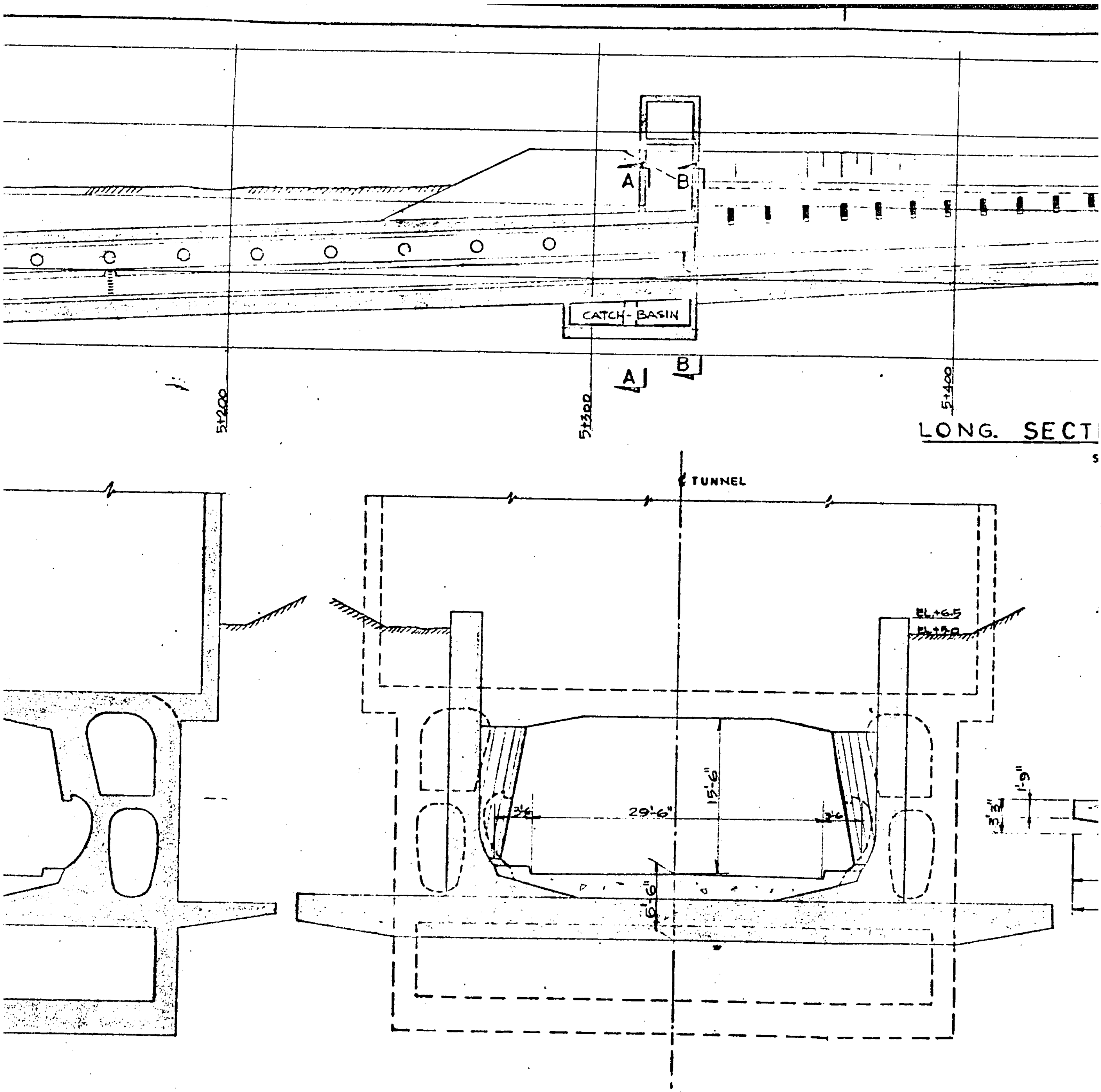
LONGITUDINAL AND TRANSVERSE SECTIONS

Fig. 6.1

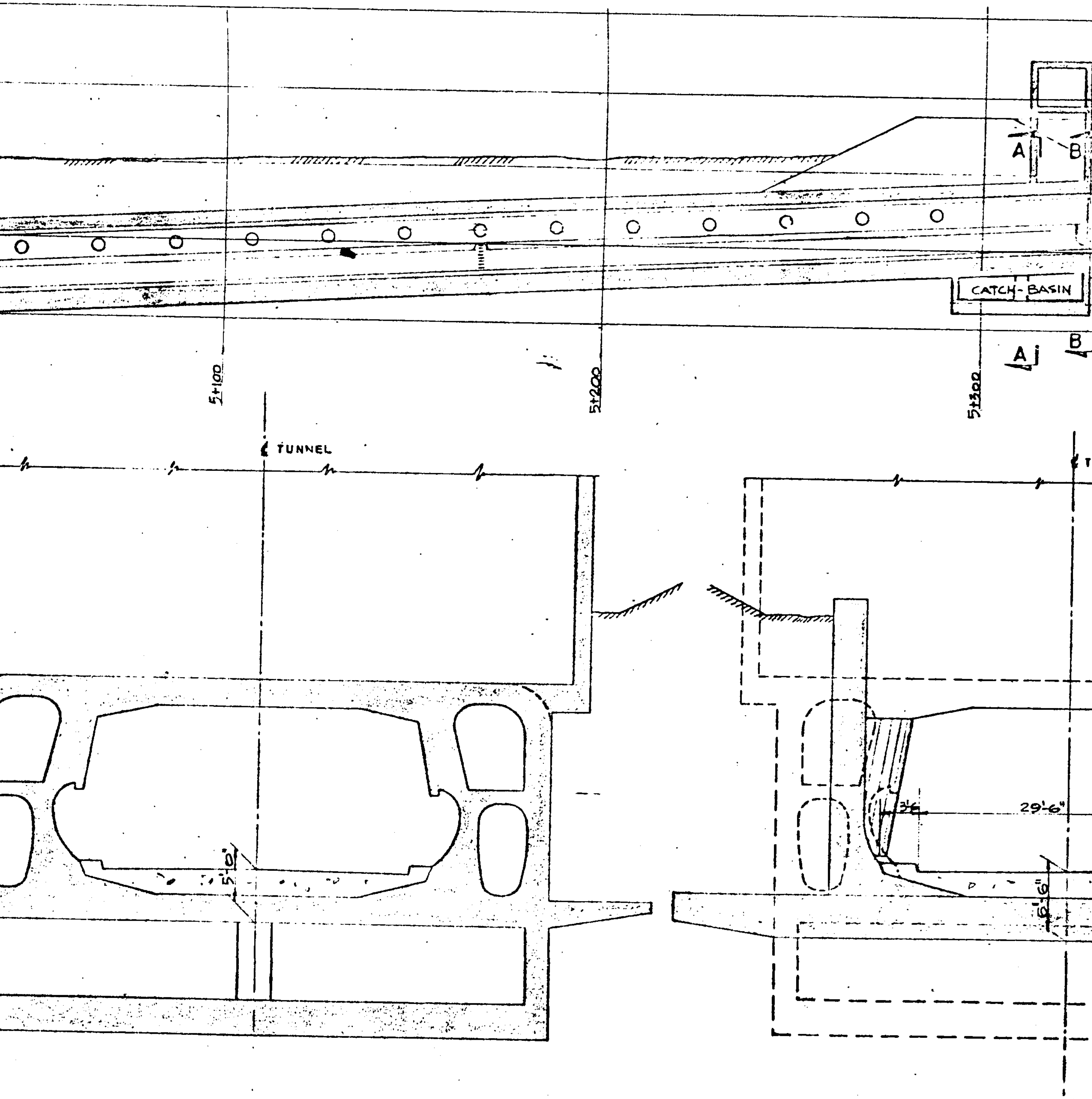




SECTION "A-A"

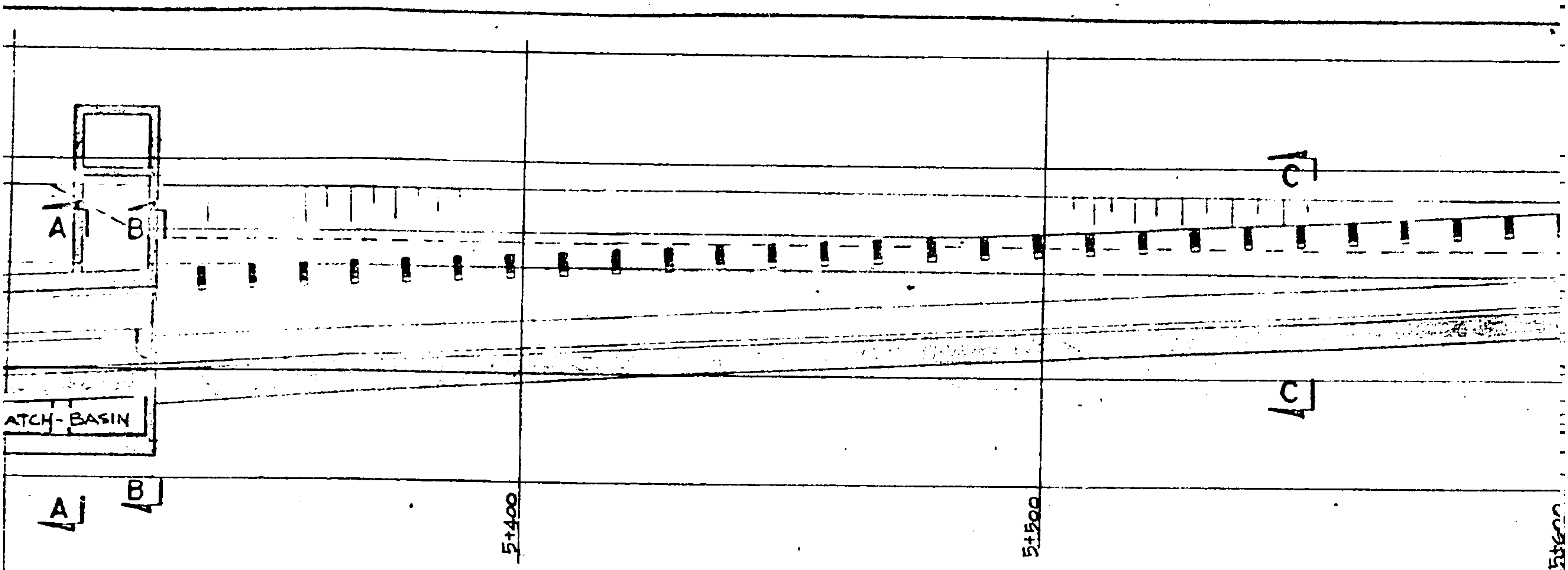

A
B
CATCH-BASIN
A
B
5+200
5+300
5+400
LONG. SECT
℄ TUNNEL
EL.+6.5
EL.+5.0
15'-6"
29'-6"
3'-6
3'-6
6'-6"
3'-3"
1'-9"

SECTION "B-B"



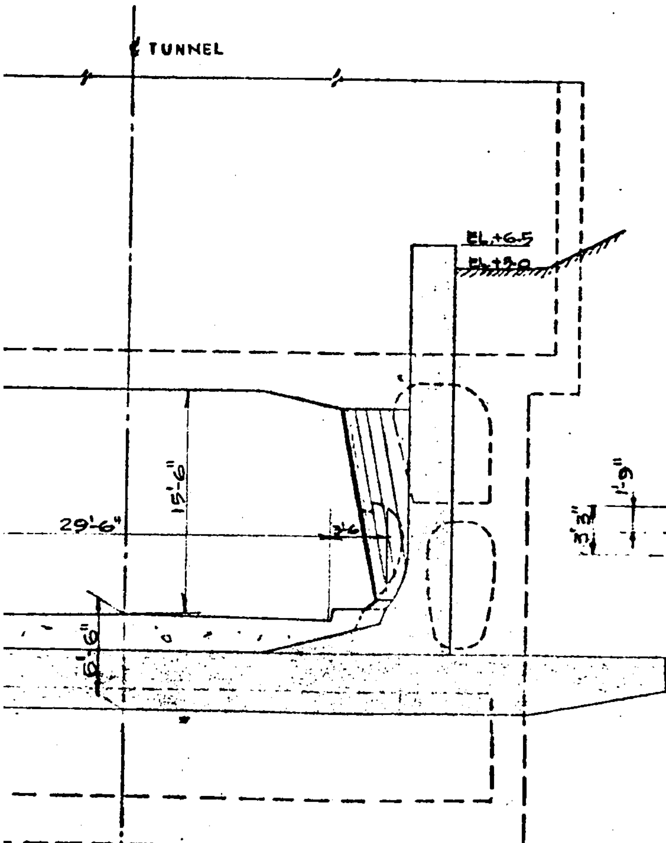
SECTION "A-A"

SECTION



LONG. SECTION OF APPROACH

SCALE 0 10 20 30 40 50 FEET



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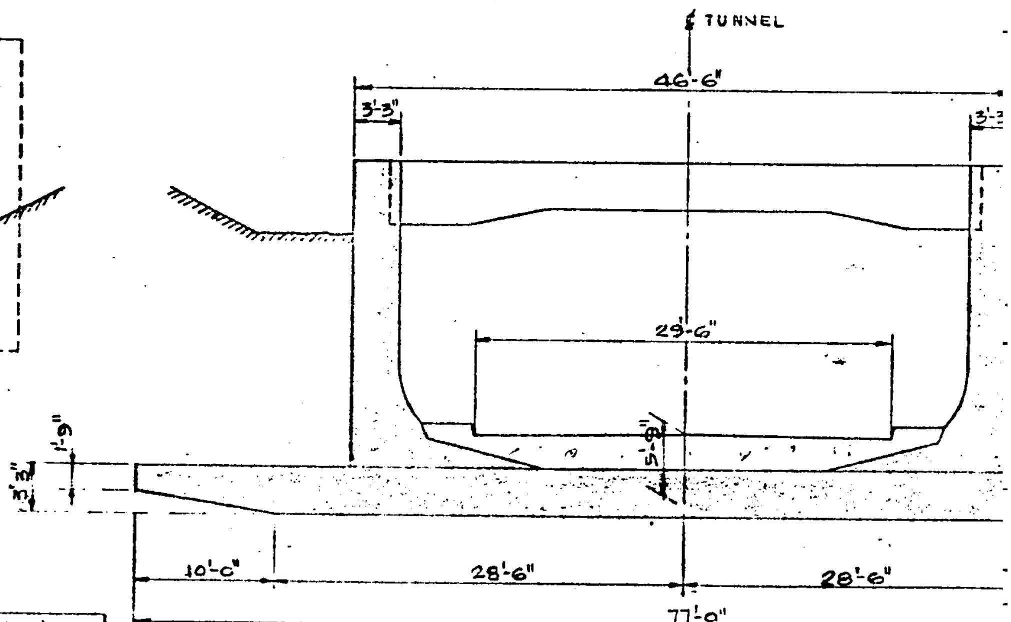
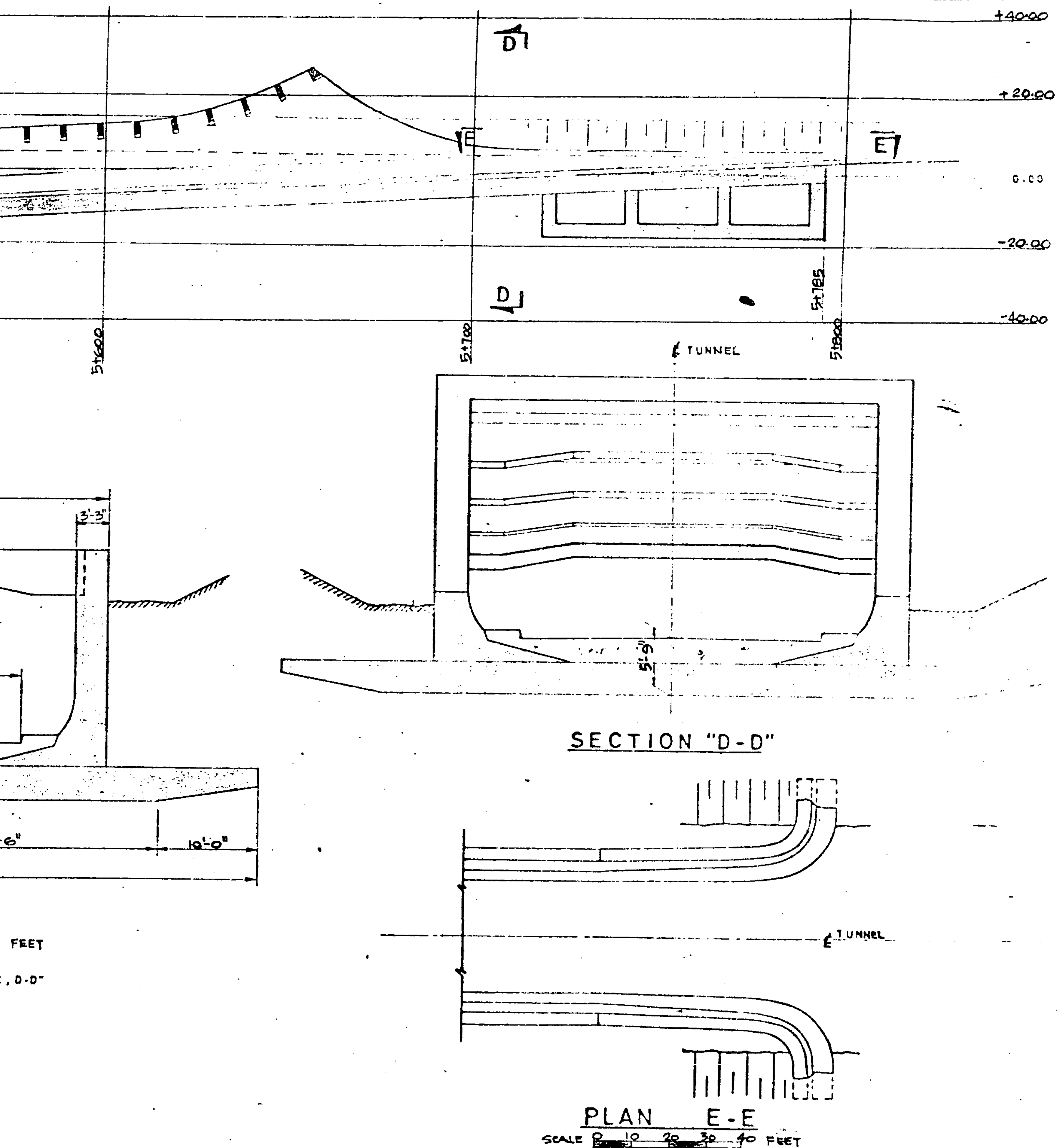


SECTION "C-C"

SCALE 0 10 20 FEET

APPLIES TO SECT. "A-A, B-B, C-C, D-D"



MATA REDONDA APPROACH

LONGITUDINAL AND TRANSVERSE SECTIONS

Fig. 6.2

6.2 Mata Redonda Approach

As on the left bank, the crown of the roadway at the right bank also slopes upwards at 5% grade from El. -20.0' at the tunnel portal to El. +3.25' at the upper end of the approach structure. On this side, however, the roadway continues at 5% grade approximately 1,000' to El. +52.50'. The extra length on this shore is required to provide a sufficiently long launchway for the fabrication of the 5 immersed tunnel elements.

The approach structure on this side, illustrated in Fig. 6.2, incorporates the launchway slab provided for the prefabrication and launching of the subaqueous tunnel elements. This side has been chosen for the fabrication site because of better site conditions pertaining to soil characteristics and the natural requirement for the long slope up to the top of the hill. The ground is underlain by sandstone with good bearing capacity and excavation of such a soil presents no difficulties.

The launchway slab, supported by the sandstone, is wider than the width required for the approach structure and will be built first in order to construct the tunnel elements. Because of this limitation, hinged joints, rather than rigid joints, between the walls and the slab must be assumed. As the bottom slab extends beyond the outer faces of the sidewalls, the volume of backfill supported by the slab would add to the stability of the structure. Lacking the sheet-pile walls acting as cut-off walls of the Tampico approach, uplift is more severe here than the other side. This condition has been designed accordingly.

CHAPTER 7

TUNNEL WATERPROOFING

One of the more serious problems troubling the designers of underwater works is the difficulty encountered in achieving watertightness where such a requirement is needed in the structure.

In the field of immersed tunnel construction, waterproofing, although essential, is as yet in a rather primitive stage of development. It is indeed unfortunate that despite all the advances achieved in other technical areas, the problem of leakage in marine structures still remains.

As a certain degree of voids exists in all substances, absolute impermeability is theoretically impossible. Together with the unavoidable tension cracks, shrinkage cracks and improper construction joints of the tunnel elements, most tunnels do leak, though in some cases, only minutely. In such cases, the small quantity of water able to penetrate through the barriers would evaporate before damage could be done. The difference between excellent and poor waterproofing is merely in the speed in which the water is able to penetrate through the waterproofing membrane, and then into the tunnel interior through the various cracks in the concrete. Hence, waterproofing is dependent on the permeability coefficients of the various substances.

7.1 Watertightness Through Concrete and Tunnel Construction

Water seepage would be significantly impeded by an appropriate choice of concrete mix and casting procedures and by developing flexible joints between abutting tunnel elements. The waterproofing on the tunnel exterior would not then need to be called upon entirely to resist water infiltration.

7.1.1 Choice of Concrete Mix

It is possible, through a thorough study, to choose a concrete mix which will produce a low modulus of elasticity, a minimum of hydration heat and shrinkage and a high degree of tensile strength and watertightness. In the design of the Metro tunnel at Rotterdam, after a study of 14 concrete mixes, a watertight concrete resistant to water pressure of about 2 atmospheres was achieved. The provision of a bitumen membrane on the outside was then only an extra precaution.

7.1.2 Use of Prestressed Concrete

Tensile cracks in ordinary reinforced concrete tunnels are difficult to eliminate as hairline cracks must exist before the reinforcing steel comes into play. This problem can be overcome by the use of a high tensile strength concrete and prestressing cables as the concrete reinforcement. This method can, through appropriate design, ensure compression on all sections of the concrete. The potential leakage points in utilizing prestressed concrete will then be restricted to the anchorage areas for the prestressing cables. However, these seepage paths are known beforehand and can be effectively waterproofed.

Shrinkage cracks are also a problem. The shrinking process takes a long time to complete although much of the shrinkage will have taken place in the first few months. As it is impossible, from a construction viewpoint, to wait for the shrinking process to complete, cracks will occur after the tunnel elements are in place and backfilled. A concrete having a low heat of hydration and low shrinkage values will help to a certain degree. It is preferable, however, to provide in addition some longitudinal prestressing steel to ensure compression to the tunnel cross section to reduce the shrinkage cracks and thus increase the effectiveness of waterproofing. At the same time, this provision will act as reinforcing for the tunnel working as a beam when differential settlements occur.

7.1.3 Choice of Waterstop for Construction Joints

Another region of possible leakage is along the construction joints which are usually made to reduce shrinkage cracks. The construction joints in the Lafontaine Tunnel at Montreal were spaced at alternately 50' and 2' intervals. The 50' sections were first cast, and the 2' sections were poured after sufficient shrinkage of the 50' sections had taken place.

The joints between adjacent pours are usually gapped with a rubber waterstop. But due to the flexibility of the waterstops which often get displaced during the concreting operations, leakage of water to the tunnel interior is highly probable if the outer waterproofing system fails. To circumvent this problem, stiffer waterstops such as those vulcanized with two steel plates at the two edges of the waterstop would provide the rigidity required to remain in place during concreting.

7.1.4 Choice of Joint Between Elements

An important factor to be considered in the design of the tunnel elements, from the standpoint of waterproofing, is the flexibility of the joints between adjacent elements. If these joints were rigid, i.e., the entire tunnel acting as a beam of infinite length, large tension cracks could develop due to the longer length available for unsupported spans; whereas, if these joints were flexible like hinges, to tolerate the rotations produced by possible differential settlements, the maximum unsupported span possible would be that of the length of the element. The maximum width of tension cracks and hence the longitudinal prestressing force required to prevent these cracks can be computed.

The flexibility required at a joint can be satisfied by using a rubber membrane as a secondary watertight seal to bridge the gap between the adjacent elements instead of using welded stiff steel plates. The primary seal is the neoprene rubber gasket which is pressed tightly against the end face of the abutting elements either hydrostatically or mechanically. The joint is then completed by pouring a shear key which will allow a certain degree of rotation but prevent relative upward movements at the joint. Such treatment will reduce the tendencies for large cracks in the concrete and simplify the waterproofing system.

7.2 Watertightness Through Applied Waterproofing

Of the many commercially available waterproofing methods existing today, not one can be said to be absolutely watertight without some reservations. Most of these methods comprise a thin layer of watertight jacket applied on and around the exterior peripheral area of the tunnel. In most submerged tunnels, this outer waterproofing membrane is used as the primary seal. The following seven methods of waterproofing tunnel elements were studied: built-up bituminous membrane, penetrating chemicals, rubber membrane liners, liquid rubber membrane, steel plate membrane, fibre-glass reinforced polyester covering and bentonite panels.

7.2.1 Built-up Bituminous Membrane

Most concrete tunnels built to date used this type of waterproofing. It comprises an asphalt primer, 3 or 4 plies of glass fibre alternating with 4 or 5 coats of asphalt. This membrane is then protected by timber sheetings, asphalt panels or mortar. A timber fendering system on the outside protects the waterproofing and the timber sheets from being damaged during handling and sinking operations.

Major tunnels such as the Lafontaine, Deas Island, Schelde, Heinenoord, Coen, Rotterdam Metro, etc. used built-up bituminous waterproofing membrane in similar applications cited above. The Lafontaine Tunnel in Montreal employed ½" Carey asphalt panels as the outer layer to protect the membrane. However, these panels had to be sprayed with white paint to prevent them from melting due to heat absorption under the hot sun. The Schelde Tùnnel at Antwerp left a 1" space between the bituminous membrane and the timber sheeting so that the motar poured into this space provided additional waterproofing safety.

Built-up bituminous membranes have been applied mainly to rectangular concrete tunnels. As a means of halting water leakage, its true performance is questionable. Under certain circumstances, asphalt can become a brittle substance and therefore is subject to cracking under impact forces. Hence the waterproofing quality disappears if such cracks develop. With the fibre-glass reinforcement, occurrence of such undesirable consequences may be rare. This remains an uncertainty because information concerning impact behaviour and permeability is unavailable.

The cost of this method of waterproofing is relatively reasonable. With all the membrane components in place, the price is approximately $.50/sq. ft. Together with the protective timber sheeting or asphalt panels and the timber fendering system, the total cost would be in the order of $.70/sq. ft. It must be remembered that conscientious site supervision is required to ensure that the various layers of the membrane are properly applied.

7.2.2 Penetrating Chemicals

This methods of waterproofing as a primary seal, has not been used for immersed tube tunnels. Although the Mont Blanc and the Ottawa Art Centre Tunnels both used chemcials for waterproofing, they are dry, bored and cut and cover tunnels and are not subject to high water pressures as are immersed tubes. The leaks caused by faulty construction joints in the Limfjord Tunnel in Denmark were repaired by applying a penetrating chemical form inside the tunnel. The components of the penetration agent consisted of cement, sand and chemicals. The chemical mixture is applied to the moist concrete surface which by virtue of capillary action existing in the moist concrete is carried well into the concrete surface. It has been claimed that penetration can reach more than 2' and that waterproofing is effective provided the cracks in the concrete do not exceed 0.01". One advantage of this method is that no protection is required on the sides of the tunnel against abrasion during backfilling.

One commercially available penetrating chemical mixture is "Vandex". In this instance, one coat of Vandex Super is used as a slurry or wash coat either on inside or outside surfaces, followed by a second coat of Vandex Premix. Another application is to use Vandex Pure with Vax Quickbinder which will be more effective when the surfaces are subject to high water pressure. The cost of materials and labour is in the order of \$.60/sq. ft.

The critical problem in using this method of waterproofing is the limitation that cracks in concrete cannot exceed 0.01". As it is possible that differential settlements will produce tensile cracks, there is no guarantee that these cracks will be within the limit.

Another commercially available chemical product is "Chemstop" waterproofing. It works on the same principle as stated above. However, instead of applying the slurry coats by brushes, the chemical is sprayed onto the concrete surface by low pressure spray. This method of waterproofing so far has been applied only to buildings and has an application life span of 8 to 10 years. The cost of material alone is approximately \$0.05/ sq. ft. Much laboratory testing must still be performed to ensure its suitability and usefulness for submerged tunnel applications. Due to its low material cost and ease of installation, perhaps further studies and tests on this material could be justified.

7.2.3 Rubber Membrane Liners

Utilizing elastic membrane sheeting as a form of waterproofing has been widely adopted for reservoirs, canals, lagoons, cooling ponds, etc. and has been found to perform satisfactorily with a life expectancy of roughly 25 years if not covered by another protective layer. These rubber surfaces can vary in thickness from 1/32" to 1/8" with normal applications calling for 1/16". The permeability of the rubber surface manufactured by "Uniroyal" is around 0.15 perms/mil thickness. This is roughly equivalent to 1 pint of water passing through 1 sq. yd. of 1/8" membrane in 10 years.

The installation of these membrane liners is relatively simple. A coat of adhesive is first applied to the concrete surface to be waterproofed. The membrane is then bonded to the concrete surface, thus providing a tight jacket of waterproofing around the peripheral surface of the tunnel.

The Limfjord Tunnel in Aalborg, northen Denmark which was opened in 1969 employed this method of waterproofing. A 1/16" butyl rubber membrane was glued to the top and walls of the tunnel by a PVC cement slurry and protected by a layer of concrete on the outside.

One major advantage of these elastic membrane liners is that they can be stretched and yet still remain watertight, to tolerate elongation of the concrete induced by hairline cracks caused by differential settlements of the tunnel units. Another feature of the membrane is that if it is protected say, by a layer of timber, no damage will result in the waterproofing even if accidentally subjected to an impact load. Had the waterproofing been composed of brittle materials like a bituminous membrane, it could have cracked resulting in the admission of water.

The cost of 1/16", 3/32" and 1/8" thicknesses of Uniroyal's "Royal-seal" waterproofing surfaces is $0.50, $0.80

and \$0.90 per sq. ft. installed, respectively. Including the timber sheeting protection layer and the timber fender system which in turn protects the timber sheeting, a total cost of approximately \$0.70 to \$0.75/sq. ft. for a 1/16" thick rubber membrane is realized.

A method of waterproofing which offers double performance in stopping water infiltration by providing two water barriers is, instead of using timber sheets and fendering system, to use a thin layer say, ½" thick of ferro-cement gunited onto a watertight membrane. This ferro-cement layer, which consists of 2 or 3 layers of chicken wire closely spaced and thoroughly penetrated with a cement-sand mortar either gunited or hand trowelled, is a waterproofing membrane in itself. It is also shatterproof so that further protection against impact and abrasion is unnecessary. Hundreds of ferro-cement ships and sea-going vessels have been built world-wide and they have been found to be waterproof even after the formation of cracks caused by collisions with rocks, or other solid objects. Granted that the draught of ships is not very great compared to the 60' to 80' of water depth in immersed tunnel construction, impermeability of ferro-cement can still be relied on, though may not be as effective as when used for ship construction. Another function of this ferro-cement layer is to protect the rubber membrane, thus extending the life span of the membrane indefinitely since the ferro-cement layer takes all the abuse so that the rubber membrane performs its duty only if the first barrier fails. This utilization then can permit specifying a thinner membrane liner to reduce cost. Using 1/16" rubber membrane and ½" ferro-cement outer layer, the total cost for this double safety waterproofing is in the order of $1.25/sq.ft.

7.2.4 Liquid Rubber Membrane

This type of sprayed rubber coating is available under the trade name "Rubson". Once the coating is applied, it cures to a continuous rubber membrane without joints. This unique coating is composed only of rubber and resin and has been used extensively in Europe, since its origin in 1955, for internal surfacing of water towers, reservoirs, underground concrete walls and foundations. It was claimed that Rubson liquid rubber, applied from inside, was used in the Marseilles Tunnel in France to seal leaks developed.

The main characteristic of the liquid rubber is that it dries on the surface but stays supple below. It will not become brittle or crack and will stay flexible indefinitely. It can stretch to 3½ to 6½ times its thickness depending on the ambient temperature without cracking. This perhaps is its outstanding feature in waterproofing concrete tunnels. With 2 or 3 sprayed layers of the rubber coating, ordinary shrinkage and tension cracks in the structure could be accommodated by the membrane without distress. It was claimed that tests showed that the coatings also offer excellent resistance to impact and abrasion.

Together with the liquid rubber, Rubson also developed a base to which the liquid rubber adheres. This base system called the "All Chlorinated Curing System", bonds to the concrete surface and subsequently cures the concrete and eliminates all other methods normally used for curing. This curing compound penetrates the honeycomb-type structure of the concrete and seals from within, thus preventing the re-entry of water. It is on this curing compound that the liquid rubber coatings are sprayed.

For possible immersed tunnel applications, perhaps more positive measures should be taken to ensure that the rubber coatings remain undamaged during handling and backfilling of the tunnel elements. To this end, the exposed surfaces may be protected by a plywood sheeting and fendering system. However, it is possible to create a mechanical key for the purpose of bonding a thin layer say, ½" of sprayed grout or ferro-cement to the liquid rubber membrane. This mechanical bond can be made by spraying silica sand onto the last coat of freshly applied liquid rubber. The sand fuses with the rubber and leaves a sandy surface having very good bonding qualities.

As the rubber forms a continuous jointless membrane, the possibility of water penetration through the seams of the membrane is eliminated. Hence, the outer layer of gunite serves only to resist impact and abrasion. Using a ½" ferro-cement layer as an outer membrane would provide two barriers against infiltration as the ferro-cement layer itself is watertight.

The cost of the curing layer and 3 sprayed or brushed layers of liquid rubber is approximately $0.60/sq. ft. Including the ½" layer of gunite, the total cost is $1.20/sq. ft.

7.2.5 Steel Membrane

This type of waterproofing is perhaps the most positive way of stopping water from penetrating into the tunnel interior. However, unless the joints between adjacent steel plates are finely inspected to ensure watertightness, complex problems can develop. The water seeping through one leaky joint can run along the minute space between the steel plate and the concrete face and enter into the tunnel interior via the numerous cracks along the length of the tunnel. If this were the situation, it would be impossible to pinpoint the origin of the leak thus rendering it very costly, if not impossible, to repair because the entire tunnel may have to be waterproofed again depending on the amount of leakage occurring.

To avoid such dangers, all welding will have to be carefully performed by highly qualified welders and tediously inspected, preferably by x-ray verification. But this is an expensive process. Resorting to automatic welding will reduce considerably the human errors, but there is still no guarantee that faulty welding and thus leakage through the seams will not occur.

To avoid the steel plates from warping due to welding, joints between adjacent plates could be vulcanized in case of rubber or sealed with bituminous or rubber strips as was done on the Lafontaine Tunnel. Just how effective are such treatments of joints, is debatable. It has been stated often that the steel plate rusts and the oxides produced will close the pores and cracks of the concrete to keep it watertight. This assumption is difficult to justify since the leakage areas do not necessarily coincide with the rusted areas of the steel plate.

In submerged tunnel applications, the steel plate is sometimes guarded by cathodic protection against electrolytic action. The San Francisco Bay submerged rapid transit tunnel is protected by such a process.

The cost of x-ray inspection of welds alone is around \$5.00/lin. ft. and the cost of the steel plate together with welding is approximately \$1.25/sq. ft. of area for a 1/8" plate. Including the x-ray inspection of all seams, but excluding the cathodic protection, the total cost of this method of waterproofing will roughly come to \$1.65/sq. ft.

In existing tunnels which used steel plates as waterproofing membranes, x-ray inspection was rarely employed. Inspection was made by selective trepanning, vacuum soap bubble testing and spot radiographic examination of seams welded by automatic processes. If this type of inspection were adopted at the sacrifice of positive assurance of watertightness, the cost would be around \$1.30/sq. ft.

7.2.6 Fibre-glass Reinforced Polyester Covering

This method has only recently been used on immersed tube tunnel waterproofing, although it has found its applications in the fibre-glass boat building industry for some time.

The Parana-Santa Fe Tunnel in Argentina is believed to be the first subaqueous tunnel to use such a process for waterproofing.

The covering as used on the Argentine tunnel is made by the synthetic fibre extrusion process which consists of the coating simultaneously onto a base substance of lengths of glass fibres and two resinous components in a variable or fixed ratio of fibres to resin. One of the resinous components acts as a catalytic agent, the other as an activating agent. The entire covering consists of 5 or 8 layers of material - a layer of base component, consisting soley of resin, one or two laminated resin-fibre layers and two sealing layers of resin. The porosity of the finished covering is electrically measured and tested to detect any pores or defects.

To prevent damage to the covering, timber sheeting and a timber fendering system would have to be installed. The cost for material and installation of a 5-layer covering is approximately \$1.60/sq. ft. With the addition of timber sheeting and fender system, the cost would become roughly \$1.80/sq. ft.

7.2.6 Bentonite Panels

This type of water barrier is available under the trade name "Volclay". It has been claimed that Volclay bentonite panels have been successfully used for 25 years to seal ponds, lagoons, reservoirs and dams. Essentially, the Volclay panel is filled with bentonite, an inorganic mineral compound which instantly forms into a mineral mastic when it comes in contact with water. The mastic consequently grows in size, expands in position and locks itself in place. As a result, water is unable to penetrate through this mastic layer.

This method of waterproofing performs well for cast-in-place sections which must be backfilled. Volclay panels were used on Thorold Tunnel which was cast in the dry. These panels would also be attractive for waterproofing in-situ tunnel approach structures built in the dry and backfilled. For the immersed sections of the tunnel, the use of these panels is impracticable as the bentonite within them expands immediately when it comes in contact with water. Subsequent backfilling of the tunnel could possibly damage these panels, resulting in the loss of their waterproofing ability. Protective timber sheeting or a concrete layer cannot be installed outside for abrasion protection due to the expansion characteristics of the bentonite compound. If these panels were used to waterproof structures built in the dry, careful backfilling would eliminate the use of abrasion protection.

The cost of this method of waterproofing, which is attractive only for cast-in-place sections, is relatively reasonable. The supply and installation of the panels amount to approximately $0.40/sq. ft.

7.3 Recommended Approach to Waterproofing

Basically, two approaches to tunnel waterproofing are possible. The first is to design the tunnel to tolerate a "defined" amount of seepage, collect this water, by means of sloping the ceiling in two directions and installing gutters at suitable locations, into the central sump at the sag of the tunnel and then pump away into the river after passing through the large sumps at the foot of the portals. If this approach to waterproofing is adopted, the watertight membrane on the outside can be much simplified. The second approach is to provide as watertight a tunnel as possible by suitable choices of concrete mix, construction joints, connecting joints and waterproofing membrane.

The first method, though less expensive, involves a large risk factor since a "defined" amount of seepage cannot be easily assessed and from aesthetic standpoint, the various wet and dry patches and gutter system on the ceiling should be hidden from the view of the motorist in order not to create an unsafe feeling. Furthermore, there is a large possibility that the water which penetrated through the structural shell will corrode the reinforcing bars.

As far as is known, most tunnels have been designed to achieve as dry a tunnel as possible. For the present purpose, the second approach to waterproofing will be adopted, since the total cost of waterproofing amounts to only approximately 2% of the entire cost of the tunnel project. However, the first approach to waterproofing certainly deserves further investigation as complicated, expensive membranes can be eliminated.

Having decided on the general approach to waterproofing the next step is to select the appropriate watertight membrane to be used. Absolute water leakage prevention is theoretically impossible. A so-called "impermeable" membrane only means that it has, relatively, a much lower permeability coefficient when compared with other materials. Every marine structure can be "waterproofed" by a certain method at a certain cost and at a certain tolerance as to the amount of water which slowly seeps into the interior and has to be drained away.

The most inexpensive method of waterproofing is by the use of penetrating chemicals. However, as the use of this technique is uncommon in the case of immersed tubes, its usage here should be considered only after more elaborate laboratory testings. The next least expensive way is the method of built-up bituminous membrane. Although many existing immersed tunnels have adopted this technique, its waterproofing ability is dubious as leaks are visible in some of these tunnels. Not much is known of the performance of the fibre-glass reinforced polyester covering in immersed tunnels as the first tunnel adopting this method is still under construction. It is expensive, which is part of the reason why this process has not been adopted before. Wrapping the tunnel with a thin steel membrane seems to be a very effective way of preventing water from entering the tunnel. The only problems are the seams between adjoining plates and the probable need for cathodic protection. If these problems can be overcome, it would perhaps be the best method of waterproofing to date. It is expensive but it can offer structural assistance to the tunnel cross section.

The method of rubber membrane liner protected by a ferro-cement layer seems to have prospects offering a high factor of safety against leakage at a medium cost, but there may be some problems in bonding the ferro-cement layer to the rubber membrane. The sprayed liquid rubber membrane seems to be the most promising waterproofing system, if also protected by a ferro-cement layer. The use of ferro-cement has penetrated well into the ship-building industry and the material has shown excellent performance as to watertightness and impact. Based on these reasons, this method of waterproofing has been contemplated for use in the present tunnel.

For the cast-in-place portions of the closed tunnel, bentonite panels will be used to line the earth faces of vertical walls.

CHAPTER 8

FOUNDATIONS

General - Experience with trench type tunnels has shown that a proper choice of the type of foundation employed to support the submerged tunnel is of paramount importance, both from structural and economic standpoints. The type of foundation adopted and the methods devised to prepare this type of foundation comprise a considerable portion of the total cost of the tunnel, these are areas of potential savings in cost and construction time. For these reasons, six feasible types of more or less conventional foundations have been studied in order to determine a particular type which when considered simultaneously with other construction works of the project, would bestow the most economical solution.

In general, finding suitable ground for the support of the tunnel is obviously not a very great problem, as it is basically true that when excavating the trench to receive the elements, the material removed will, in most cases, have a greater weight than the submerged elements placed in the bottom of the trench since when fully ballasted the tunnel will have an overall specific gravity of only 1.1. In an actual tunnel structure, the most severe loading condition will occur under the extreme case of flooding of the tunnel, either accidentally or purposely.

Tampico Tunnel - According to preliminary soils investigation, the subsoil along the chosen tunnel alignment changes appreciably from point to point. From the Tampico shore to the Mata Redonda shore, the underside of the tunnel goes through alternating layers of sand and clay with constantly varying compositions and compressibilities ranging from very soft mud to sandstone. The sandstone occurs at some distance inward from the Mata Redonda shore. Hence, the only location suitable for the fabrication of the tunnel elements is either within a cofferdammed dry dock or a sloping launchway.

For the given physical conditions, several foundation methods are technically feasible. These are as follows:

1. Piled
2. Sandjetted
3. Screeded crushed stone
4. Grouted
5. Sand-asphalt

Each one of these methods can yield a safe structure at a certain cost. Some of the above methods are, under the given conditions, obviously more expensive than others and may be eliminated prior to a detailed cost comparison. However, for completeness, they will likewise be analysed in some detail in order to compile a table for cost comparison purposes.

8.1 Piled Foundation

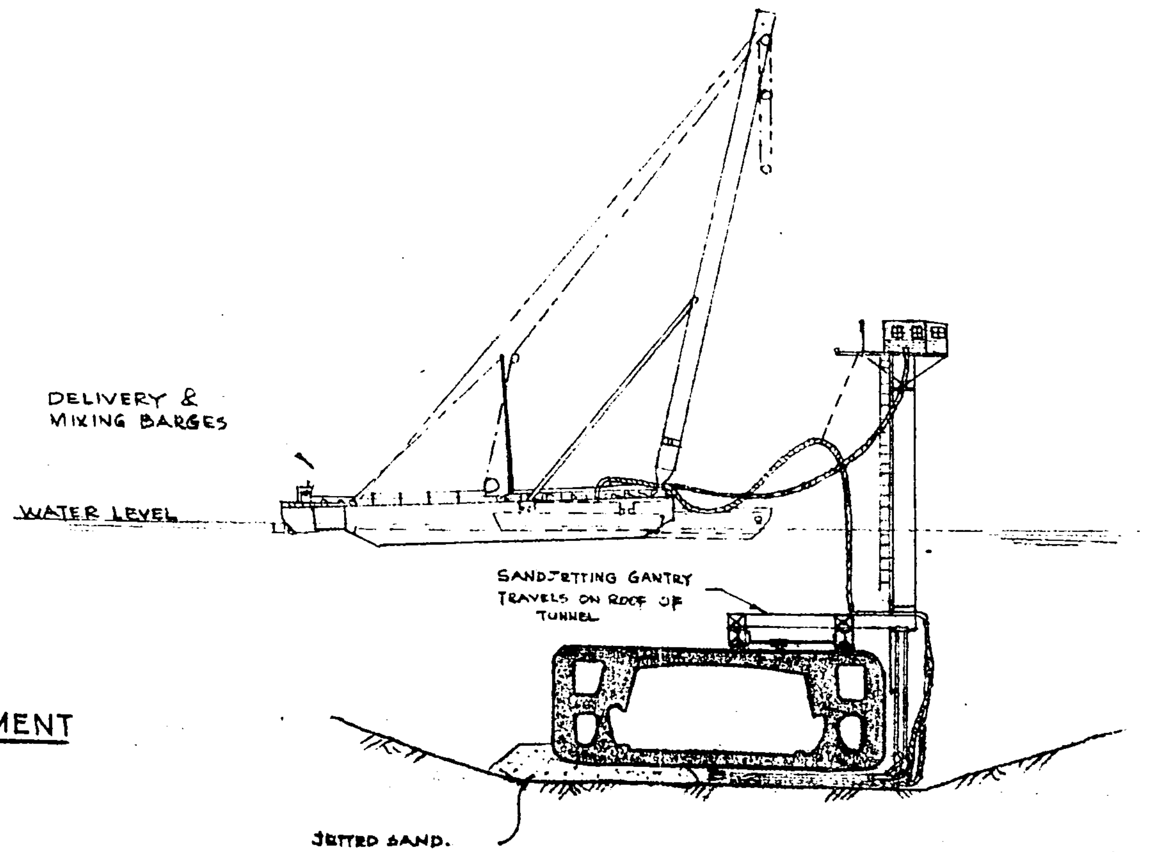
Due to the extremely adverse soil condition on the centreline of the tunnel, this type of foundation would be expensive. The subsoil between the shores is mainly mud underlain by silt and clay with an N-value of less than 10 for considerable depth except at the centre sag of the tunnel. Piles would have to be driven in some instances down to El. -150' to obtain reasonably good bearing. In this case, there would be a total pile length of 160'. The use of friction piles has to be discounted because dubious values for skin friction would have to be assumed and these values would be of such small magnitude as to render friction piles impracticable.

Of course, standard construction equipment may be used in driving the piles, thus eliminating the need to invest in specially built equipment such as a sandjetting rig for the sandjetted foundation. Another advantage is that differential settlement along the length of the tunnel is virtually eliminated. This means that no special flexibility will have to be built into the tunnel, resulting in some savings.

Theoretically, scour protection could possibly be eliminated as long as it could be ascertained that the scour does not reach too deep below the bottom of the tunnel where the stability of the piles would be endangered, and as long as there would be no danger of anchors dragging from passing ships doing damage to the tunnel. Isolated scour holes formed under the tunnel elements could widen and deepen quickly. Thus, elimination of scour protection of the tunnel necessitates the assurance that the depths of scour holes remain within the design limit. As there can be no guarantee that such favorable conditions can be satisfied at all times, the use of top tunnel protection will have to be effected to maintain the same magnitude of safety. In the Tampico Tunnel under study, scour action is very severe and unpredictable. Therefore, for safety against the uncertain, it is advisable to provide scour protection as on other types of foundations.

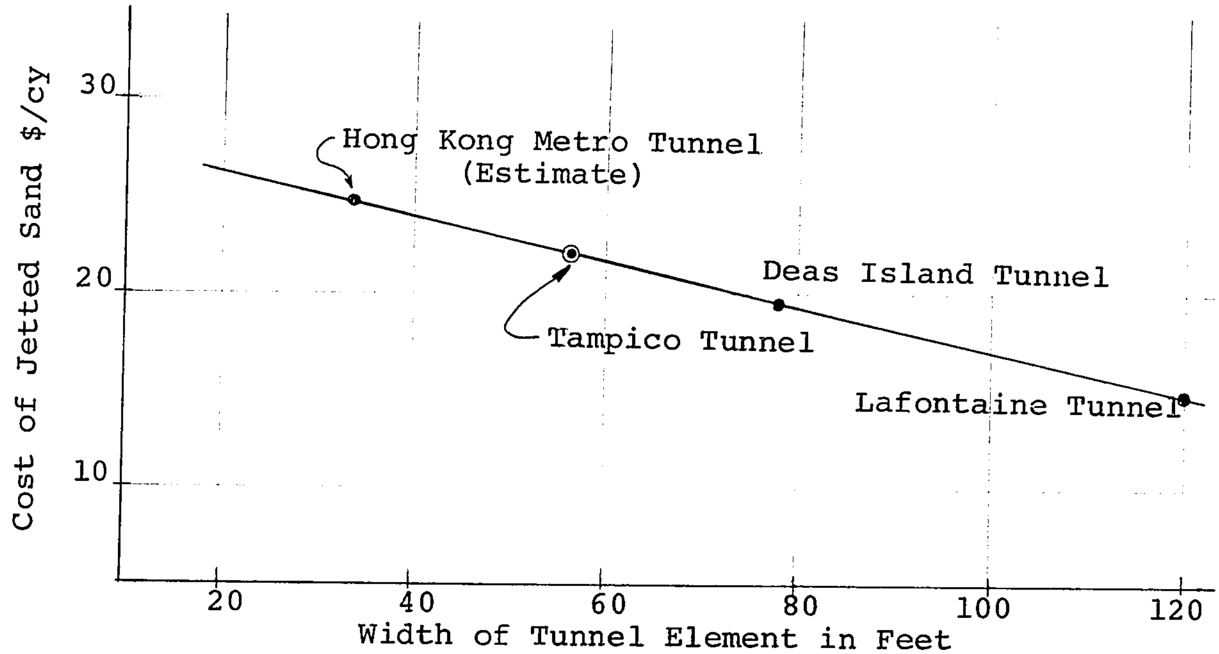
8.2 Sandjetted Foundation

This type of foundation, devised by Christiani and Nielsen, a Danish construction firm, was first used in the building of the Maas Tunnel in Rotterdam, Holland in 1941. In this method, sand fill is mixed with water and jetted under the tunnel elements which are resting on temporary support blocks, to form a uniform and reliable support. Sandjetting is especially applicable to wide rectangular cross sections. Many foundations for rectangular tunnels have used this classical method of sandjetting to yield a uniform support for the tunnel elements. Both of Canada's immersed tunnels, the Deas Island and the Lafontaine, were constructed using this method of foundation preparation. The essence of this method is shown in Fig. 8.1. The most expensive item is the specially built sandjetting rig. Thus, it is obvious that the cost per unit volume of the jetted sand decreases with the increase in tunnel width as an almost identical rig is required regardless of the width of the tunnel cross section. From experience, this type of foundation is not economical for narrow cross sections. The graph in Fig. 8.2 shows the variation



SANDJETTING EQUIPMENT

Fig. 8.1



COST OF SANDJETTING AS A FUNCTION OF TUNNEL WIDTH

Fig. 8.2

of cost as a function of the width of the tunnel cross section.

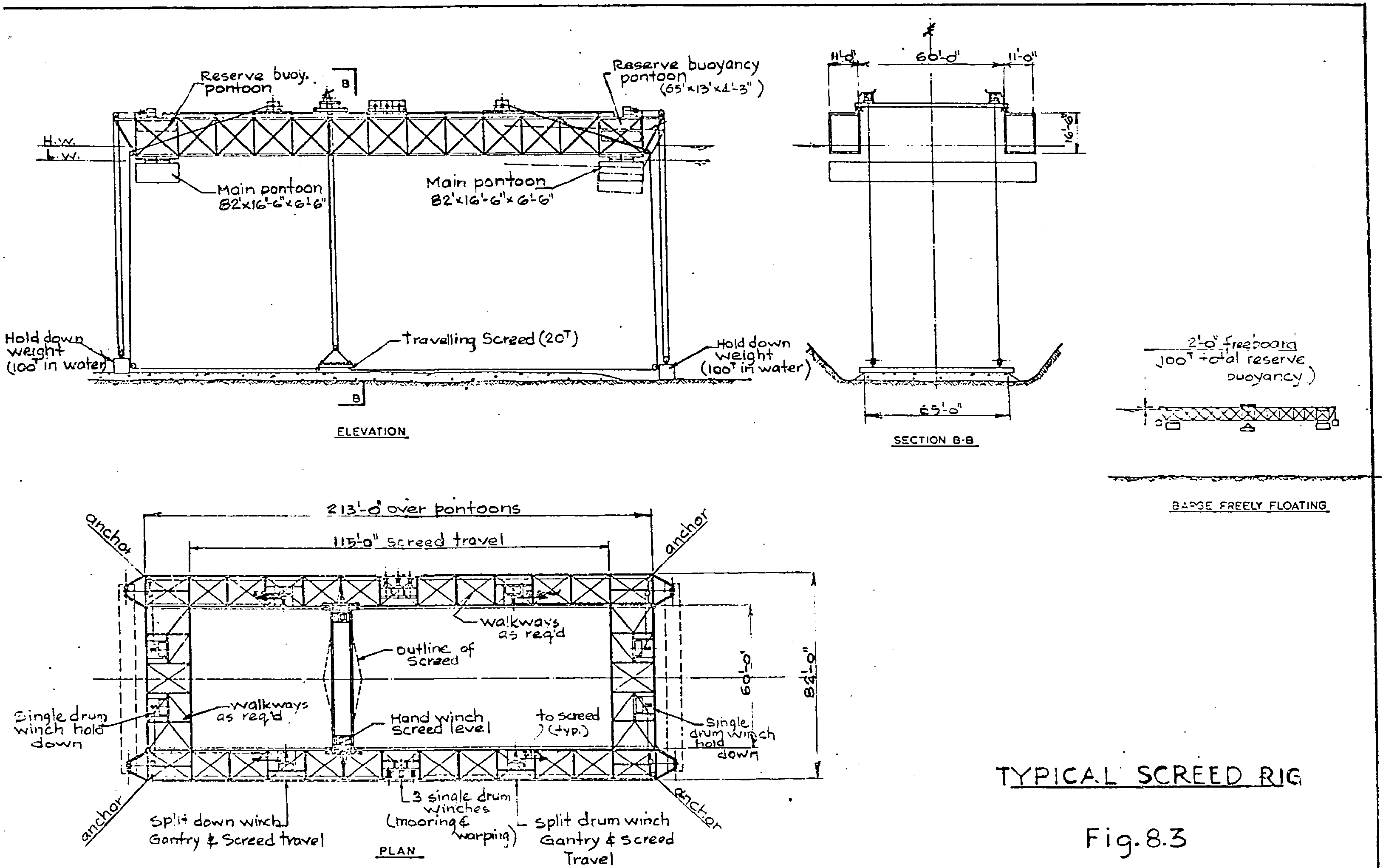
It has often been found that the final quantities of sand used in the sandjetting process are somewhat higher than the estimated quantities. The reason is probably due to the higher compaction actually achieved, to the washing away of part of the sand and to the scouring and then sucking up of the supporting soil by the jetting and suction pipes of the sandjetting rig respectively. The scouring action caused by the jet is more pronounced when the supporting soil is weak as is the case in the Rio Panuco. This undesirable effect could be eliminated by protecting the supporting soil by a layer of say 2' of gravel before sandjetting. It is quite possible that even this precaution is inadequate and that a filter may be necessary.

Sandjetted foundations require that the bottom of the tunnel cross section be provided with a steel plate as a waterproofing membrane because differential settlements will invariably occur causing tensile stresses in both the longitudinal and transverse directions. Unlike tunnels supported by piles, where scour protection could be eliminated under certain conditions, tunnels bearing on sandjetted foundations must be protected against scour action lest the scour damage the foundation.

8.3 Screeded Crushed Stone Foundation

The preparation of this type of foundation consists of dredging a trench at a stable slope in the river bed in which the tunnel elements are to be placed. The accuracy of the suction type dredge is usually in the order of ±1'. The bottom of the trench is then levelled off with crushed stone or gravel by means of a specially built screed barge as shown in Fig. 8.3. As the tunnel elements will be supported by this gravel bed, it should be levelled to an accuracy of about ±2". This kind of fine screeding under deep water is difficult and therefore expensive. This type of foundation does not give truly uniform support to the tunnel elements. However, for structural purposes, it has usually been assumed as providing uniform support.

Like the sandjetted foundation, this type of founda-



TYPICAL SCREED RIG

Fig. 8.3

tion is also prone to differential settlements due to the greatly varying soil conditions along the length of the tunnel. A steel plate membrane has to be provided to prevent water leakage. Again, whether the tunnel cross section is wide or narrow, a practically identical screed barge is required, therefore cost per unit volume of prepared foundation decreases with increase in tunnel width. As with the sandjetted foundation, scour protection is a necessity.

In Tampico, the cost of gravel is higher than the type of sand required for sandjetting. As the cost of a sandjetting rig is similar to that of a screed barge, it is obvious that a screeded foundation is not competitive with sandjetting if it is assumed that the same quantity of founding material is present in both foundation types. Unfortunately, the quantity of sand required is normally more for sandjetting than the gravel required for the screeded foundation.

8.4 Grouted Foundation

This type of foundation requires a trench to be dredged as in all other types of foundation. The tunnel elements are then placed in the trench and supported by temporary foundations. The tunnel elements have vertical grout holes through the tunnel walls or the tunnel floor. Grout pipes are inserted through these openings and the space between the bottom of the tunnel floor and the dredged trench bottom is then filled with grout. However, because the trench is dredged under water and at great depth, the bottom of the trench is initially quite uneven. This large tolerance of the trench bottom implies that considerable quantity of grout is required to completely fill the voids. As the grout has to be placed under water and under tremendous hydrostatic pressure, when grouting from inside the tunnel tube through the sealed floor openings, this method of preparing foundation is normally expensive. Grouted foundations are used only if the quantity of grout required is small or if special performance is required, such as extra load carrying capacity or watertightness.

A variation of the grouted foundation is a combination of a roughly screeded crushed stone foundation topped with perhaps 6" of grout between the tunnel elements and the crushed stone bed. It is relatively inexpensive to rough screed the gravel bedding because such an operation does not require any specially built equipment. Rough screeding can be effected by attaching a screed beam to the under side of the suspended tunnel element and running the beam back and forth over the gravel bedding. With this version of grouted foundation, only a small quantity of grout is needed and some savings in cost are possible. To date, only the Tingstad Tunnel in Sweden has used this type of grouted foundation and then in a modified form. Although the original plans called for a roughly screeded grouted foundation similar to the type discussed above, problems due to the rapid siltation of the tunnel trench required that the grouting be used in conjunction with pile clusters and grout bags atop these pile clusters. The first try at the roughly screeded and grouted foundation then, was not altogether successful.

In river crossings where siltation does not present problems, this type of foundation could be economically utilized. Similar precautions regarding differential settlements, thus tensile cracks, and scour protection of the foundation material must be exercised.

8.5 Sand-asphalt Foundation

This type of foundation is a further refinement of the roughly screeded grouted foundation mentioned above. The 6" of grout is replaced by 6" of sand-asphalt. The method of placing the sand-asphalt is similar to that used for placing the grout as both materials have the consistency of a dense fluid.

This method of foundation preparation has evolved from the fact that the cost of grout in place is considerably higher than the cost of sand-asphalt in place. Tunnel elements founded on a sand-asphalt layer and gravel bedding would truly be supported on elastic foundations as the sand-asphalt layer itself is compressible. This relatively high compressibility of the sand-asphalt layer results in the tunnel elements being supported uniformly along the entire length of the tunnel. Although the roughly screeded gravel bedding may result in some differential settlement, the more compressible sand-asphalt layer will tend to reduce this differential settlement by compressing more the areas which did not settle, that is, provided that the settlement is not appreciable, the sand-asphalt acts as a thick layer of rubber. Rubber has the tendency to flatten out any small unevenness, thus providing a more uniform support to the structure. To reduce excessive settlement of the foundation below the sand-asphalt, well-graded gravel fill is used for the underlying bedding and screeded to the required grade.

This type of foundation, although giving more uniform support to the structure than the sandjetted or the grouted foundations, also requires a steel plate membrane on the underside of the tunnel element for extra safety against water leakage.

8.6 Cost Comparison of Foundation Types

The six types of foundation discussed above may now be summarized and compared from the cost standpoint. It must be understood that the least expensive foundation type suitable does not necessarily mean that it is the most economical type geared to reduce cost of the project as a whole. To induce overall economy, the choice of method of foundation preparation must take into consideration other related factors of the entire engineering works. For example, in the Tampico Tunnel, portal cranes which are to be used for the fabrication of the tunnel elements can be used for placing and rough screeding the gravel bedding and injecting the grout or the sand-asphalt layers. Thus, these portal cranes serve as multipurpose equipment and considerable savings in cost and time can be foreseen.

Table 8.1 compares only the relative cost of the various types of foundations and the values shown do not necessarily represent the absolute costs.

Table 8.1

COST OF FOUNDATIONS

(per lin. ft. of tunnel)

Item	Piled	Sand-jetted	Screeded	Grouted	Roughly Screeded, Grout	Roughly Screeded, Sand-asphalt
Dredging	415	443	391	400	397	397
Piles (65')	442	-	-	-	-	-
Gravel layer	-	18	255	-	30	30
Rough screeding	-	10		-	10	10
Sand	-	250	-	-	-	-
Grout	-	-	-	345	95	-
Sand-Asphalt	-	-	-	-	-	69
Steel membrane	-	-	39	-	-	-
COST (Can.$)	857	721	685	745	532	506

It can be seen that the roughly screeded foundation topped off with either a grout or sand-asphalt layer results in the most economical foundation type studied. The screeded foundation with the sand-asphalt layer was chosen for the

Tampico Tunnel as this type of foundation can be easily prepared with the aid of the two portal cranes available.

Recently, a founding method, traded named "Vibroflotation" has been adopted for an immersed tube tunnel. This method is suitable mainly for tunnels with round cross sections. The foundation is prepared essentially by compacting, by means of vibration, the cohesiveless fill dumped around the circular cross section. The fill, of course, must be confined so that the voids in the fill can decrease by the resulting settlement of the fill material.

The latest tunnel employing this type of foundation is the Parana-Santa Fe Tunnel in Argentina. This tunnel, with a circular section, is at present under construction.

The cost of this type of foundation is very competitive with the roughly screeded foundation shown in the Table above. However, as the Tampico Tunnel is rectangular in section, founding by vibroflotation is not a practical solution.

CHAPTER 9

RIVER SCOUR

General - Subaqueous tunnels crossing under a body of water where scour action is pronounced, caused by either the high currents or obstruction of the tunnel to the natural flow of the water, require protection to the tunnel and river bed both upstream and downstream of the crossing. Where scouring is slight, protection to the river bed can be omitted, however, the tunnel itself should remain protected in all cases. Scouring caused by propeller wash and damage from anchors of ships passing over the crossing will then be unable to endanger the safety of the tunnel structure and its foundations.

Scour protection can be effected by several methods. One method is to provide a flexible stone mattress to cover the tunnel and the river banks. Should the soil under the mattress be eroded by scour action, the stone mattress would merely settle to resume the surface contour of the new river bottom. Thus provided that scour action does not reach too deep below the design scour depth producing an unstable slope for the rock mattress, the tunnel and river banks remain protected. Another method of scour protection for the tunnel is to enclose the tunnel foundation and the tunnel side backfill by steel sheet pile walls driven well below the maximum scour depth. Scour action will then be halted outside of the enclosure. However, as the soil outside the enclosure is scoured away, the vertical steel sheet pile walls become exposed on the outside and accentuate the scour action due to the turbulence caused by the exposed wall tending to stop the river flow. This condition must be investigated in the actual design.

Tampico Tunnel - In the present Tampico crossing, the tunnel and river bed upstream and downstream of the tunnel will be protected by a stone mattress as mentioned above because of its greater flexibility. Its main purpose is to protect the tunnel itself against undermining, but it also provides a gradual guidance of river flow from the natural river section to the wider and shallower tunnel section and back to the river section, almost eliminating the effects of turbulence downstream.

9.1 Scour Protection

As indicated by the river bed survey of October, 1955 and by recent velocity computations, protection of the tunnel against undermining is essential. Filter and rock protection will be provided to prevent undermining of the tunnel and erosion of the river banks.

During high floods the tunnel structure as presently designed will function as a submerged weir and thereby accentuate erosion downstream. This situation will be intensified if the scour protection is displaced or damaged during the brief periods of such floods.

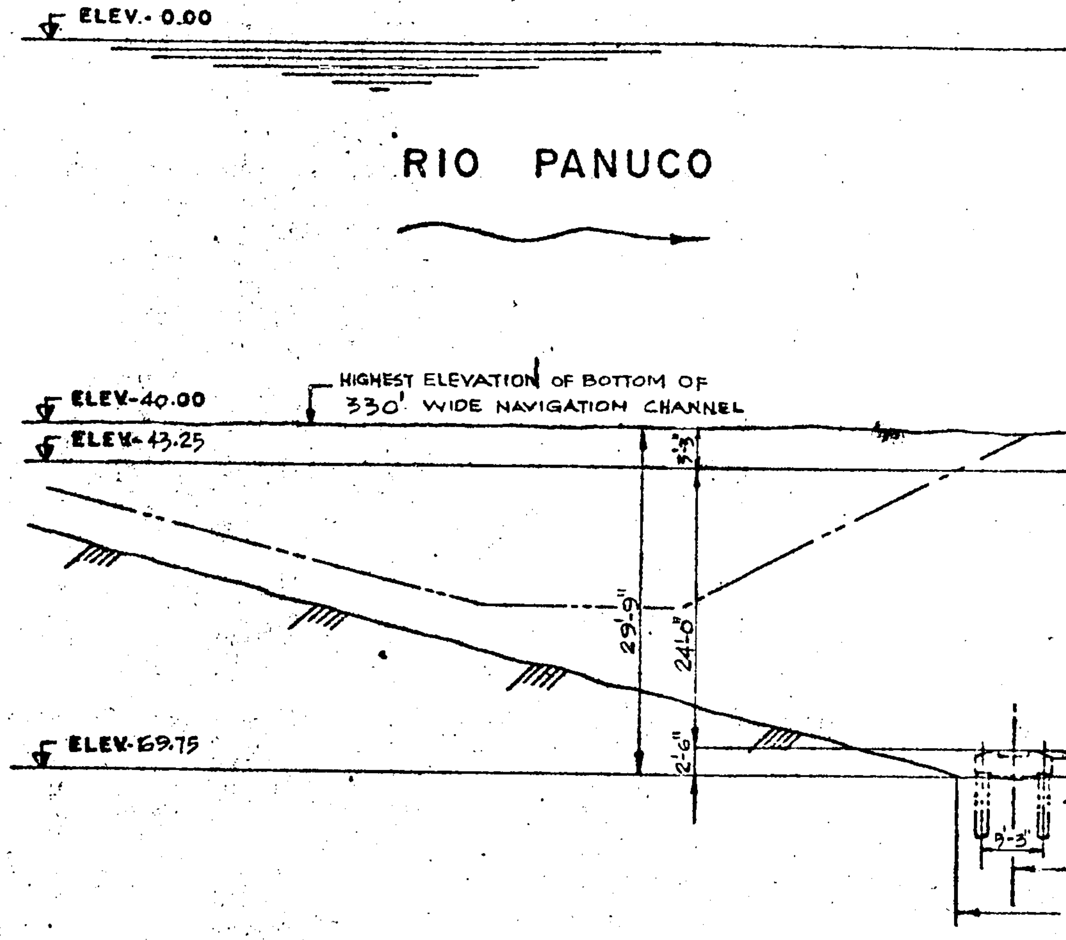
In the design of scour protection, scour depths as disclosed by the survey of October, 1955, were assumed for bottom protection, El. -75.0'; for sidebanks at Tampico, El. -33.0';

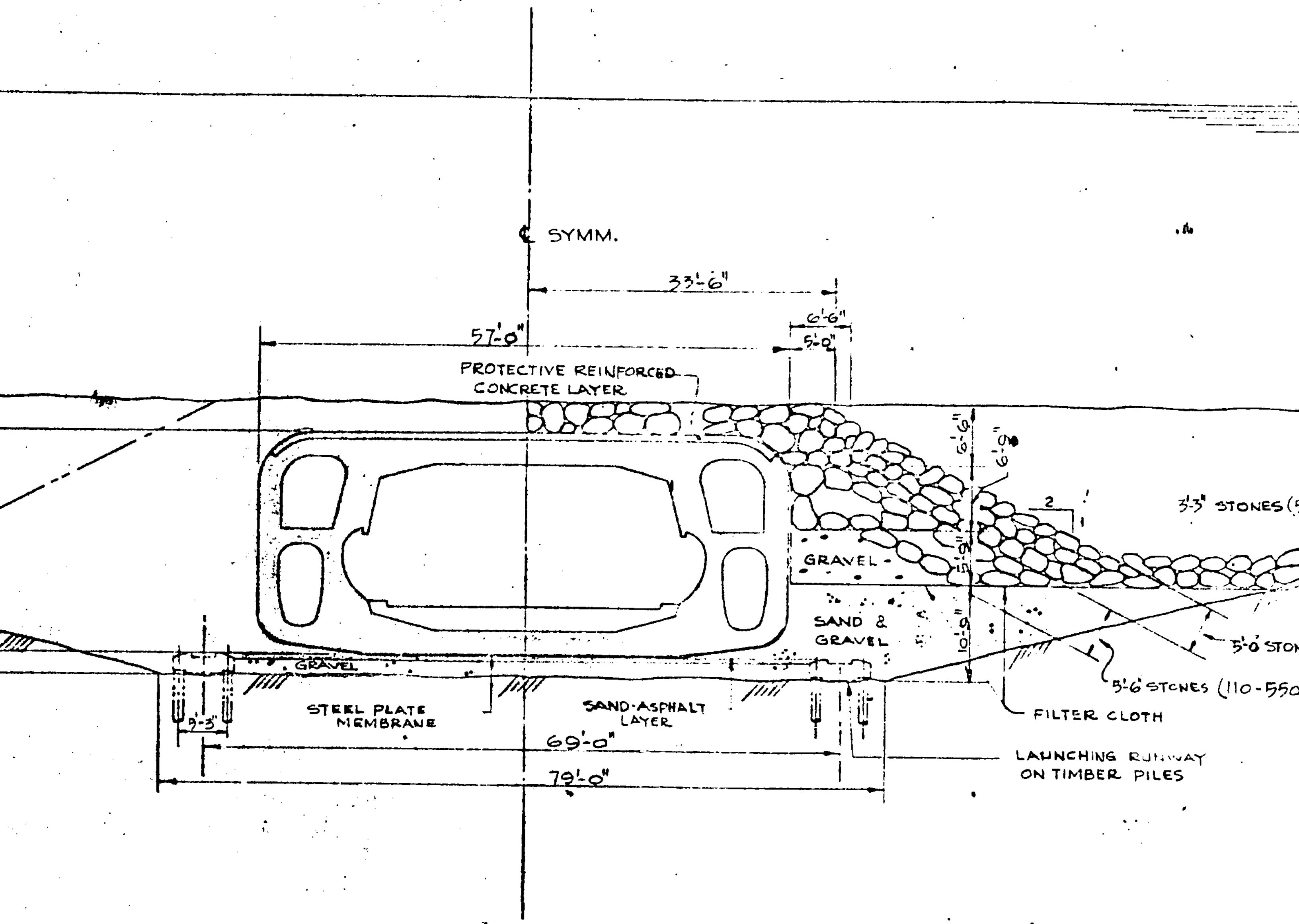
and at Mata Redonda, El. -65.0'. The overall width of the bottom protection, measured normal to the longitudinal centreline of the tunnel, varies.

The stability of the protection and the weights of stone armour for stable slopes are based mainly on the results of hydraulic model investigations concerning the stability of a submerged rock weir, proposed for the St. Clair River, Michigan, U.S.A. Cross sections of the scour protection indicating the various filter layers and sizes of stones for the tunnel portion and river banks are shown in Fig. 9.1 and Fig. 9.2 respectively. An important feature of the proposed design is the use of flexible filter cloth which has been used extensively in Holland for protecting dykes in the Delta Project. The filter cloth not only protects the sand and bed material against erosion but also enables the outstanding layer of rock to accommodate to scour conditions by ensuring its continuity and allowing it to modify its upward slope and to assume a downward slope to suit the prevailing scouring conditions.

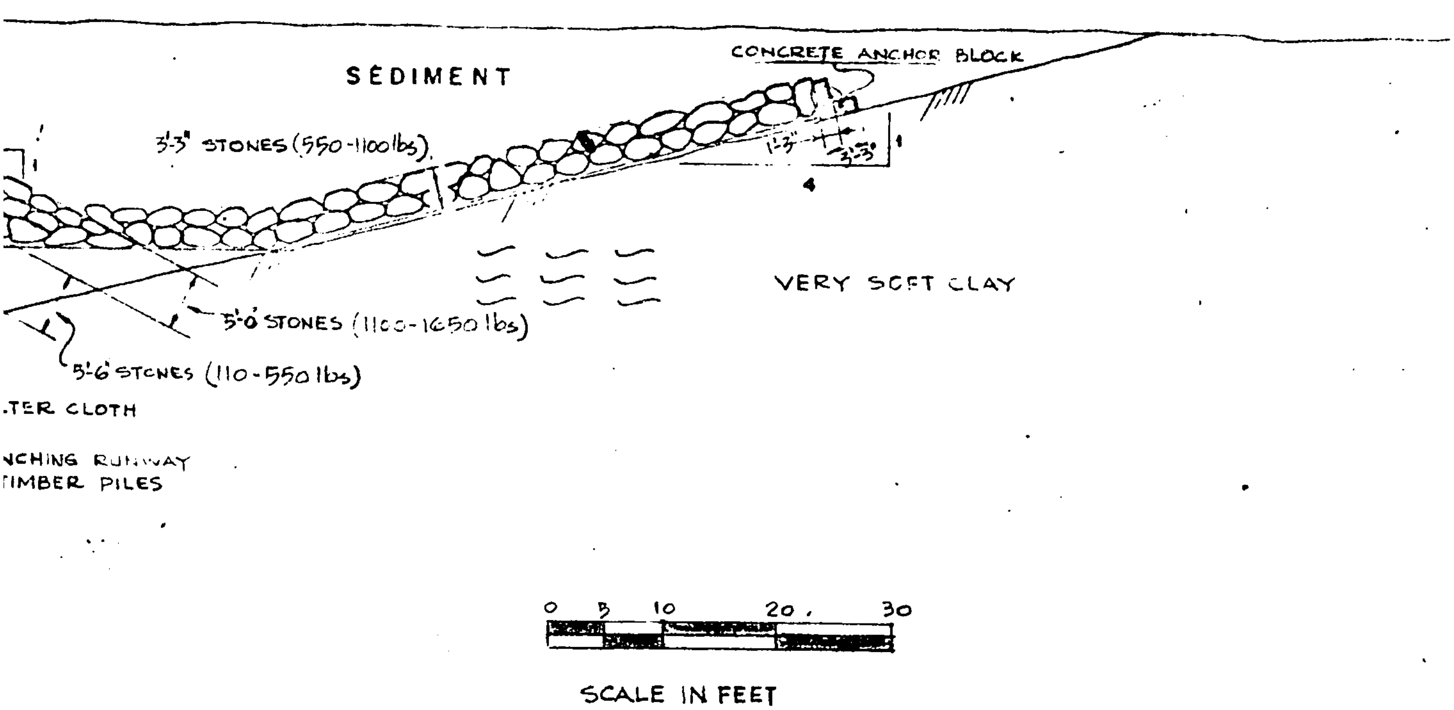
9.2 Transition

The transition from the relatively wide cross section of the river at the tunnel crossing with sidebanks at 1 on 4 slopes, to the narrower natural river section having gentler sidebank slopes, is made by gradual change of width and slope over a distance of 1720' on the Mata Redonda bank, i.e., 930' upstream and 790' downstream of the tunnel crossing, and 1600' on the Tampico bank, i.e., 530' upstream and 1070' downstream.


ELEV.- 0.00
RIO PANUCO
HIGHEST ELEVATION OF BOTTOM OF
330' WIDE NAVIGATION CHANNEL
ELEV-40.00
ELEV-43.25
29'-9"
24'-0"
2'-6"
ELEV-69.75
9'-3"



℄ SYMM.
33'-6"
6'-6"
5'-0"
57'-0"
PROTECTIVE REINFORCED CONCRETE LAYER
6'-6"
6'-9"
2
1
3'-3" STONES
GRAVEL
5'-9"
SAND & GRAVEL
10'-0"
5'-0" STO
5'-6" STONES (110-550
FILTER CLOTH
GRAVEL
STEEL PLATE MEMBRANE
SAND-ASPHALT LAYER
9'-3"
69'-0"
79'-0"
LAUNCHING RUNWAY ON TIMBER PILES



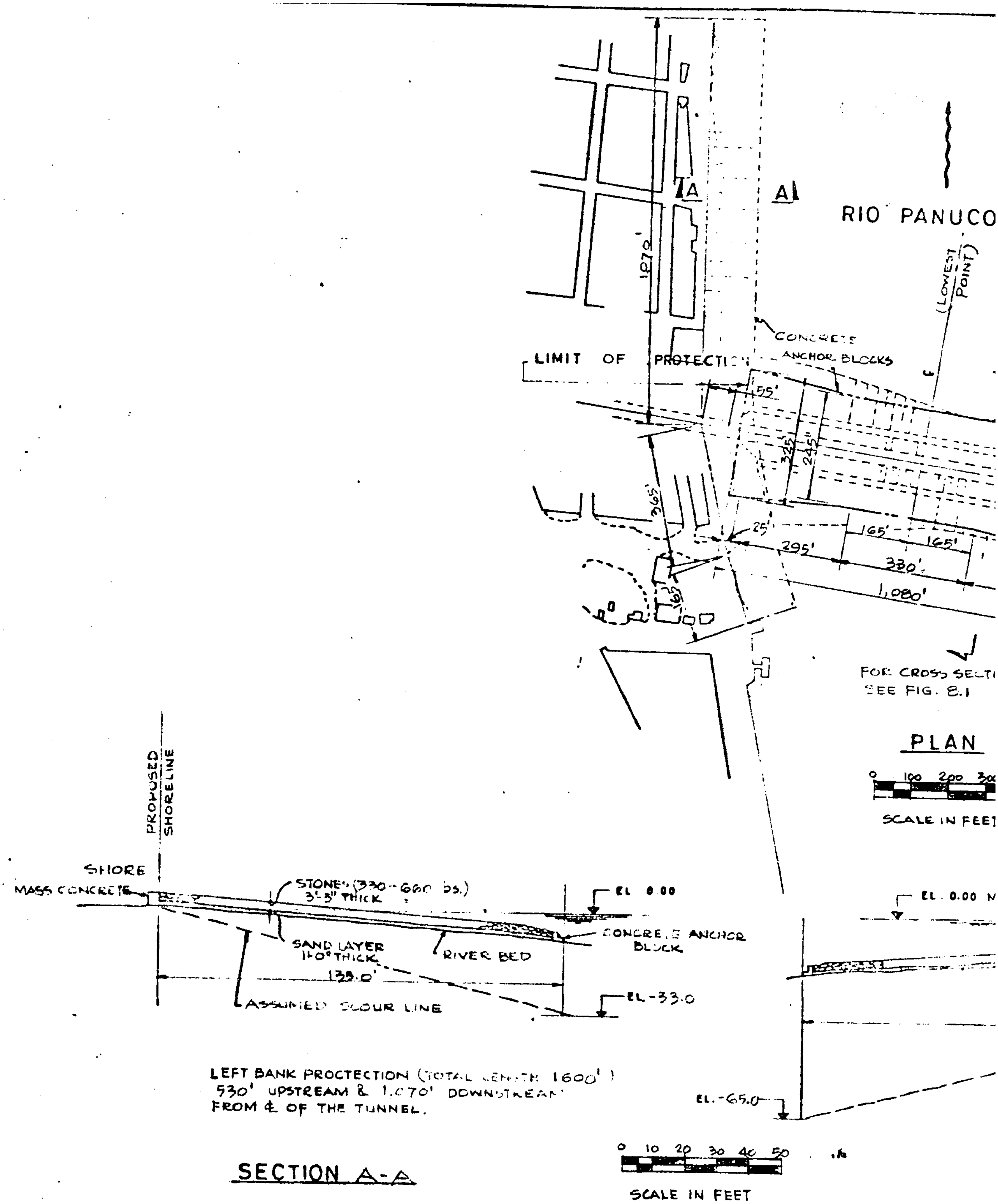
CROSS SECTION OF TRENCH

FINISHED STRUCTURE

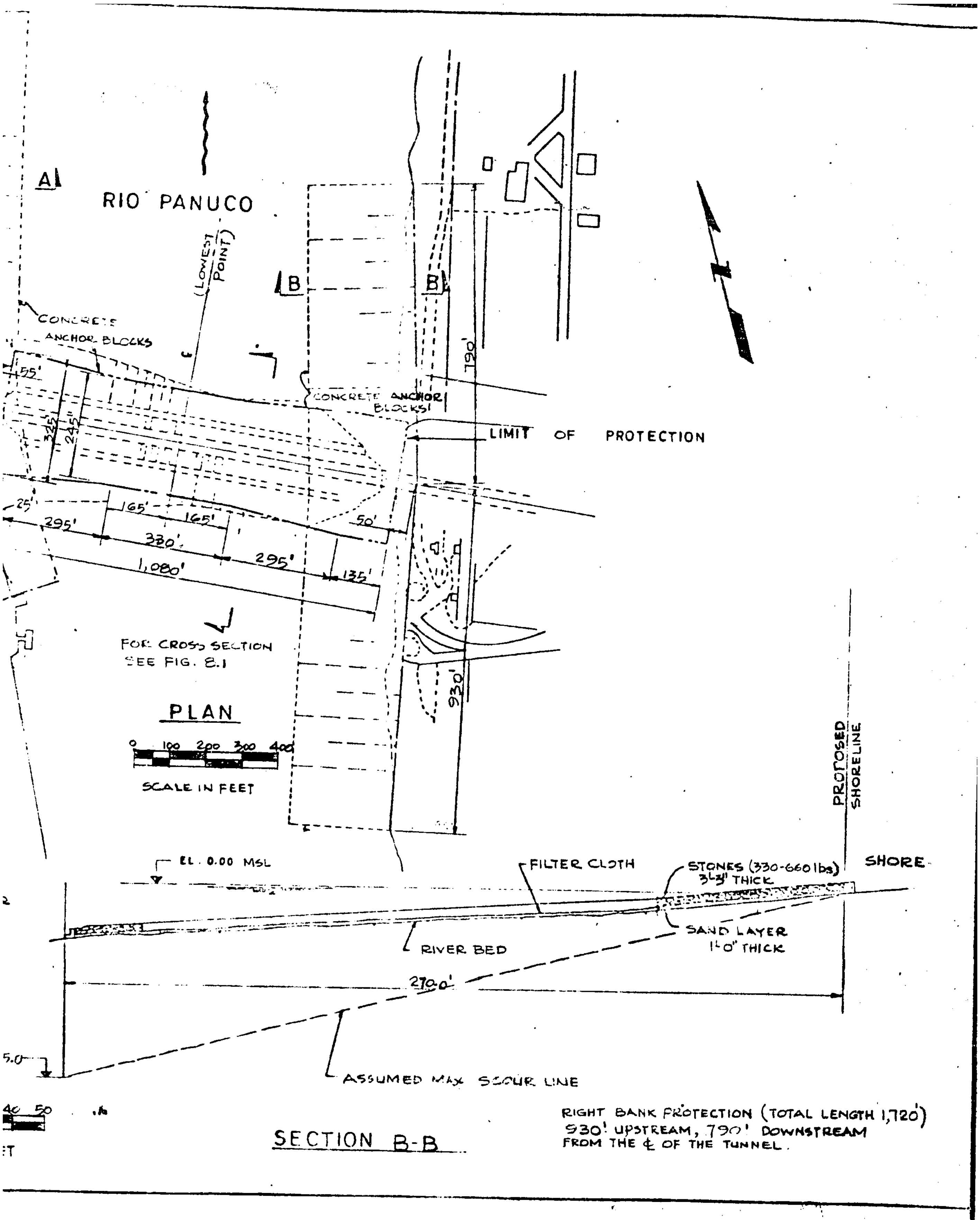
Fig. 9.1

SHOR

MASS CONCRETE



RIO PANUCO
(LOWEST POINT)
A
A
1,070'
LIMIT OF PROTECTION
CONCRETE ANCHOR BLOCKS
55'
325'
245'
365'
25'
295'
165'
165'
330'
1,080'
FOR CROSS SECTI
SEE FIG. 8.1
PLAN
0 100 200 300
SCALE IN FEET
PROPOSED SHORELINE
SHORE
MASS CONCRETE
STONES (330-660 lbs.) 3'-3" THICK
SAND LAYER 1'-0" THICK
RIVER BED
EL 0.00
CONCRETE ANCHOR BLOCK
135.0'
EL -33.0
ASSUMED SCOUR LINE
EL. 0.00
EL. -65.0
LEFT BANK PROCTECTION (TOTAL LENGTH 1600')
530' UPSTREAM & 1,070' DOWNSTREAM
FROM ℄ OF THE TUNNEL.
SECTION A-A
0 10 20 30 40 50
SCALE IN FEET



SCOUR PROTECTION

Fig. 9.2

9.3 Protective Dykes

In conformity with the proposed method of constructing the approach structures and cast-in-place tunnel sections in the dry, temporary protective dykes are required to enclose the construction sites. These dykes are shown in Fig. 11.3 and Fig. 11.4 in Chapter 11. They will be modified and integrated in the finished project as permanent dykes to safeguard the tunnel and approaches against flooding. The crest of both temporary and permanent dykes will be at El. +15.0' allowing 3'-3" as a safety margin above the maximum flood level on record at Tampico.

Because of their proximity to the edge of the river bank, maximum velocities in the order of 10'/sec. may be expected at the outer face of the length of temporary dyke parallel to the river during an extraordinary high flood. Accordingly, the slopes of the portion of the dyke between Stas. 4+920 and 5+230 will be protected with a 3'-3" thick layer of 330-660 lb. stones, similar to that of the river bank protection. The toe of this portion of the dyke will also be protected against scour with a 30' wide blanket of stones. This protection, as the dyke is used only for construction purposes at this stage, is deemed to be adequate. In the remainder of the dyke, i.e., the portion inland of Sta. 5+230, will be protected by a 1'-6" layer of stones weighing at least 110 lbs. This slope protection will be extended up to El. +15.0'.

After the completion of the tunnel, the temporary dykes will be removed and reconstructed from approximately Sta. 5+250 landwards to avoid too much protrusion into the stream of river flow during flood periods.

Because these permanent dykes are located at higher elevations and because of the greater degree of resistance to flow on the shores, the scouring aspects are less important. The maximum flow velocity at the outer face of the portion of dyke parallel to the river for the 1955 flood was in the order of 2'/sec. In this computation, possible backwater effects due to the structure were considered to be negligible.

Protection of the slopes of the portion of permanent dykes landwards of Sta. 5+450 will be identical to that of the typical slope of the temporary dykes. At the portion between Stas. 5+250 and 5+450, the thickness of the rock layer will be increased to 3'-3" and the weight of stones to 110-330 lbs. This layer will be extended for a width of 65' at the toe to provide protection against possible scour due to backwater effects and flow around the tip of the dyke.

CHAPTER 10

TECHNICAL INSTALLATIONS

Any highway tunnel functions as a section of road similar to the roads it connects. It has the same technical requirements for traffic and emergency situations. However, as tunnels are more complex structures than bridges or ordinary stretches of road, it is within reason to expect that tunnels would require more severe design parameters. In the field of technical installations, there are, among many, two essentials which are unique for tunnels in contrast to open bridges and roads. These are ventilation and illumination.

10.1 Ventilation

The purpose of tunnel ventilation is to control the pollution of the interior atmosphere produced by the internal combustion engines of motor vehicles so that the concentration of carbon monoxide (CO) and other noxious or nauseous combustion products remain within acceptable limits under all operating conditions. Operating experience indicates that if the CO is kept to a satisfactory level, the other products of combustion would also be kept within acceptable levels. Under normal conditions, the most dangerous gas to be controlled is CO. The effect of CO on human beings is illustrated in Fig. 10.1.

For short tunnels, natural drafts may be adequate to ventilate the tunnel and no special treatment for ventilation

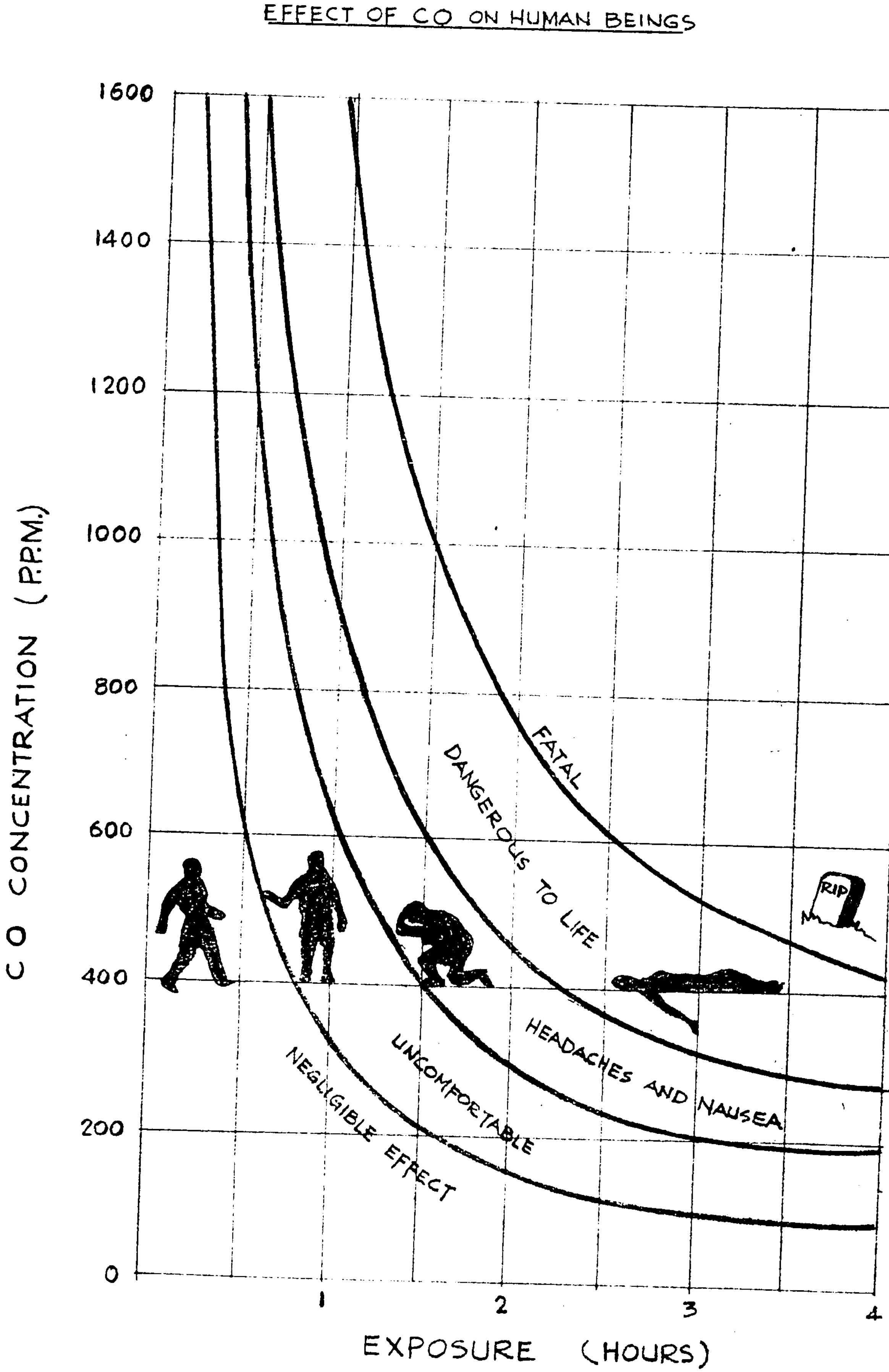

EFFECT OF CO ON HUMAN BEINGS
1600
1400
1200
1000
800
600
400
200
0
CO CONCENTRATION (P.P.M.)
1
2
3
4
EXPOSURE (HOURS)
FATAL
DANGEROUS TO LIFE
HEADACHES AND NAUSEA
UNCOMFORTABLE
NEGLIGIBLE EFFECT
RIP

Fig. 10.1

may be necessary. It is generally agreed, however, that tunnels with a length in excess of 1000' from portal to portal need some form of forced ventilation. In principle, three ventilation systems, with various combinations, are technically feasible:

1. Transversal
2. Semi-transversal
3. Longitudinal

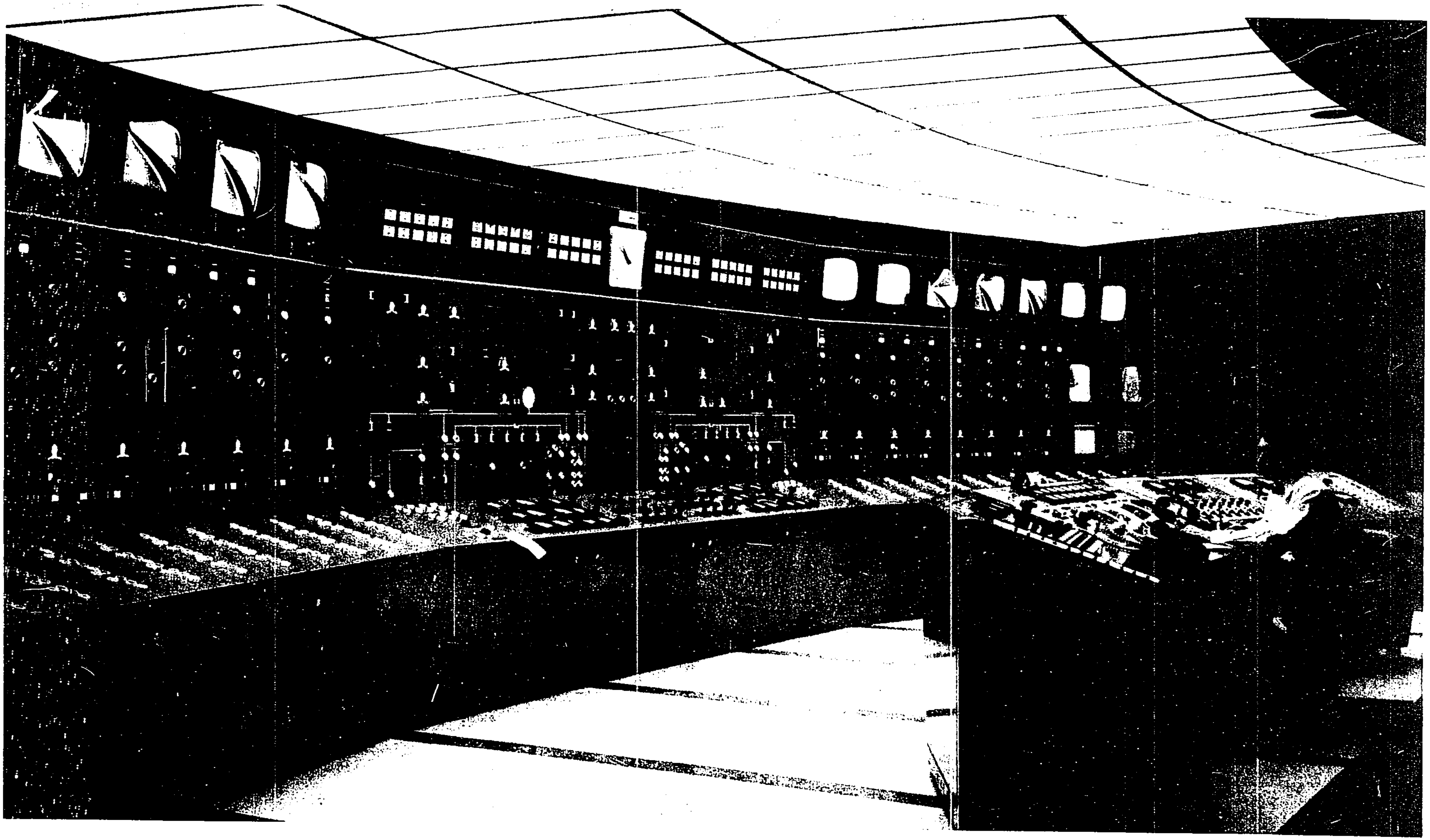
For very long tunnels, the only method of air removal is the transverse system of ventilation, where fresh air is forced through openings in the tunnel walls at regular intervals and the vitiated air exhausted through openings in the opposite wall, or at the top, when fresh air is blown in at the bottom. This system of ventilation was adopted for the 8,690' long Parana-Santa Fe Tunnel presently under construction in Argentina.

For shorter tunnels, up to 1000', the traffic when running at normal speed, can produce the ventilation required, provided there is one-way traffic in each tube. For such tunnels, it is necessary to install artificial ventilation only to deal with special circumstances. In Europe, tunnels in the 1,600'-3,000' range have used this system of ventilation. The Heinenoord, Benelux and Limfjord Tunnels, for example, rely on natural ventilation caused by the traffic movement. Booster fans, mounted on the ceiling, come into operation only when stalled traffic causes an abnormal rise in the CO content of the air in the tunnel.

The first variation from the established principle of transverse ventilation used for shorter tunnels is the semi-transverse ventilation system. Here, fresh air is forced under pressure to ventilation ducts from whence it passes, through regular openings, to the tunnel tubes. This air is then mixed with the air in the tunnel and removed through the portals by the increased pressure caused by the incoming air and the air current created by the traffic. Both of Canada's tunnels, the Deas Island and the Lafontaine, adopted the semi-transverse system of ventilation. In fact, in the case of the Lafontaine Tunnel, it was the first time that this system had been used on a tunnel of this size. A photograph illustrating the master control room in the north ventilation tower of the Lafontaine Tunnel is shown in Fig. 10.2.

Longitudinal ventilation is the simplest of the available systems. This very simple system was used for the Schelde Tunnel near Antwerp. Fresh air is blown directly into the traffic tube as an aid to the ventilation created by the traffic and moves longitudinally along the length of the tunnel up to the point where it is evacuated. In this case, no ducts parallel with the tunnel tube are necessary. However, for tunnels carrying two-way traffic within one traffic tube, longitudinal ventilation is not a logical solution, since the opposing traffic flow will constantly disturb and change the direction of longitudinal air flow through the tunnel.

As previously mentioned, tunnels over 1000' normally requires some kind of forced ventilation. The tunnel under the



THE LOUIS-HIPPOLYTE LAFONTAINE BRIDGE-TUNNEL

CONTROL ROOM,
LOCATED IN NORTH VENTILATION TOWER.
THE WHOLE TUNNEL IS CONTROLLED FROM THIS ROOM

Fig.10.2

PER HALL ASSOCIATES

Rio Panuco will need artificial ventilation since it has a portal to portal distance of 1,945'. The requirement for a forced ventilation system necessitates a set of design parameters upon which the design of the system must be based. Of primary importance is the carbon monoxide content which the ventilation system must reduce. These design criteria are listed in Appendix C.

In addition to its normal function, the ventilation system should also be designed to be of maximum assistance in the event of fire or other emergency. Experience in the operation of tunnels indicates that the chances of a fire occurring are extremely remote. Nevertheless it is deemed advisable to provide a means for evacuating smoke or other noxious fumes from the traffic tube and to supply fresh air, to assist people in leaving the tunnel and to permit fire-fighting equipment access to the fire.

In a two-lane tunnel such as that under consideration, in which opposing traffic flows generate opposing pressure to the air flow, it becomes desirable to install a completely flexible ventilation system which can be effectively controlled to satisfy changing traffic densities and the emergency situations which occur.

The recommended system has a large capacity longitudinal duct in the lower half of the wall on each side of the tunnel with adjustable openings to the traffic tube at intervals of 20'. Each main duct is bulkheaded at mid-length. Equipped with 4 two-speed reversible fans, one at each end of each duct, the system may be controlled to function as shown in Fig. 10.3.

For normal traffic, the transversal method of operation is reccmmended, with fresh air supplied to each traffic lane on the up-grade, i.e., at diagonally opposite ends of the tunnel, where the exhaust fumes from acceleration and increased power are the greatest. The vitiated air will be exhausted, in part, form each traffic lane on the down-grade, opposite the supply ducts, and in part, through the tunnel portals.

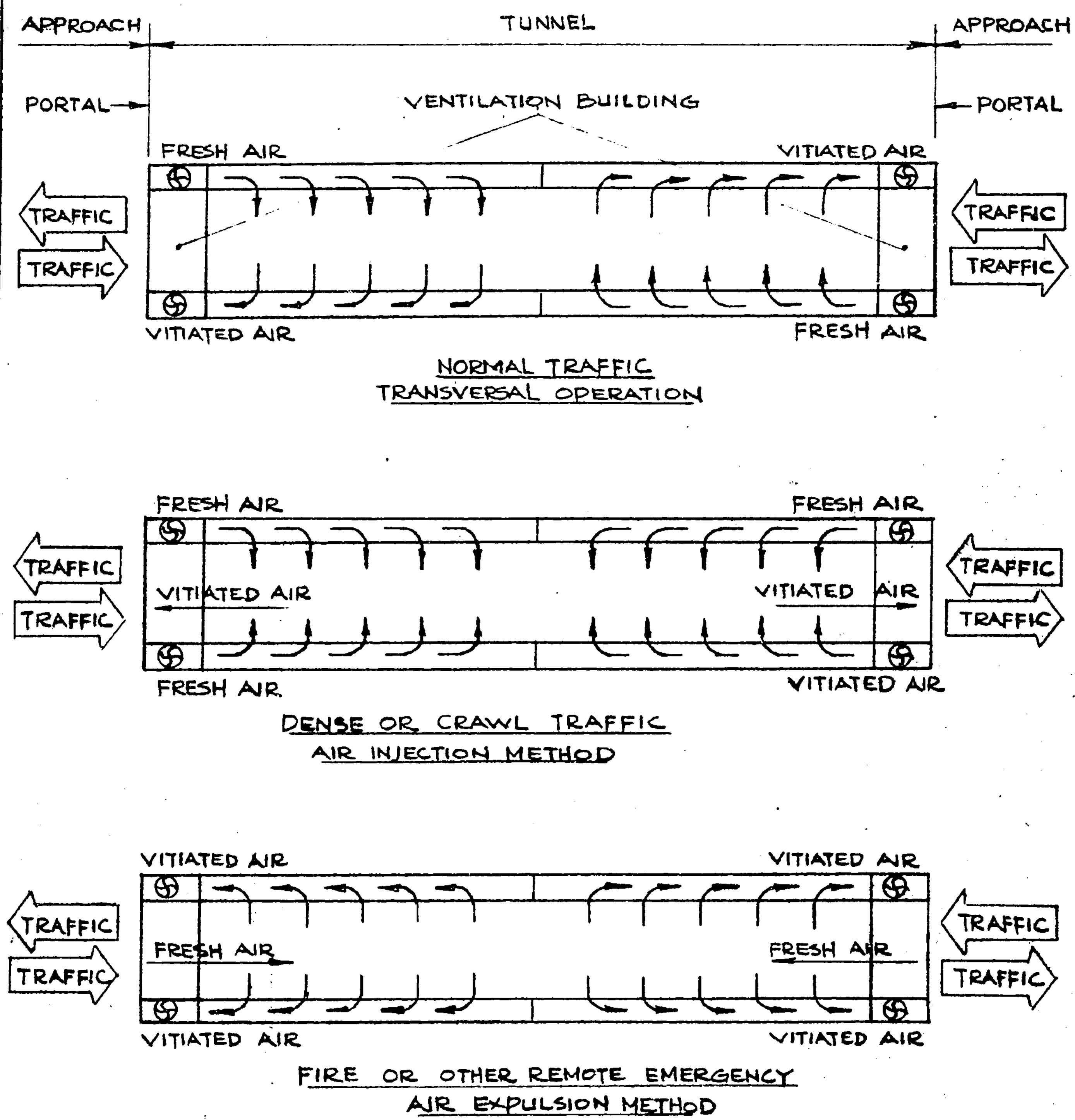
For dense, slow-moving or stalled traffic, the air injection method of operation is recommended whereby fresh air will be supplied from each side throughout the length of the tunnel and vitiated air exhausted through the portals.

In the case of fire or other remote emergency, the fans will be reversed to exhaust smoke and noxious fumes from each side throughout the length of the tunnel, with fresh air being drawn in through the portals.

A more detailed design of the ventilation system discussed above is presented in Appendix C.

10.2 Illumination

It is well known that motorists entering a tunnel from bright daylight will be completely blinded for the first few seconds even if the tunnel is artificially illuminated. This may cause accidents and various measures have been taken to



VENTILATION SYSTEMS

Fig. 10.3

counteract it. The objective of tunnel lighting, therefore, is to provide visibility and comfortable surroundings for the drivers, under both daytime and nighttime conditions. The entrance to the tunnel on bright days requires particularly careful treatment for a satisfactory effect.

The lighting system must be dependable under all emergencies, including a failure of the power supply, in order to avoid danger of accidents which might accompany a sudden blackout.

The eye normally requires a minimum of 10 seconds to adapt from outside brightness to tunnel interior and a 15-second period for safety and comfort. For a design speed of 45 mph, this corresponds to an entrance zone of 950'. This zone can be partly an open section with louvres to cut out direct sunlight and partly a tunnel section with extra lighting near the entrance.

The division of the total adaptation zone into louvred and artificially lit sections is partly an economics problem. Experience indicates that a louvred length of 330' to the tunnel portal plus a tapered brightness in the first 620' of the tunnel gives good results. The sunscreen louvres are arranged so that they provide 100% shade on the road for all positions of the sun, otherwise the driver would experience discomfort in driving through the black and bright patches on the road surface.

Driving from outside direct sunlight to the shade under the screen is comparable to driving from an open field with brightness say 80,000 lux, into a shaded spot under a tree, where the brightness may be 18,000 lux. This kind of a transition is rather sudden but acceptable and occurs in practice on many occasions. In fact, the human eye can withstand a reduction ratio of 10 to 1 between adjacent lighting levels without inconveniences. The effect is reduced by employing sun-screens with a high reflectance at the beginning, then gradually reducing the reflectance of screens and walls. A closer spacing of the slats in the screens can also be used. The brightness is gradually reduced towards the tunnel portal.

For the interior part of the tunnel, on clear days, a lighting intensity at the roadway surface of about 150 lux is recommended. At night, this intensity should be reduced to approximately 50 lux to avoid the "blackhole" effect when emerging from the tunnel onto the lower illumination level of the open streets.

In summary, therefore, motorists will drive from an outside brightness of 80,000 lux to a louvred stretch of 8,000 lux. In the 330' louvred length, the brightness tapers off from 8,000 lux to about 1,500 lux. After passing through the tunnel portal, the brightness is gradually reduced over a length of 620' to 150 lux. The foregoing treatment of lighting intensities for the tunnel entrance applies equally well for the tunnel exit.

It has been found desirable to have a continuous source of light throughout a tunnel; otherwise, drivers experience a flicker effect as they go past individual fixtures. The most suitable type of light from this point of view is the fluorescent lamp which can be mounted in a continuous row. It is proposed that two rows, one above the centreline of each traffic lane, be installed at the tunnel roof; and that one row be installed on each of the side walls to highlight the lower concave section.

The fixtures recommended comprises clear pyrex glass tubes 12' long with cast bronze mountings and housings for the transformers at each end. Such units are adjustable as to the degree of illumination output and provide the best possible resistance to corrosion and high humidity.

The switching of lighting should be designed to permit varying intensities of lighting inside the tunnel as required by the level of outside illumination. This can be achieved by turning off a portion of the system of preferably, by dimming.

In the event of a blackout, a number of battery powered lights will switch on automatically in the tunnel proper as well as in the various service areas. The batteries will provide lighting for about 20 minutes during which time the emergency diesel generators will be brought into service. The design of the lighting system is given in Appendix D in more detail.

10.3 Traffic Signals, Supervision and Instrumentation

<u>Traffic Signals</u> - The provision of traffic lights is a necessity to achieve complete control of traffic going through the tunnel. Such installation will enable possible guidance of all traffic to go in one direction, for example, in rush hours or in evacuation of the city population in times of emergency. Therefore, overhead traffic signals operated from the master control room will be provided for each lane.

<u>Television Cameras</u> - Closed circuit television with cameras installed at about 500' intervals will permit continuous viewing of traffic conditions in the tunnel from the master control room. Such monitoring has proven to be the fastest and most economical means of quickly identifying problems such as accidents and inadequate visibility.

<u>Carbon Monoxide Detectors</u> - As an overdose of CO can be fatal, it is important that some kind of detection system to measure the CO level be installed such that it is possible to continuously monitor the air in the various parts of the tunnel. With such an installation, an increase in CO concentration will automatically increase the ventilation and will activate an alarm circuit when the concentration approaches the permissible maximum.

<u>Opacity Meters</u> - To determine the visibility within the tunnel, opacity meters should be considered although existing meters have proved difficult to maintain without the services of a qualified technician.

Traffic Counters - To forecast future expansion of traffic facilities and to obtain accurate records of traffic volumes carried by the tunnel, automatic traffic counters will be installed for each lane of traffic, registering in the main control room.

10.4 Pumping Plant

It is obvious that all tunnel installations must include a pumping plant for the removal of storm water, drainage water from the approaches, possible leakage into the tunnel and water used for cleaning the walls of the tunnel.

In the present tunnel, a large catch basin extending the full width of the roadway, with its top at about El. -20', is to be installed at each portal. It is intended that this basin should collect roadway drainage from a distance of about 680' where the 5% grade will attain El. +15.0'. Beyond this limit, drainage will be the responsibility of the municipalities.

The capacity of each catch basin will be sufficient to store one hour runoff from the drainage areas contained within the approach structure and protective dykes, computed for the heaviest rainfall likely to be experienced. The maximum rainfall intensity for a storm of one hour duration has been estimated at 4" per hour.

Catch basins will also be installed at intervals throughout the length of the tunnel to drain rainwater brought in by vehicles and washwater from tunnel cleaning. These catch basins will drain to a sump below the lowest elevation of the tunnel. Automatic pumps will maintain the water levels in the large catch basins and the sump at predetermined elevations.

Fig. 4.1 shows that the drainage area discharging water onto the tunnel approach on the Mata Redonda side is much larger than that on the Tampico side because excavation for the launchway extends well beyond the point where the road level reaches El. +15.0'. It has been assumed, however, that the tunnel drainage facilities need not accommodate runoff from the approaches beyond El. +15.0' level. This means that a storm water catch basin of substantial size will be built by others near the point where the road elevation on the Mata Redonda side reaches El. +15.0'. The remaining flow to be handled by the tunnel drainage facilities will then be approximately equal to that computed for the Tampico side, requiring the installation of storage basins and pumps of the same sizes and in similar locations. The drainage water in these catch basins will be pumped into the river.

The maximum flow to the mid-tunnel sump pit has been assumed to be equal to the combined capacity of the two $1\frac{1}{2}$" service pipelines and two $2\frac{1}{2}$" municipal hose-lines. As the sump at the mid-point of the tunnel cannot easily be enlarged beyond the periphery of the precast tunnel element, it is proposed to keep the sump small and provide a pumping capacity about 35% larger than the expected maximum flow. The waste water is to be pumped to one of the catch basins at the tunnel portals from whence it will be pumped into the river.

10.5 Fire Stations

Only rarely do fires occur in tunnels. However, fire fighting equipment should be installed at regular intervals regardless, in case circumstances dictate its use. The Tampico Tunnel will have fire stations located in the traffic tube at 275' intervals. Each station will provide a fire hose, hand dry-chemical extinguishers, sand and an emergency telephone. A $6\frac{1}{2}$" waterline will supply the fire hose stations.

Equipment will be arranged that the use of the fire hose or the extinguisher will automatically transmit an alarm to the control room and to the Municipal Fire Department. However, should there not be opportunity to utilize the above equipment for combatting fire, the tunnel would also be equipped with automatic fire alarms which would be activated by the presence of excessive heat.

10.6 Emergency Services

As the tunnel is part of a highway system, the usual highway emergency services of police and ambulance should be available, with special services for fire. Special provision is also required for towing disabled vehicles clear of the tunnel section.

10.7 Acoustics

As modern day driving demands more comfortable surroundings, the need for noise control is of importance. Observations have indicated that pedestrians and maintenance crews experience considerable discomfort whenever the noise level, due to continuous traffic and reverberation, is high. To diminish the noise, various methods have been used. One method is to install special resonators on the tunnel ceiling. These resonators fraction the sound and eliminate reverberation, resulting in a less agitating type of noise. Another method is to provide some form of acoustic lining to the ceiling, if the standing sound waves are reflected up to the ceiling, to absorb as much traffic noise as possible and reduce the reverberation. The echo time, which in an unfaced tunnel is 7-10 seconds, is reduced to around 2 seconds. This decrease gives the driver a better impression of the distance between his vehicle and the other traffic.

In the Tampico Tunnel, the concave portion of the walls as well as the ceiling will be treated with sound absorbent material to produce less noisy surroundings for the drivers and maintenance crews.

10.8 Electric Power Supply

Highway tunnels should preferrably have two independent main sources of electrical power as well as emergency diesel generators which can start up automatically in the event of complete power failure. In addition, storage batteries should come into operation immediately upon a primary power failure and function to start the diesel generators and to supply energy for emergency direct-current lighting, traffic signals and emergency controls for about 20 minutes.

For the present tunnel, the power for the tunnel operation is to be supplied from the Tampico side. For this reason, it will be convenient to locate the primary substation and all major electrical equipment and controls in the north ventilation building located on the Tampico shore.

Preliminary studies indicate the probability that a transformer of 2,000 KVA capacity will satisfy the peak power loads for lighting the tunnel, operation buildings, etc.; for the operation of traffic lights, closed circuit television, and other services; for the operation of ventilation fans; and for the operation of fire pumps and storm-water drainage pumps.

The stand-by diesel generators, each rated 200 KW, will provide energy for the operation of storm-water pumps at full capacity and for lighting, ventilation, and the operation of other services, at approximately 50% of normal capacity.

The arrangement of substation and electrical equipment in the principal ventilation building at the Tampico portal is illustrated in Fig. C.1.

CHAPTER 11

FABRICATION AND CONSTRUCTION METHODS

General - In the construction of a concrete submerged tunnel crossing, the cost of fabricating, casting, sinking and founding of the immersed tunnel elements can be divided into approximately 40-50% in the fabrication of the structural cross section and 50-60% in the method of casting (for example, within a casting basin or on slipways), placing and foundation preparation. As shown in Chapter 5, the approximate saving in cost in using the most economical cross section is relatively small. However, it seems possible that substantial economy can be achieved by devising more efficient and economical casting, placing and founding techniques.

To date, the essentials of virtually all tunnels built by the immersed-tube method are similar. The construction procedures may be summarized as follows:

(1) Excavating a trench underwater to an accurate profile.

(2) Constructing the tunnel elements with bulkheads at each end, in a casting basin, or behind cofferdams or on slipways.

(3) Flooding the casting basin or cofferdams or launching the elements from the slipways and then towing the sections either afloat or submerged, to the tunnel site and sinking in the correct location by means of sinking rigs.

(4) Joining the sunken section to the previous one with the aid of divers.

(5) Removing the bulkheads and completing the sealing of the joints between the sections.

The above procedure so far has been followed by most tunnel engineers, partly because the physical site conditions may not permit more economical methods and partly because any new techniques devised always have the uncertainty that they may not work as well as envisaged.

Tampico Tunnel - To achieve a better insight into the choice of fabrication and construction methods, two entirely different approaches were studied in great detail to obtain comparisons between cost and construction time. A choice was then made followed by further detailed analysis of the selected scheme. The schemes investigated were the conventional dry dock (casting basin) and the launchway methods of fabrication and construction.

The proposed tunnel crossing comprises a number of components, all to be constructed in or near their final locations within the project site. The two-lane roadway included within the project limits stretches over a total length of about 4,000' between points on the two shores at which the approach ramps of 5% gradient reach El. +15.0'. On the Mata Redonda side, however, a length of 820' extends past this point of intersection to provide space required for the launchway method of fabrication and construction. For the dry dock scheme, this length of roadway will be built as an open roadway.

11.1 Dry Dock Scheme

Generally, it is advantageous to build as much of the tunnel as possible in the dry, which requires the cofferdamming and dewatering in stages of the river bed along the centreline of the tunnel. This procedure is costly as the river under consideration does not avail itself favourably to this approach due to the poor soil conditions in the river bottom to support dykes for dry construction. Consequently, nearly all construction activity will be restricted to the shores.

The centre 5 sections of the tunnel each 328' long will be prefabricated within a dry dock. This length, chosen to suit the in-situ tunnel section of 155' on each shore to maintain the portal to portal length of 1945', prevents the protective dykes from protruding into the river. At the same time, the 328' length is very close to the optimal length as a shorter length would increase the number of elements, thus the cost and more importantly, the time in placing operations. A longer length would not be practicable for a sinking rig to handle or would require one with bigger capacity because of the larger forces acting on the element. Maintaining the submerged weight of the element to around 3½% negative buoyancy for stability in "floating-equipment" placing, an extra 1.5 tons per increased foot of tunnel is added to the supporting cables of the sinking rig. Upon completion of the 5 elements, the dry dock will be flooded and the elements taken out and placed in their position.

The construction work for the immersed sections of the tunnel may be visualized as being divided into two logical phases. The building of the north approach, the installation of scour and river bank protections, the finishing work on the tunnel, etc., will be discussed under the subsection "Launchway Scheme", as it is the scheme finally chosen.

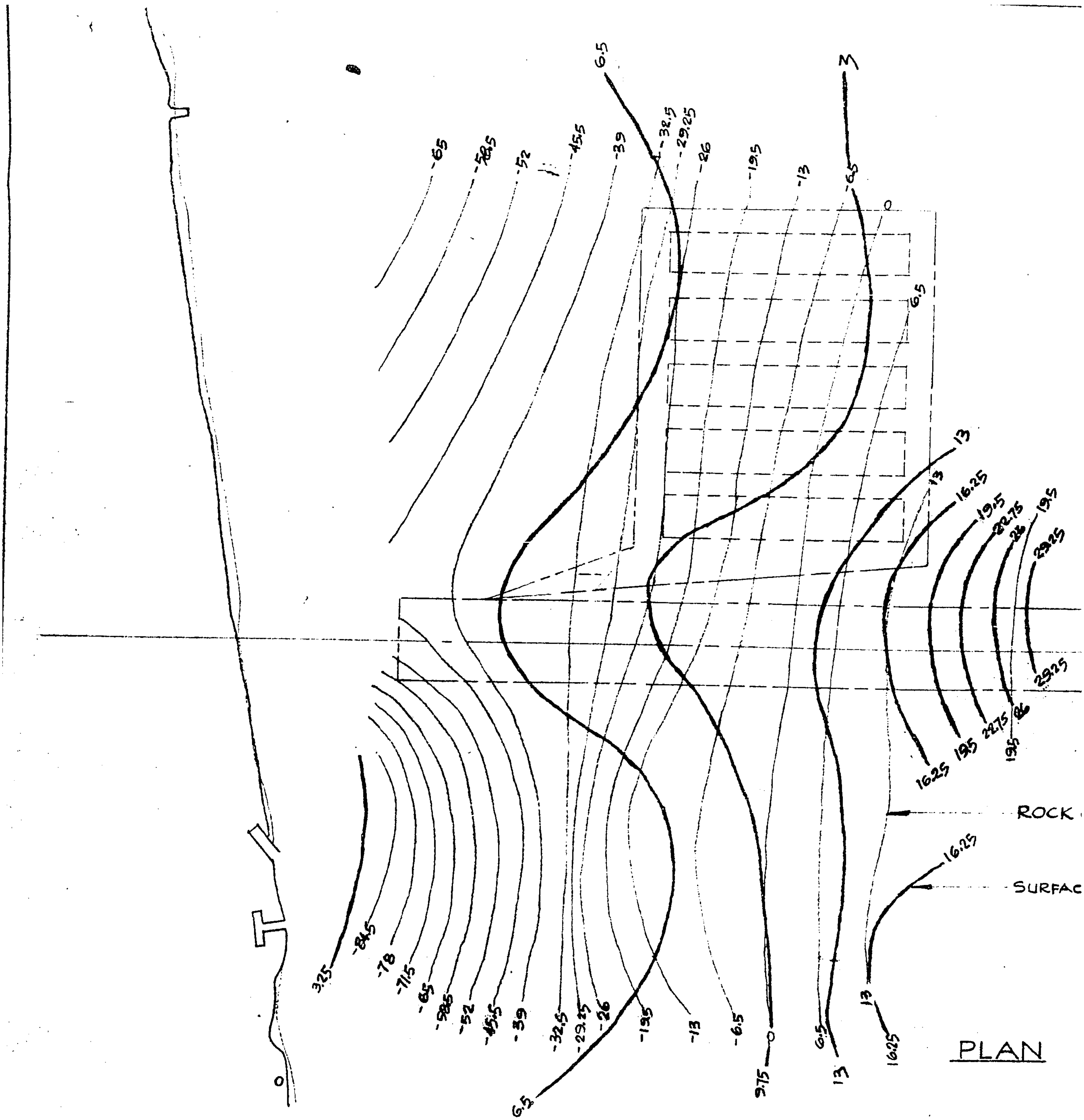
11.1.1 Phase I -- Dry Dock Construction

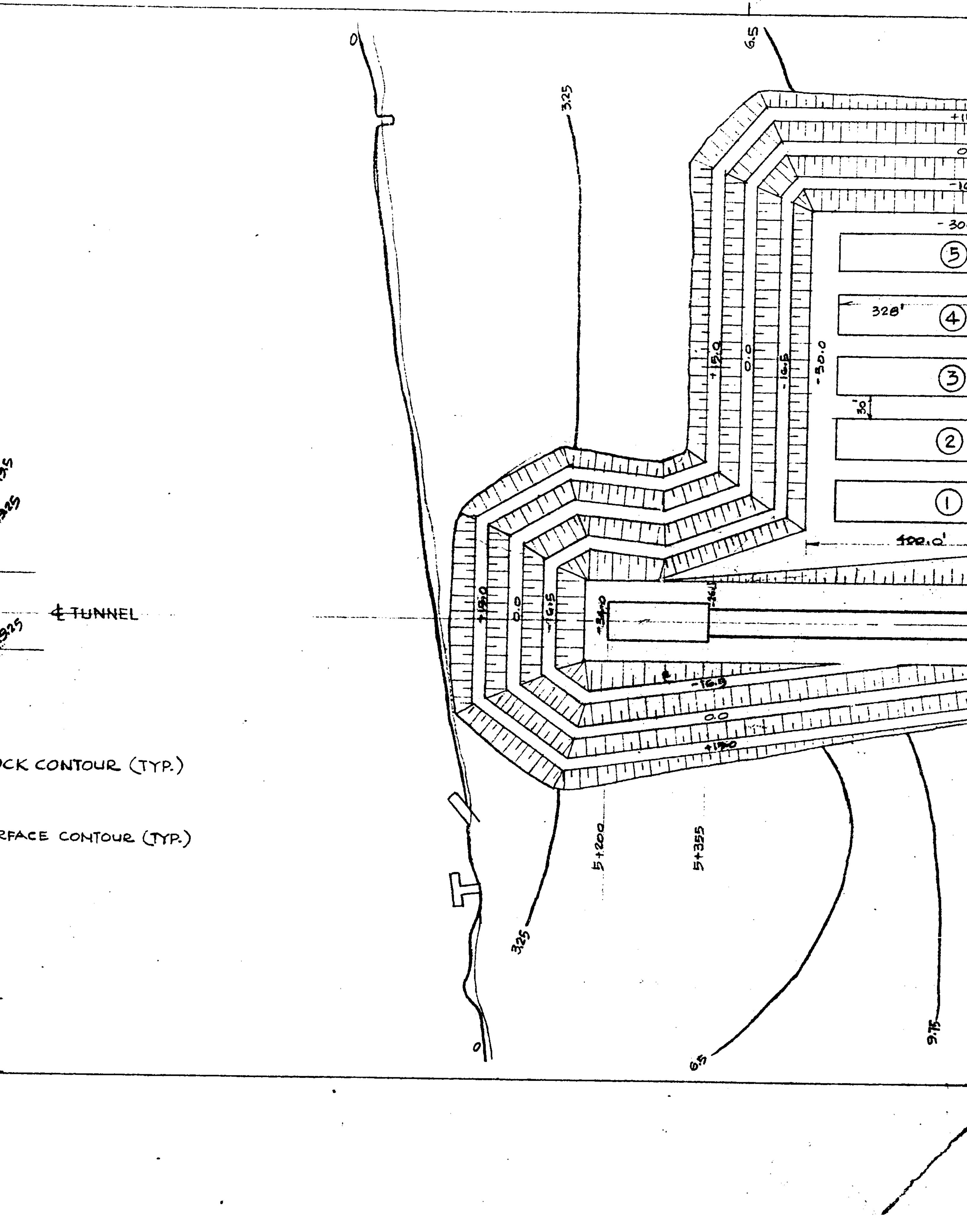
Dry Dock - The dry dock is to be located at Mata Redonda where the rock surface is not too deep below the natural ground.

The dry dock will accommodate the construction of the 5 prefabricated tunnel elements, each roughly 57' by 328' in plan, and the south approach. The exact location of the dry dock is governed by soil conditions as the bottom must be capable of supporting the freshly poured concrete without differential settlement. Consequently, the dry dock is located as shown in Fig. 11.1 with the bottom underlain by the soft sandstone formation.

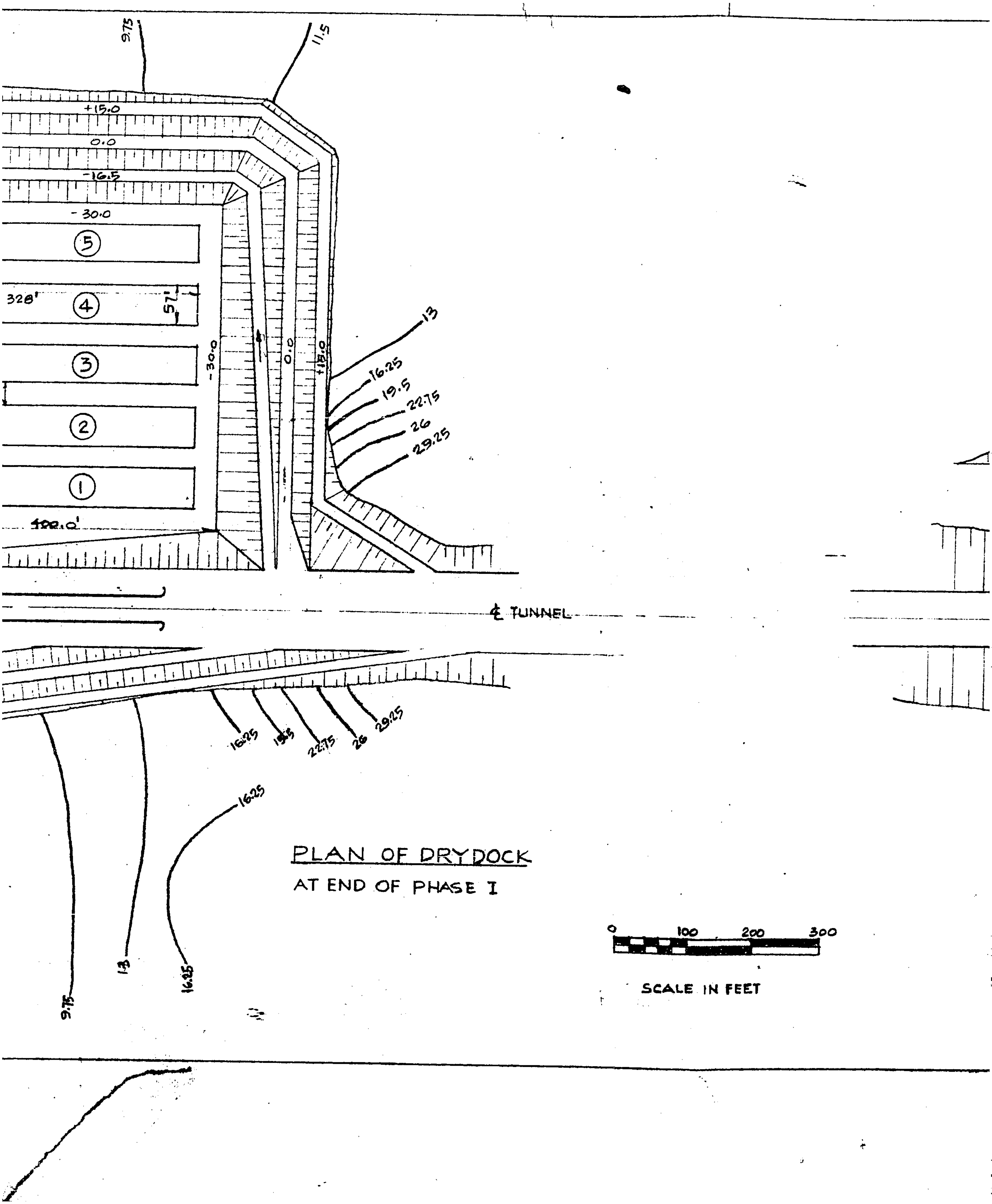
The part which accommodates the precast units will be excavated in the dry to an elevation which will enable all elements to be towed out while completely submerged. Therefore, the bottom of this part will be finished at El. -30.0'. The remaining part, which accommodates the construction of the south approach, will be excavated in the dry along a 5% grade enabling the approach structure to be built directly in place.

The dry dock bottom will be finished to the exact grade by a 1'-6" layer of crushed stone for buoyancy forces to act on


ROCK
SURFAC
PLAN

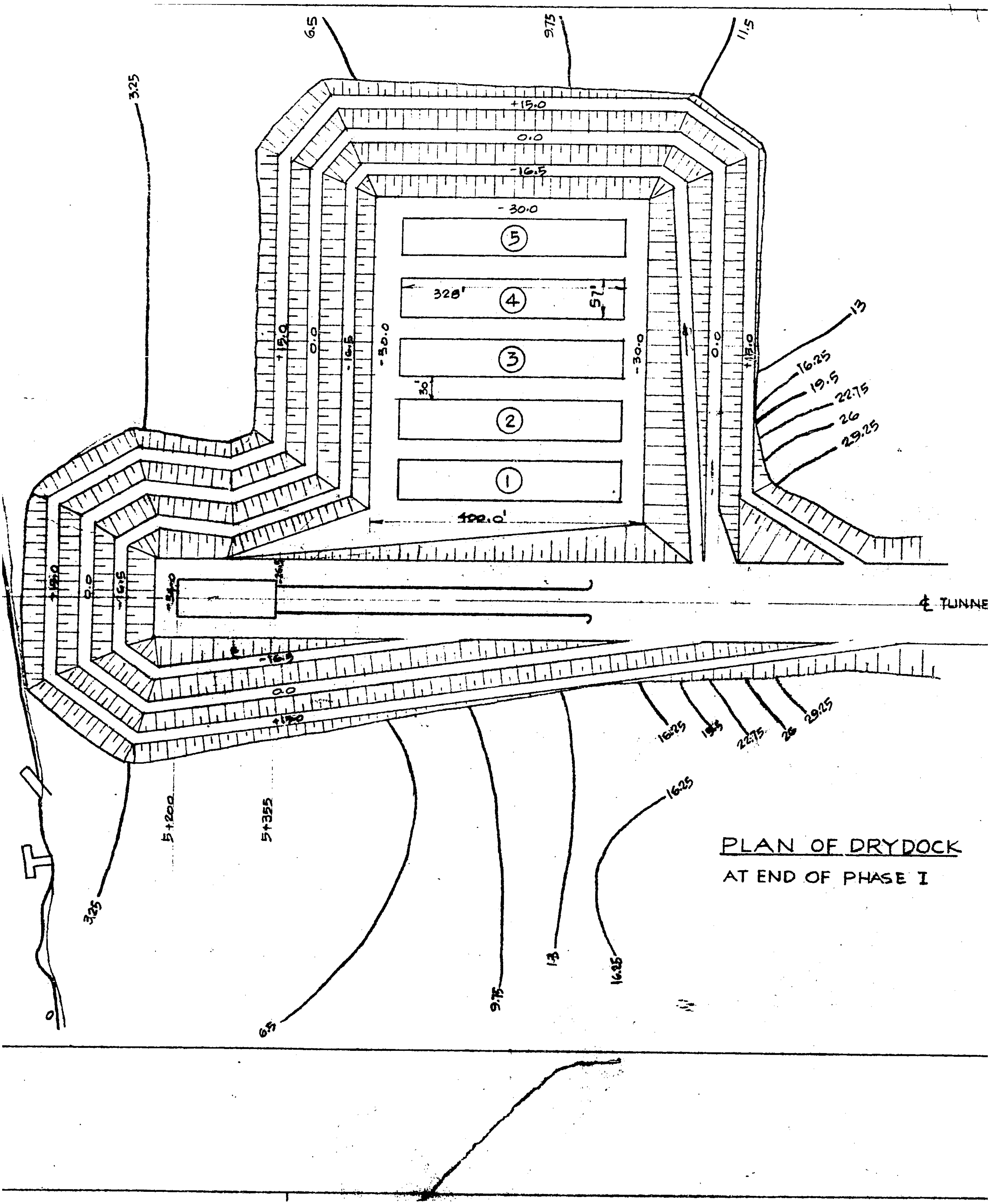


℄ TUNNEL
CK CONTOUR (TYP.)
RFACE CONTOUR (TYP.)
0
3.25
6.5
9.75
+15.0
0.0
-16.5
-30.0
328'
30'
400.0'
5+200
5+355
1
2
3
4
5



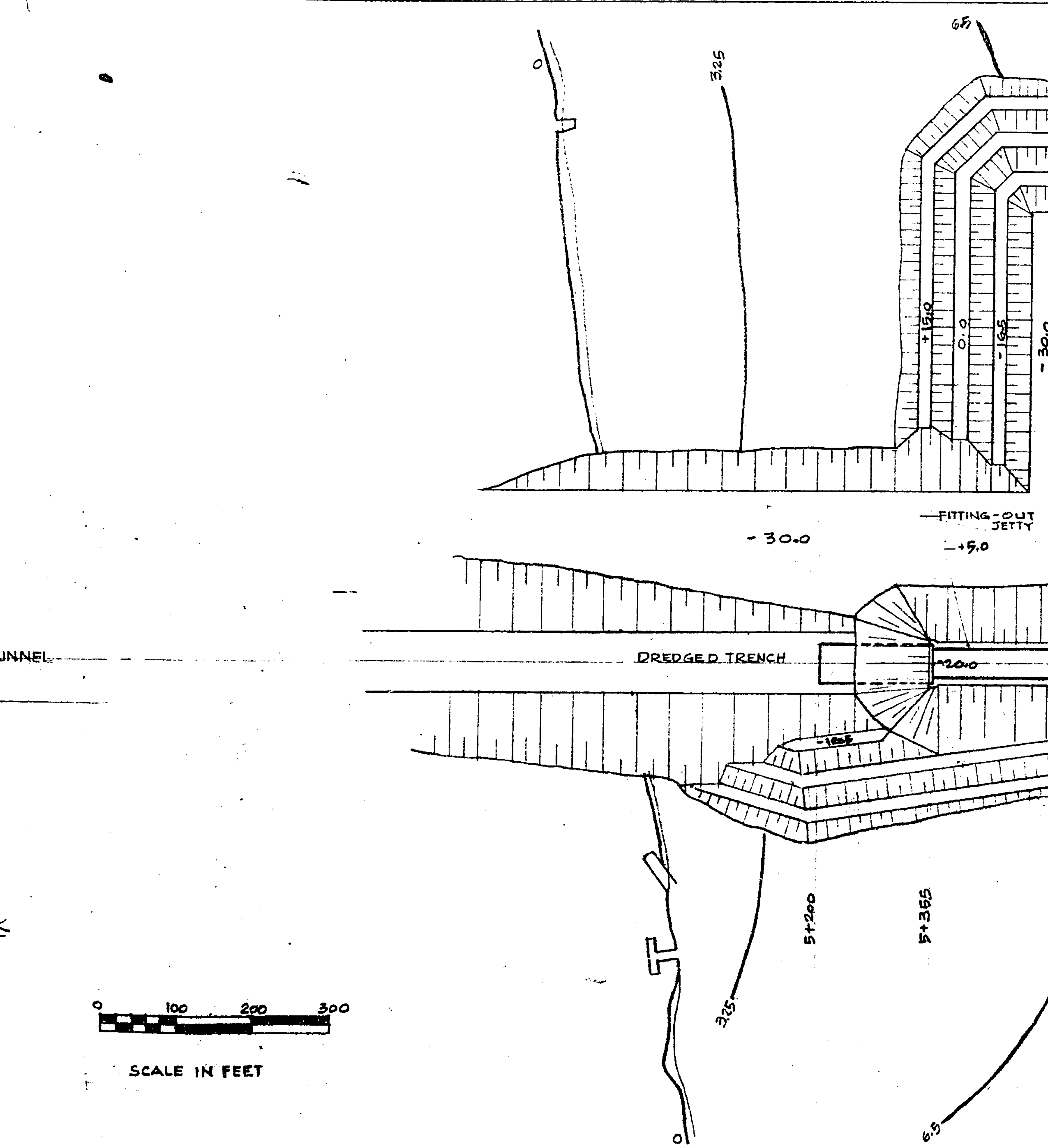
PLAN OF DRYDOCK

AT END OF PHASE I

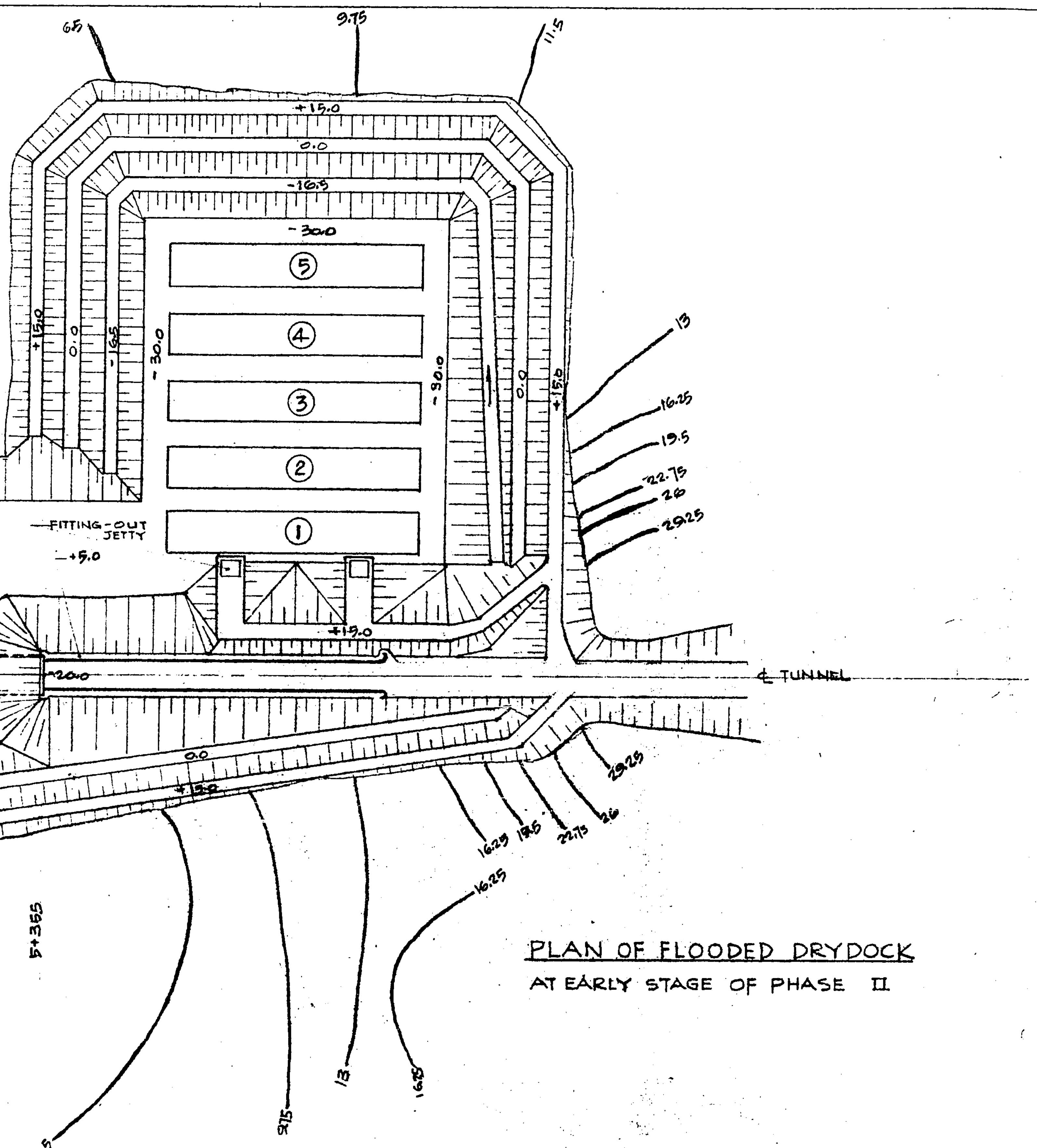


PLAN OF DRYDOCK

AT END OF PHASE I



0
325
65
+15.0
0.0
-16.5
FITTING-OUT
JETTY
-30.0
+5.0
UNNEL
DREDGED TRENCH
-20.0
-16.5
5+200
5+355
325
6.5
0
0
100
200
300
SCALE IN FEET



PLAN OF FLOODED DRYDOCK

AT EARLY STAGE OF PHASE II

DRYDOCK

MATA REDONDA

Fig. 11.1

the bottom of the elements after the inundation of the dry dock. During the fabrication of the tunnel units, inflow of ground water into the basin must be prevented. To this end, a number of drainage systems are proposed as follows:

(a) A series of vertical drains, drilled into the artesian water bearing layers and designed to relieve the artesian pressure from these layers.

(b) A deep well system with a header pipe installed at El. 0.00' along the periphery of the dry dock.

(c) A deep well system with a header pipe installed at El. -16.5', placed on a 10' wide horizontal berm in the side slopes of the dry dock.

The last two systems are designed to lower the groundwater table to below the side slopes and the bottom of the casting basin.

It is anticipated that the construction of the dry dock will start on February of Year 1, immediately after the awarding of the contract, and be completed for element fabrication by May, thus allowing 4 months for this phase of construction.

<u>Tunnel Elements</u> - At the completion of construction and dewatering of the dry dock, the south tunnel approach and the 5 tunnel units can then be built by standard methods in the dry from June to the middle of November of Year 1. This timing will permit working through the hurricane season occurring from July to October.

While the elements are in the dry dock, measurements, observations and weight adjustments of the units will be carried out under controlled conditions with little or no risk of damaging the elements. In order to speed up the actual sinking and placing operations to suit the 8 month hurricane-free period, as much preparatory work as possible will be carried out while the elements are still in the dry dock.

After the completion of all construction work in the dry dock, except for the addition of the 8" protective concrete layer on top of the elements, water will be pumped into the basin to permit the checking of the weight and floating characteristics of the elements. Adjustments in the location of the centres of gravity of the elements by means of ballasting will then be made. Eventual leaks which become evident in the bottom or sides of the elements will also be repaired.

After completing the above work, the water level will be lowered to enable the elements to rest once more on the bottom of the dry dock. The 8" protective layer of reinforced concrete will then be poured while additional lean-concrete ballast will be placed inside the elements to obtain the exact weight for sinking. It is expected that the minimum submerged weight, i.e., in water with a density of 64.0 pcf, will be in the order of 200 tons per element. This negative buoyancy will stabilize the element against forces from stream currents and wave action during the transportation of the elements to site. Each element displaces approximately 440,000 cu. ft. of water. In fresh water with density of 62.4 pcf, then, an element when fully submerged will have a submerged weight of roughly 550 tons. The supporting cables of the sinking rig must have at least this capacity.

After the above preparation, all elements will be ready for the next construction phase, i.e., the placing of the finished units into the pre-dredged trench. The dredging and cleaning work, which started 2 weeks earlier immediately after the hurricane season will be in progress.

During the period of activity in the dry dock, the north approach will have been constructed and the timing so scheduled that work on both the north and south shore will be completed simultaneously.

11.1.2 Phase II -- Placing of Submerged Elements

This phase involves the placing of the precast tunnel elements into their final position in the trench dredged along the tunnel centreline ahead of the placing operations. While most of the activity under Phase I was confined to the shores, the field of action now moves to the river itself to take advantage of the 8 month hurricane-free period.

Dredging - The trench to house the 5 elements will be dredged from the south approach structure to the north approach structure. The material to be encountered is mainly very soft mud to soft sandy clay and clayey sand of medium density. On the south shore, the bottom of the tunnel may reach into a sandstone layer which must be scraped or if necessary, blasted.

Drilling the blast holes and placing the explosive charges can be done from within the dry dock. The explosives can be ignited after the dry dock has been flooded to avoid the sudden inflow of water due to the sudden rupture of the dyke. Similar procedure was used in the construction of Canada's Lafontaine Tunnel near Montreal.

Due to the great depth of the river channel required for navigation, the dredging equipment will have to reach down to a maximum depth of 72' below mean sea level. At present, no suction dredge is available on the coast of the Gulf of Mexico capable of dredging to a depth greater than 65' at the same efficiency. It is possible to dredge deeper, but at a reduced efficiency and consequently higher unit cost. At such depth, siltation is appreciable, and at the reduced efficiency, the dredge may be incapable of coping with the silting. For this reason, and depending on the amount of siltation, a clam shell dredger may have to be used for depths in excess of the 65'.

To determine the amount of siltation, it is recommended that a test trench be dredged on the centreline of the tunnel. After the results of such a test are available, the specifications for the dredging equipment can then be established.

The preparation of the foundation base will follow the dredging operation with a slight time lag so that the placing of the tunnel elements can be performed immediately afterwards to avoid excessive siltation of the trench. The foundation base consists of 2' of well-graded gravel to reduce the probability of large differential settlements after the tunnel elements are in place. The gravel will be placed from floating barges and through suspended elephant trunks in order to obtain more accuracy in locating the trench bottom for the gravel bedding. This gravel layer is to be roughly screeded when the tunnel element has been warped over its exact location. The screeding operation will be economically and efficiently performed by a travelling screed beam suspended from the underside of the tunnel element above.

In addition to the dredging of the trench for the placing of the elements, some dredging will be required along the river for the diversion of the shipping channel towards the left bank at the time of placing of the elements near the right bank and vice versa. Dredging will also be required for an exit channel for towing elements out of the drydock. The dredging of diversion and exit channels is within reach of standard equipment and will not cause any undue difficulties.

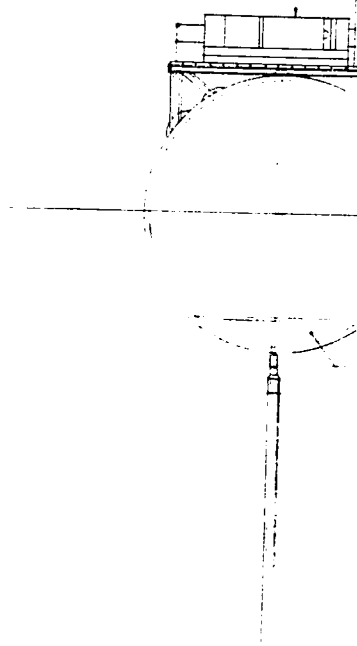
<u>Sinking</u> - By this time, in the middle of November of Year 1, the tunnel elements are totally submerged and are resting on the bottom of the dry dock with only short access shafts protruding just above the maximum high tide level of El. +3.25'. Depending on the salinity of the water at the instant concerned, the effective weight of the elements may vary from 200 tons to 550 tons corresponding to specific gravities of water of 1.02 or 1.00 respectively.

To handle the elements, a special "sinking rig" must be built, capable of lifting and lowering a 550-ton element into its final position. A typical sinking rig is illustrated in Fig. 11.2. It consists of two separate units, one at each end of an element. Each unit will be equipped with winches for the placing operation.

Whereas the actual sinking operation only takes a matter of a few hours, the elements, after being picked up by the sinking rig, must be pulled to the fitting-out area where some time is spent in waiting to be fitted with the necessary equipment. A control tower together with an access shaft will be placed on the extreme south end of the element while a second tower equipped with a secondary shaft, required for alignment, will be installed on the extreme north end as shown in Fig. 11.2. The control cabin on top of the control tower contains two rooms, placed one above the other and has a panoramic view. The control room directs the sinking operation and the other room is occupied by the surveyors and their instruments.

In order to control the exact alignment and profile of the immersed elements along the tunnel centreline, a laser beam generated on one shore and sent across the Rio Panuco to the opposite shore will be used. Lining up the alignment markings on the element with the laser beam will indicate the exact location of the element.

Prior to sinking, so called "Dutch" anchors are placed.



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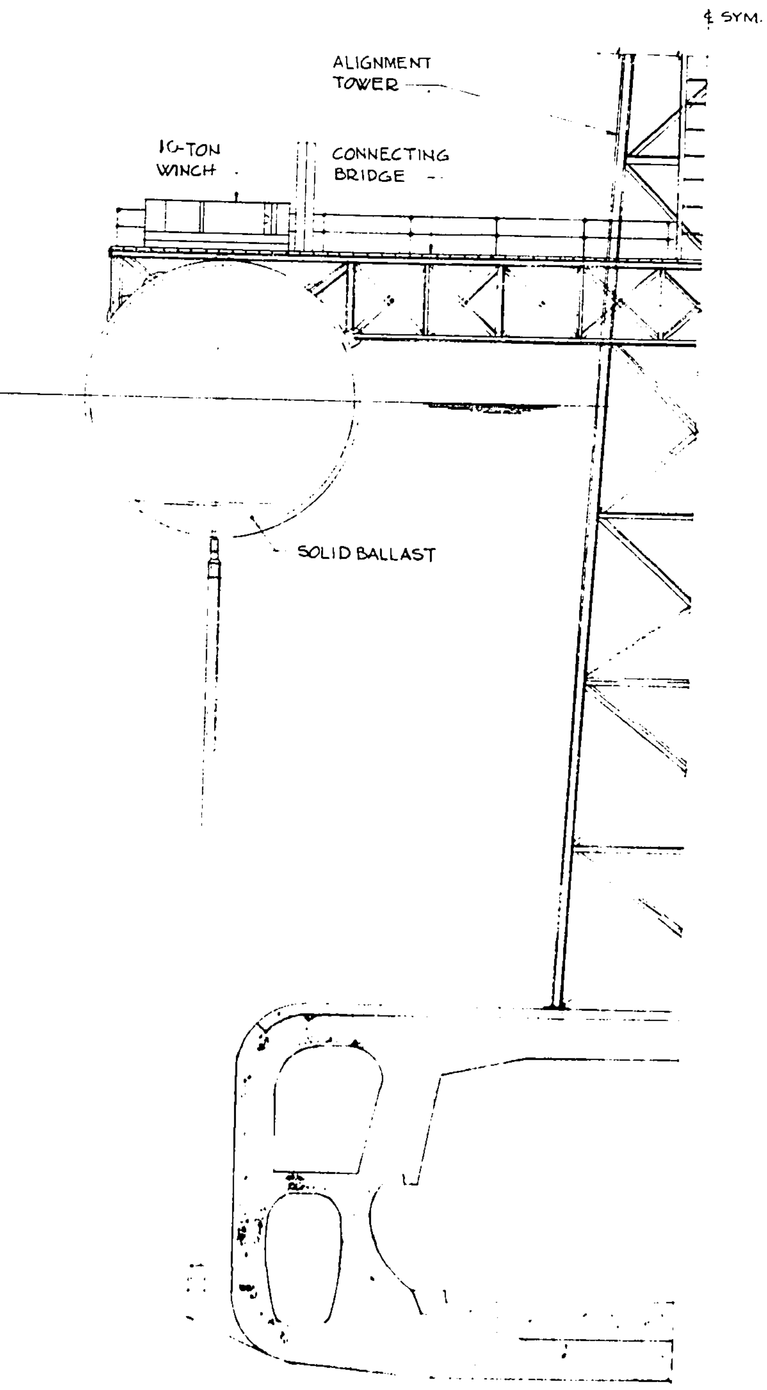
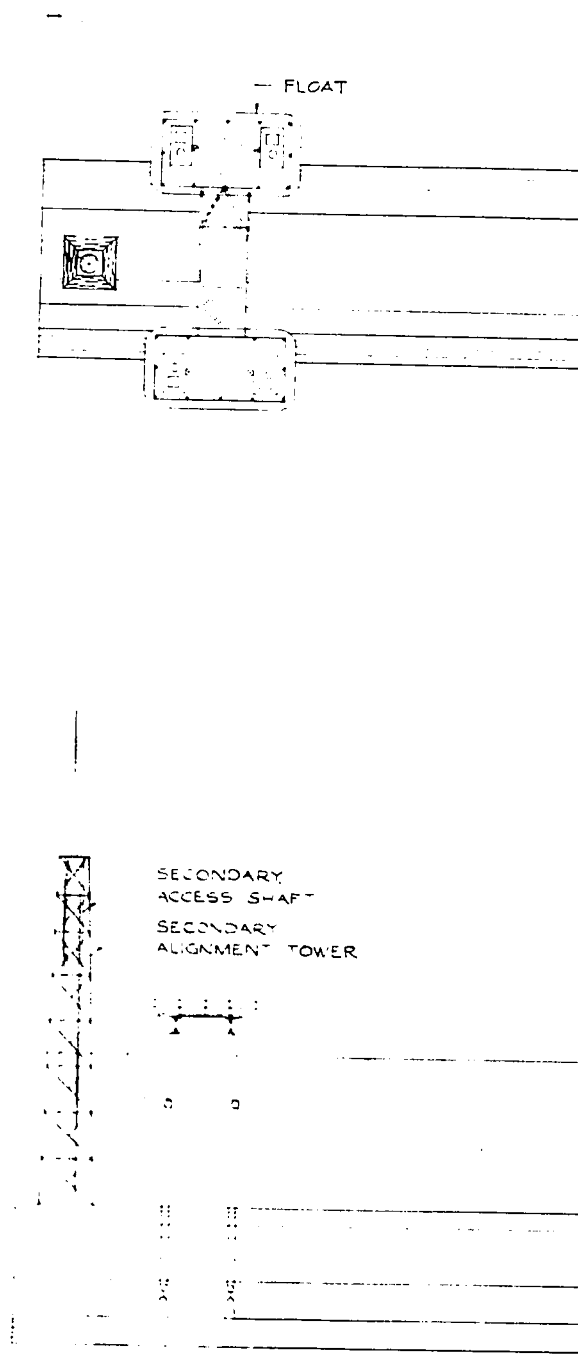
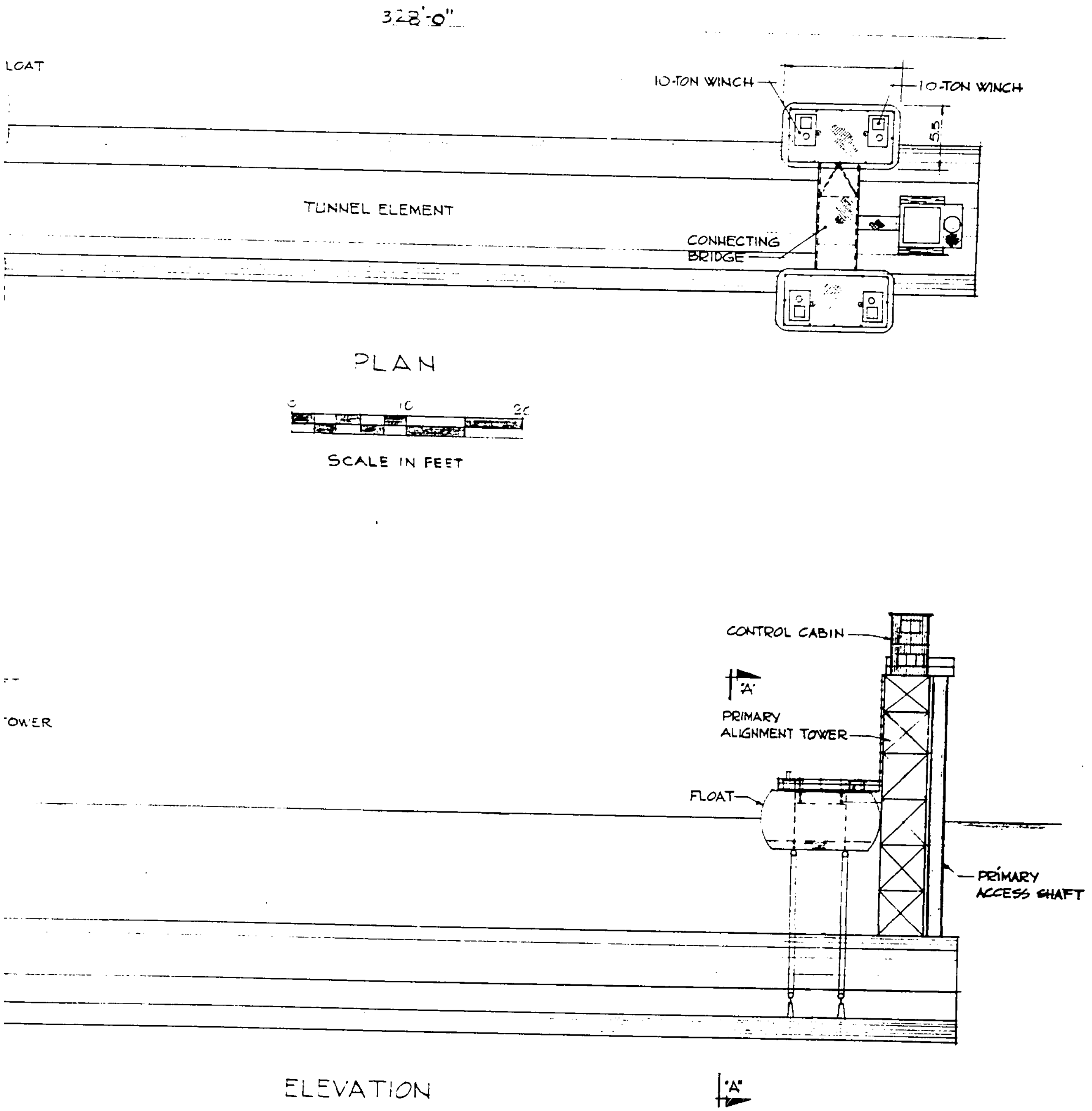

℄ SYM.
ALIGNMENT TOWER
10-TON WINCH
CONNECTING BRIDGE
SOLID BALLAST

SECTION "A-A"


FLOAT
SECONDARY ACCESS SHAFT
SECONDARY ALIGNMENT TOWER



SINKING RIG

Fig. 1.2

These anchors are driven to a considerable depth into the river bed by means of pile-driving equipment and are capable of resisting forces in the order of 100 tons. The transporting of elements from the fitting-out area to the tunnel site is done by towing and warping. Six warping lines with winches placed on shore will be adequate to place an element. Each warping line is guided through a sheave block suspended from a buoy, anchored by a Dutch anchor. Having arrived at its final location, the element is moored by winch lines for horizontal control during sinking. The winch stations, including the winches for vertical movement, are connected by telephone to the control room, as are the surveyor stations.

When all preparations for sinking are complete, the element is warped into the correct position and then lowered into the trench by means of the appropriate winches. While the longitudinal alignment and gradient of the element are controlled by laser beams, the exact location on the tunnel centreline can be verified either by triangulation or by a direct, electronic distance measuring device.

It must be remembered that while still in the dry dock, the elements were ballasted to sink under any condition which may occur, even if a salt water wedge unexpectedly moved up the river, changing the salinity of the water. To ensure optimal safety, however, the sinking operation should be performed only at a time when a minimum of 4 hours is available with a current velocity not more than 1.5 fps to avoid tipping of the element due to strong stream forces.

Joining - Many ingenious methods have been developed to join adjacent tunnel sections. In the one adopted here, a neoprene gasket is installed at one end of the section while in the dry dock. When the element is sunk to the desired elevation, the built-in cantilever arm on one end of the section unavoidably comes to rest on the supporting brackets protruding from the sides of the previously placed element. The longitudinal tackle can then pull the new element against the previous one.

A hydraulically operated pulling device installed through the end bulkhead of the previous element is now engaged with the hooking system on the bulkhead of the new element. Operating from within the placed tunnel section, the new section together with the neoprene gasket is pulled against the flat end surface of the former section, thus effecting a watertight seal. When initial watertightness has been obtained, the water in the space between the two bulkheads will be drained into the tunnel thus reducing the pressure in this space to the atmospheric pressure existing in the elements. The full hydrostatic pressure at the free end of the new element will press the joint together to ensure complete seal.

After the primary seal, the vertical tackle of the sinking rig, which up to this point is still supporting this end of the element, may be removed. The grout bags which had been installed at the other end of the element at the fabrication stage in the dry dock, will then be filled with cement grout to obtained firm support for this end to enable the removal of the tackle of the sinking rig.

After the grout in the grout bags at the north end of the element has completely set such that firm support for the end is ascertained, sand will be dumped simultaneously through suspended tremie pipes along both sides of the element. This procedure will increase the resistance, because of the friction forces existing between the tunnel walls and backfill, against the uplifting forces of the tunnel element.

To complete the support under the prefabricated elements, a 6" sand-asphalt layer will be pumped under the elements through pipes embedded in the bottom slab of the elements and spaced in a 10' x 10' pattern. Placing operations of the sand-asphalt will start after the removal of the supporting tackles. The average pressure in the pumped sand-asphalt should not be too much above the water pressure prevailing locally to prevent uplifting of the elements. A safeguard against this danger is to increase the submerged weight of the elements by ballasting before the application of the sand-asphalt layer.

After the tunnel element is supported over its full area, scour protection can then be placed on the necessary elements to prevent undermining of the tunnel foundations.

It is anticipated that the placing of the 5 elements, started in the middle of November, Year 1 will be completed in the second week of April of Year 2. The preparation of the foundation and the placing of scour protection will follow behind the sinking operations such that the necessary river works will be completed before the hurricane season begins again.

11.2 Launchway Scheme

The method of immersed-tube tunnel construction envisaged here for the Tampico Tunnel departs markedly from the conventional method. Similar methods may have been proposed by engineers in tunnel feasibility or preliminary studies, but such concepts are usually confined to within the engineers' design offices. Christiani & Nielsen of Copenhagen proposed for the Chunnel tunnel, a crossing of the English Channel, a method for placing tunnel elements which differed from the conventional sinking rig methods of placing. It was unique in that the elements were to be brought to the site by rails along the tunnel centreline by submarine machines. However, this was a proposal as the English Channel crossing has not yet been built.

Launching of tunnel elements from slipways has been used, but after the elements are afloat, they need to be towed to site and sunk using normal procedures. The steel shells of the Hampton Roads Tunnel in Virginia were launched and then towed to an outfitting site for concreting and finishing. The tubes were then towed to the final tunnel site and sunk by pouring more concrete. The tunnel elements for the Hong Kong Cross Harbour Tunnel presently under construction, are fabricated adjacent to a slipway jutting into the harbour. These elements are to be launched one by one and then towed to the site and sunk into positon as in the conventional method.

It should be realized that not every tunnel site is suitable for the method of element fabrication and construction as contemplated for the Tampico Tunnel. This method is preferred because the site on the Mata Redonda side is unerlain by solid sandstone and the tunnel roadway emerges at 5% gradient intersecting the existing roadway system at the top of the hill, thus providing ample space for a launchway and prefabricating yard for the five 275' long submerged elements. The precast units for this scheme are each 53' shorter than those for the dry dock scheme. This length was chosen to reduce the dead weight of the units because each unit has to be picked up by portal cranes in the actual placing operations. The remaining lengths of the tunnel are compensated for by constructing 2 much longer cast-in-place sections. From experience, it is usually regarded that the conventional dry dock operation of fabricating, towing, and sinking of tunnel sections is costly for a two-lane narrow tunnel.

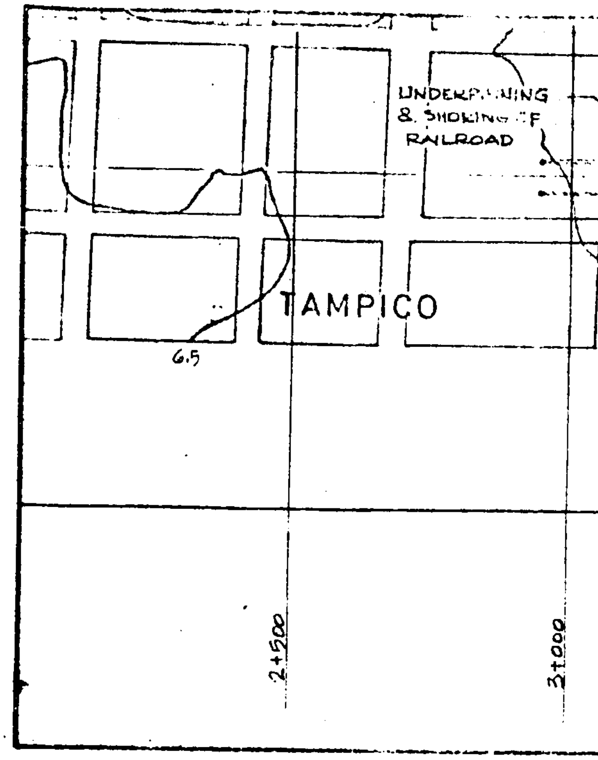
The proposal contemplates the progressive fabrication of the 5 elements in the orderly alignment and sequence in which they will be rolled down the grade for launching and moved to final positions on a prepared foundation in a trench dredged below the river bed. Specially designed mobile gantry cranes, straddling the tunnel elements, will be used for the fabrication of the elements and subsequently, in modified form, for the progressive driving ahead, of timber piles across the river and placing thereon of precast concrete runways upon which the gantries will advance. The gantry cranes will also be used for the placing of gravel fill and a compressible sand-asphalt topping for the tunnel foundations. The crane runways, contoured to the profile of the tunnel foundation, will be used as screed guides. The cranes will finally be used to lift and transport the submerged tunnel elements from the launchway to their respective locations in the river, and to lower them into final position.

This procedure will not only reduce costs substantially but also effect an appreciable saving in time to assure completion of subaqueous operations within the 8-month season available for favourable construction in the river.

The method of tunnel construction depicted above minimizes construction hazards during the annual hurricane season and permits execution of the optimal amount of work on dry land above flood level or within protective dykes. Nevertheless, some 30% of the project embraces subaqueous construction work which must be completed in the river during the favourable period.

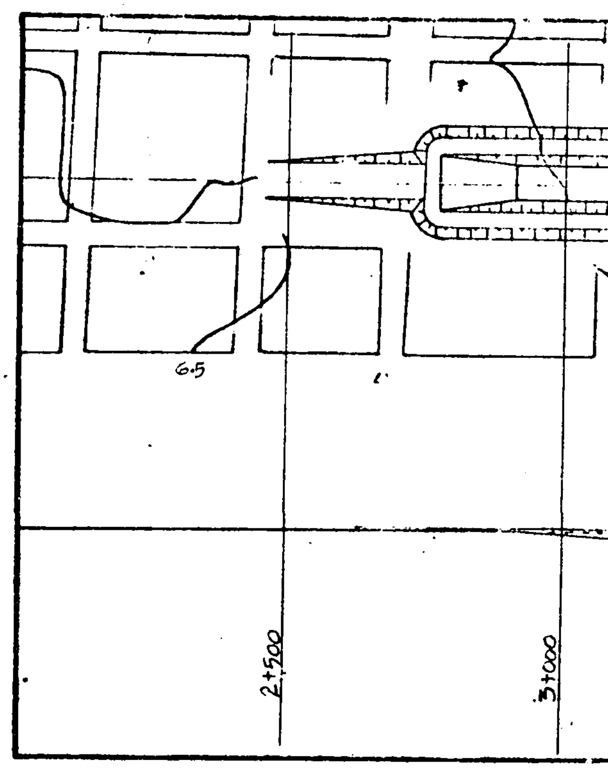
Construction operations may be conveniently considered in 3 distinct phases of which Phase II includes all work in the river while Phase I and III describe the sequence of construction operations on dry land. Assuming that the contract will be awarded in January of Year 1, and work will start immediately, 30 months has been allowed for the entire construction programme. The sequence of construction operations in 8 progressive stages is shown in Fig. 11.3 and Fig. 11.4.

As this is the method of construction finally selected



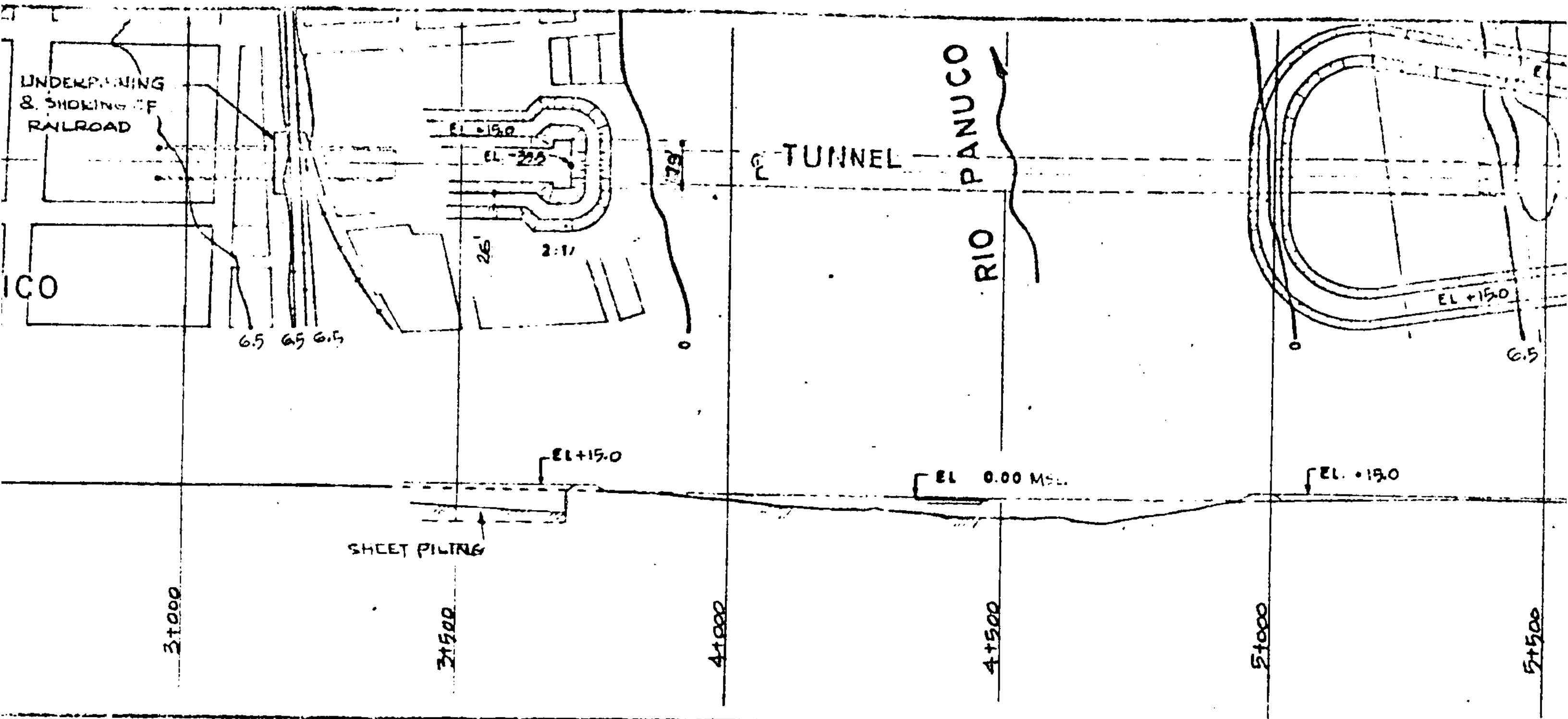
TAMPICO APPROACH

UNDERPINNING AND SHORING OF RA
DEMOLITION OF EXISTING BUILDIN
DRY EXCAVATION OF APPROACHE
CONSTRUCTION OF PROTECTIVE D



TAMPICO APPROACH

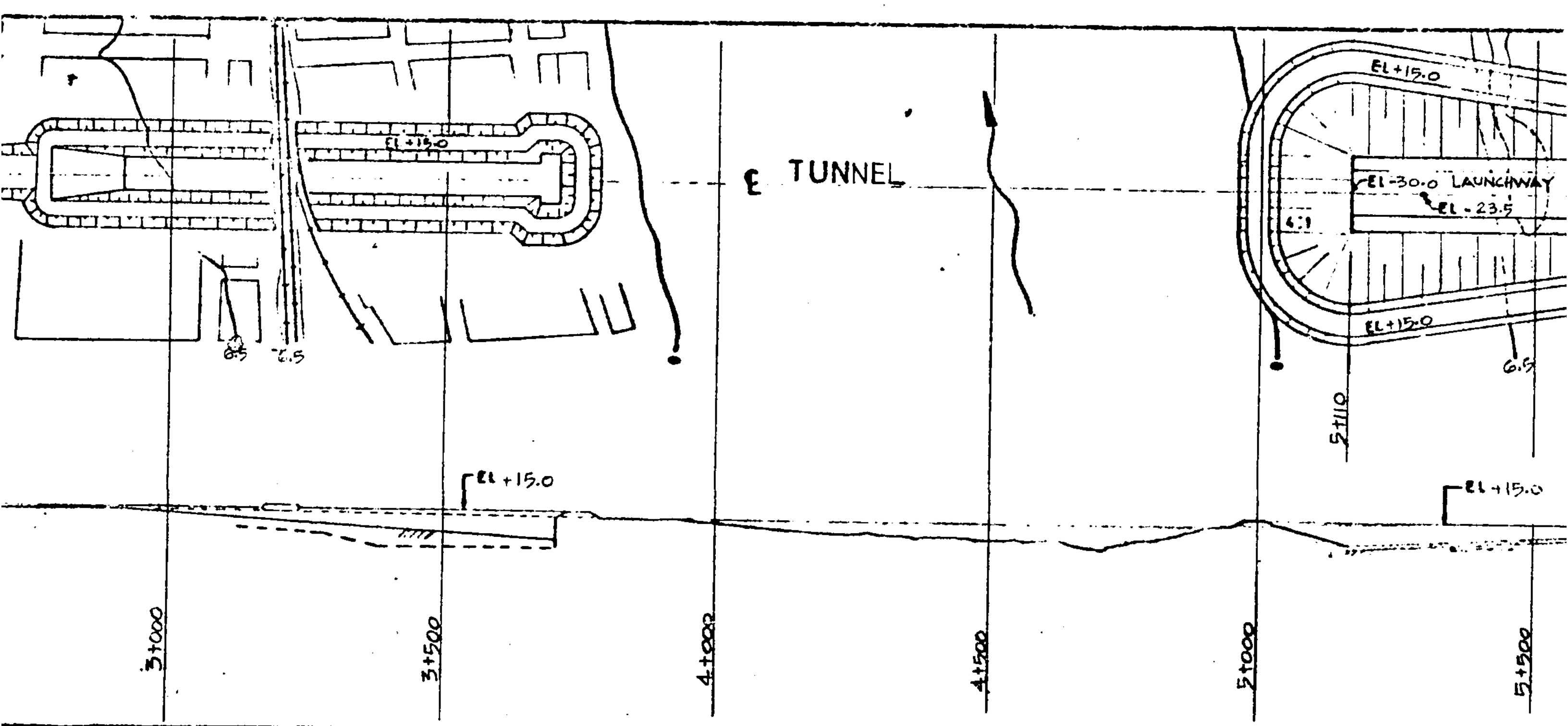
CONSTRUCTION OF PROTECTIVE DY
INSTALLATION OF STEEL SHEET PIL
AND DRY EXCAVATION FOR APPROA



PHASE I - STAGE 1

(YEAR 1 - FEBRUARY & APRIL)

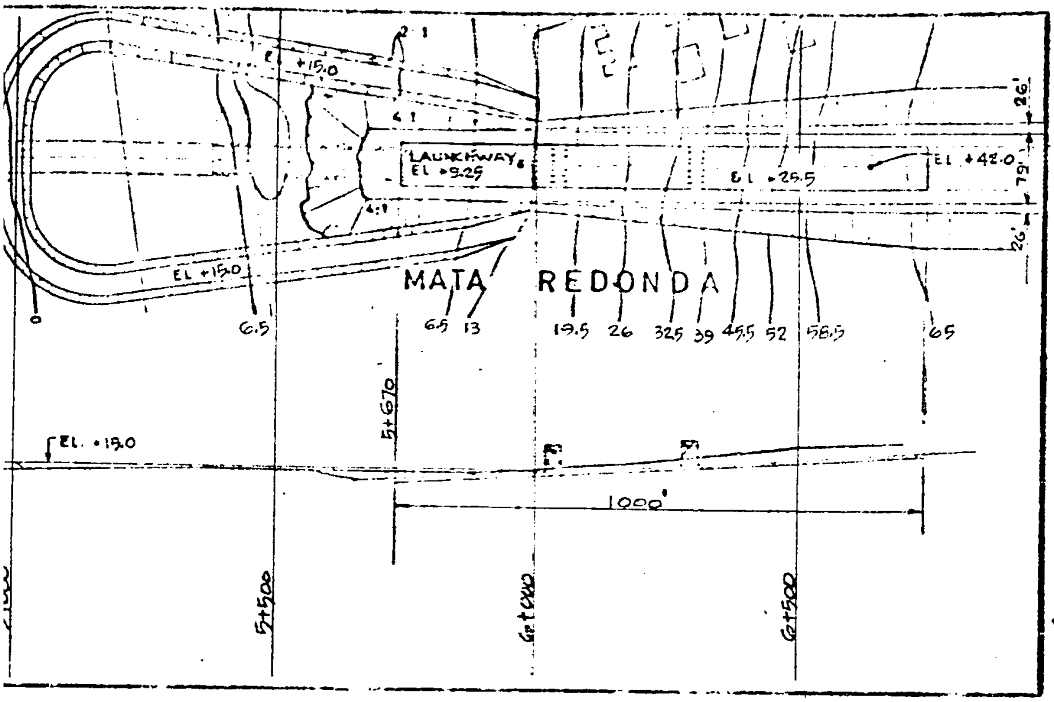
ACH
AND SHORING OF RAILROAD COMPLETED.
EXISTING BUILDINGS COMPLETED.
N OF APPROACHES COMMENCED.
OF PROTECTIVE DYKE COMMENCED.

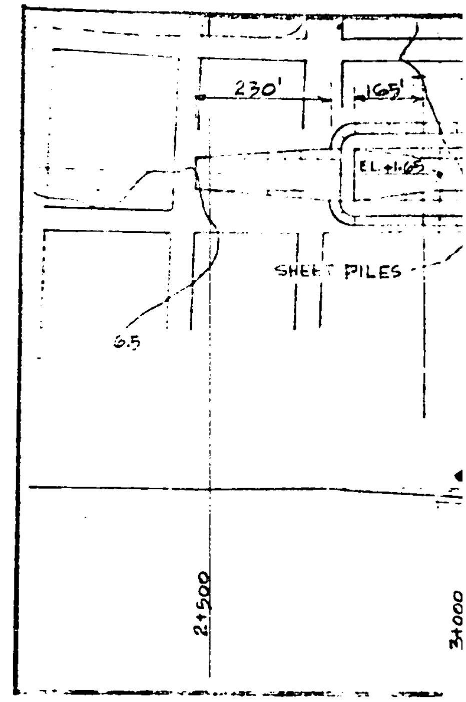


PHASE I - STAGE 2

(YEAR 1 - MAY & JUNE)

OACH
F PROTECTIVE DYKE COMPLETED.
STEEL SHEET PILE COFFERDAM
ATION FOR APPROACH IN PROGRESS.



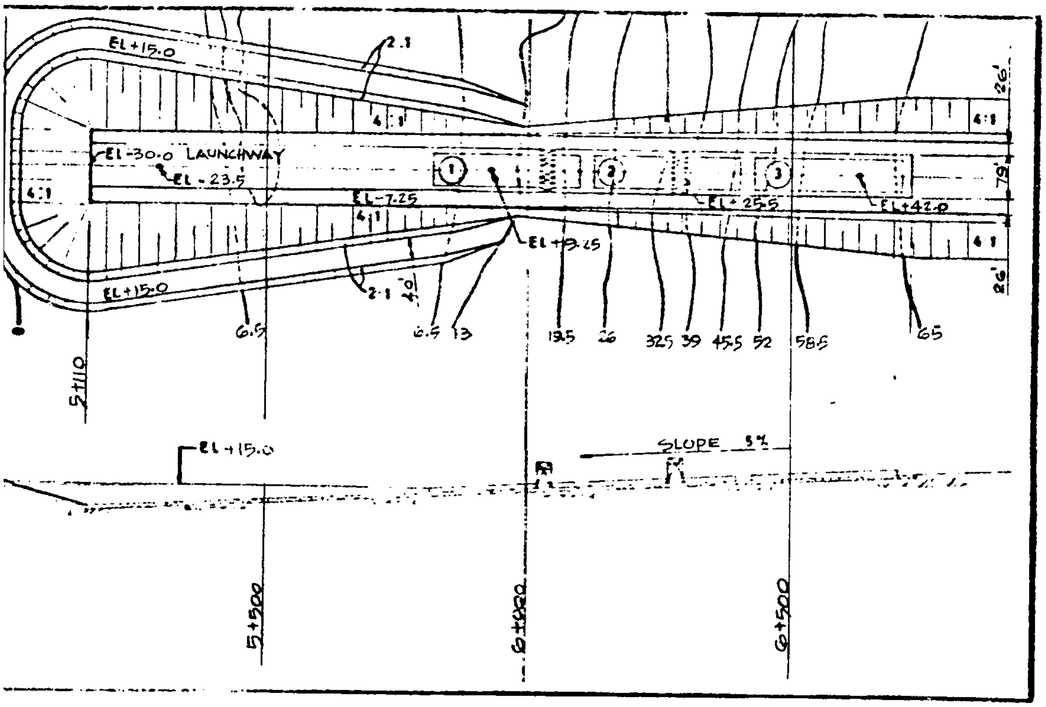


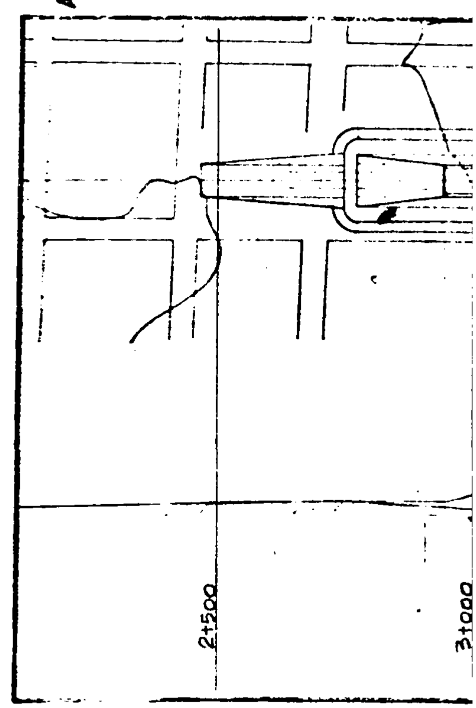
MATA REDONDA APPROACH

DRY EXCAVATION AND UPPER 1,000' OF LAUNCHWAY COMPLETED.
FIRST SECTION OF GANTRIES ERECTED.
CONSTRUCTION OF PROTECTIVE DYKE COMPLETED.
DEWATERING AND EXCAVATION IN PROGRESS.

TAMPICO

DEWATER
FOR APP



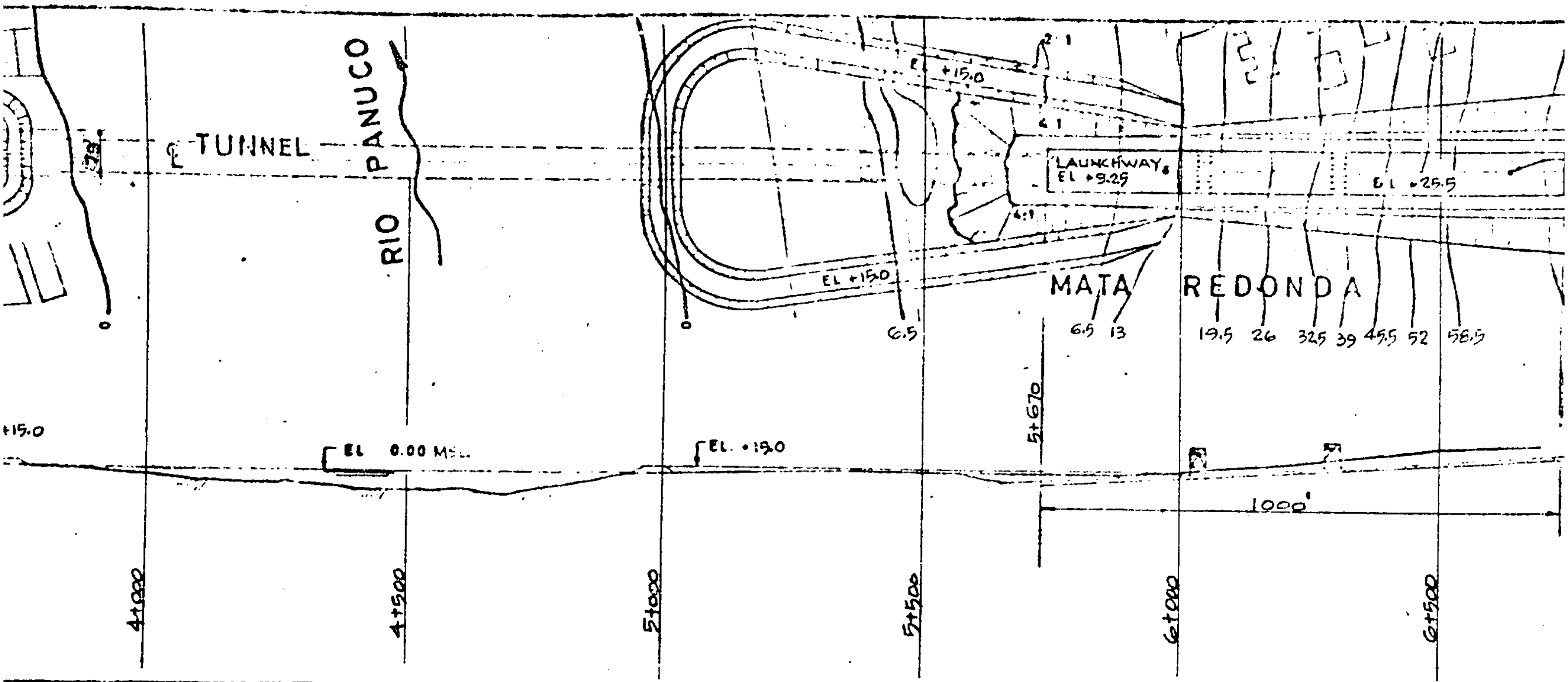


MATA REDONDA APPROACH

PROTECTIVE DYKE COMPLETED TO EL.+15.0.
DEWATERING AND DRY EXCAVATION FOR APPROACH AND LAUNCHWAY COMPLETED.
CONSTRUCTION OF TUNNEL ELEMENTS IN PROGRESS.

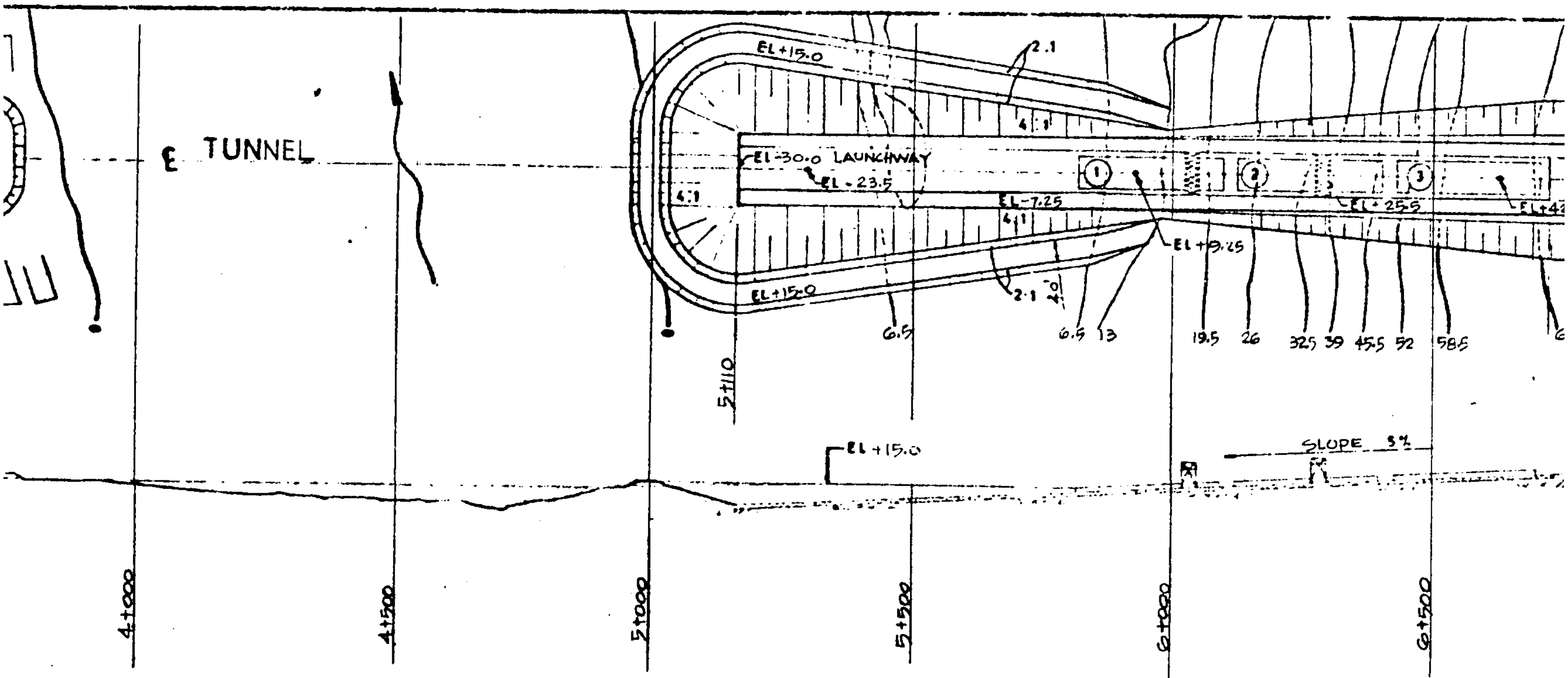
TAMPICO APPROACH

APPROACH STRUCTURE COMPLETED.
285' CAST-IN-PLACE TUNNEL SECTIO
VENTILATION BLDG. COMPLETED TO
52' OF BASE FOR TUNNEL COMPLE
PERMANENT RAILWAY BRIDGE ACROSS



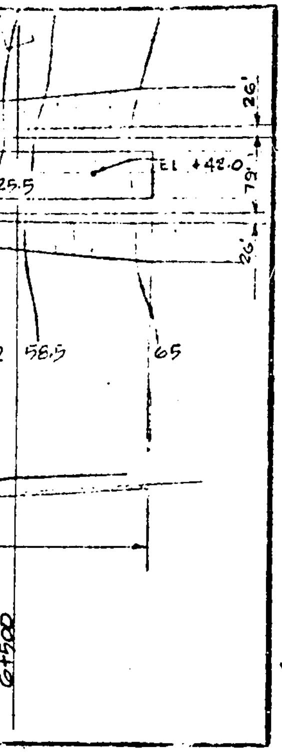
PHASE I - STAGE 1
(YEAR I - FEBRUARY & APRIL)

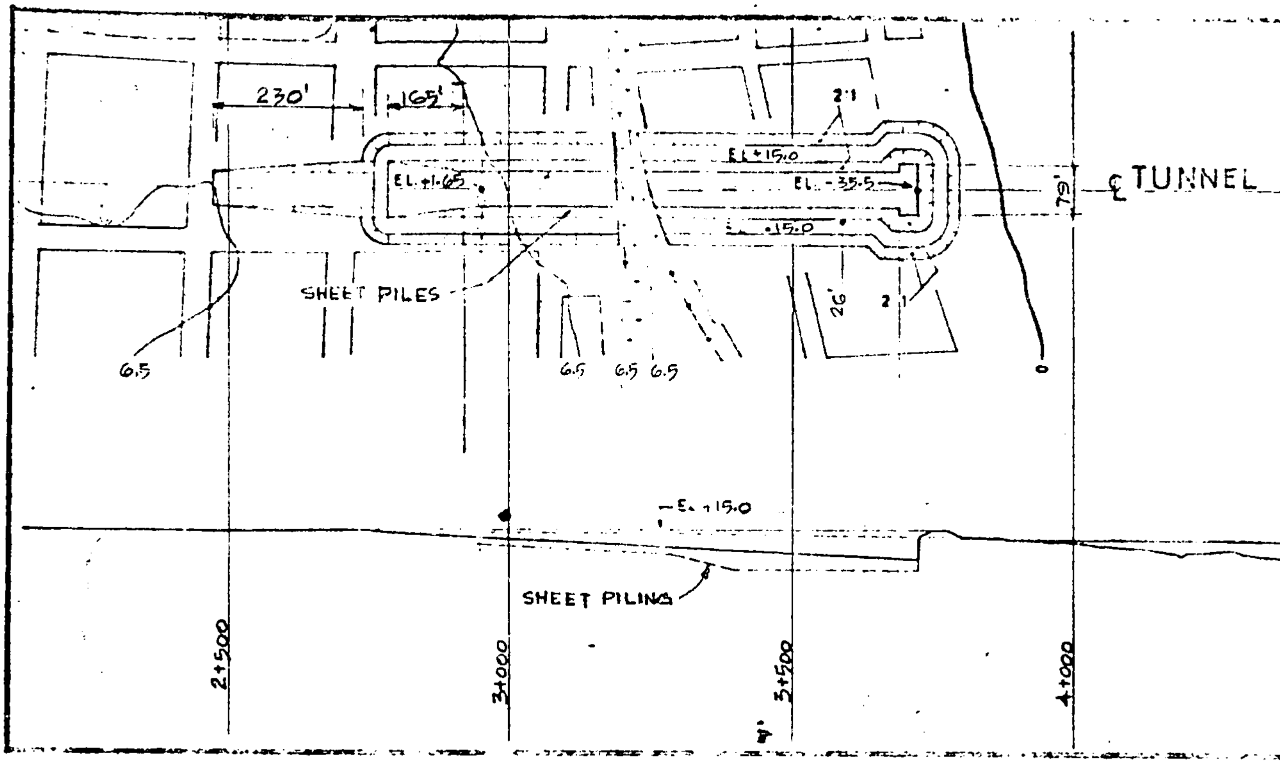
MATA REDONDA APPROACH
DRY EXCAVATION AND UPPER 1,000' OF LAUNCHWAY COMPLETED.
FIRST SECTION OF GANTRIES ERECTED.
CONSTRUCTION OF PROTECTIVE DYKE COMPLETED.
DEWATERING AND EXCAVATION IN PROGRESS.



PHASE I - STAGE 2
(YEAR I - MAY & JUNE)

MATA REDONDA APPROACH
PROTECTIVE DYKE COMPLETED TO EL.+15.0.
DEWATERING AND DRY EXCAVATION FOR APPROAC
AND LAUNCHWAY COMPLETED.
CONSTRUCTION OF TUNNEL ELEMENTS IN PROGR





F

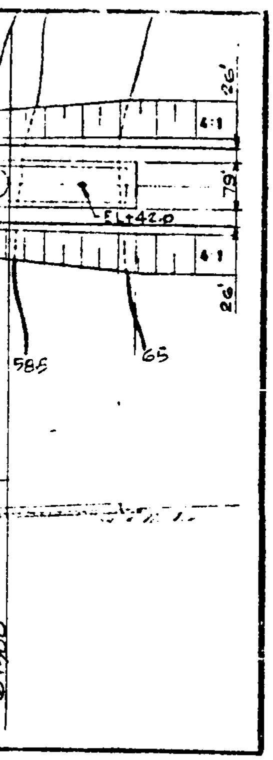
COMPLETED.
RESS.

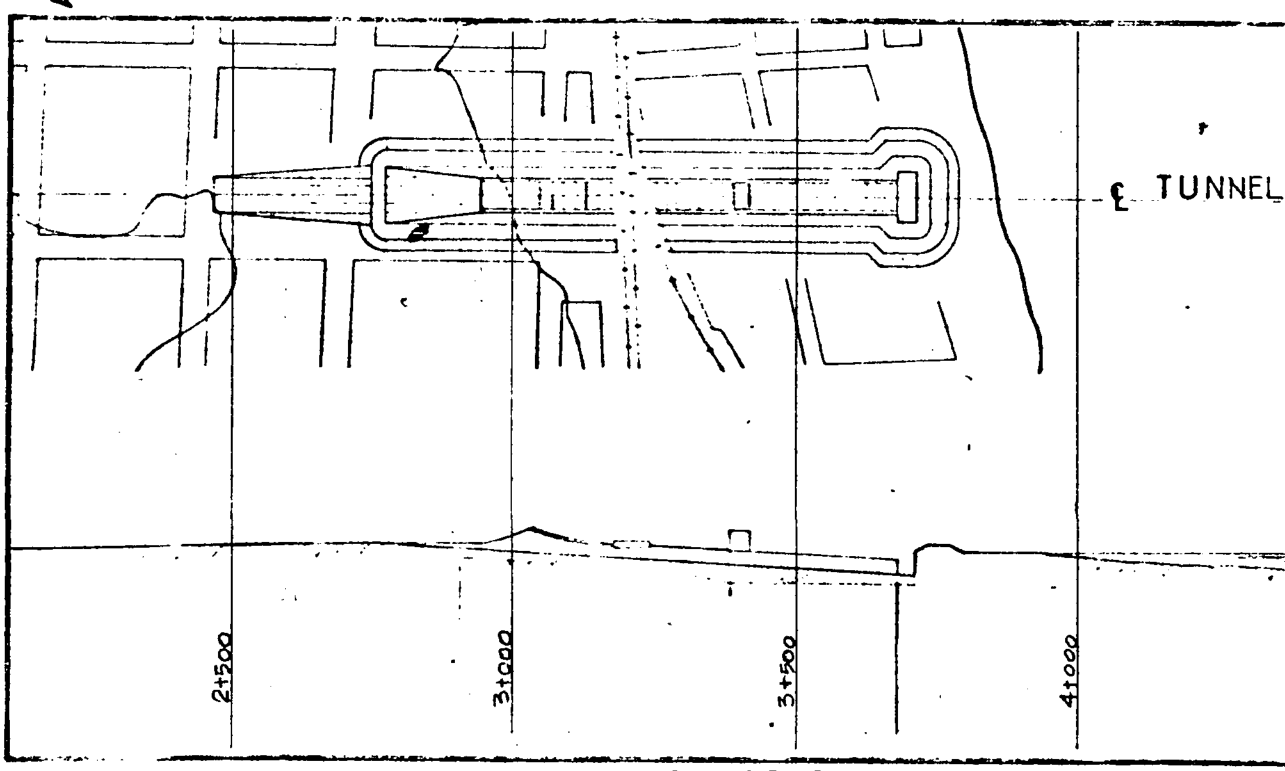
TAMPICO APPROACH

DEWATERING AND DRY EXCAVATION
FOR APPROACH COMPLETED

PHASE I - ST

(YEAR I - JULY





PHASE I - STAGE 4

(YEAR I - SEPT. & OCT)

.+15.0.
OR APPROACH

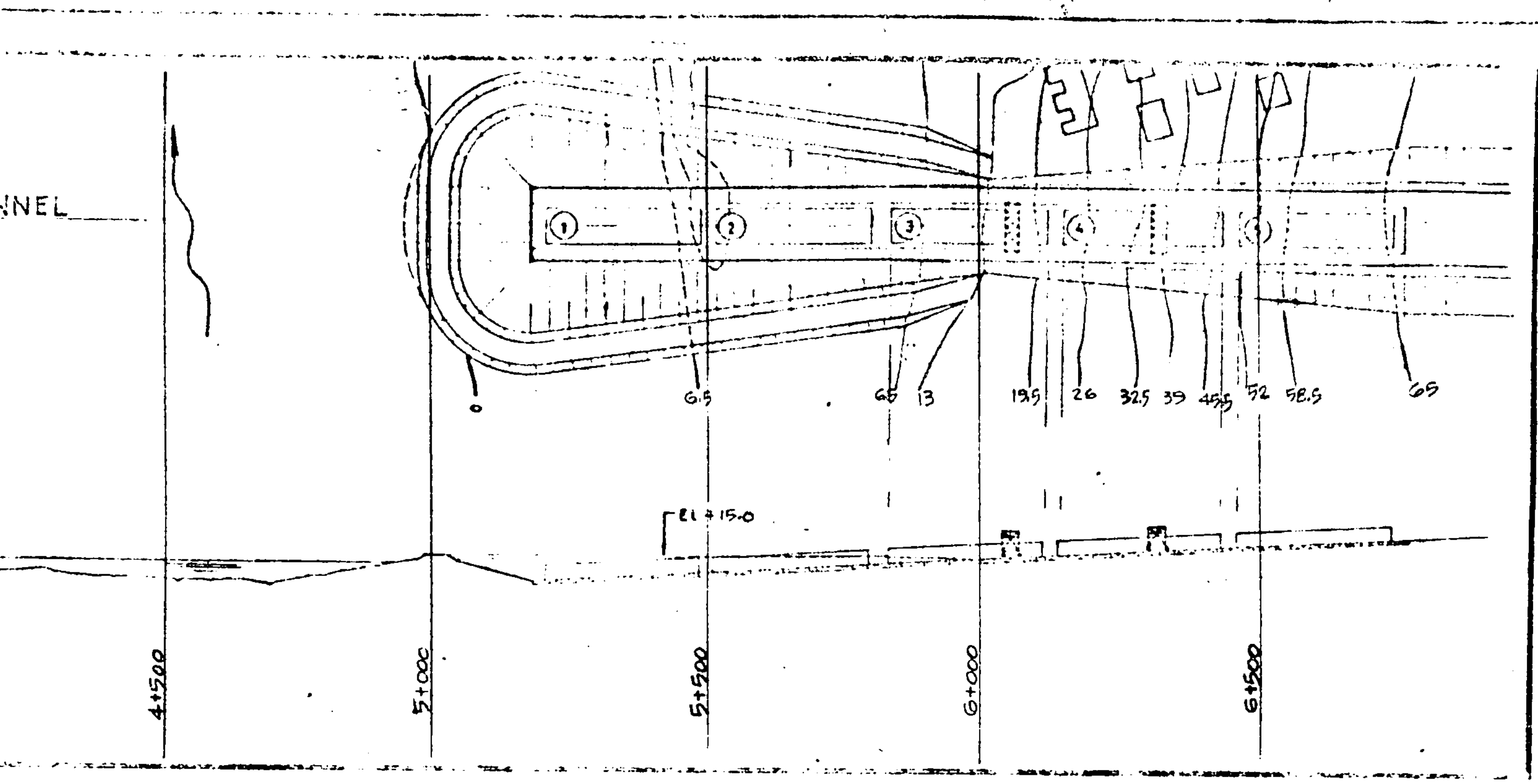
TS IN PROGRESS.

TAMPICO APPROACH

APPROACH STRUCTURE COMPLETED.
285' CAST-IN-PLACE TUNNEL SECTION COMPLETED.
VENTILATION BLDG. COMPLETED TO EL. + 15.0
52' OF BASE FOR TUNNEL COMPLETED.
PERMANENT RAILWAY BRIDGE ACROSS APPROACH COMPLETED.

MATA

ALL T
READ
MODI



STAGE . 3

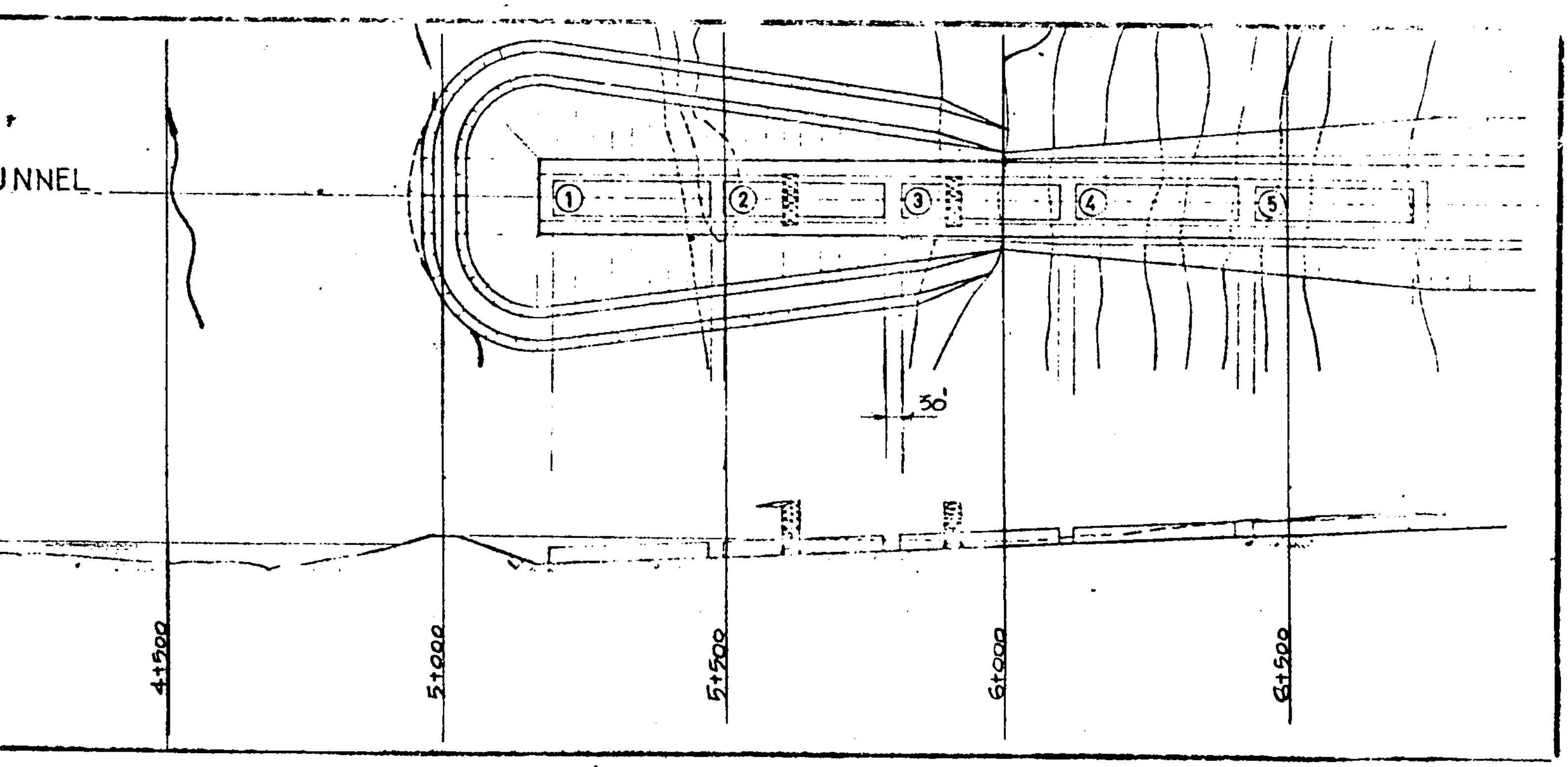
JULY & AUGUST)

MATA REDONDA APPROACH

CONSTRUCTION OF TUNNEL ELEMENTS IN PROGRESS.

0 50 100 200 300 400

SCALE IN FEET

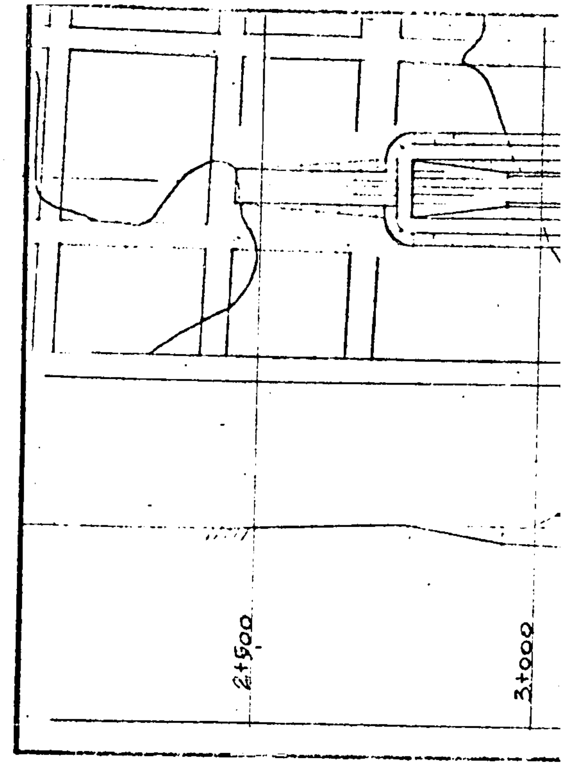


MATA REDONDA APPROACH

ALL TUNNEL ELEMENTS COMPLETED AND READY FOR LAUNCHING.
MODIFICATION OF GANTRIES COMPLETED.

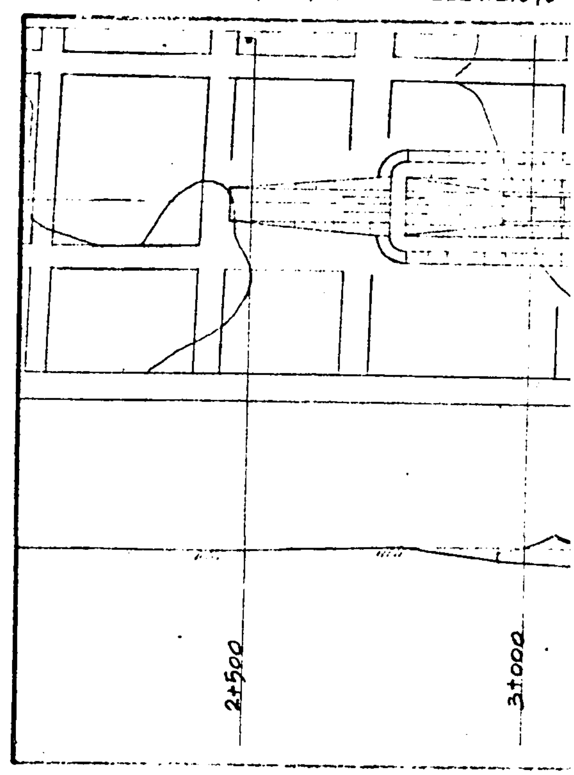
TUNNEL CONSTRUCTION PROCEDURE I

Fig. 11.3



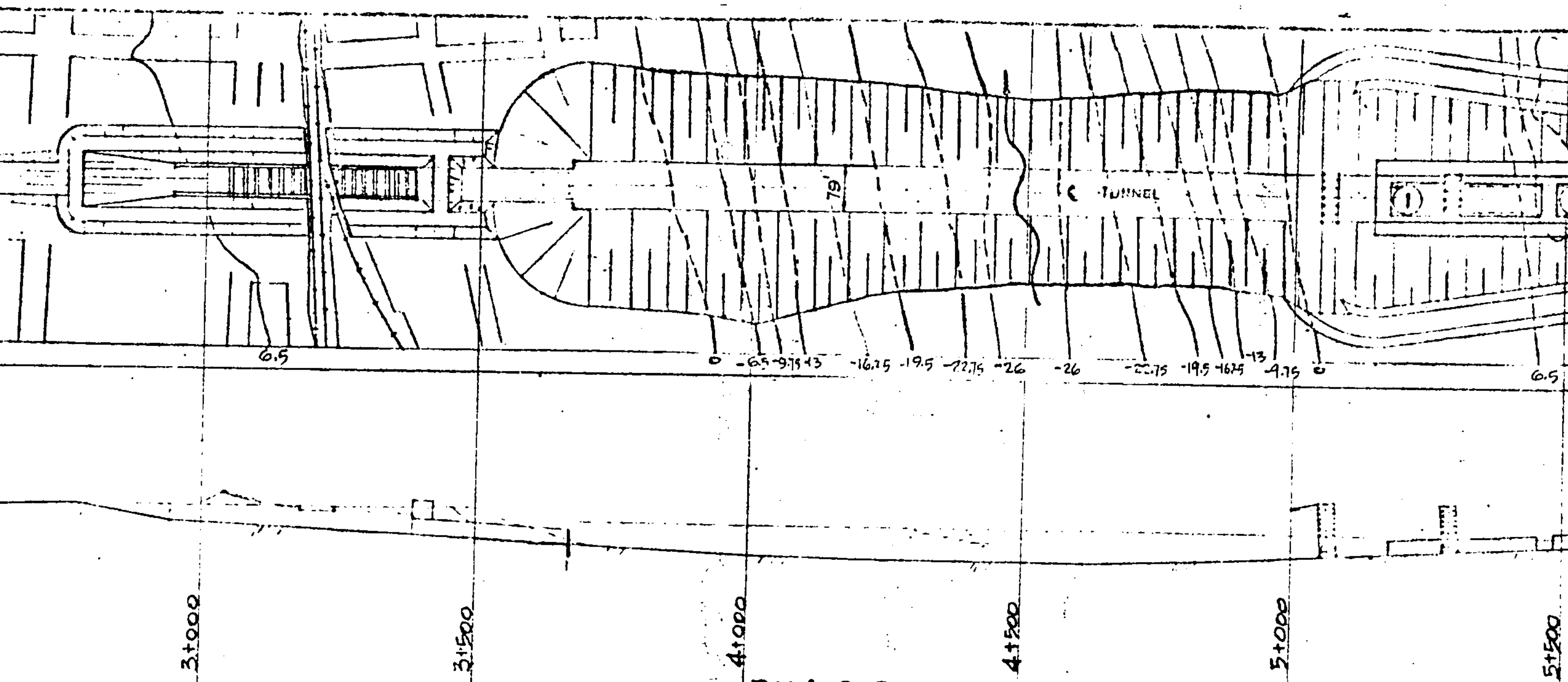
TAMPICO APPROACH

CUT-OFF DYKE INSTALLED.
RIVER END OF PROTECTIVE DY
DREDGING COMPLETED & REA
OF FIRST TUNNEL ELEMENT.



TAMPICO APPROACH

FINISHING WORK IN PROGRES
VENTILATION BUILDING COMP



PHASE II STAGE 5

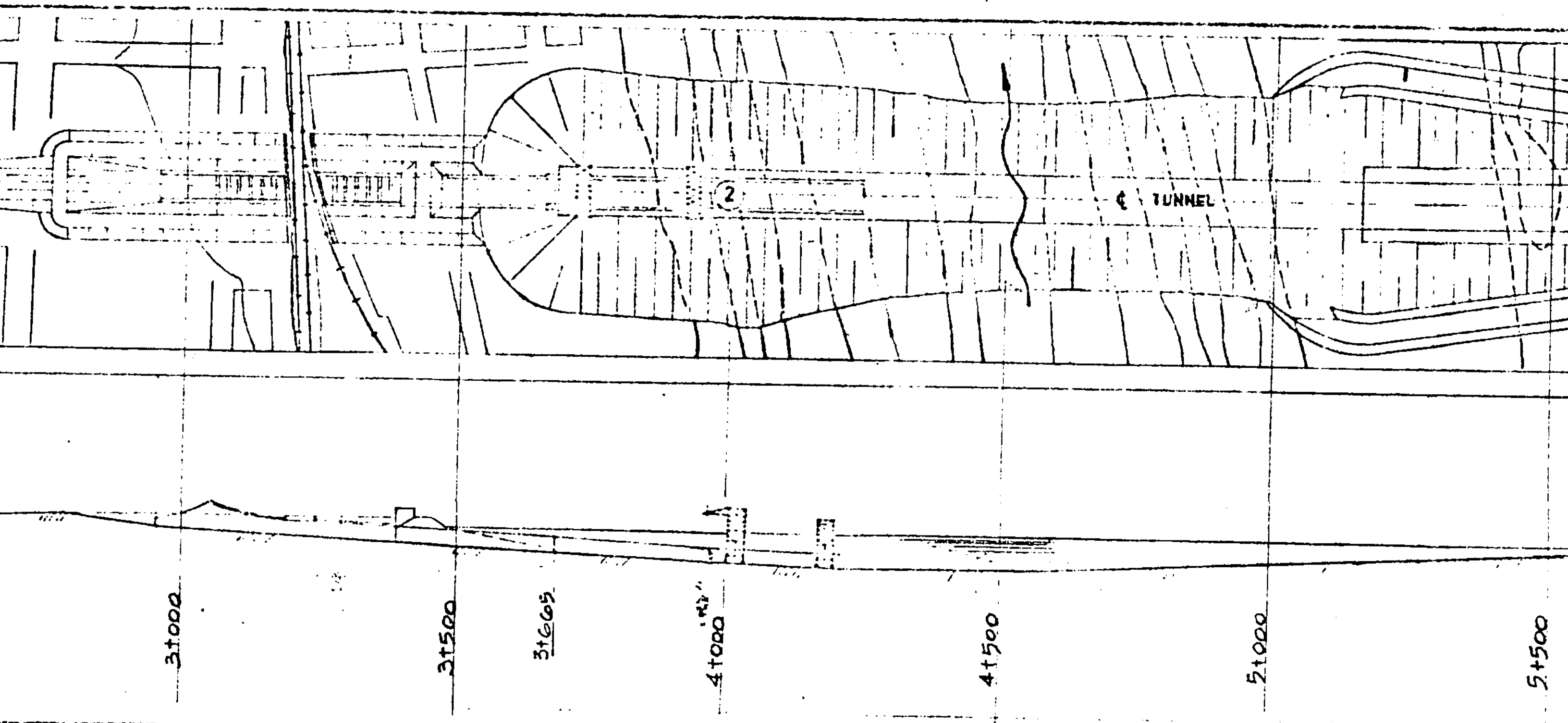
YEAR 1 NOV. - DEC.
YEAR 2 JAN. - FEB.

PPROACH

YKE INSTALLED.
OF PROTECTIVE DYKE REMOVED.
COMPLETED & READY FOR PLACING
TUNNEL ELEMENT.

RIVER

DREDGING OF TRENCH FOR TUNNEL COMPLETED.
WORK ON GANTRY RUNWAYS IN RIVER COMPLETED.
FOUNDATION FILL & ASPHALT TOPPING COMPLETED.



PHASE II STAGE 6

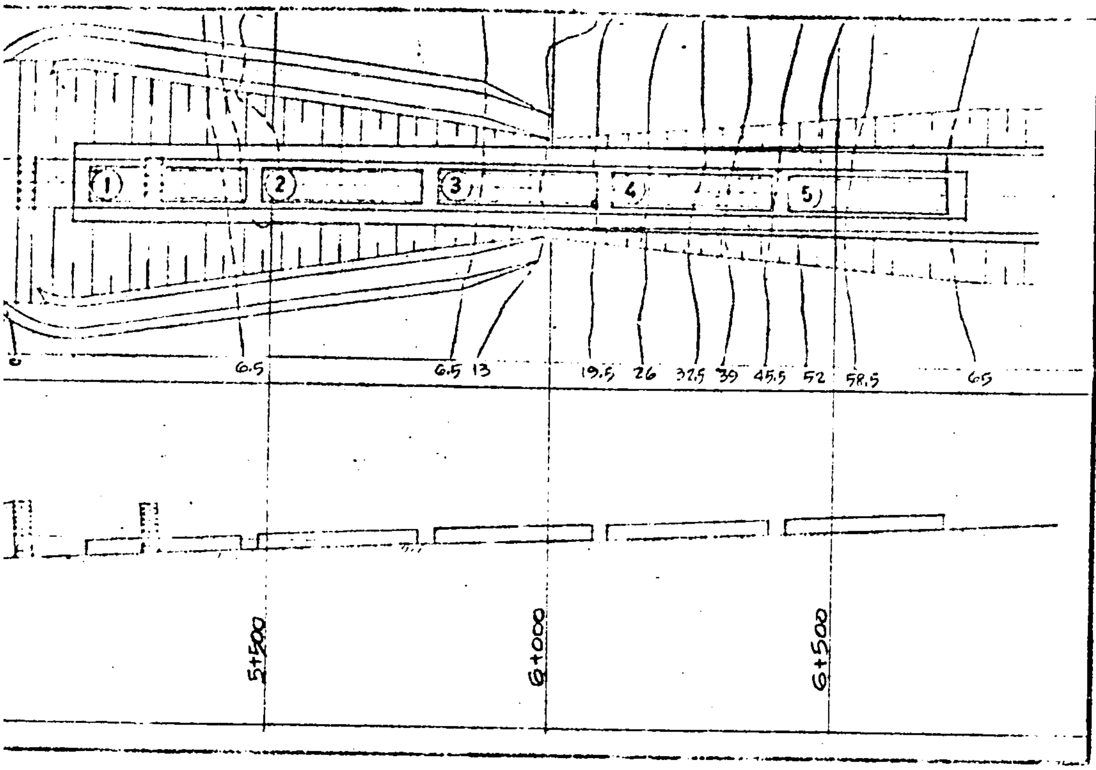
YEAR 2 MARCH - APRIL

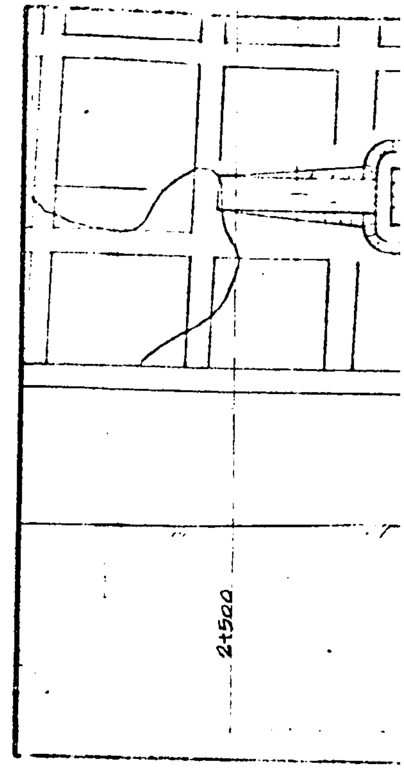
APPROACH

WORK IN PROGRESS.
N BUILDING COMPLETED.

RIVER

TUNNEL ELEMENT 1 PLACED, CONNECTED TO
TAMPICO APPROACH.
PLACING OF TUNNEL ELEMENT 2 IN PROGRESS.
BACKFILL AND SCOUR PROTECTION COMMENCED.



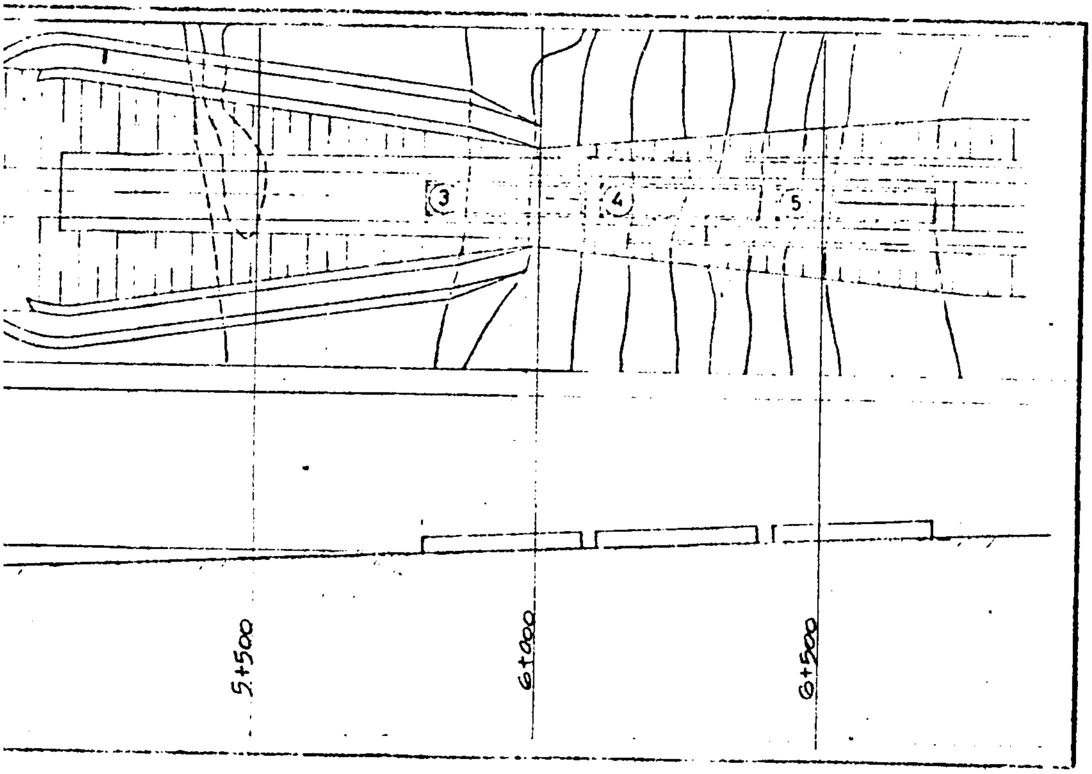


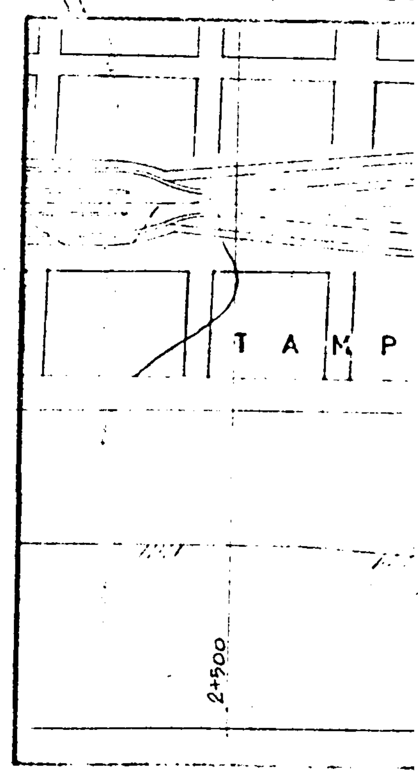
ETED.
MPLETED.
MPLETED.

MATA REDONDA APPROACH

APPROACH FLOODED.
RIVER END OF DYKE REMOVED BY DREDGE.

TAMPICO APPR
FINISHING W
ROADWAY CC
CITY STREETS
MECHANICAL



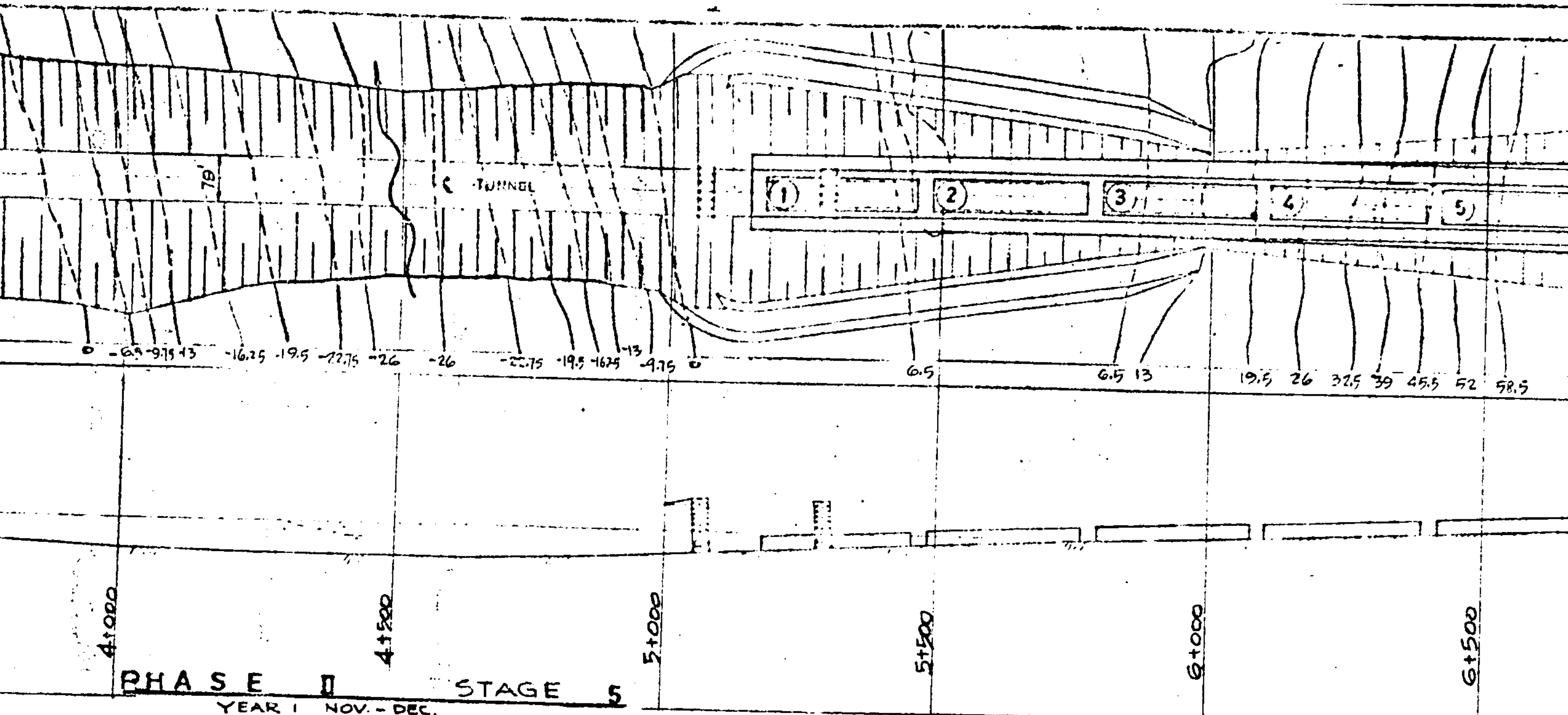


TO

OGRESS.
MMENCED.

TAMPICO APPROACH

APPROACH COMPLETED, INCLUDING
DYKE TO ELEVATION + 15.0
MECHANICAL AND ELECTRICAL
COMPLETED.
LANDSCAPING COMPLETED.



PHASE II STAGE 5

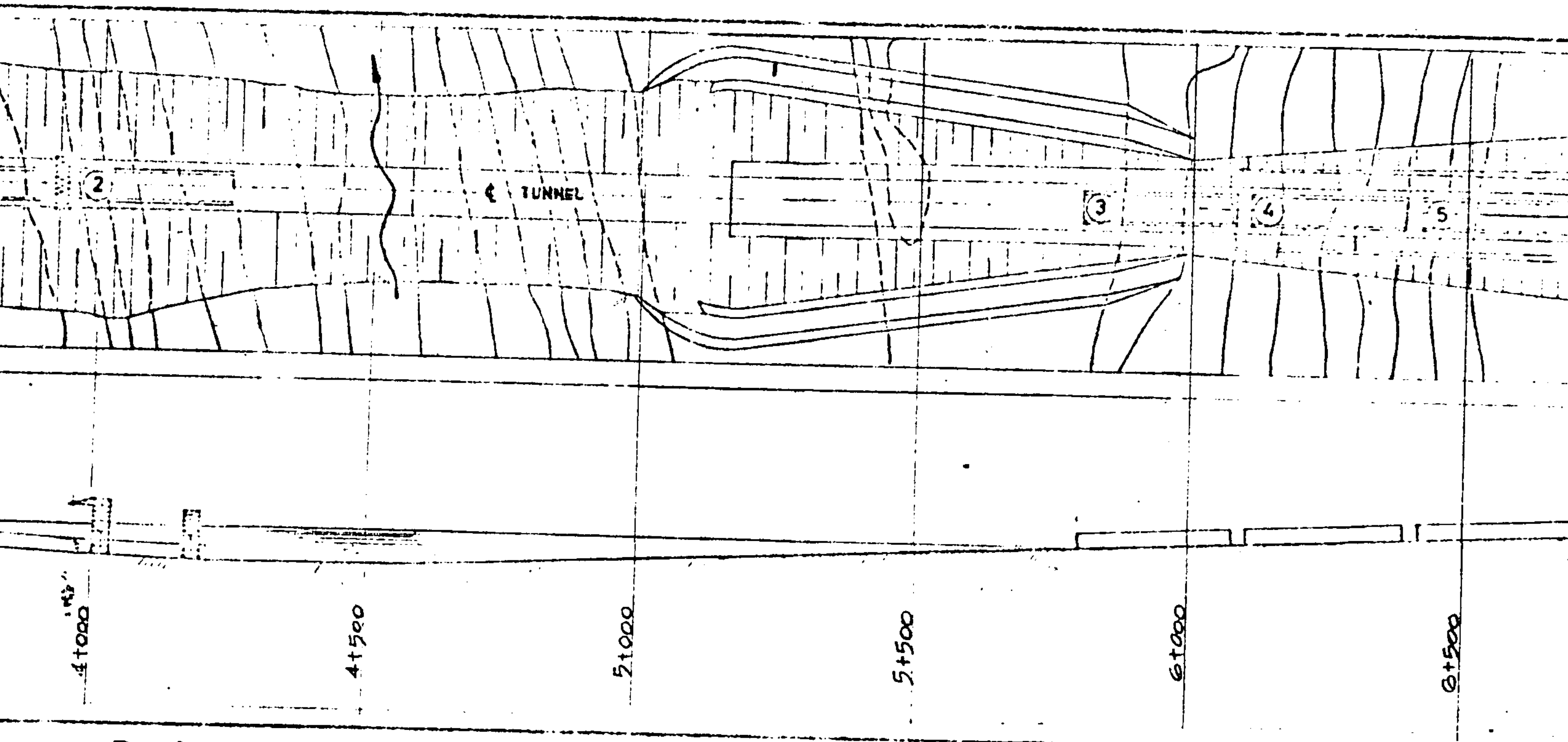
YEAR 1 NOV. - DEC.
YEAR 2 JAN. - FEB.

RIVER

DREDGING OF TRENCH FOR TUNNEL COMPLETED.
WORK ON GANTRY RUNWAYS IN RIVER COMPLETED.
FOUNDATION FILL & ASPHALT TOPPING COMPLETED.

MATA REDONDA APPROACH

APPROACH FLOODED.
RIVER END OF DYKE REMOVED BY DREDGE.

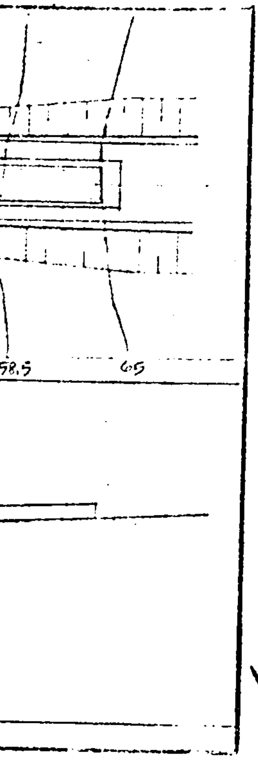


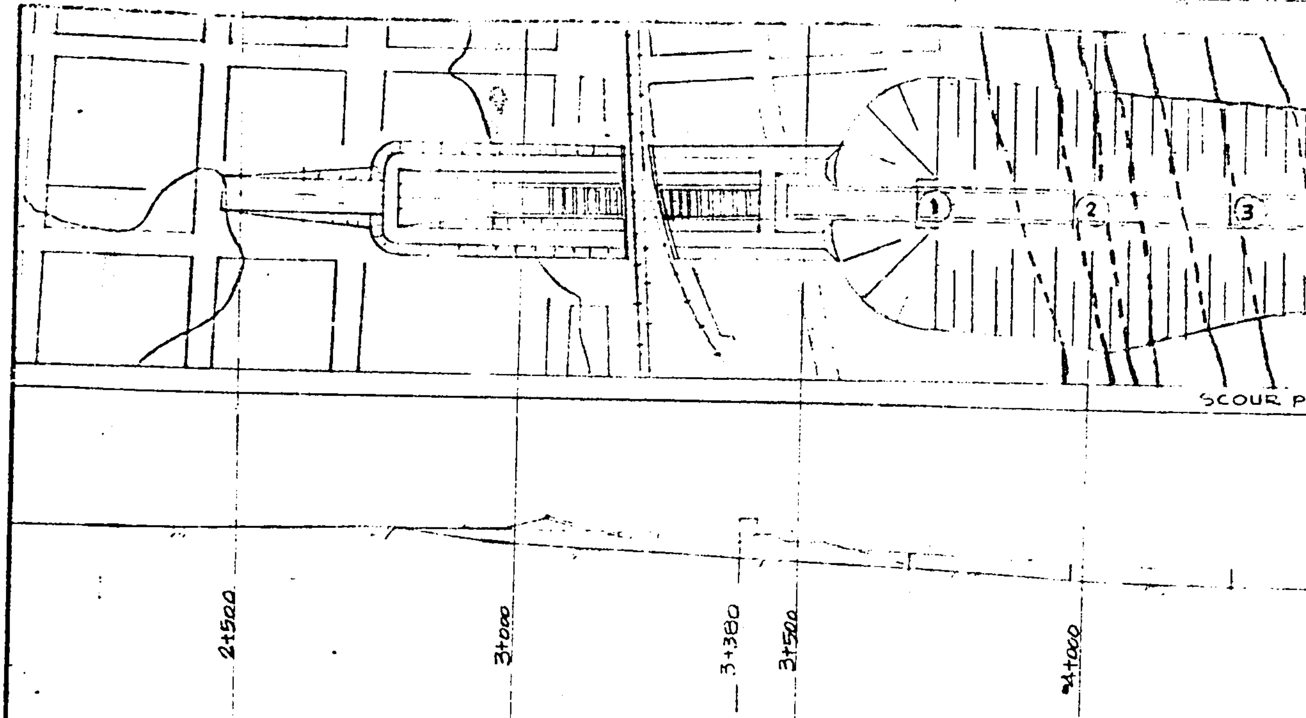
PHASE II STAGE 6

YEAR 2 MARCH - APRIL

RIVER

TUNNEL ELEMENT 1 PLACED, CONNECTED TO TAMPICO APPROACH.
PLACING OF TUNNEL ELEMENT 2 IN PROGRESS.
BACKFILL AND SCOUR PROTECTION COMMENCED.





PHASE II

YEAR 2

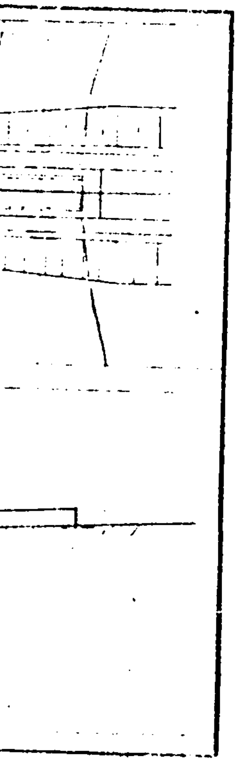
EDGE.

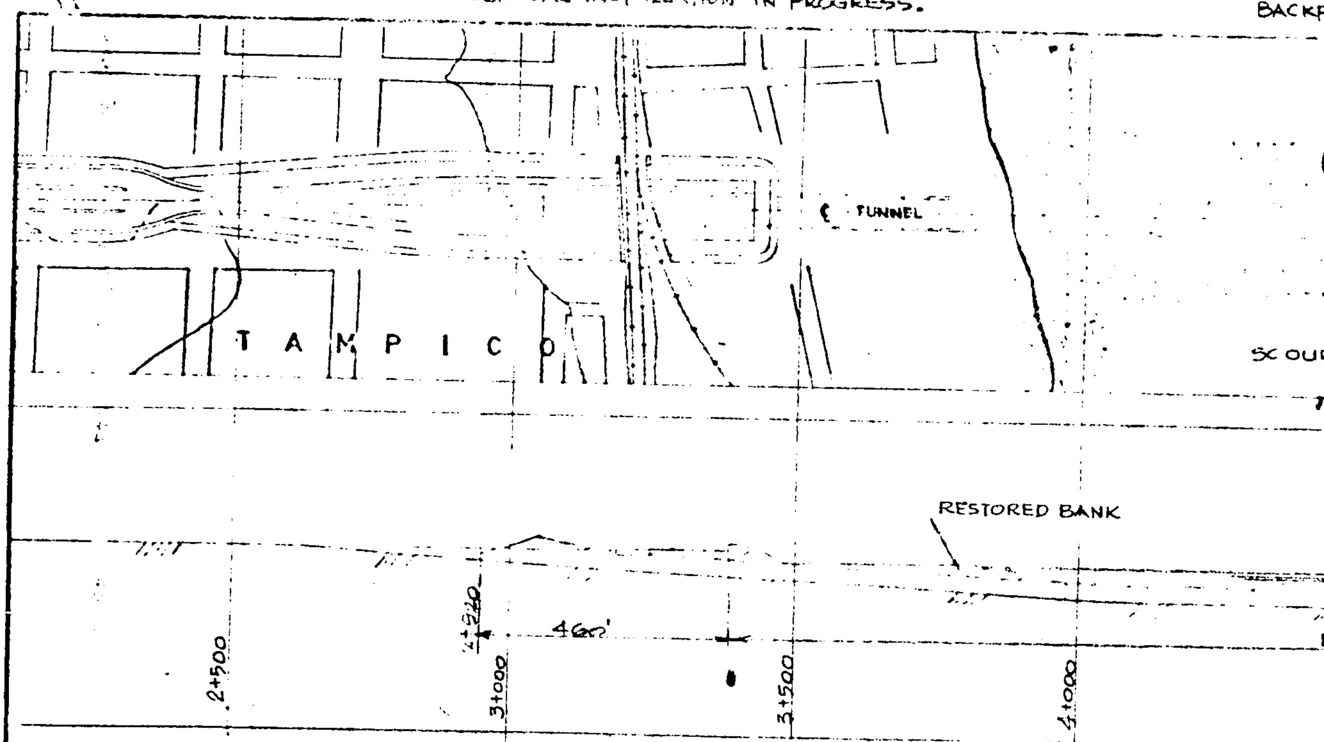
TAMPICO APPROACH

FINISHING WORK IN PROGRESS.
ROADWAY COMPLETED AND CONNECTED TO CITY STREETS.
MECHANICAL & ELECTRICAL INSTALLATION IN PROGRESS.

RIVE

ALL 5
WITH
REDO
BACKF





PHASE III

YEAR 2 J
YEAR 3 JA

TAMPICO APPROACH

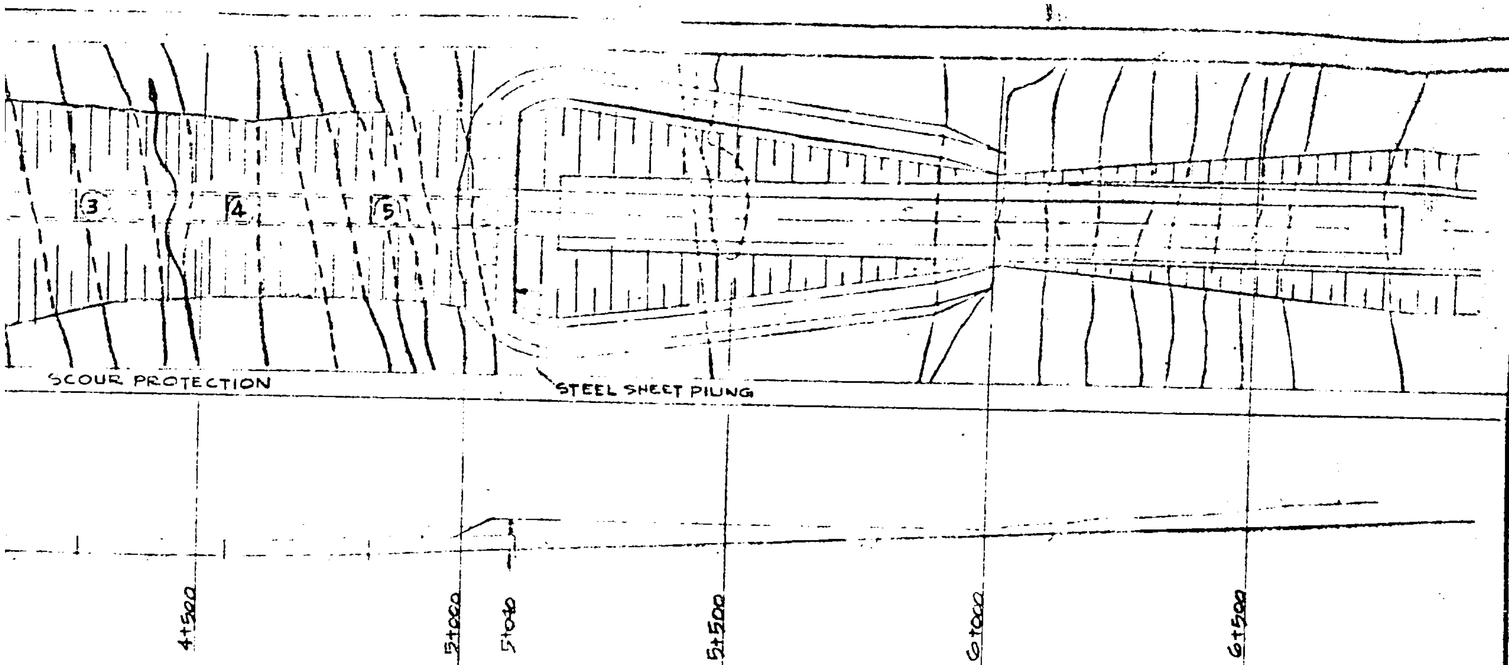
APPROACH COMPLETED, INCLUDING PROTECTIVE DYKE TO ELEVATION + 15.0
MECHANICAL AND ELECTRICAL INSTALLATIONS COMPLETED.
LANDSCAPING COMPLETED.

RIVER

MECHANICAL AND ELECTRICAL INSTALLATIONS COMPLETED.

MATA REDONDA APPROAC

APPROACH COMPLETED W
DYKE TO EL.+15.0 IN PE
MECHANICAL AND ELECTR
COMPLETED.
ROADWAY COMPLETED.
LANDSCAPING COMPLE



HASE II STAGE 7

YEAR 2 MAY & JUNE

RIVER

ALL 5 ELEMENTS IN PLACE AND CONNECTED, WITH EXCEPTION OF CONNECTION TO MATA REDONDA APPROACH.

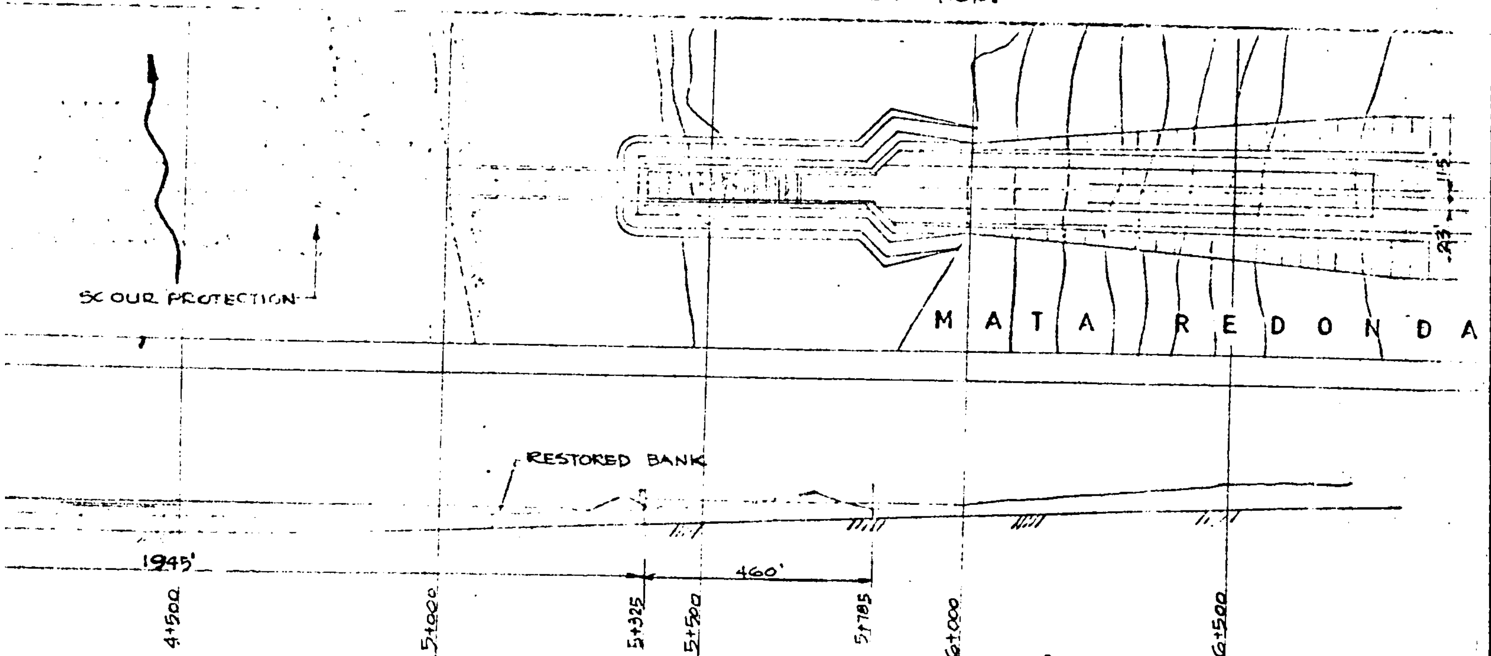
BACKFILL AND SCOUR PROTECTION COMPLETED.

MATA REDONDA APPROACH

PROTECTIVE DYKE WITH SHEET PILING RESTORED. APPROACH DEWATERED, READY FOR CONSTRUCTION OF 285' CAST-IN-PLACE TUNNEL SECTION.

0 50 100 200 300 400

SCALE IN FEET



SE III STAGE 8

YEAR 2 JULY – DECEMBER
YEAR 3 JAN. – FEBRUARY

DA APPROACH

OMPLETED WITH PROTECTIVE
+15.0 IN PERMANENT LOCATION.
AND ELECTRICAL INSTALLATIONS
D.
OMPLETED.
ING COMPLETED.

TUNNEL CONSTRUCTION

PROCEDURE 2

Fig. 11.4

much effort will be spent in examining this scheme in great detail. The reasons for the choice of the launchway scheme over the dry dock scheme willbe more apparent after review of the cost estimates in Chapter 12.

11.2.1 Phase I -- First Dry Land Construction

This phase, including 4 stages, extends over a period of 9 months from February to the end of October of Year 1. During the first 5 months of this phase (Stages 1 and 2), preparatory work is to be carried out in both Tampico and Mata Redonda, including the construction of protective dykes to permit continuing construction work in the dry throughout the following 4 months (Stages 3 and 4) of the hurricane season.

To allow a full 8 months of favourable weather for river construction, it is imperative that by the end of October, Year 1, the Tampico approach structures be completed and made ready to connect the first precast tunnel element, whereas the Mata Redonda launchway, casting yard and special plant will have been installed in time to assure completion of the 5 precast tunnel elements ready for launching.

Preparatory Work -- Mata Redonda - It is anticipated that the principal base of operations will be established on the Mata Redonda side of the river; construction roads, storage areas for materials, warehouses, shops, construction offices and probably a temporary wharf with apron facilities to transfer equipment and materials across the river by ferry between the construction sites on the two shores will be built. An office for the resident engineering staff, inspectors and test facilities will also be required on the Mata Redonda side.

Concurrently with the preparation of temporary facilities, work will be started on the excavation for the Mata Redonda approach ramp, the construction of the upper 1,000' of launchway, and the construction of the protective dyke, and consequently on dewatering and excavation within the dyke. These activities are shown in Fig. 11.3, Stage 1, Year 1, February to April. During Stage 2, Year 1, May and June, the dewatering, excavation and the construction of the launchway within the protective dyke will have been completed.

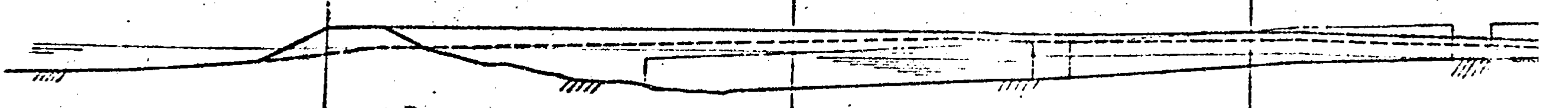
Preparatory Work -- Tampico - Initially, construction on the Tampico side will be limited to the demolition of buildings, necessary relocation of existing road facilities, addition of construction roads, temporary wharf facilities on the river bank similar to those on the Mata Redonda side, materials storage area, warehouse and construction office, all commencing April, Year 1. In the same period, temporary underpinning and shoring of the 3 railway tracks which cross the approach will be installed. Construction of the protective dyke with top at El. +15.0' which will enclose the approach area, will begin. These events can be seen in Fig. 11.3, Stage 1, Year 1.

During Stage 2, Year 1, May and June, construction of the protective dyke will have been completed. Installation of the two parallel steel sheet-pile walls and the river-end-closure wall which will be driven in the soft clay on the Tampico shore will commence. Following the excavation of the material from between the 2 sheet-pile walls, these walls will then be used as external formwork for the cast-in-place tunnel section and the cast-in-place approach structure. These efforts are shown in Fig. 11.3, Phase I, Stage 2, and Fig. 6.1.

Fabrication of Elements -- Mata Redonda Yard - The production line method for fabricating the 5 tunnel elements - each 275' long, 57' wide and 24' high - is illustrated in Fig. 11.5. The figure shows 6 progressive stages of fabrication corresponding to the tunnel construction stages shown in Fig. 11.3 and Fig. 11.4.

Various methods of fabricating the tunnel elements, besides the dry dock scheme depicted earlier, have been investigated, some in more detail than others. However, the solid sandstone formation on the Mata Redonda side, on which the bottom slab of the approach ramp can be directly supported, favours the launchway method which is a practical one considering the relatively small size and weight of each element - approximately 12,000 tons gross. This method is also highly economical as it can be combined with the procedure for launching, transporting and placing the submerged elements which are properly trimmed and ballasted to a maximum negative buoyancy of roughly 350 tons in fresh water, by means of the mobile portal cranes modified for this purpose.

The launchway consists of a reinforced concrete slab, 80' wide and 1475' long, constructed to a grade of 5%. The

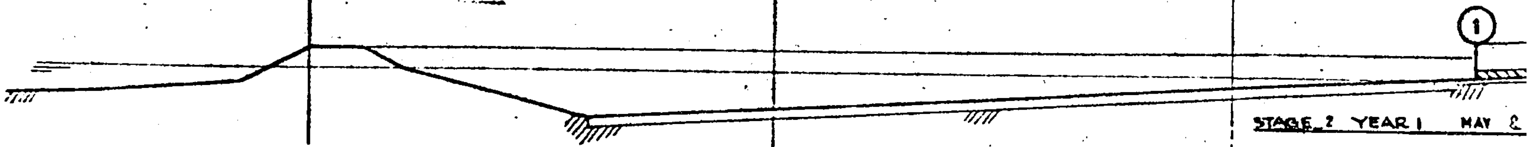


STAGE 1 YEAR 1 F

UPPER 1,000' OF LA
TWO PORTAL CRANE
FOR OPERATION.

PROTECTIVE DYK
EXPLAINED UNDER M
APPROACH STAGE I

DEWATERING, EXCA
OF LOWER END OF L

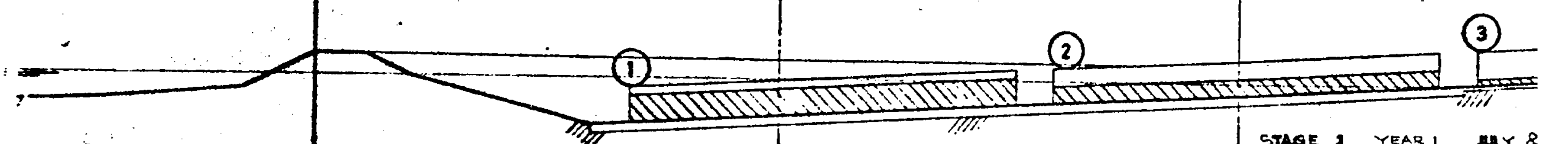


STAGE 2 YEAR 1 MAY &

DEWATERING & EXCAV
PORTION OF MATA REDON

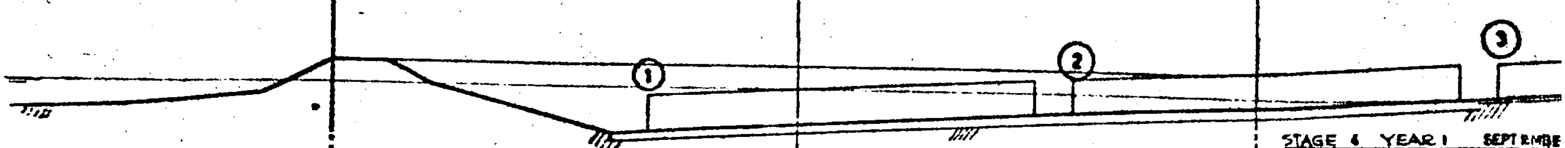
APPROACH BOTTOM
APRONS EXTENDING TO

BOTTOM SLABS FOR
COMPLETED & READY T
END OF LAUNCHWAY.



STAGE 3 YEAR 1 JULY &

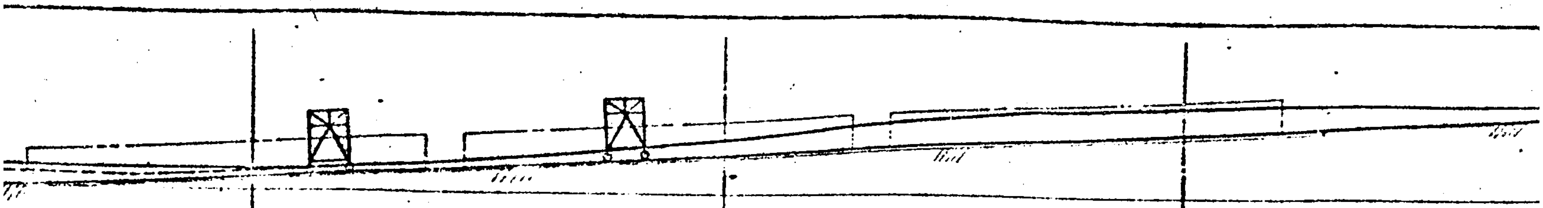
ELEMENT 1 COMPLETED
ELEMENTS 2 & 3 PAR
BOTTOM SLAB OF ELE



STAGE 4 YEAR 1 SEPTEMBE

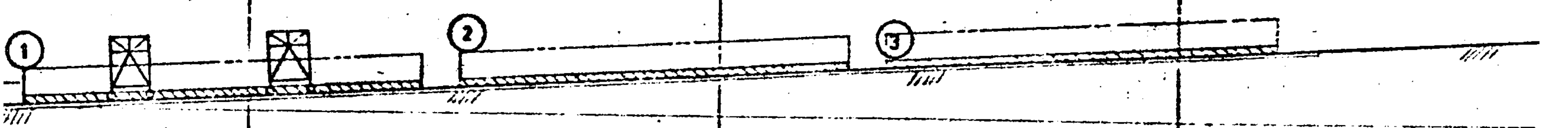
FIVE ELEMENTS COMPL
FOR CONTROLLED LAUNC

PORTAL GANTRIES CO
USE IN OUTFITTING ELEM
WORK.



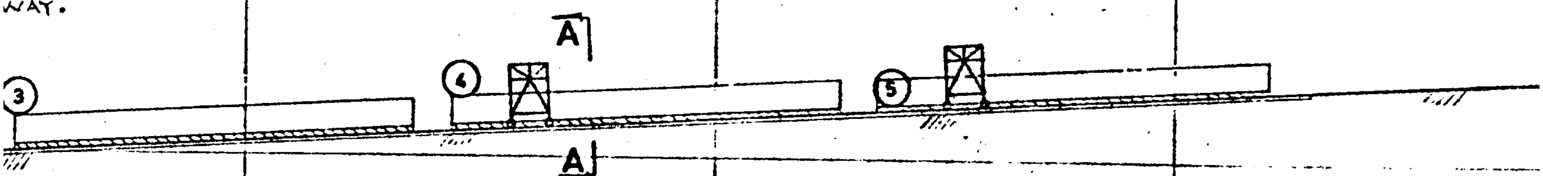
FEBRUARY – APRIL

OF LAUNCHWAY COMPLETED.
CRANES ERECTED, READY
N.
E DYKE IN PLACE AS
IDER MATA REDONDA
TAGE 1.
3, EXCAVATION & CONSTRUCTION
) OF LAUNCHWAY IN PROGRESS.



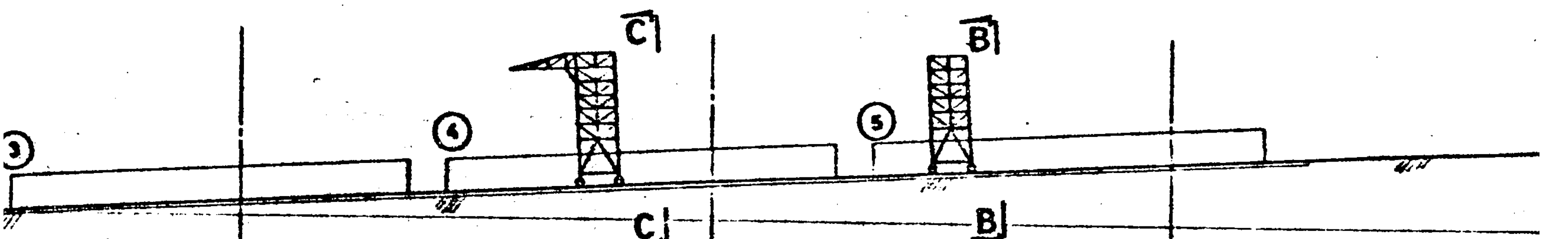
MAY & JUNE

. EXCAVATION OF LOWER
REDONDA APPROACH COMPLETED.
OTTOM SLAB CONCRETED WITH
NG TO FULL WIDTH OF LAUNCHWAY.
S FOR ELEMENTS 1, 2 & 3
EADY TO MOVE TO LOWER
WAY.



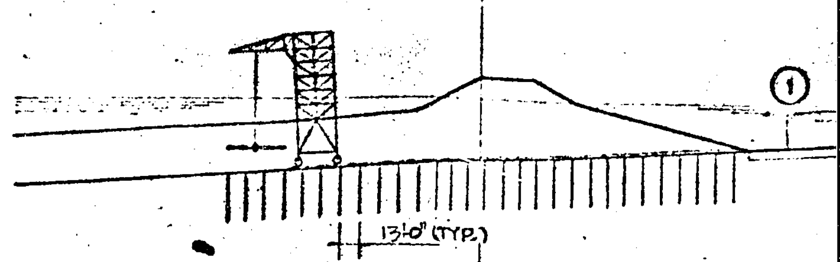
JULY & AUGUST

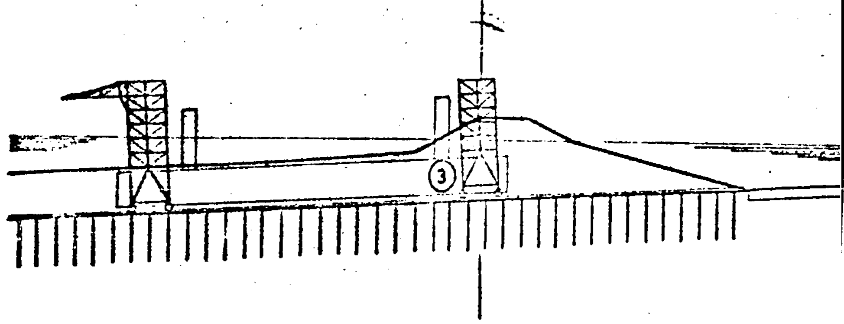
IPLETED.
3 PARTIALLY COMPLETED.
OF ELEMENTS 4 & 5 COMPLETED.

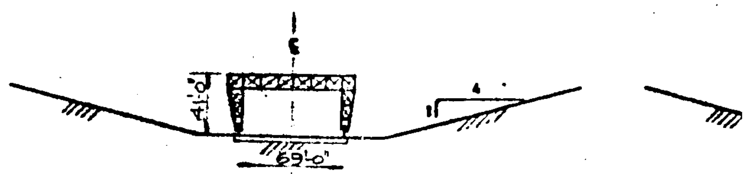


EPTEMBER & OCTOBER

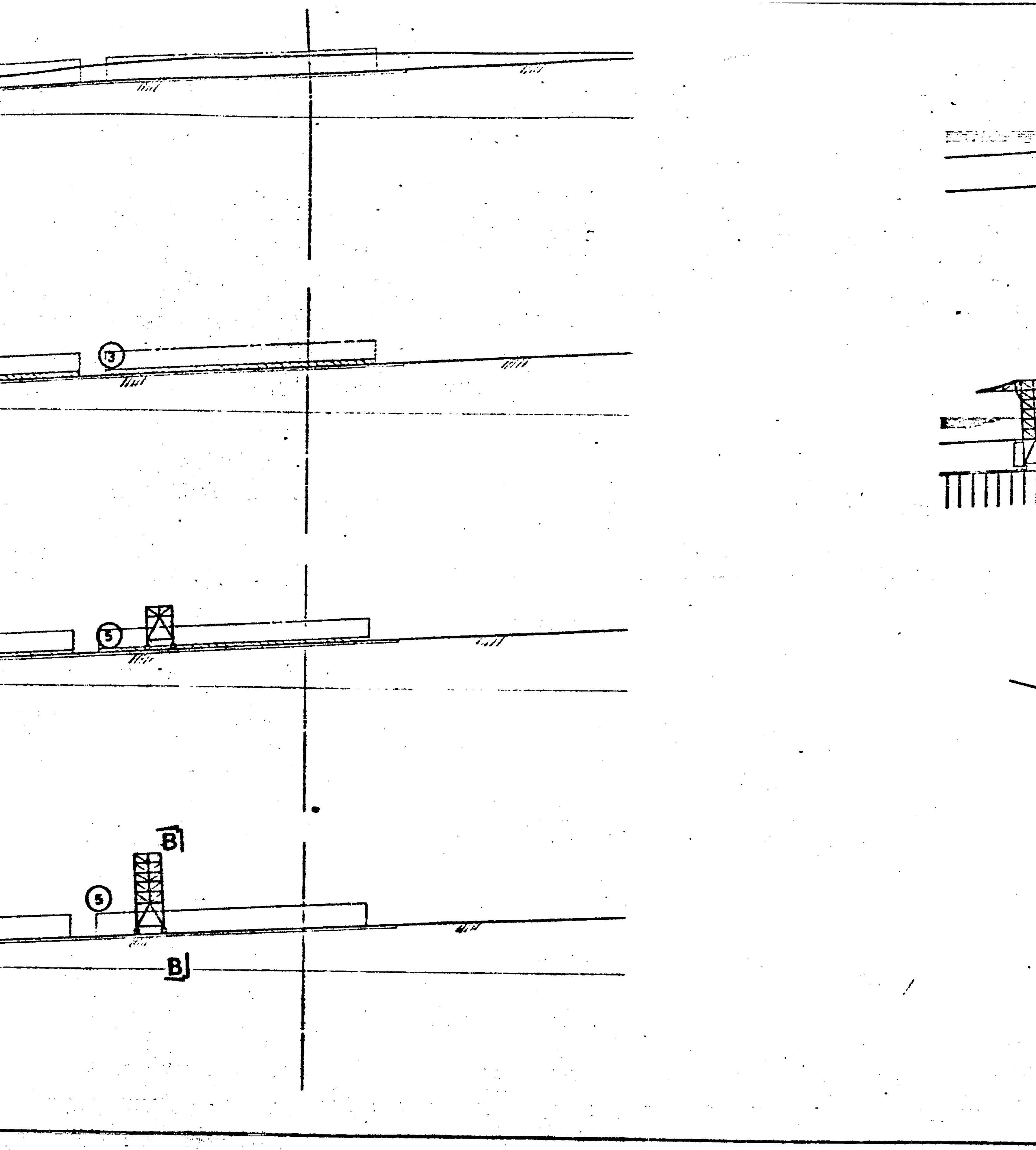
COMPLETED & READY
LAUNCHING.
ES CONVERTED FOR
ELEMENTS & FOR RIVER



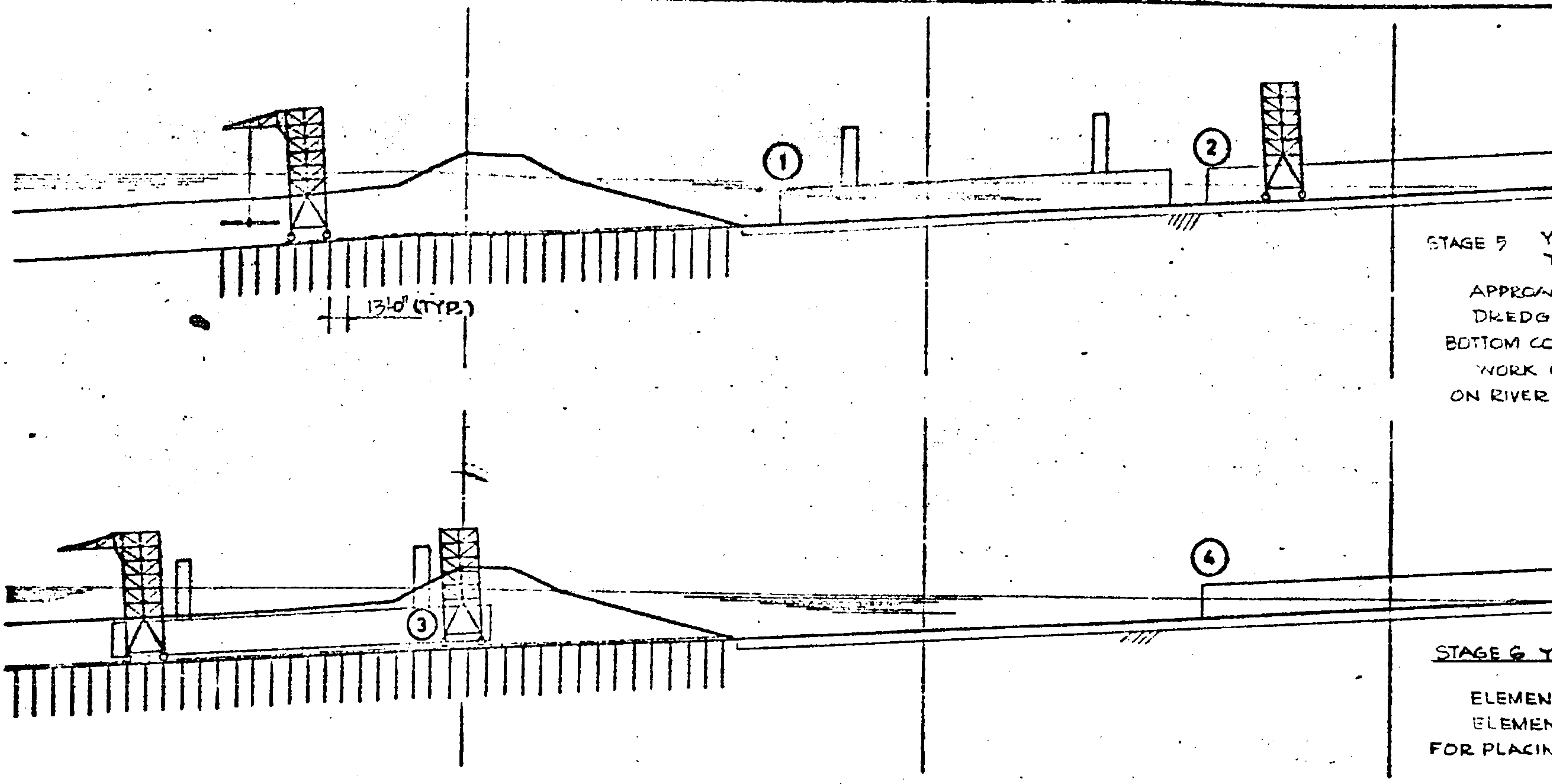


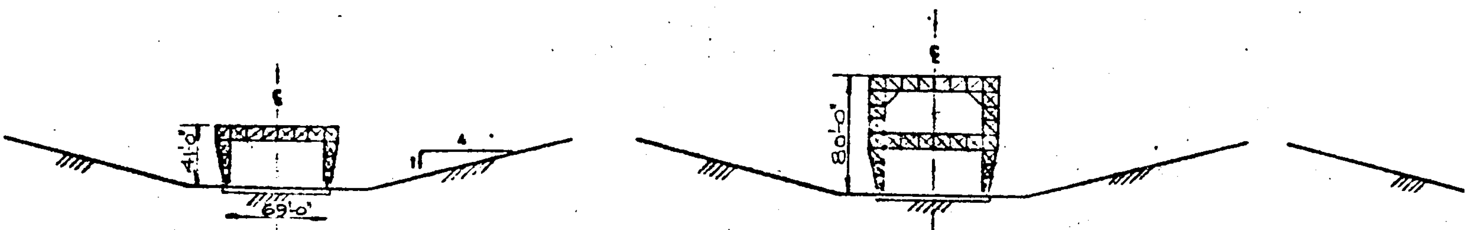


SECTION A-A



13
5
5
B
B

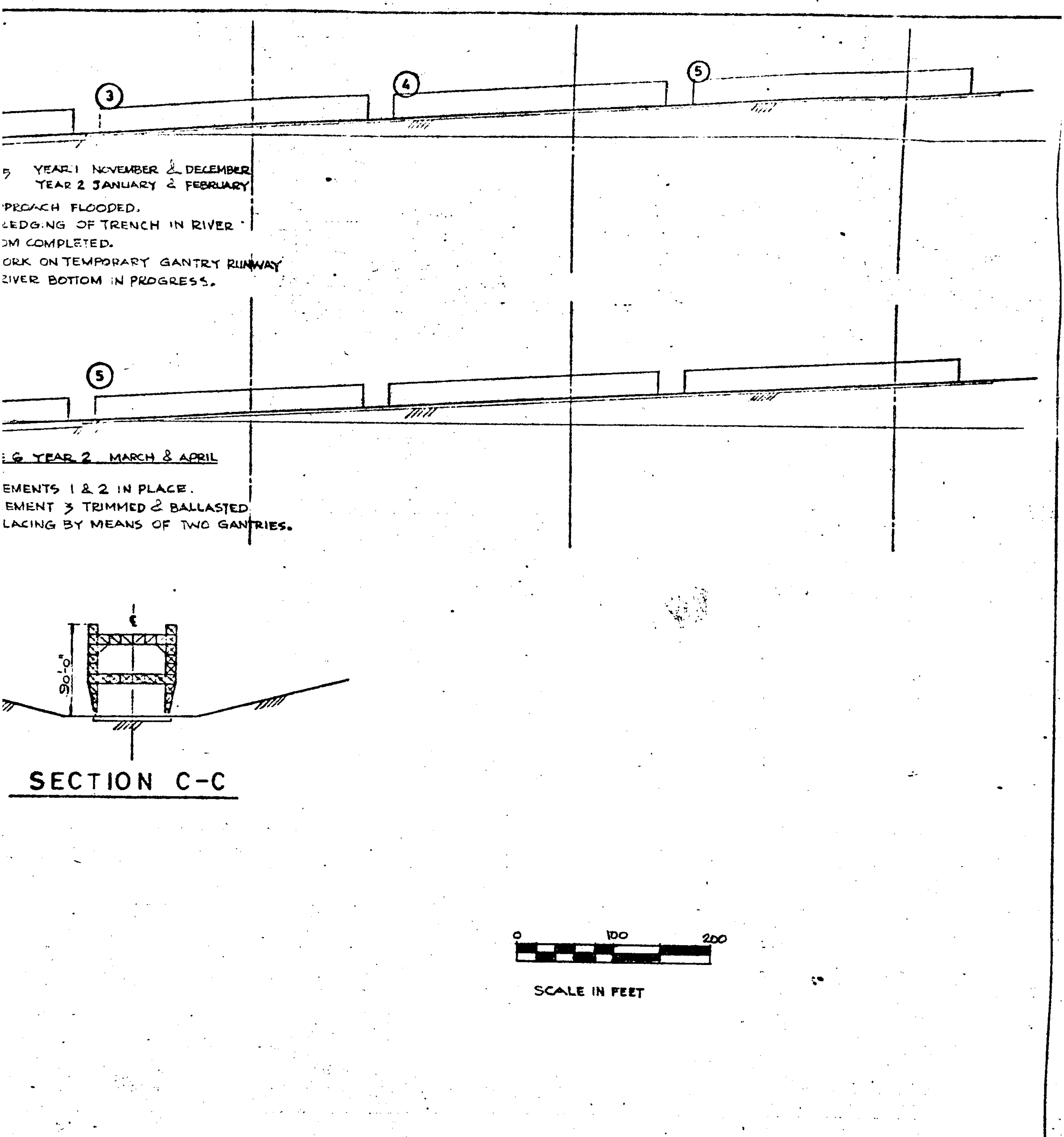




SECTION A-A

SECTION B-B

S



FABRICATION STAGES OF THE SUBMERGED TUNNEL ELEMENTS

Fig. 11.5

concrete slab is necessary to smooth out the sandstone surface, to distribute the roller loads uniformly to the sandstone and to reduce seepage of water caused by the artesian pressure of the sandstone layer. The upper 820' with an 8" slab thickness, will ultimately serve as part of the approach roadway. It lies above flood level and therefore, can be constructed in the dry. Excavation for this section will be in sandstone overlain by about 3' to 7' of soft clay. Excavation can be performed by ripper and scraper but some of the sandstone may require blasting.

The lower 655' of the launchway with slab thickness varying from 3'-3" to 4'-0" slopes downward to a maximum depth of 33' below low water level. This will necessitate the progressive dewatering and excavation of clay followed by the construction of the launchway slab, all within a protective cofferdam. This section of the reinforced concrete launchway will ultimately form, in part, the bottom of the in-situ tunnel section. The much thicker slab is therefore necessary to satisfy strength and stability requirements.

Immediately after the construction of the upper 1,000' of the launchway, the fabrication of the bottom slabs for 3 of the 5 precast tunnel elements will start. Three compacted sand beds 8" thick and approximately 60' x 280' in plan area spaced over the 1,000' launchway will first be prepared. The sand cushion will be transversed by 8" x 1'-6" openings spaced 16'-6" apart. These 60' long openings are to be formed by inverted U-shaped steel sheets into which a total of 120 roller skids per element each of 100-ton capacity will be inserted. The skids are also in the order of 8" in height so that they are just short of touching the bottom of the element slabs to be cast. The weight of the slab will initially be supported by the compacted sand bed. However, when the slabs are ready to be moved down to the bottom of the launchway to make room for the fabrication of the other 2 elements, the supporting sand beds will be washed away by high pressure water jets thus permitting the slabs to rest on the roller skids spaced evenly along the openings. The U-shaped steel sheets act only as forms to contain the roller skids.

The movement of the slabs and later, the completed elements, down the 5% grade must be controlled. For this purpose, rock anchor clusters spaced approximately 200' apart and about 12' above the slab and throughout the whole length of the launchway will be installed into the side slopes of the sandstone formation. A block and tackle system will be provided at each rock anchor cluster. Lines will be extended to connect to the slabs or the finished elements to be moved. During the actual moving process, 4 lines, 2 on each long edge, will always be supporting the specimen. By means of winches, the lines will be gradually let out to enable the element to roll down the 5% incline. The total capacity of the 4 lines and consequently, the 4 rock anchor clusters engaged must be greater than the tangential component, which amounts to approximately 600 tons, of the dead weight of the finished element.

The above movement control system will have been installed simultaneously with the fabrication of the bottom slabs of the first 3 elements and completed in time for the move.

The 2 downhill bottom slabs with a total of 240 roller skids will first be moved about 600' down to the bottom of the launchway, guided by the tackle lines, for further concreting. Once at their predetermined locations, the first slab will be jacked up and supporting stiffened steel beams inserted 16'-6" apart under the slab. This way, the rollers can then be retrieved and used to move the third slab. After the second and third elements have been moved and sunk, the 240 rollers are retrieved and inserted into the openings of the sand beds of the final 2 elements for moving down to the bottom of the launchway. The bottom slabs of the 2 last tunnel elements will be fabricated on the upper part of the launchway at the same time as the walls and tops of the first 3 elements are built on the lower part of the launchway.

These last 2 elements will later be moved down one after the other to the same spot that the first element now occupies, before being picked up by the gantry cranes for sinking. Each element must pass through this position after the protective dyke is breached in order to change the 12,000-ton element dry weight into a 350-ton element submerged weight for easy gantry pick up. This weight is less than that proposed for the sinking rig scheme because the submerged gantries are more stable against force fluctuations than floating equipment and hence less weight required for stability.

The procedure for completing the fabrication of all

5 elements, as shown in Fig. 11.5, allows for a continuous operation by the use of collapsible movable frames for the support of precast or prefabricated belt-forms described below.

Each 275' element will be formed and concreted in 5 sections 53' long with 2'-6" gaps between the sections; the gaps to be completed with an expanding concrete after a period of shrinkage of the 5 sections. The interior forms will comprise curved precast tubular forms for the ventilation ducts and emergency pedestrian passageways, removable steel forms for the ceiling and corrugated transite forms for the upper sections of the interior walls.

The schedule allows a total period of 4 months for completing the fabrication of the walls and top slabs of the 5 elements above the previously completed bottom slabs.

In reinforcing the elements, an interesting feature is the provision of the temporary vertical tendons between the top and bottom slabs. These tendons, post-tensioned after a period of curing and used to prevent the tunnel cross section from bulging due to the large transverse prestressing force, will be burned off gradually as the elements are being submerged and subjected to increasing hydrostatic pressure.

After the fabrication of the structural cross section is completed, temporary end bulkheads required for submergence of the element, neoprene gaskets along the perimter of the tunnel and hydraulic jacks and couplers for effecting end connections to adjoining tunnel units will be installed.

Special Equipment -- Mata Redonda - For the fabrication of the 5 elements on the launchway, 2 portal cranes will be used. These cranes will span 70' in width and provide a vertical clearance of 30' above the launchway slab.

Following the completion of the elements near the end of Phase I, Stage 4, the cranes will be modified as illustrated in Fig. 11.5 for use in the river operations. One crane will be equipped to perform the following 3 functions:

(1) Drive ahead of the crane 2 parallel double rows of timber piles across the river with their tops to be cut off accurately 2'-3" below the profile of the tunnel foundation. The double rows will be at 70' centres, equidistant from the longitudinal centreline of the tunnel. The piles in each row will be spaced transversely at 5'-3" centres and longitudinally at 13' centres.

(2) Place precast concrete runways, 40'long, on top of each double row of piles. Each concrete runway, with one end mortised to the previously placed 40' section, will then rest on 6 timber piles.

(3) Dump sufficient well-graded gravel to completely fill the space between the concrete runways.

The other crane will be modified to perform the following operations:

(1) Screed the foundation fill dumped by the first crane to the profile of the foundation using the

concrete runways as screed guides.

(2) Place a uniform layer of compressible sand-asphalt on top of the screeded gravel fill.

In addition to the foregoing duties, the 2 portal cranes working in tandem will be used for transporting and lowering the 5 submerged tunnel elements into their final position. This operation requires a vertical lifting power of 100 tons applied at each of 4 points on the element. These points are 56' from each end and 20' each side of the longitudinal centreline. The spacing of the support points results in approximately equal positive and negative bending moments at midspan and supports of the element respectively. The submerged weight of an element will be about 350 tons maximum. Two cylindrical access shafts, 3'-3" in diameter, will be erected on the top slab of the element when the gantries are in place to pick up the element.

Construction of Tampico Approach - When the steel sheet-pile cofferdam built within the protective dyke on the Tampico shore has been dewatered, the construction of the 285' cast-in-place tunnel section and the 460' approach structure may proceed to completion, as explained above for the fabrication of precast elements. These steps can be seen in Fig. 11.3, Stage 4.

The lower end of the in-situ tunnel section has to be closed by a watertight bulkhead to avoid flooding the approach and to provide the same type of joint between it and the adjoining precast tunnel element as between other abutting submerged elements. This operation will terminate all Phase I construction operations on the Tampico side at the end of the hurricane season late in October of Year 1.

11.2.2 Phase II -- Subaqueous Operations

This phase, including Stages 5, 6 and 7, extends over a period of 8 months between the hurricane seasons of Year 1 and Year 2, when climatic conditions favour construction works in the river. During this phase, it is necessary to complete all subaqueous construction, including dredging of the cross river trench for the tunnel, preparation of foundations, placing and connecting of 5 precast elements, rehabilitation of river banks and protective dykes and placing of scour protection.

Dredging of Trench for Tunnel - At the end of October, Year 1, dredging of the cross river trench for the founding of the 5 precast tunnel elements will be commenced. As a total quantity of approximately 730,000 cu. yd. of material must be removed within a period of 3 months or less, this operation requires the use of an hydraulic cutter dredge with a capacity of 260,000 cu. yd. per month at a depth ranging from 30' to 65'. Pumping capacity is required to dispose of the spoil by pipeline to an area being reclaimed in a lagoon northwest of the Tampico approach.

The maximum depth of the trench will be 72' for a navigation channel 40' deep and 328' wide. Dredging below 65' depth may require special equipment which may not be available locally should the rate of production of local clam shell dredges be inadequate to meet the construction schedule.

A bottom width of the trench of 80' or more is required for placing the tunnel elements. The length of the trench will be 1,375', the overall length of 5 precast submerged elements, between the cast-in-place tunnel section of the Tampico shore and the bottom slab of the cast-in-place section on the Mata Redonda shore.

Dredging River Banks - Included as part of the dredging operation is the removal of short sections of the river banks and the portion of the temporary protective dykes parallel to the river, to expose the lower end of the cast-in-place tunnel section at Tampico and thus provide space for the later connection of the first tunnel element, and to expose the lower end of the bottom slab of the in-situ tunnel section at Mata Redonda to permit the mobile gantry cranes to proceed with the construction of temporary runways and the tunnel foundation across the river. The protective dykes should not be breached until the enclosures have been flooded to river level. This procedure is again to avoid the sudden inflow of water. See Fig. 11.4, Stage 5.

Placing and Connecting of Tunnel Elements - Following the completion of the foundation for the tunnel elements, the 2 gantries will move to the submerged part of the launchway on the Mata Redonda approach to pick up Element No. 1, which will first be ballasted to a submerged weight of 350 tons in fresh water and trimmed to a grade corresponding to that of the foundation upon which it will be placed. Suspended from the 2 gantries, the submerged element will be transported across the river, lowered into final position on the foundation and joined to the cast-in-place tunnel section. The joining of the tunnel elements is similar to the procedure described before for the dry dock scheme.

As before, each element is closed off by temporary bulkheads set back 2'-6" from each end, designed to withstand an hydrostatic pressure corresponding to the depth of water in which the element is placed on the prepared base at the bottom of the dredged trench.

The ventilation ducts are temporarily divided into 8 watertight compartments by means of temporary bulkheads. Water ballast can be pumped in and out of these compartments to compensate for changes in density caused by variations in salinity and to accurately control longitudinal and transversal trim of the element during placing operations.

The element is brought to within 8" of the end of the previous element which is provided with a protruding ledge at the bottom designed to support the abutting end of the new element and to assure its correct vertical and horizontal alignment. The outer end of the new element remains suspended by the gantry until closure is achieved. The elements are then connected by means of 2 automatic train couplers activitated by hydraulic jacks capable of pulling the elements together with a force corresponding to 1,000 lbs/ft. of perimeter of the element. The jacking operation compresses the neoprene gasket fitted to the end of the element being placed and thus seals the abutting ends of the two elements forming a 5'-0" watertight compartment between the bulkheads. By draining the water from the compartment to the inside of the element which is equipped with air and access shafts, the pressure between the bulkheads is quickly reduced to atmospheric pressure with the result that the full hydrostatic pressure acting on the free end of the element drives the 2 units tightly together and further compresses the neoprene gasket providing a safe water seal.

The first element must be placed in March, Year 2 and remaining 4 at 2-week intervals, so that scour protection can be completed by the end of June, Year 2, prior to the hurricane season.

When Element No. 2 is in place and the joint between Element No. 1 and the cast-in-place tunnel section has been sealed by a secondary watertight rubber membrane stretched over the joint and fastened to the concrete, it will be safe to remove the temporary bulkheads from this location. Thereafter, a concrete shear ring will be poured over this rubber membrane, thus forming a flexible joint as suggested in Chapter 7 for better waterproofing.

The above procedure is to be repeated for the remaining 4 joints between elements. The last joint between Element

No. 5 and the cast-in-place tunnel section of the Mata Redonda approach will require special treatment such as jacks to maintain compression in the tunnel and will normally have to be sealed from the outside to facilitate concreting.

Placing of Scour Protection and Rehabilitating of River Banks - Backfill and scour protection will be placed over Element No. 1 when Element No. 3 is in place, to give the first 2 units time to settle into permanent position on the compressible sand-asphalt base.

Work on the rehabilitation and regrading of the Tampico River bank, and the subsequent placing of scour protection, can be commenced immediately after the completion of scour protection for Element No. 1.

The arrangement of backfill and scour protection for the tunnel is shown in Fig. 9.1 and scour protection of the river banks is shown in Fig. 9.2. As indicated, a sandfill is placed on each side of the tunnel before the filter cloth, temporarily stored in rolls on top of the tunnel element, is picked up by a barge and lowered on top of the sandfill over 100' wide areas on each side of the element and held in place at the outer ends by concrete anchor beams. Gravel, rockfill and armour stones are dumped in sequence on the filter cloth and on top of the tunnel as shown in Fig. 9.1. A similar procedure is followed for the placing of scour protection on the river banks, as illustrated in Fig. 9.2.

Supplementary Cofferdam -- Mata Redonda - The connection of the fifth element to the cast-in-place Mata Redonda tunnel section is to be made in the dry. Fig. 11.4, Stage 7 illustrates this step.

When Element No. 5 has been sunk, the river bank will be restored thus closing off the launchway from the river. To avoid piping along the element, a sheet-pile wall is to be placed against a collar around the lower end of the element in order to assure watertightness of the cofferdam which forms a part of the restored river bank.

The scour protection along the whole Mata Redonda side should be placed by the end of the dry season. The reconstruction of the cofferdam closing the launchway, and the installation of scour protection on the Mata Redonda side, terminate work scheduled for completion under Phase II by the end of June, Year 2.

11.2.3 Phase III -- Second Dry Land Construction

This phase extends from July 1 of Year 2 to the termination of all construction work ready for the opening of the tunnel sometime between March and the end of June of Year 3.

The works of this phase (Stage 8) can be performed during and after the rainy season, subsequent to the installation of the 5 submerged elements.

Construction of Mata Redonda Approach - The reconstructed cofferdam will be dewatered to complete fabrication of the tunnel section and the approach structure in the dry on the bottom slabs used as parts of the launchway. As there will be no possibility of maintaining hydrostatic pressure against the end bulkhead of Element No. 5 following dewatering, the element has to be kept under compression by flat jacks so as to avoid any relaxation and leaking of the joint between Elements Nos. 4 and 5.

Upon completion, the cast-in-place tunnel section and a part of Element No. 5 will be buried in the restored river banks.

Installation of Electrical and Mechanical Equipment - At any convenient time following completion of the Tampico approach structures and ventilation building, but not later than early June of Year 2, the installation of the substation, lighting system, electrical equipment, ventilating fans, pumping stations, electrical controls, etc., may be commenced. The installations are to be scheduled to suit remaining construction works, so that the equipment may be tested and be ready for operation by the end of Stage 8.

Paving of Roadway Surfaces - The tunnel and approach roadways within the limits of the project are to be paved with asphalt to comply with the standard highway specifications.

Finishing and Clearing of Site - It is contemplated that this work will be continued progressively throughout the period of construction and be finally completed by the end of Year 2.

Landscaping - The rehabilitation and improvement of the construction site, sodding and planting of shrubs on the banks of the protective dykes and other such works, preferably will be completed by the end of Year 2, but not later than the official opening ceremonies scheduled for June of Year 3.

11.3 Comparison Between Construction Schemes

The detailed breakdown of the estimated cost of the dry dock and launchway construction schemes described above will be shown in Tables 12.1 and 12.2 respectively. From the comparison of these two Tables which will be elaborated in Chapter 12, the reasons for the selection of the latter scheme for casting, sinking and founding will become apparent.

As far as construction time is concerned, both schemes have been geared to the short 8-month hurricane-free period in which work can be performed favourably in the river. Assuming that Year 1 begins at the awarding of the construction contract in January, both schemes will enable the tunnel to be opened to traffic in the middle of Year 3. Essentially, in the 8 months of the year that are hurricane-free, construction is so scheduled that all river works will be in full swing. Once the hurricane season sets in, all construction efforts again move back to both shores. It is vital to maintain the dates for the completion of the various phases of work as scheduled in order to have the tunnel inaugurated on time.

Two important features differ between the two construction schemes. Firstly, the casting method of the dry dock scheme requires the construction of a dry dock or casting basin with the top of the protective dykes at El. +15.0' to prevent inundation should a flood equal to the 1955 one recur. After the 5 tunnel elements have been towed out of the dry dock, it might have to be backfilled and regraded to the original ground level for future development of the region unless Mata Redonda plans to use the basin as a marina for pleasure vessels. Although this problem of backfilling does not exist in the launchway method of casting the tunnel elements, the launchway must still be guarded against inundation by protective dykes. In addition, wider excavation and extra concreting of slab for 1,050' are necessary in order to accommodate the fabrication of the 5 immersed tunnel elements. This provision results subsequently in a wider roadway after the tunnel is completed. Nevertheless, the cost of the latter method of casting is much less than the cost of casting by the method of the dry dock scheme as will be apparent in the following chapter. Secondly, the dry dock scheme necessitates the use of floating equipment and laser beams in the transportation and sinking of the tunnel elements and consequently causes more interference to the shipping lane. The one-million dollar expenditure on the specially-built sinking rig is an expensive item for such a relatively short and narrow tunnel. The launchway method of construction comprises mainly underwater work and eliminates all major floating equipment. This feature induces less interference with the navigation channel. Because the elements are brought out to the site by portal cranes running on rails, positioning the elements for sinking becomes less a problem and laser beams for alignment purposes may be eliminated. It is possible that the precast concrete rails upon which the portal cranes travel will be clogged by sediment depositions and continuous cleaning of the rails may have to be effected.

As a whole, both schemes of construction have their advantages and disadvantages but the major factor influencing the decision to choose the launchway scheme was its much lower cost.

CHAPTER 12

COST ESTIMATES

The need for the construction of any river crossing arises from traffic requirements necessitated by the connection to the national highway system or local traffic volume increase due to present or future development of either or both of the two regions connected by the crossing. Hence, the functional aspects to the regions concerned determine the necessity of a crossing. But it is the economic aspects of the crossing structure which decide the building of this structure. Therefore, a cost figure is always attached to any pre-investment or feasibility investigation or preliminary design study.

The accuracy of the cost figure in preliminary design stages is only within 10-15%. However, be it as approximate as it is, the figure still gives an excellent indication of the range of cost of the actual structure. It usually happens that the estimated cost must be revised upwards with the lapse of time due to capital depreciation, interest rate appreciation, labour and material cost increase, etc. These considerations are fields of interest to the specialists of cost trend forecasting, a relatively new field. Nevertheless, cost figures based on current prices will give a fair indication of the cost of the structure and with a reservation factor, will suggest an approximate cost for the future.

The cost estimates which follow have been established on a unit price basis, according to the level of prices currently prevailing in the heavy construction industry. No provision has been made for escalation or for interest charges during construction.

The quantities shown are estimated net quantities of materials essential to and remaining in the work, and the estimated measures of services necessary to the execution of the work.

The unit prices and lump sum amounts shown provide due allowances for unavoidable waste and for all costs relating to materials furnished and work performed, including labour, equipment, consumables, taxes, contractors' overheads and profits and financing, and appropriate contingency allowances averaging approximately 10%.

The estimate establishes the "order of magnitude" capital costs of the civil, electrical and mechanical engineering works for the complete tunnel project. They do not include, however, the costs of engineering design, supervision or construction management.

The total estimated costs in Canadian dollars for both the dry dock and launchway construction schemes are summarized in Tables 12.1 and 12.2 respectively for a 40' deep navigation channel for uncontrolled river flow.

Inspection of Tables 12.1 and 12.2 indicates that the cost of civil work, excluding the fabrication of the tunnel elements, on the Mata Redonda shore for the dry dock scheme and launchway scheme is approximately 4.0 and 2.32 million dollars respectively. This difference represents a saving of 70% by the use of the launchway scheme. The cost of fabrication for the 5 tunnel units is 3.11 and 3.06 million dollars for the dry dock and launchway scheme respectively. The difference exhibited under this item is relatively insignificant.

In the placing operations, the dry dock scheme, which necessitates the use of a sinking rig, costs approximately 1.83 million dollars while the launching of the elements totals only 1.03 million dollars. This difference represents another saving of almost 80%.

In the comparison of the various items above, it must be kept in mind that the precast elements of the dry dock scheme are longer and the cast-in-place tunnel section shorter than those of the launchway scheme. Nevertheless, the figures are fairly representative of comparisons based on equal grounds.

As a whole, the total difference between the two casting, sinking and founding methods is in the order of 2.25 million dollars. The total cost of the project being in the order of 11.3 million dollars, excluding the 10% contingency, the above figure represents a saving of approximately 20%.

Based on this finding, it seems obvious that the selection of the launchway scheme was justified.

Table 12.1

ESTIMATED COSTS OF DRYDOCK SCHEME

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
1. MATA REDONDA				
Drydock & Approach -				
Temporary protective dyke	c.y.	32,800	1.15	37,720
Earth excavation	c.y.	428,000	0.90	385,200
Rock excavation	c.y.	389,660	4.35	1,695,000
1.5' gravel base	c.y.	20,800	3.00	62,400
1' gravel slope protection	c.y.	7,280	3.00	21,840
1' sand slope protection	c.y.	7,220	2.60	18,770
Drainage, operate & maintain	sum			50,000
Wellpoints, "	sum			200,000
Concrete	c.y.	9,910	28.70	284,420
Reinforcing steel	tons	720	343.00	247,000
Formwork	s.f.	62,000	1.35	83,700
Precast forms for ducts	ft.	250	97.00	24,250
Emergency stairs & ventilation bldg.	sum			45,000
Sun screen & grid system	s.f.	12,800	1.45	18,600
Fitting-out jetties	c.y.	24,000	1.15	27,600
Flooding & dewatering	sum			100,000
Excavate exit channel	c.y.	83,000	0.90	74,700
Backfill approach	c.y.	63,000	1.15	72,450
Backfill drydock	c.y.	512,000	0.90	460,080
Rebuilt permanent dyke	c.y.	12,600	1.15	14,490
Surface drains	sum			21,000
Finishing & landscaping	sum			72,800
				4,018,020

Table 12.1 - cont'd

	Item	Unit	Quantity	Unit Price	Total Cost (dollars)
2.	TAMPICO				
	Temporary protective dyke	c.y.	60,500	1.15	69,580
	Steel sheet cofferdam	tons	510	437.00	222,870
	Excavation & bracing	c.y.	82,000	2.10	172,200
	Progressive dewatering	sum			77,500
	Preparation of base	s.f.	30,000	0.09	2,700
	Concrete	c.y.	9,000	28.70	258,300
	Reinforcing steel	tons	450	343.00	154,350
	Formwork	s.f.	32,700	1.35	44,150
	Precast forms for ducts	ft.	250	97.00	24,250
	Operate movable form support frames	each	27	410.00	11,070
	Emergency stairs & ventilation bldg.	sum			87,000
	Railway bridge	s.f.	2,730	17.00	46,410
	Sun screen & grid system	s.f.	12,830	1.45	18,600
	Backfill	c.y.	78,200	1.15	89,930
	Displace protective dyke	c.y.	21,430	1.15	24,650
	Surface drains	sum			21,200
	Finishing & landscaping	sum			72,800
					1,395,560
3.	FABRICATION OF ELEMENTS				
	Hot asphalt coat	s.f.	97,300	0.22	21,410
	1/8" bottom steel membrane	tons	256	760.00	194,560
	Concrete	c.y.	30,000	28.70	861,000
	Reinforcing steel	tons	900	343.00	308,700
	Formwork	s.f.	184,500	1.35	249,080

Table 12.1 - cont'd

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
Precast forms for ducts	ft.	3,290	97.00	319,130
Supply & operate movable form supports	sum			65,000
Prestressed cables (in place)	tons	580	550.00	319,000
Anchors for prestressed cables	each	2,090	40.00	83,600
Waterproofing & abrasion protection of walls	s.f.	75,000	1.25	93,750
Waterproofing roof	s.f.	80,000	0.50	40,000
8" protective layer	c.y.	1,740	28.70	49,940
Side Openings	each	200	100.00	20,000
Emergency stairways	each	14	300.00	4,200
Sound-absorbent liners	s.f.	100,000	0.60	60,000
End bulkheads	each	11	6,530	71,830
Coupler & jacks	each	5	2,730	13,650
Neoprene seal	ft.	944	45.00	42,480
Testing watertightness	each	5	1,000	5,000
Temporary access towers	ft.	80	45.00	3,600
Portal cranes	each	2	120,000	240,000
				3,105,930

4. <u>PREPARATION OF TRENCH</u>

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
Remove protective dykes on both approaches	c.y.	38,400	0.90	34,560
Dredging: (above El.-65)	c.y.	653,000	0.50	326,500
(below El.-65)	c.y.	75,000	1.70	127,500
				488,500

Table 12.1 - cont'd

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
5. PLACING OF ELEMENTS				
Fitting-out wharf	sum			30,000
3'-3" screeded gravel layer	c.y.	11,000	4.65	51,150
6" sand-asphalt	c.y.	1,640	30.00	49,200
Screeding equipment	sum			100,000
Trimming (bulkheads, pumps & pipes)	sum			37,000
Steel access shafts	tons	13	750.00	9,750
Sinking rig	sum			1,000,000
Temporary anchors	each	5	10,000	50,000
Placing operations	each	5	25,000	125,000
Joints between elements	ft.	822	38.00	31,240
Remove end bulkheads	each	12	1,900	22,800
Construct final joint	sum			13,000
Restore river bank	sum			110,000
Roadway ballast	c.y.	9,510	20.00	190,200
Roadway paving	tons	1,040	10.00	10,400
				1,829,740
6. SCOUR PROTECTION				
Tunnel Section -				
Gravel	c.y.	11,700	3.00	35,100
Sand	c.y.	24,700	2.60	64,220
Filter cloth	s.f.	265,500	0.65	172,580
Stones	c.y.	49,100	5.50	270,000
Reinforced concrete blocks	"	520	50.00	26,000
River Banks -				
Sand layer	c.y.	25,000	2.60	65,000
Filter cloth	s.f.	81,900	0.55	450,450

Table 12.1 - cont'd

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
Stone blanket	c.y.	83,400	5.50	458,700
Reinforced concrete anchor blocks	c.y.	1,485	42.50	63,110
Dykes -				
Temporary dyke	c.y.	15,360	5.50	84,480
Permanent dyke (re-use of stones)	c.y.	4,290	3.50	15,020
				1,704,660
7. ELECTRICAL & MECHANICAL INSTALLATIONS				
200 HP axial reversible fans	each	4	90,000	360,000
CO detection meters	sum			30,000
Opacity meters	sum			9,000
Closed circuit TV	sum			18,500
Automatic drainage pumps	sum			90,000
Lighting	sum			310,000
Traffic signals	sum			15,000
Emergency telephone & fire protection	sum			72,000
Control panel & desk	sum			24,000
Standby power diesel generator	each	2	27,000	54,000
Wet batteries	sum			4,500
Electrical wiring & service	sum			45,500
				1,022,500
				13,564,910
CONTINGENCY 10% (approx.)				1,335,090
Total Cost of Tunnel =				14,900,000

Table 12.2

ESTIMATED COSTS OF LAUNCHWAY SCHEME

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
1. MATA REDONDA				
Approach -				
Temporary protective dyke	c.y.	32,800	1.15	37,720
Earth excavation	c.y.	251,000	0.90	225,900
Rock excavation	c.y.	12,480	4.35	54,200
Dewater, install & operate wellpoint system	sum			218,000
Preparation of base	s.f.	8,440	0.09	7,600
Concrete	c.y.	12,240	28.70	351,300
Reinforcing steel	tons	890	343.00	305,270
Formwork	s.f.	78,600	1.35	106,000
Precast forms for ducts	ft.	710	97.00	68,870
Emergency stairs & ventilation bldg.	sum			45,000
Displace protective dyke	c.y.	21,440	1.15	24,650
Permanent cofferdam	c.y.	107,200	1.15	123,280
Backfill	c.y.	155,000	1.15	178,250
Sun screen & grid system	s.f.	12,830	1.45	18,600
Surface drains	sum			21,000
Finishing and landscaping	sum			73,000
				1,858,640
Launchway -				
Rock excavation	c.y.	75,400	4.35	328,000
Earth excavation	c.y.	58,800	0.90	52,920

Table 12.2 - cont'd

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
Preparation of base	s.f.	116,000	0.09	10,440
Concrete	c.y.	1,720	28.70	49,360
Reinforcing Steel	tons	55	343.00	18,870
				459,590
2. TAMPICO				
Temporary protective dyke	c.y.	60,500	1.15	69,580
Steel sheet cofferdam	tons	660	437.00	288,420
Excavation & bracing	c.y.	91,700	2.10	192,570
Progressive dewatering	sum			77,500
Preparation of base	s.f.	35,400	0.09	3,190
Concrete	c.y.	11,230	28.70	322,200
Reinforcing steel	tons	600	343.00	205,800
Formwork	s.f.	47,300	1.35	63,860
Precast forms for ducts	ft.	710	97.00	68,870
Operate movable form support frames	each	27	410.00	11,070
Emergency stairs & ventilation bldg.	sum			87,000
Railway bridge	s.f.	2,730	17.00	46,410
Sun screen & grid system	s.f.	12,830	1.45	18,600
Backfill	c.y.	78,200	1.15	89,930
Displace protective dyke	c.y.	21,430	1.15	24,650
Surface drains	sum			21,200
Finishing & landscaping	sum			72,800
				1,664,650
3. FABRICATION OF ELEMENTS				
Roller skids	each	240	325.00	78,000
Misc. structural steel	tons	45	500.00	22,500

Table 12.2 - cont'd

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
Hot asphalt coat	s.f.	81,500	0.22	17,930
1/8" bottom steel membrane	tons	215	760.00	163,400
Concrete	c.y.	25,100	28.70	720,370
Reinforcing steel	c.y.	760	343.00	260,680
Formwork	s.f.	154,600	1.35	208,710
Precast forms for ducts	ft.	2,760	97.00	267,720
Supply & operate form supports	sum			65,000
Prestressed cables (in place)	tons	487	550.00	267,850
Anchors for prestressed cables	each	1,750	40.00	70,000
Waterproofing & abrasion protection of walls	s.f.	63,000	1.20	75,600
Waterproofing roof	s.f.	67,000	0.50	33,500
8" protective layer	c.y.	1,740	28.70	49,940
Side openings	each	200	100.00	20,000
Emergency stairways	each	14	300.00	4,200
Sound-absorbent liners	s.f.	85,000	0.60	51,000
End bulkheads	each	11	6,530.00	71,830
Coupler & jacks	each	5	2,730.00	13,650
Neoprene seal	ft.	944	45.00	42,480
Testing watertightness	each	5	1,000.00	5,000
Temporary access towers	ft.	80	45.00	3,600
8" sand cushion placed and removed	c.y.	1,950	70.00	136,500
Portal cranes	each	2	200,000.	400,000
Move elements 1,2 & 3 600' down launchway	each	3	2,000.00	6,000
Rock anchor clusters and winches	sum			80,000
				3,145,460

Table 12.2 - cont'd

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
4. PREPARATION OF TRENCH				
Remove protective dykes on both approaches	c.y.	38,000	0.90	34,560
Dredging: (above El. -65)	"	653,000	0.50	326,500
(below El. -65)	"	75,000	1.70	127,500
				488,500
5. PLACING OF ELEMENTS				
Modification of portal cranes	sum			250,000
Drive timber piles	each	420	110.00	46,200
Precast reinforced concrete rails	c.y.	1,100	35.00	38,500
3' foundation gravel fill	c.y.	11,000	3.50	38,500
Rough screeding	s.f.	90,000	0.13	11,700
6" sand-asphalt layer	c.y.	1,640	30.00	49,200
Trimming (bulkheads, pumps & pipes)	sum			37,000
Steel access shafts	tons	13	750.00	9,750
Launching & placing	each	5	18,200	91,000
Joints between elements	ft.	822	38.00	31,240
Remove end bulkheads	each	12	1,900.00	22,800
Construct final joint	sum			13,000
Restore river bank	sum			110,000
Roadway ballast	c.y.	9,510	20.00	190,200
Roadway paving	tons	1,040	10.00	10,400
				949,490
6. SCOUR PROTECTION				
Tunnel Section -				
Gravel	c.y.	11,700	3.00	35,100

Table 12.2 - cont'd

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
Sand	c.y.	24,700	2.60	64,220
Filter cloth	s.f.	265,500	0.65	172,580
Stones	c.y.	49,100	5.50	270,000
Reinforced concrete anchor blocks	c.y.	520	50.00	26,000
River Banks -				
Sand layer	c.y.	25,000	2.60	65,000
Filter cloth	s.f.	81,900	0.55	450,450
Stone blanket	c.y.	83,400	5.50	458,700
Reinforced concrete anchor blocks	c.y.	1,485	42.50	63,110
Dykes -				
Temporary dyke	c.y.	15,360	5.50	84,480
Permanent dyke (re-use of stones)	c.y.	4,290	3.50	15,020
				1,704,660
7. ELECTRICAL & MECHANICAL INSTALLATIONS				
200 HP axial reversible fans	each	4	900,000	360,000
CO detection meters	sum			30,000
Opacity meters	sum			9,000
Closed circuit TV	sum			18,500
Automatic drainage pumps	sum			90,000
Lighting	sum			310,000
Traffic signals	sum			15,000
Emergency telephone & fire prevention	sum			72.000
Control panel & desk	sum			24,000

Table 12.2 - cont'd

Item	Unit	Quantity	Unit Price	Total Cost (dollars)
Standby power diesel generator	each	2	27,000	54,000
Wet batteries	sum			4.500
Electrical wiring & service	sum			45,500
				1,022,500
		Sub-total		11,274,640
CONTINGENCY 10% (approx.)				1,125,360

Total Cost of Tunnel (Can. $) = 12,400,000

CHAPTER 13

CONCLUSIONS AND RECOMMENDATIONS

Throughout the dissertation, much emphasis has been made on cost. The cost-benefit relationship, i.e., extra safety and comfort to be derived from higher expenditure, has also been stressed. Various alternatives of tunnel cross sections, methods of waterproofing, types of foundations, methods of fabrication and construction have been studied in sufficient detail to warrant cost comparisons to be made such that the most economical solution, bearing in mind the cost-benefit criterion, can be chosen with a better insight. It can be seen that certain alternatives selected were not the most economical alternatives investigated. In these cases, an extra cost, in the vicinity of half a million dollars, was sacrificed for better safety, functional and comfort features. This figure, representing about 4% of the total cost of the tunnel, seems to be a justifiable price to be paid for the many added extras achieved.

It has been stated earlier that the cost of fabrication, placing and founding can be proportioned roughly as 40-50% for the fabrication of the structural skeleton and 50-60% for the placing and founding expenses when based on conventional methods of tunnel construction. Inspection of Tables 12.1 and 12.2 indicates that the fabrication of the cross section of the 5 tunnel elements costs approximately 3 million dollars while the placing and founding of these elements amount to roughly 4 million dollars. This difference in cost places the proportioning of costs to 43% fabrication and 57% placing and founding. By devising better placing and founding methods as was done for the Tampico Tunnel, the latter cost was reduced to 3 million dollars. Consequently, the proportioning becomes 50% for fabrication and 50% for placing and founding.

In Chapter 5, it was seen that the most economical cross section in terms of least materials cost 10% less than the cross section which offered the added features for safety and aesthetics. Over the entire tunnel length of 1945' the savings incurred by the adoption of the Type I cross section would only have been 3.5% of the total cost of the tunnel project. Cross sections having the same minimum internal requirements differ even less on the total cost. Based on these revelations and on experience gained from the design of other tunnel structures, it can be concluded that the savings induced by the selection of the most economical cross section alone is negligible compared to the possible savings which can be obtained elsewhere.

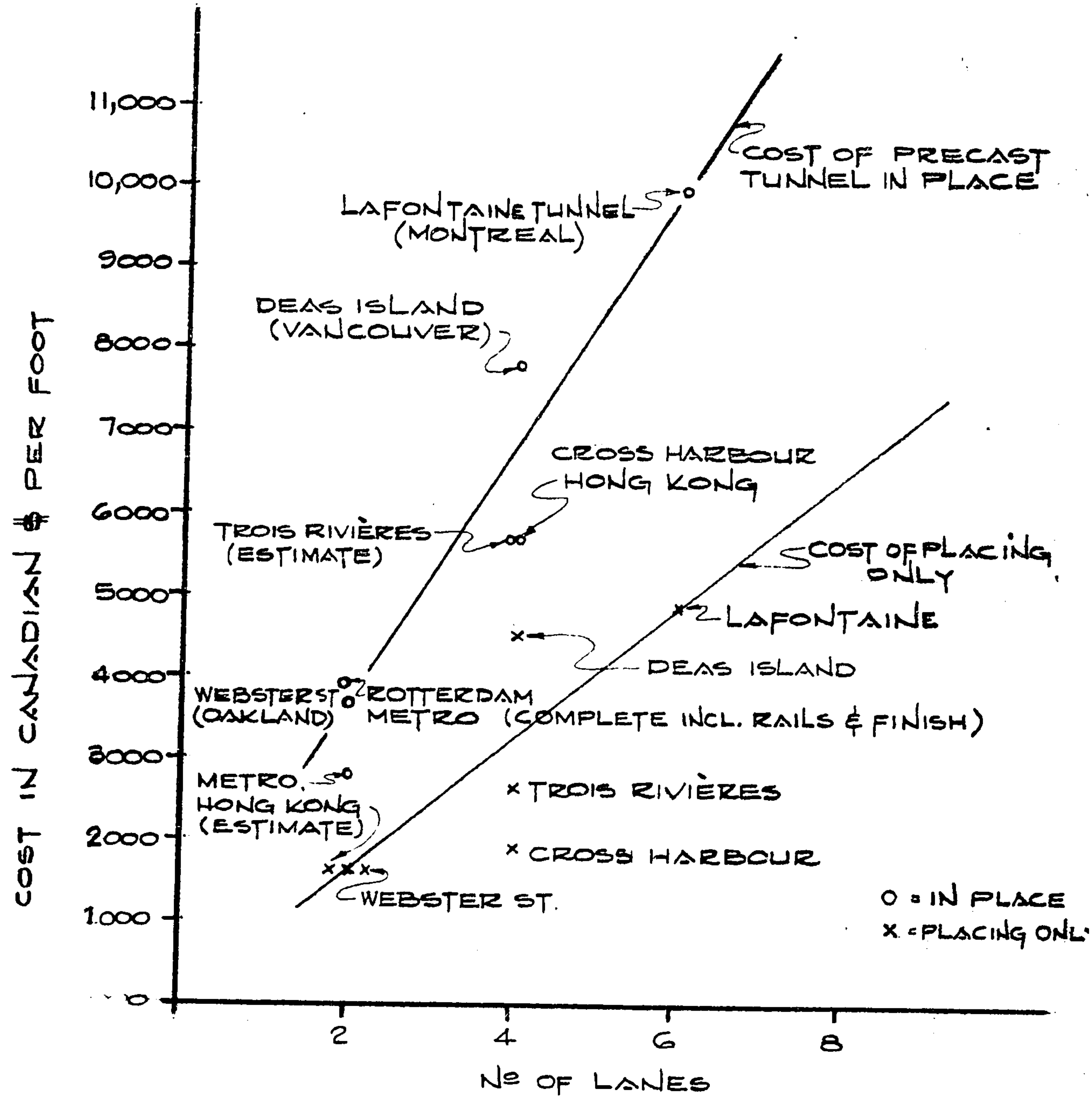
In Chapter 7, the cost of various types of applied waterproofing was compared. The dry tunnel approach to waterproofing was chosen to ensure the utmost dryness in the tunnel. This approach has been adopted in most tunnels built to date. The acceptance of such an approach by tunnel engineers is by no means surprising as a wet tunnel is psychologically "unsafe" regardless of how structurally sound the tunnel is. The wet tunnel approach, though involving a bigger risk factor which can be minimized by more experimental data, definitely deserves a more thorough investigation both theoretically and experimentally to assess its true value. If it can be shown that such an approach to tunnel waterproofing is feasible both economically and structurally, the outer waterproofing membrane can be of a much simpler form. The savings achieved by the adoption of such an approach, however, would only be of an insignificant amount as the cost of the waterproofing membranes constitutes only 2% of the cost of the entire project. The major consolation here is the fact that a new engineering principle in tunnel waterproofing would have been established.

The Table of foundation cost in Chapter 8 indicates an average difference of around 50% between the conventional foundations and the foundation type selected for use in the present tunnel. This difference occurs not only in the Tampico Tunnel but also in tunnels studied for other projects. Hence, it can be inferred that substantial savings in cost is possible by devising an economical founding technique for the tunnel as was done here. For the rectangular tunnel cross section adopted, the roughly screeded foundation topped with a layer of sand-asphalt seemed to be the most economical solution to the founding problem. Had the cross section been rounded, other methods such as vibroflotation and free dumping might possibly have been more suitable.

Reviewing Tables 12.1 and 12.2 again reveals that the areas for substantial savings, besides the preparation of foundations, was in the method of placing. Between the dry dock and launchway placing methods, a difference of 0.8 million dollars, representing 80% of the placing cost, in favour of the latter was accomplished. Considering all aspects of the tunnel project, a 20% savings in cost was achieved by the use of a launchway method of construction over that of the conventional dry dock method of casting, sinking and founding.

It can be summarized that in order to effect savings the designer should direct much of his effort towards the fields of casting and placing of the submerged elements as well as the selection of the most economical type of foundation and methods to place this foundation. These three areas are prospects for major savings both in cost and construction time. The possible savings in the areas of tunnel cross sections and waterproofing techniques are relatively insignificant and thus their selection can be less strict.

A graph showing the costs of some of the precast tunnel sections is presented in Fig. 13.1.



COSTS OF PRECAST TUNNEL SECTION

ADJUSTED TO 1968 PRICES

NOTES:

1.) COSTS DO NOT INCLUDE VENTILATION, ELECTRICAL INSTALLATIONS, COSTS OF APPROACHES, VENTILATION BUILDINGS, FINISHING.
2.) PLACING COSTS INCLUDE DREDGING, BEDDING MATERIALS, BULKHEADS, LAUNCHING, SINKING, NORMAL TOP OF TUNNEL PROTECTION.
3.) BANK PROTECTION FOR DEAS ISLAND TUNNEL NOT INCLUDED.

Fig. 13.1

A P P E N D I C E S

APPENDIX A

PORT DEVELOPMENT AND NAVIGATION REQUIREMENTS

Given the problem of constructing an underwater crossing beneath a navigation channel, it is a necessity to study the effects on the underwater structure due to the limitations naturally or unnaturally imposed on the upper and lower reaches of the watercourse; and conversely, the aftermath caused by a crossing structure on the areas both upstream and downstream of the crossing site.

A review of Admiralty charts and sailing directions for the Port of Tampico indicated two parallel breakwaters jutting out into the Gulf of Mexico serving as a protection for the entrance to the river. The width of the river channel is approximately 330'. A sand bar, termed "subject to frequent changes" extends into the Gulf of Mexico from about 2,000' to 5,000' beyond the outer end of the entrance channel breakwaters. The depth of the bar varies from approximately 40' to as little as 26' over random areas due to shifting sands.

The entrance channel to the Port is shown to have been dredged to depths of 33' in 1947, 31' in 1955 and 33' in 1968. The channel could be maintained at a depth of 40' or greater, at reasonable cost, if the results of an economic cost-benefit study reveal the desirability of such an improvement.

The lower section of the river channel, extending downstream from the proposed tunnel crossing approximately 3.5 miles to the inner end of the entrance channel, varies in width from 650' to 1,000' and in depth from 26' to 31'. This section of the river basin lends itself to an extensive expansion of harbour facilities by the excavation of shipping basins and the deepening of the river channel.

The upper section of the river channel, upstream from the proposed tunnel site, is narrow (490'), shallow (26' to 31') and contains a 90° bend of about 3,000' radius. Considerable expenditure is required to improve this section to provide a navigation channel of greater than 33' depth.

Questions naturally arise concerning the provision of a 40' channel as related to future port development and concerning possible measures which might be adopted in controlling and regulating hydraulic flow to provide a higher tunnel profile. The modification of either of these features could substantially influence the cost of the tunnel. Therefore, it would seem discreet to first complete a comprehensive, fully co-ordinated and integrated study of the various interrelated factors affecting present and future port requirements before firmly adopting a particular depth for the final design of the tunnel.

At the present, a somewhat obvious solution would be that the section of river upstream from the proposed tunnel crossing be restricted to facilitate general cargo vessels carrying containerized, palletized, pre-slung and other unit-load commodities which require not more than 40' depth of channel. This limitation could be justified as current development of berths for similar types of shipping at numerous locations in the world provide a depth of only approximately 39' as can be seen from Table A.1. Coastal vessels, conventional merchant ships, roll-on/roll-off vessels and LASH (lighter aboard ship) scows require appreciably shallower channel depths. Should further investigations indicate that berths of greater depth be required in the Port of Tampico, they may readily be established downstream from the proposed tunnel crossing. The adoption of this solution will permit a maximum flood flow of over 600,000 cfs at the crossing site. This provision results in specifying the top elevation of the tunnel protective cover to be at El. -40.0'.

If flood control measures were implemented to reduce the maximum flood discharge at the crossing, the tunnel profile may be raised. This represents, of course, further restrictions as to the type of vessels which may enter the upstream section of the river; namely, coastal vessels, conventional merchant ships, roll-on/roll-off vessels and LASH scows, types which require only 33' depth of channel, could be accommodated.

In any case, whether or not flood control measures will be adopted prior to tunnel construction, continual dredging of the shipping lane in order to provide the necessary channel depth seems unavoidable. The institution of flood control, as far as the navigation channel is concerned, only means lesser dredging, provided the various vessels using the channel require only this depth. If deeper-draught vessels had to enter the section of river upstream from the tunnel site, as may be the case should studies indicate that deep port development downstream of the crossing site is not feasible, the shipping lane would still have to be dredged to the depth required to accommodate these vessels. Flood control then, would not lift the tunnel profile in any way but would only reduce the scour and silting action of the river and the risk of inundating the harbour area experienced during the 1955 flood. The former effect will be reflected in the amount of scour protection required and the quantity of material to be dredged for this deeper channel.

Table A.1

CHANNEL DEPTHS OF RECENTLY CONSTRUCTED AND/OR PLANNED PORT FACILITIES

General Cargo Berths	Container and Unit-load	Roll-on/ Roll-off
Amsterdam	33'	
Antwerp	39'	
Bremen	35'	
Felixstowe	33'	
Gothenburg	36/39'	23'
Hamburg	33'	
Le Havre	39'	
London (Tilbury)	44'	23'
	31'	32'
Rotterdam	39'	
Baltimore	33'	
Boston	35'	
Long Beach	37'	
Los Angeles	35'	
New York (Elizabeth)	35'	
San Francisco	40'	
Seattle	40'	
Osaka	39'	
Yokohama	36'	
	39'	
Melbourne	31'	26'
	35'	25'
Sydney	36'	33'

APPENDIX B

DESIGN PARAMETERS AND COMPUTATIONS

B.1 General

For preliminary planning and design development purposes, it is sufficient to analyse the structures to ascertain that their floatation and stability requirements and that major stresses due to the worst loading conditions are satisfied. Normally, a cross section proportioned to meet the negative buoyancy requirements will fulfill the stress requirements since the major enigma in most tunnel designs is the problem of providing enough mass to prevent the tunnel from lifting up under buoyancy forces.

It is recognized that the singular climatological, hydrological and physiological conditions that prevail at Tampico impose special problems of design which will require more intensive analyses than those developed herein. Nevertheless, the present studies have been carried out to a degree that, based on past experience, is sufficient to assure the adequacy and economy of the proposed structures as a basis for preliminary estimates of cost.

B.2 Design Criteria

Design criteria for structural analysis used herein follow current engineering practices as used in Canada and the United States in the design of highway bridges and reinforced and prestressed concrete structures.

Throughout the design, much use has been made of the "Standard Specifications for Highway Bridges adopted by the American Association of State Highway Officials, 9th edition, 1965" (ASSHO), and notably loads and allowable stresses used are in accordance with the ASSHO requirements. These requirements are substantially in agreement with the Canadian Standards Associtation specifications "CSA Standard S6 - 1966, Design of Highway Bridges". Whenever needed, additional specifications from the American Concrete Institute's Building Code and the National Building Code of Canada were adopted.

Unit Weights:

Concrete	150	pcf
Steel	490	"
Pavement	150	"
Water (fresh)	62.4	"
Water (salt)	64.0	"
Granular Backfill (e=40%) -		
Solid Weight	170	"
Dry Weight	105	"
Saturated Weight	130	"
Moist Weight	115	"
Submerged Weight (fresh water)	67.0	"
Submerged Weight (salt water)	66.0	"
River Silt -		
Saturated (w=42%)	110	"
Submerged (fresh water)	48.0	"
Submerged (salt water)	47.0	"

Stresses:

Concrete (placed in the dry)	$f'_c = 4{,}500$ psi
Concrete (placed under water)	$f'_c = 3{,}000$ "
Reinf. Steel, int. grade	$f_y = 40{,}000$ "
Prestressing Steel	$f'_s = 235{,}000$ "

B.3 Earthquake Loading

Several modes of earthquake damage are possible and have been investigated.

(1) Relative Surface Displacement Along a Fault - Little can be provided in the structure to prevent damage due to fault movements. There seem to be no records of faulting in the Tampico region and no known faults across the proposed tunnel. Designing the cross section for flooded tunnel condition was considered to be adequate in that the tunnel would be repairable in the extremely remote case of cracks and flooding caused by fault movements.

(2) Ground Accelerations - Intense ground shaking induces inertia forces which the tunnel structure must resist. By applying the formulae given for a simple static analysis, it was found that the inertia forces to be allowed for amounted to about 25% of the vertical loads of the structure. These forces may tend to shear or open up the joints between the tunnel elements. Should this happen, the tunnel would be flooded. This probability was found not to be critical either in transverse shear stresses or in unacceptable release of the longitudinal compression at the joint.

(3) Ground Deformations - As the ground is relatively stiff compared to any structure built within it, the structure is compelled to conform to the ground distortions. Thus, the criterion for design is one of ability to absorb imposed deformations, rather than a requirement of strength to resist

applied forces. In this respect, tunnels with flexible joints between tunnel elements obviously have some advantages.

Seismic shear waves cause curvature distortion and shearing distortion. The former represents the direct sinusoidal curvature of the shear wave, whose amplitude increases as the ground becomes softer. However, even in the poorest soils, the radius of curvature is of the order of 100 miles or more. The buried structure can deform to this curvature without distress. The latter results from a time lag in movement between the overburden and the base rock analogous to shaking a bowl of jelly. For reasonably competent ground, the angle of shearing distortion is quite small and is generally within the elastic deformation capacities of concrete structures. For very soft soils, the distortion may approach 0.5% which could cause cracks in corner joints.

As no special soil studies were made for the purpose of earthquake analysis for the Tampico Tunnel, extra reinforcing steel and prestressing steel will be provided to reduce such possible cracks.

(4) Soil Liquefaction - No testing was done to determine whether the soil surrounding the tunnel would liquefy when subjected to oscillatory loadings. However, soils such as clay with coarse sand or gravel are very rarely found to become liquefied. Thus for design purposes, and subject to test verification, the tunnel foundation which consists of clay mixed with sand and gravel, will be assumed not to become

quick but the backfill at the tunnel sides will be assumed, because of its loose state, to liquefy to become a fluid with specific gravity of 1.9. This earthquake consideration was combined with other loadings to determine the governing loading case.

B.4 Analysis

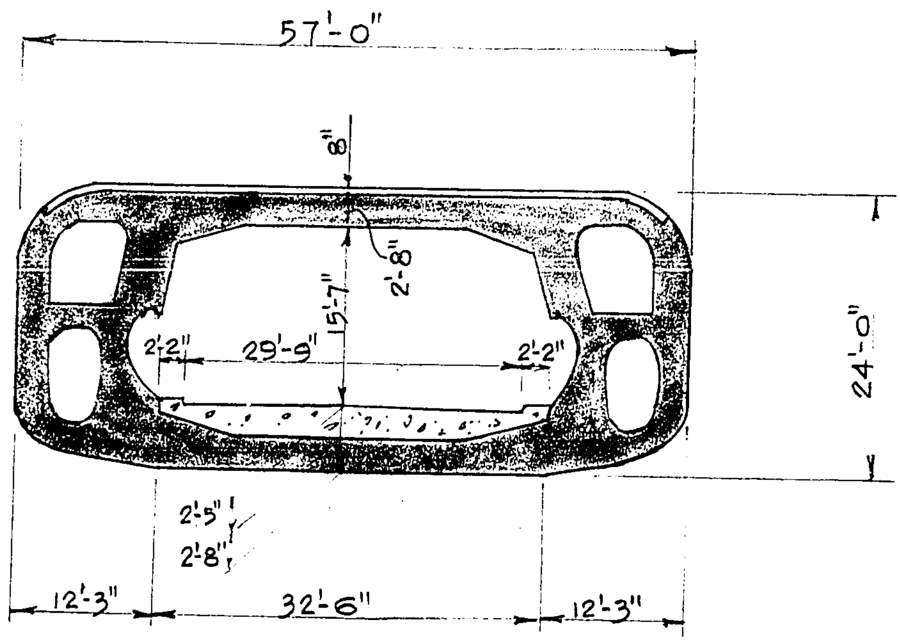
The design of the typical cross section of the closed portion of the tunnel structure has been based on the assumption that the concrete cross section would behave like a flat discretized rectangular Vierendeel frame, with braced sidewalls and supported on an elastic foundation. A commercially available computer programme, ICES 'STRUDL' developed by MIT has been used to compute the bending moments, shear forces and thrusts at various discrete points along the assumed centrelines of the different members constituting the cross section.

Buoyancy analysis for the typical tunnel cross section, which differs from normal design of engineering structures, is illustrated by design briefs. However, since structural design of reinforcement for the cross section followed standard practice of reinforced and prestressed concrete design, computational procedures will be omitted. It suffices to list the various loading combinations considered and to indicate the loading condition which governed the design.

Tunnel Proper

(A) **Buoyancy Analysis**: Typical Section at Sta. 4+000

(Ref. Fig. 5.1)



consider 1 lineal foot of section

Dead Wt. of structure:

Concrete --

Volume -	outer rectangle 57x24		= 1370.0 cf
	rounded corners		=- 34.8
	8" concrete protective layer		=- 35.8
		outer volume	= 1299.4 cf
Deduct volume of	walkways	= 90.5	
	vent. & service ducts	= 64.5	
	traffic tube	= 655.0	
	walkway opngs	= 2.8	
	vent. louvres	= .5	
	stair recesses	= .8	- 814.1 cf
	Total Vol. of Concrete		= 485.3 cf

Wt. of concrete $0.15 \times 485.3 = 73.0^k$

Wt. of 8" conc. protective layer $0.15 \times 35.8 = 5.4$

Wt. of roadway ballast $0.15 \times 81 = 12.1$

Total Wt./ft. $= 90.5^k$

Buoyancy force:

$$U = 0.064(1299.4+35.8) = 85.5^k$$

Factor of Safety:

a) Permanent F.S. vs Uplift $= \dfrac{90.5}{85.5} = 1.058$ 1.05 OK

b) With 3'-3" rock protection (wt.$=3.25 \times 57 \times 0.066 = 12.2^k$)

$$\text{F.S. vs Uplift} = \frac{90.5+12.2}{85.5} = 1.20 \quad \text{OK}$$

Floatation:

Wt. without roadway ballast = 78.4 k/lin. ft.

Total length per precast element is 275'.

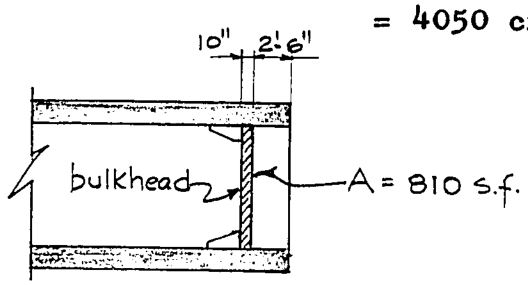
Assume end bulkheads placed 2'-6" from the end of the element, with 10" bulkhead thickness.

Wt. of bulkheads $= 2 \times 0.833(655+64.5+90.5)(0.15)$
$= 202.5$ k/element

Wt. of element (with 8" protective layer)
$= 275(73+5.4)+202.5$
$= 21{,}802.5^k$

Buoyancy force/element --

Loss of buoyance vol. $= 2 \times 2.5 \times 810$
$= 4050$ cf



a) In fresh water (w_w = 62.4 pcf)

$$U = ((1299.4+35.8)275-4050)(0.0624) = 22,650^k$$

$$\text{Exposed volume} = \frac{22,650-21,802}{275x0.0624} = 49.5 \text{ cf}$$

$$\text{Freeboard} = \frac{49.5}{50} = 1'\text{-}0" \quad \text{(minimum)}$$

b) In salt water (w_w = 64.0 pcf)

$$U = (367,000-4,050)(0.064) = 23,200^k$$

$$\text{Exposed volume} = \frac{23,200-21,802}{275x0.064} = 79.5 \text{ cf}$$

$$\text{Freeboard} = \frac{79.5}{49} = 1'\text{-}6" \quad \text{(maximum)}$$

Ballast Required --

For placing of elements, it is desirable to have a positive weight of not more than 350 tons as the elements will be supported by portal cranes.

Total ballast needed (in fresh water) = 22,650-21,802.5 +700

$= 1,548^k$

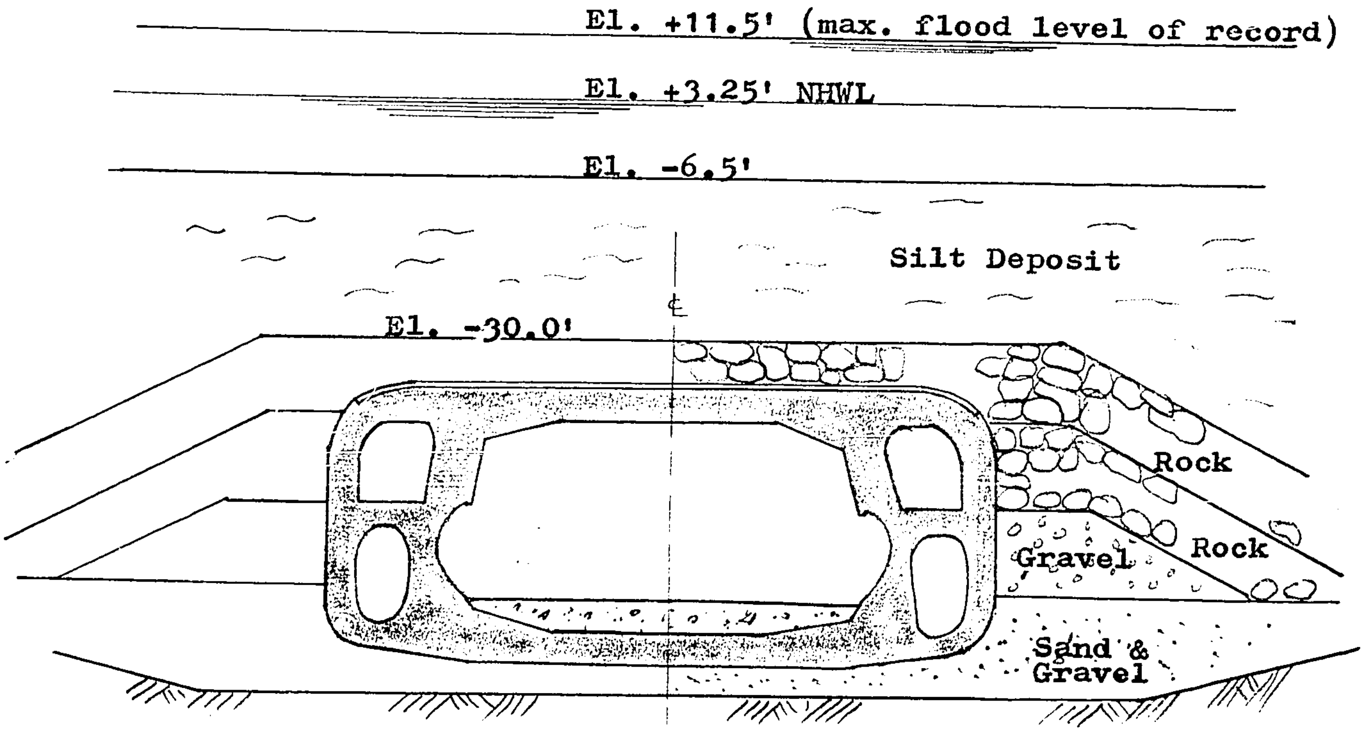
$$\text{or} = \frac{1,548}{275-2x3.33} = 5.75 \text{ k/ft. length of enclosed by bulkheads.}$$

This is equivalent to adding $t = \dfrac{5.75}{29.75x0.15} = 1'\text{-}3"$ thick roadway ballast before launching the elements (design thickness of roadway ballast = 2'-5").

For salt water conditions:

Wt. of element (incl. 1'-3" ballast)	= $23,350^k$
Buoyancy force in salt water	= 23,200
Downward force on cables	= 150^k OK

(B) Design for Reinforcement:



Typical Cross Section
(Sta. 4+000)

For the purpose of estimating the quantity of materials and to ascertain structural safety of the tunnel proper, 2 cross sections were analysed by computer techniques developed by MIT called ICES 'STRUDL' program. A critical cross section was analysed for checking the adequacy of the tunnel cross section adopted and a typical cross section, representative of the tunnel proper, was analysed by the program to obtain an estimate of the quantity of reinforcing steel required.

It was found that the tunnel cross section adopted satisfied all the stress requirements under the loadings of the critical cross section at Sta. 3+800.

The typical cross section was analysed for the following loading combinations:

1. Dead load + ballast @ 100% of basic allowable stresses.
2. D.L. + water pressure (water at El. +3.25') + granular backfill + rock and silt surcharge @ 100% of basic allowable stresses.
3. D.L. + ballast + water pressure (water at El. +11.5') + granular backfill + rock and silt surcharge @ 110% of basic allowable stresses.
4. D.L. + ballast + water pressure (water at El. +11.5') + granular backfill + rock and silt surcharge + earthquake @ 133 1/3% of basic allowable stresses.
5. D.L. + ballast + water pressure (water at El. +11.5') + granular backfill + rock and silt surcharge + inside of tunnel flooded @ 133 1/3% of basic allowable stresses.

A 10% increase in the basic allowable stresses was allowed arbitrarily for loading combination 3 as the occurrence of the maximum flood water level is not frequent and as the depth of siltation becomes much shallower at this flood water level.

Inspection of the computer output indicated that the most unfavourable loading is loading combination 3, and the design of the cross section has been based on this loading condition.

APPENDIX C

TUNNEL VENTILATION

Ventilation requirements for highway tunnels are largely governed by the allowable CO content in the traffic tubes. In normal conditions, the amount of CO in the tunnel is generated largely by the exhausted fumes of combustion engines. The CO contents from these fumes may reach 0.023% with good carburettor setting and 0.096% with improper mixtures and carburettor settings. In rare occasions of tunnel fires or explosives, the CO content obviously increases to a much higher value.

The CO content emitted by diesel engines is insignificant because of surplus air facilitating complete combustion in the cylinders. However, recent studies have indicated that diesel fumes do contain acrolein, a gas which is also highly detrimental to health.

Health authorities report that CO contents less than 0.04% do not affect human beings. An allowable limit, however, should be set at 0,02%-0.025% to avoid problems arising from impaired visibility.

In general, some sort of mechanical ventilation is warranted for tunnels carrying highways where natural ventilation can not be depended upon to remove all of the polluted air. It is obvious that the volume of air required depends on the allowable concentration of air pollution. It is also obvious that the allowable air pollution varies for each poisonous gas and is in inverse proportion to the period of time spent in such environment, i.e., the allowable concentration may be higher if a lesser length of time is spent in such surroundings. For this reason, consideration should be given to servicemen or maintennance crew who would have to stay longer in the tunnel while performing their duties than would the travelling public.

In the design of ventilation, it is customary to compute air volume requirements to satisfy CO emission rates by taking into account the effects of grade, traffic composition, i.e., truck/car ratio, vehicle spacing at design speeds and at crawl speed, and anticipated traffic densities. As the future traffic composition likely to use the Tampico Tunnel is not known, empirical air volume requirements have been based upon actual measurements of air supplies and CO concentration carried out over a period of some 12 months in the heavily travelled Lafontaine Tunnel at Montreal, Canada. The truck/car ratio was approximately 10/90. CO contents and air supplies were observed for stalled conditions and for dense traffic movements at a speed of about 20 mph. The resulting observations indicated an air requirement for stalled traffic of 5,500 cf/h/ft. of lane and for dense traffic, adjusted from 1,750 to 2,000 vehicles/hr., an average for up-grade and downgrade of 6,000 cf/h/ft. of lane.

To compensate for the reduced effectiveness of the fresh air supply to the two-lane Tampico Tunnel with its opposing traffic movements as compared with the separated one-way lanes of the Lafontaine Tunnel, the air requirement was increased arbitrarily by 25%. However, in the absence of information regarding the future traffic forecast at Tampico, the ventilating fans have been selected to permit a further increase of 20% in the air supply.

The above method of obtaining design data may appear to have been based on superficial grounds, nevertheless, certain assumptions had to be made in order to arrive at a cost figure as the necessary relevant design information is not available.

C.1 Design Criteria

The design criteria and computation outlined in this appendix were developed to satisfy the basic ventialtion requirements referred to in Chapter 10 and the alternative methods of operation depicted schematically in Fig. 10.2. The general arrangement of the ventilation equipment including structural, mechanical and electrical, is shown in Fig. C.1.

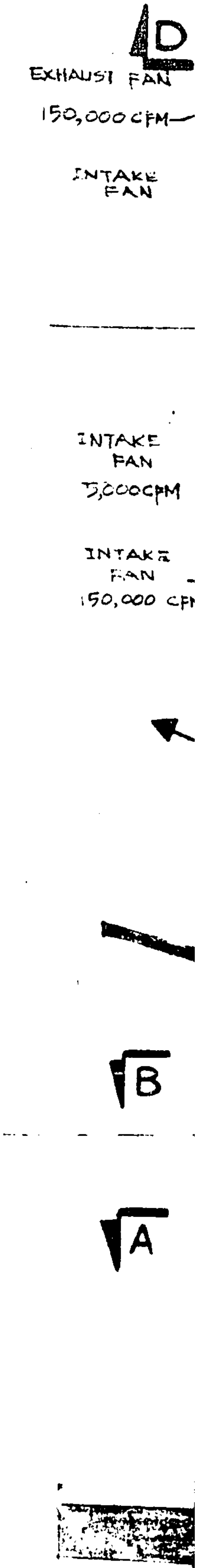
The following recommended design criteria are based on accepted practice and operating experiences adjusted so that they are reasonably applicable to the present tunnel:

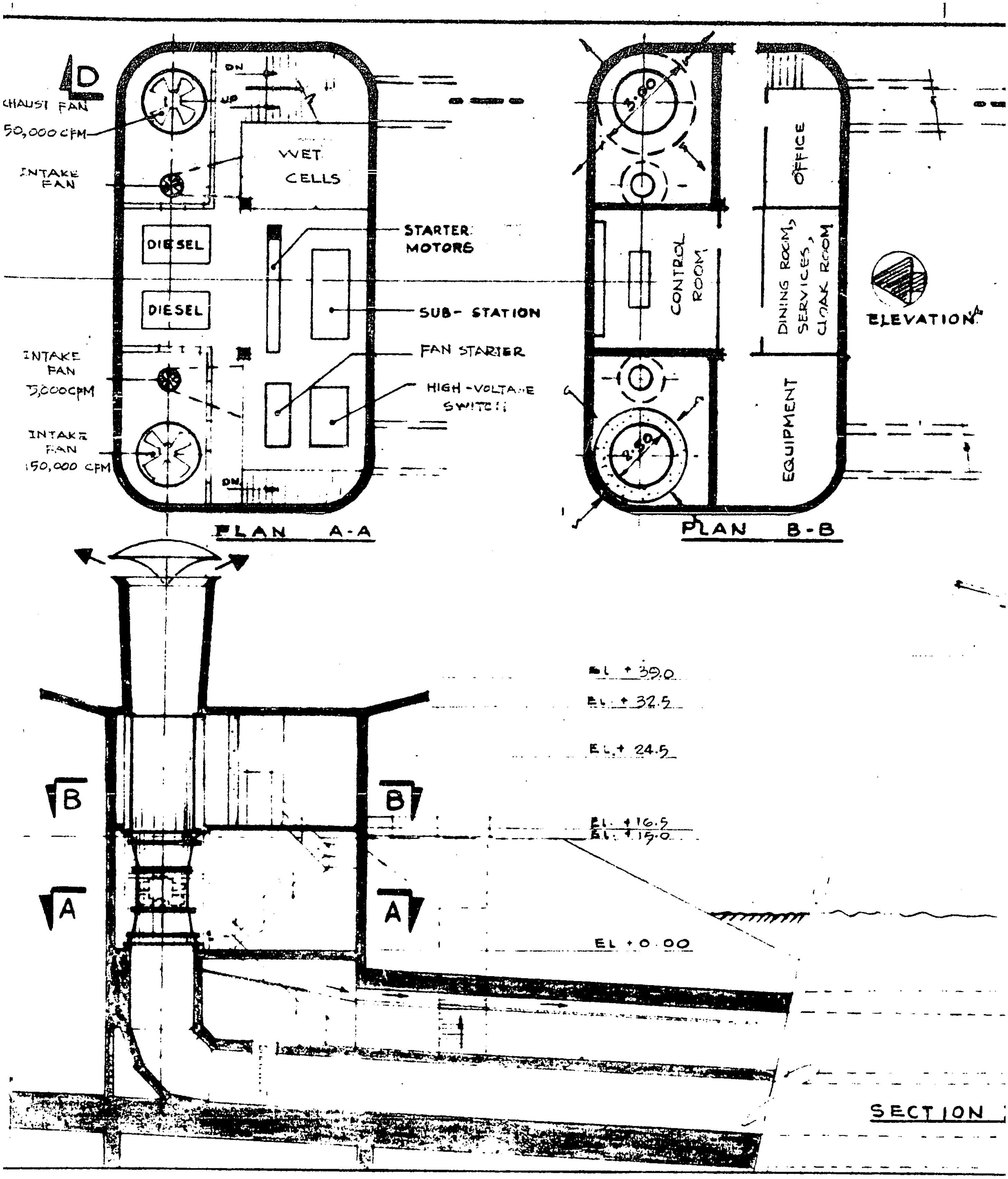
(1) Permissible CO concentration -

Normal, free running traffic (45 mph)-----200 ppm

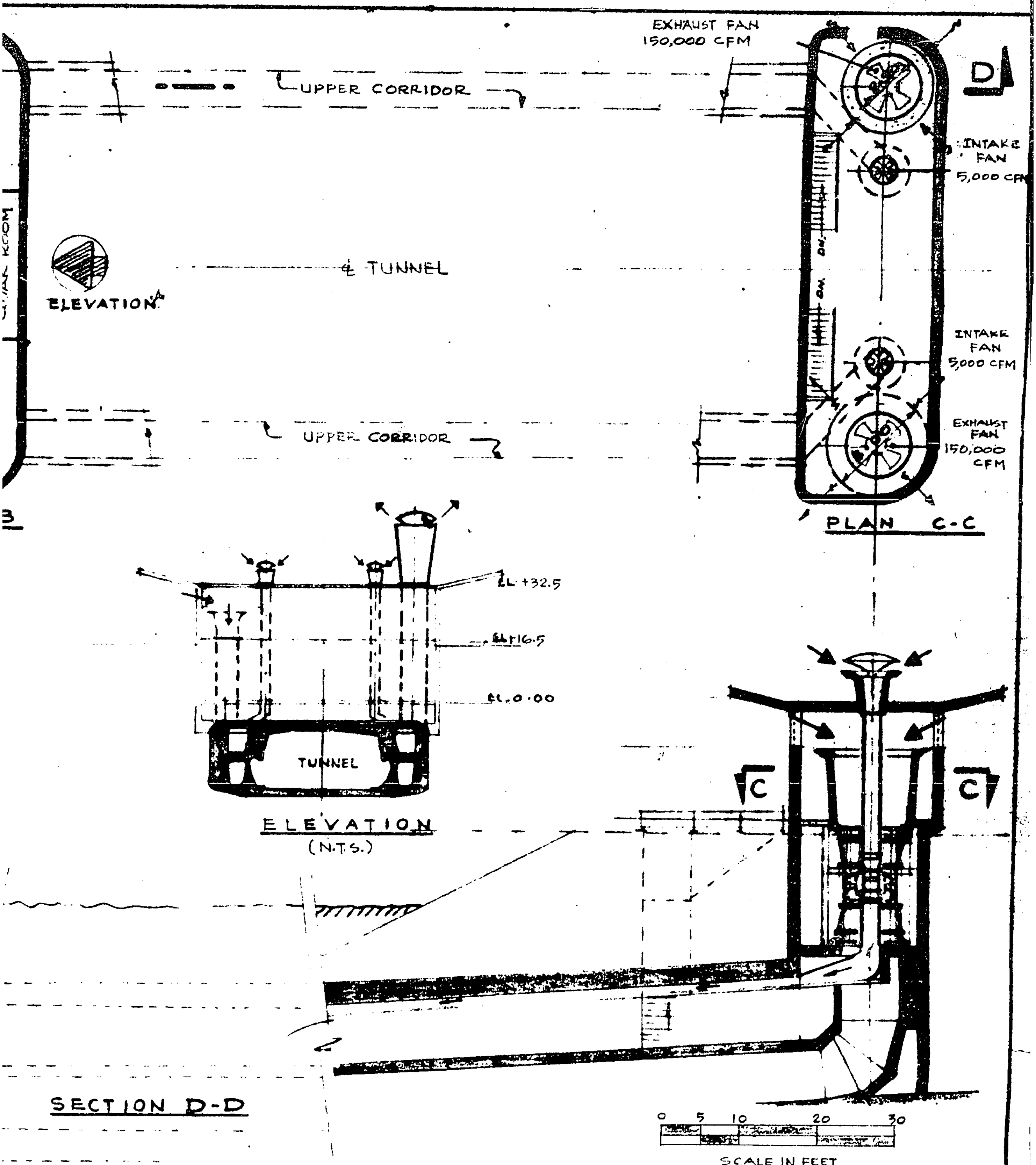
Heavy, free running traffic (25 mph)------200 ppm

Stalled or crawling traffic -------------400 ppm


D
EXHAUST FAN
150,000 CFM
INTAKE FAN
INTAKE FAN
5,000 CFM
INTAKE FAN
150,000 CF
B
A



D
CHAUST FAN
50,000 CFM
INTAKE FAN
WET CELLS
DN
UP
STARTER MOTORS
DIESEL
DIESEL
SUB- STATION
FAN STARTER
INTAKE FAN
5,000 CPM
HIGH-VOLTAGE SWITCH
INTAKE FAN
150,000 CFM
DN
PLAN A-A
3.00
OFFICE
CONTROL ROOM
DINING ROOM, SERVICES, CLOAK ROOM
ELEVATION
EQUIPMENT
2.50
PLAN B-B
EL + 39.0
EL + 32.5
EL + 24.5
B
B
EL + 16.5
EL + 15.0
A
A
EL + 0.00
SECTION



LAYOUT OF MECHANICAL AND ELECTRICAL EQUIPMENT IN THE VENTILATION BLDGS.

Fig. C.1

(2) Air supply, by forced ventilation, computed at a rate of 7,500 cf/h/lin.ft./lane with provision for a 20% increase should truck/car ratio exceed 10/90 percent assumed.

(3) Traffic tube air velocity limited to 30 mph on an empty tube basis.

(4) Air duct velocity limited to 55 mph with provision for 20% increase.

C.2 Design Computations

It suffices here to indicate only the essentials necessary for a primitive understanding of tunnel ventilation.

Traffic Tube - The following computations for ventilating fan requirements have been based upon the foregoing criteria and relevant tunnel dimensions and characteristics:

Length of 2-lane tunnel, portal to portal	2,000'
Total air requirement at 7,500 cf/h/ft./lane	30×10^6 cf/h
Air requirement per fan:	
Stalled traffic	7.5×10^6 cf/h
Dense traffic at 15 mph	7.5×10^6 cf/h
Contingency requirement at 20% increase	9.0×10^6 cf/h

Fan selection:
4 axial flow fans, each 9×10^6 cf/h; 5" water static pressure; 6.5' diameter; 200 HP; 2-speed reversing.

Traffic tube air velocity (air injection method of operation):
Area of traffic tube = 540 sq. ft. (approx.)
Air velocity = $2 \times 9 \times 10^6/540 \times 5280 = 6.3$ mph

Air duct velocity:
Area of duct = 26 sq. ft.
Air velocity - Design condition = $7.5\text{x}10^6/26\text{x}5280 = 55$mph

Pedestrian Passageway - The pedestrian passageway will normally be sealed to prevent passage to the traffic lane, with doors so arranged that, in the event of an emergency, they can be opened by the public entering from the traffic lane.

Ventilation will be sufficient to allow for 100 persons in each passageway. This will require:

4 axial flow fans, each 3×10^6 cf/h; 3" water static pressure; 15" diameter; 5 HP.

APPENDIX D

TUNNEL LIGHTING

Tunnel lighting requirements are of paramount importance as modern day motor travelling places more and emphasis on driving comfort. Comfort, in this sense, certainly must not be thought of as a provision signifying luxury; rather, it implies the need to construct the structure with all the facilities needed such that it can be driven over with a minimum of physical and psychological strain on the part of the motorist.

D.1 Introduction

In daylight hours, a motorist approaching the tunnel entrance cannot see an object in the tunnel if its luminance and that of its immediate surroundings is much lower than the luminance to which the eye of the motorist is still adjusted at the moment. This phenonmenon is known as the "black hole effect". The reverse is true at the exit of a tunnel which appears as a brilliant "white hole" to the emerging motorist. In practice, if the lighting requirements at the exit are identical to those at the entrance, the light adaptation will be satisfactory.

At night, the tunnel exit appears as a "black hole" when the road outside the tunnel has a lower luminance than the tunnel itself. Therefore the lighting has to satisfy requirements similar to those which apply to the entrance in daytime. The lighting in the tunnel must be reduced as the luminance of the open road is lower than that of the tunnel interior.

For long tunnels, the entrance zone is usually followed by an interior section where the lighting seen in length is unchanged. The entrance lighting must always allow the motorist to adapt himself to the luminance in the tunnel. Due to the special characteristics of a tunnel, the small space, the pressure of traffic and the great danger of jams, a higher average luminance than night lighting of open roads is required. As a rule, an illumination of 100 to 200 lux may be regarded as a minimum for the interior of a tunnel, provided that in all other respects the tunnel is well lit and that the entrance lighting meets the requirements.

The average experienced motorist in normal circumstances needs to give only a small portion of his conscious attention to get along in traffic. However, if unexpected or unusuall problems occur, normally only a conscious decision can lead to the correct actions under the circumstances. Therefore, to ensure safety, no abrupt changes in the pattern of behaviour should occur, so that no interruptions in the logical sequence of events would happen.

Optical guidance such as road markings, curbs and a line up of the light sources through all zones of the tunnel are important aids. The attention of the motorist may also be distracted by the high noise level and in the long reverberation times occurring in tunnels. One helpful improvement is to install a sound absorbent layer on the walls and ceiling as will be done in the present tunnel. The unavoidable monotony in long tunnels usually slackens the general alertness of the driver. To remedy such an effect, the walls of the Tampico Tunnel will be provided with coloured panels as well as portholes in the upper parts.

To prevent a catastrophe in the tunnel in the wake of sudden lighting failure, at least part of the lighting system must always function. Therefore, an emergency supply is necessary which comes into action automatically and instantly. This supply must be capable of also providing power for the ventilation system as discussed in Chapter 10.

To eliminate the flicker effects, continuous lines of fluorescent tubes are now frequently used in the lighting of tunnels. They have two disadvantages however, i.e., both the specific luminous flux and the minimum ignition voltages depend on the ambient temperature of the lamp and design of ventilation system and fittings should take into account this temperature dependence characteristic. To avoid a change in luminance level due to this temperature dependence, a continuously adjustable dimming device which is controlled by a photocell can be used. Controlled silicon rectifiers which can be used in dimming installation have a very high efficiency and a very long life. They are maintenance free so that there is negligible difference in costs between dimming and extinguishing the lighting, an operation which is necessary in night time. Such rectifiers will be used in the Tampico Tunnel to dim the fluorescent tubes.

For daytime lighting of tunnels, natural daylight can be subdued at the entrance zone to provide for the transition. The screens or grid system used to screen the daylight must absolutely satisfy the condition that under no circumstances whatever direct sunlight should strike the road surface below the screen; otherwise, the regular patterns of shadows present a very disturbing effect. Proper design of the grid system must of course take into account the solar altitudes of various days and of the coordinates of the particular location. As a rule, the grids are made 1.5' to 3.5' high and are constructed of aluminum or concrete.

D.2 Lighting Design

With the above brief introduction to the various factors which influence the lighting of tunnels, the principles will now be applied to the tunnel under consideration.

As daylight illumination is equivalent to 80,000 to 90,000 lux which is almost impossible to simulate because of prohibitive costs, transition zones to reduce this high luminance level to that of the interior lighting level are necessary. The various zones required for comfortable eye adaptation are shown in Fig. D.1.

The distance from the adaptation to the tunnel entrance

TERMINOLOGY

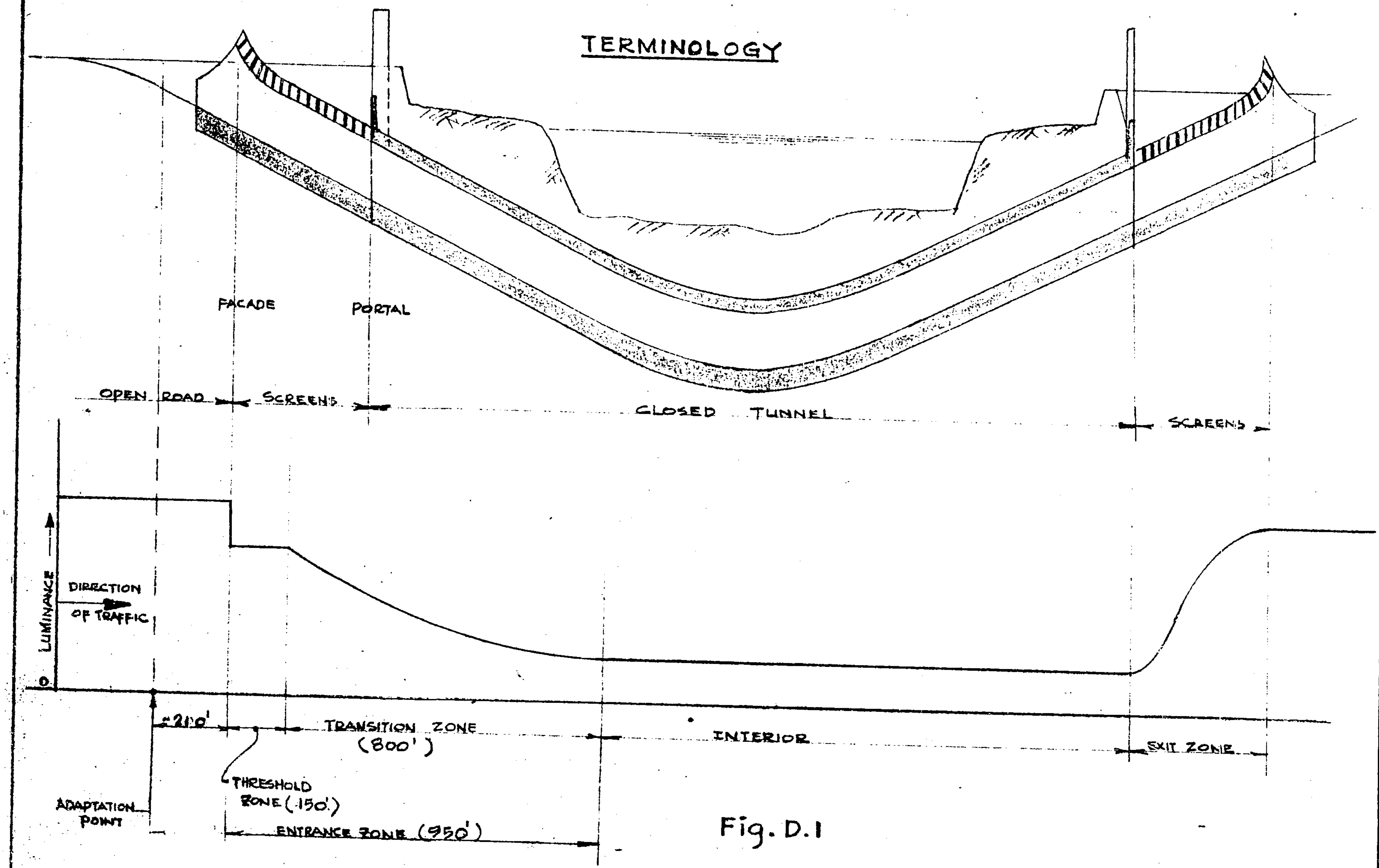


Fig. D.1

is chiefly determined by the construction and dimension of the access zone and the tunnel facade. Adaptation only begins to be affected when the last part of the facade has disappeared behind the upper rim of the car windshield. The average shielding angle of the windshield is about 7^{o}. Therefore, the higher the facade, the further the adaptation point is from the entrance. For a well-designed approach structure, the distance of the adaptation point from the tunnel entrance is around 200' as shown in Fig. D.2. The present tunnel entrance has a height of 26', therefore, the adaptation point is 210' from the entrance.

The length of the threshold zone shown in Fig. D.1 depends on the distance 'd' at which the critical object has to be visible. To this length should be added a stretch of about 65' which serve as background for obstacles. The distance 'd' varies depending on the speed of the car. For speed limit of 45 mph, it can be assumed as 300' which is considerably greater than the minimum braking distance. It is chosen extra large for town tunnels with speed restrictions. Therefore the length of the threshold zone is 365' less the distance between the adaptation point and the entrance of the tunnel structure, i.e., a total distance for the threshold of 155'. In most cases, 150' to 250' will be sufficient.

As the driver passes the adaptation point, his eyes begin to adapt themselves to the dark area of the tunnel entrance. If the lighting levels between the adjacent zones do not exceed a reduction ratio of 10 to 1, inconvenience is chiefly determined by the construction and dimension of the access zone and the tunnel facade. Adaptation only begins to be affected when the last part of the facade has disappeared behind the upper rim of the car windshield. The average shielding angle of the windshield is about 7°. Therefore, the higher the facade, the further the adaptation point is from the entrance. For a well-designed approach structure, the distance of the adaptation point from the tunnel entrance is around 200' as shown in Fig. D.2. The present tunnel entrance has a height of 26', therefore, the adaptation point is 210' from the entrance.

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LOCATION OF ADAPTATION POINT

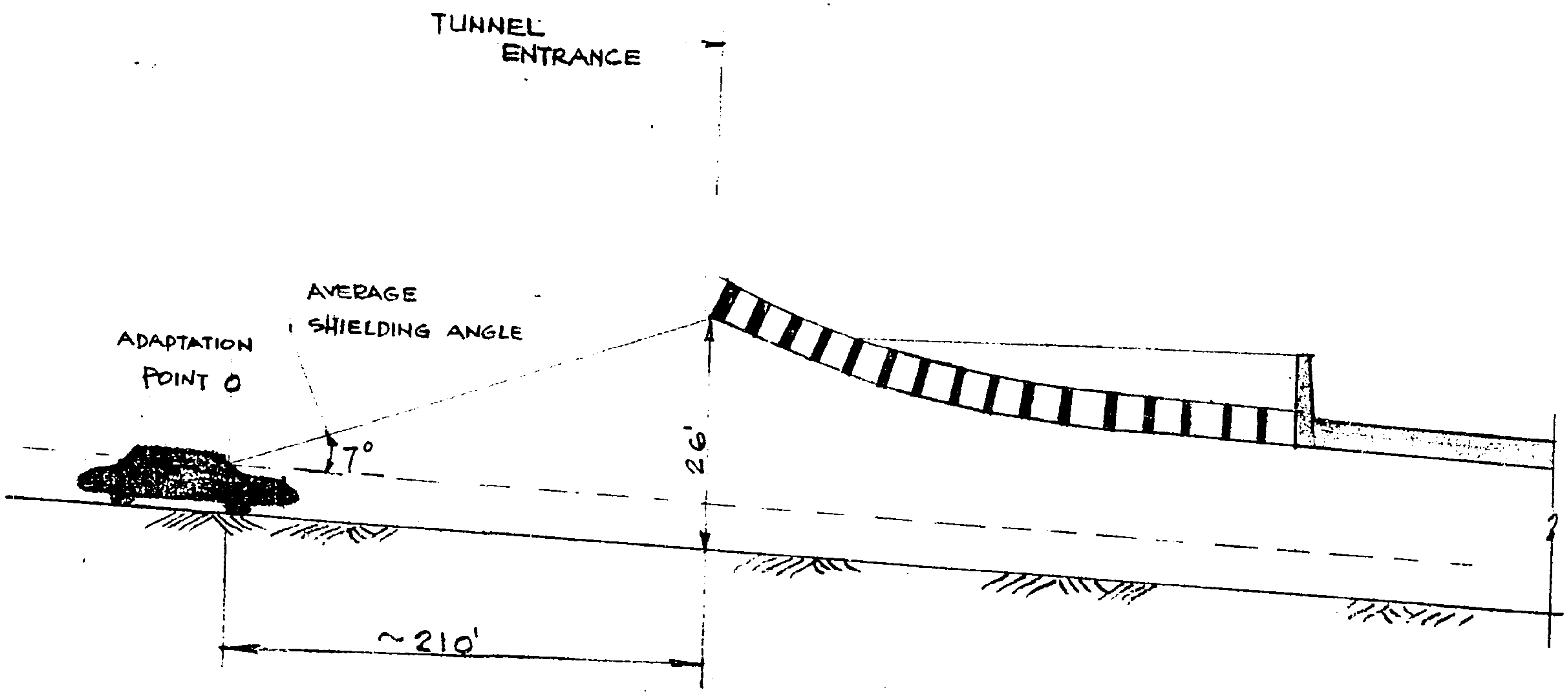


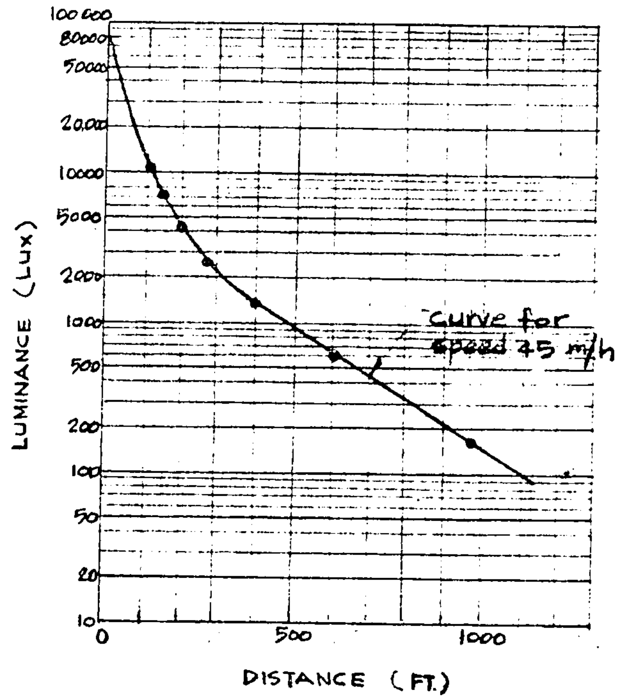
Fig. D.2

caused by after images is prevented. Many tunnel lighting systems have been designed to satisfy this requirement. Schreuder(11) developed experimentally a curve relating the luminance of field of vision and the distance in the tunnel necessary for satisfactory adaptation for a speed of 45 mph. This curve is shown in Fig. D.3a. He recommends attaching this curve to the end of the threshold zone.

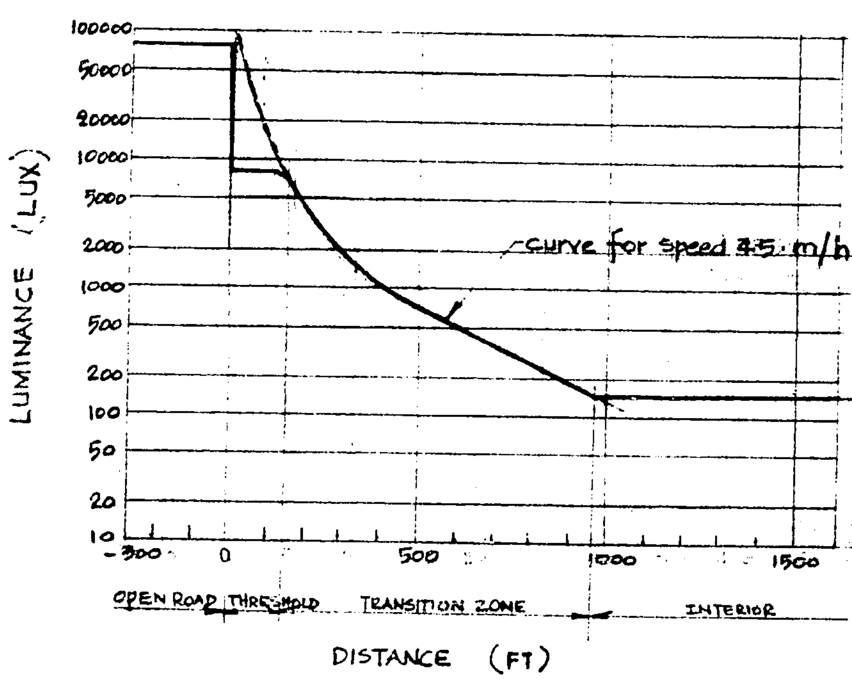
According to the above luminance course, the present tunnel which is designed for a speed of 45 mph must have a transition zone of about 800' if the luminance in the interior is to have 150 lux as the lighting level at daytime must be at least 100 lux and preferably 150 to 200 lux. This results in a total length for the whole entrance zone of about 950'. Of course, another choice of driving speed results in another length of the transition zone. Fig. D.3 can still be used in the manner described except that the scale of the abscissa has to be changed. The length intervals of Fig. D.3a may be converted back to time interval as was originally recorded. For a chosen speed, these time intervals may then be changed back to length intervals to form a new curve for attaching to the end of the threshold zone. This has been done for the Tampico Tunnel as shown in Fig. D.3b.

Traffic Tube - The entrance to the traffic tube will be illuminated to an intensity of 1,500 lux, gradually reducing over a distance of 620' to 150 lux on clear days as indicated by the curve in Fig. D.3. At night, luminance level in the

LUMINANCE AND DISTANCE REQUIRED FOR SATISFACTORY ADAPTATION



(a)



(b)

Fig. D.3

tunnel will be reduced to 50 lux. The pyrex encased fluorescent luminaires will be mounted on the underside of the top slab of the tunnel above each traffic lane.

Concave Wall Section - To highlight the concave lower section of each sidewall, as an aid to driving comfort and safety, a continuous row of low power wattage fluorescent luminaires will be installed.

Pedestrian Passageway - The lighting of each pedestrian passageway will be provided by the installation of mercury vapour lamps 50' apart. This economical type of illumination may be manually controlled for use when required.

Ventilation and Service Ducts - These ducts in the lower sections of the sidewalls will be illuminated by incandescent lamps with explosion-proof fixtures spaced about 50' apart.

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