CHAPTER 8

PRELIMINARY STRUCTURAL DESIGN

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8.1 INTRODUCTION

The preliminary structural design is carried out based on the following basic approach:

- Establish preliminary design policy
- Establish preliminary design criteria
- Make a full appreciation of existing conditions and constraints from site visits, reference to satellite data, desk studies of available data and from photographic records at critical sections
- Establish extent and impact of other existing and proposed works in the vicinity of the structures
- Determination of engineering characteristics of sub-soils from geotechnical investigations and desk studies
- Establish operational clearance requirements of existing and proposed traffic lanes from site visits, topographic surveys and the highway alignment design to determine structure spans and substructure layout and location
- Establish operational and construction clearance requirements at railway lines to determine structure spans and substructure layout and location
- Determine impact of method of construction on traffic management and railway operations
- Establish alternatives and make comparative studies for selection of scheme for preliminary design
- Make proper reference to GARBLT counter part team and other concerned bodies in establishing and comparing alternatives for consideration in the preliminary design

8.2 PRELIMINARY DESIGN POLICY

The policy adopted for the preliminary structural design is as follows:

- Selection of structure types to minimize construction period with rapid bridge construction techniques and reduce impact on traffic during construction
- Cost competitive construction
- Focus on structure types that will result in minimal maintenance obligations in the future
- Pre-cast or prefabricated construction over railway lines
- Foundation types selected to minimize disruption during construction

- No detrimental effect on existing structures
- Minimize expansion joint and bridge bearing locations
- Clean structure lines where possible to enhance visual impact

8.3 PRELIMINARY DESIGN CRITERIA

The preliminary design in the Study is undertaken based on the Egyptian standards. Egyptian standards are supplemented where necessary by AASHTO and Japan Road Association (JRA) standards.

8.3.1 Design Loads

(1) Dead Load

The following unit weights are used for the preliminary design.

Table 8.3-1 Unit Weight of Materials

Items	Unit Weight
Steel	77.0 kN/m^3
Reinforced Concrete	24.5 kN/m^3
Plain Concrete	23.0 kN/m^3
Asphalt Concrete	22.5 kN/m^3

(2) Live Load

Live load on a notional major road lane of width 3.0m shall be a 60 ton design truck load, comprised of 3-axle loads of 20 ton each, together with 500kg/m^2 uniformly distributed load on the remaining lane area.

Live load on an adjacent notional road lane of width 3.0m shall be a 30 ton design truck load, comprised of 3-axle loads of 10 ton each, together with 300kg/m^2 uniformly distributed load on the remaining lane area.

All remaining areas shall receive 300kg/m^2 uniformly distributed load.

The position of the notional lanes and the location of the two design truck loads are selected to give the greatest effect for the design.

Refer to Figure 8.3-1 and Figure 8.3-2 for the arrangement of truck and uniformly distributed loads respectively.

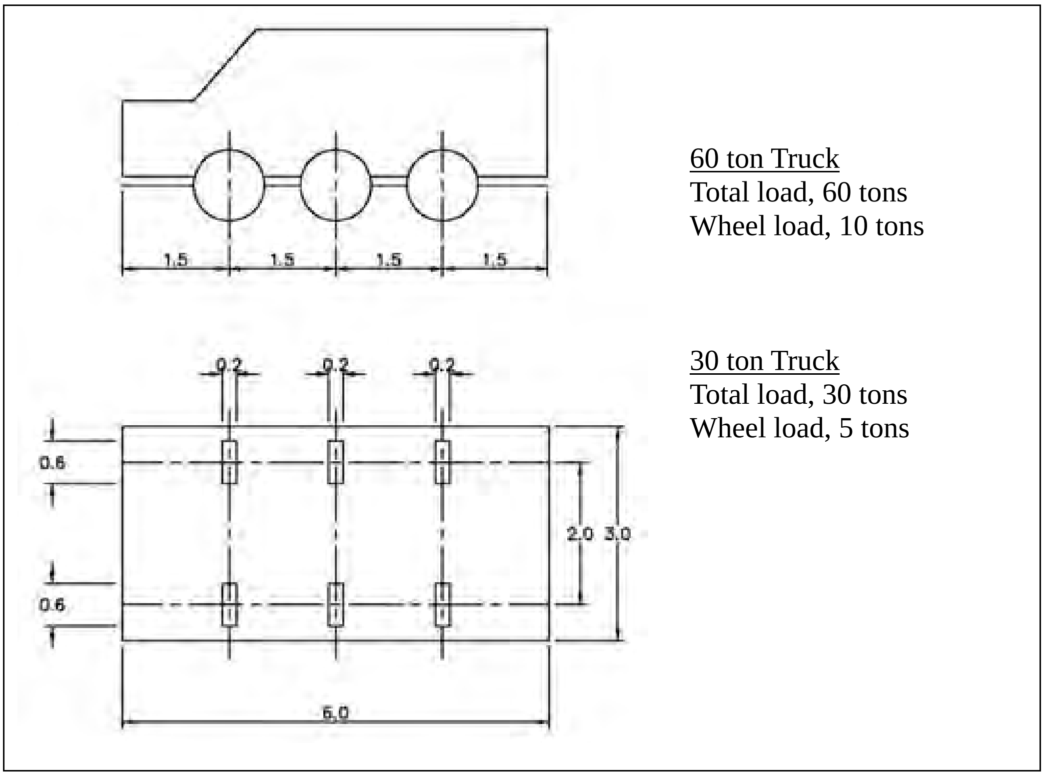


Figure 8.3-1 Truck Load

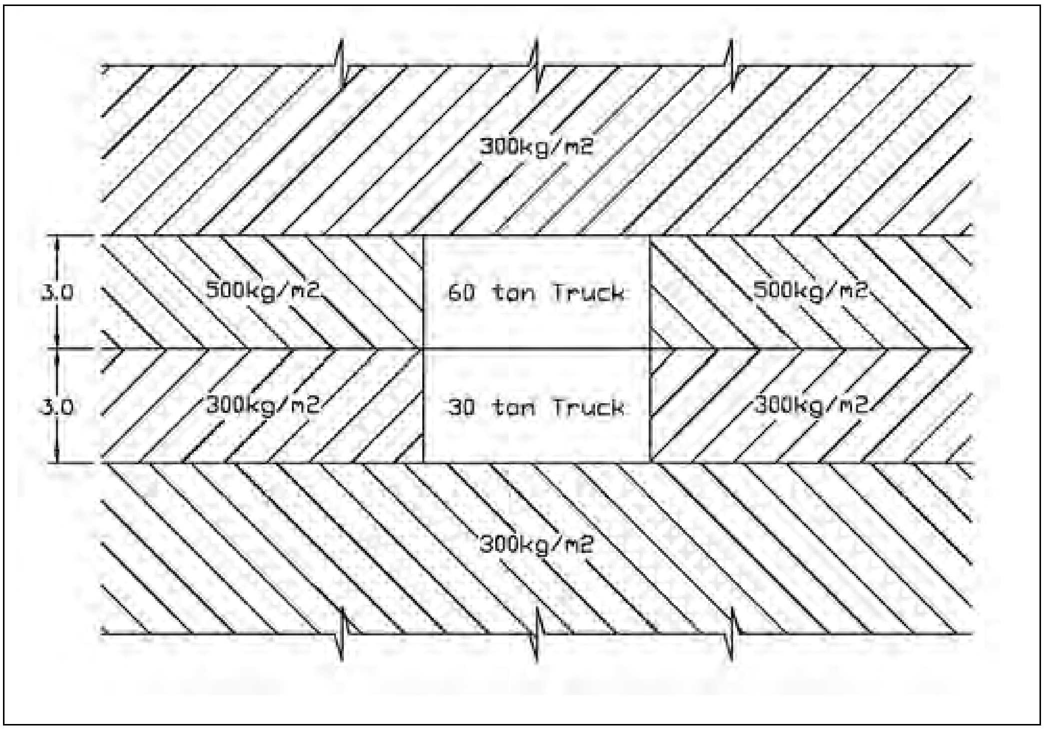


Figure 8.3-2 Uniformly Distributed Load

(3) Impact

Impact effect is added only to the live load in the major lane and not to the load in the adjacent lane or the load on the remaining bridge deck surface.

The impact load is calculated as follows:

Impact Factor: $I = 0.4 - 0.008 \times L_I$

L_I is the loaded length for impact calculation determined as follows:

Concrete Bridges:	A. In case of direct load, L_I is equal to the span length B. In case of indirect load, L_I is equal to the longest span length C. In case of loading in both directions, L_I is equal to the shortest span length
Steel Bridges:	L_I is taken with respect to the major lane over the length giving the maximum effect on elements to be designed.

Impact effect is not considered for foundation design.

(4) Pedestrian Load

A uniformly distributed pedestrian live load of 600kg/m^2 shall be applied to sidewalk areas. Impact effect is not considered for uniformly distributed pedestrian load.

If the height of the curb is less than 40cm (thus allowing the mounting of a vehicle) the structural elements of the sidewalk shall be checked for the effect of a vertical or horizontal point load of 4 tons acting alone (including dynamic effect) in the position giving the maximum stress.

The expressway sections specifically do not include pedestrian sidewalks and therefore do not feature curbs.

In the cases where bridge structures will feature curbs, such as where existing structures with sidewalks have to be widened to accommodate the expressway, curb height will match existing conditions

(5) Centrifugal Force

For curved roadway bridges, centrifugal forces shall be applied in any two notional lanes at 50m centers, acting in radial direction at the surface of the road and parallel to it. The nominal centrifugal force “C” shall be taken as follows:

$C = 3000/R + 150$ (in metric tons)

where *R* is radius of curvature of the lane in meters.

With each centrifugal force there shall also be considered a vertical live load of 30tons distributed uniformly over the notional lane (3m width) for a length of 5 meters.

(6) Temperature Change Effects

The average design temperature of bridges shall be taken as 20^{O}C. The design temperature change from average conditions shall be ± 20^{O}C for concrete bridges and ± 30^{O}C for steel bridges.

The design temperature gradient between upper and lower surfaces of bridge decks shall be ± 5^{O}C for concrete bridges and ±15^{O}C for steel bridges.

The coefficient of expansion shall be taken as 1.2×10^{-5} for steel and 1.0×10^{-5} for concrete.

(7) Braking Load

Braking load shall be taken as 25% of the traffic load in the major lane and shall not be taken to be less than 18tons and more than 90tons. Braking forces shall act at the level of the crown of the bridge deck.

Impact effect is not considered for braking load.

(8) Wind Load

When the bridge structure is unoccupied by live load, the maximum design wind pressure shall be 200kg/m^2. When there is live load on the bridge structure, the design wind pressure shall be 100kg/m^2 and shall be assumed as acting on the exposed surfaces both of the structure and of the live load. The wind pressure on the live load shall be considered as acting at the center of the exposed area.

For the windward girder, the net exposed area in normal projected elevation of the girder, truss girder and parapet shall be assumed to be exposed to the wind effect.

For the leeward girder, the following fractions of the net exposed area in normal projected elevation shall be assumed in calculating the leeward exposed area:

- [n/16] when the windward girder is a plate or concrete girder
- [n/8+0.5] when the windward girder is a truss

  where: n = ratio of distance center to center, between the windward and outer most leeward girder, to the depth of the windward girder.

The leeward suction should always be less than the windward pressure.

When there are more than two girders, only that fraction of the area of the outermost leeward girder shall be taken into consideration.

The unscreened wind area for live load on any bridge shall be taken as a single vertical plane surface having a continuous height of 2.0m above the roadway and of such length along the bridge as to produce maximum stresses in the members under consideration.

The maximum effect from the wind blowing in the lateral direction either on the loaded or unloaded structure shall be taken.

A longitudinal wind force equal to half the total lateral wind force as calculated above shall be taken. This force shall be combined with half the lateral wind force as calculated above.

(9) Earthquake Load

The following equivalent static earthquake loading will be considered in the preliminary design, based on findings as given in Appendix A8.

For short to medium span bridge structures, earthquake load "EQ" shall be taken as:

- For firm soil conditions, $EQ = 8.0\%(Dead\ Load + 20\%\ Live\ Load)$
- For medium conditions, $EQ = 9.0\%(Dead\ Load + 20\%\ Live\ Load)$
- For soft soil conditions, $EQ = 10.0\%(Dead\ Load + 20\%\ Live\ Load)$

For cable supported bridge structures, earthquake load "EQ" shall be taken as:

- For all soil conditions, $EQ = 6.0\%(Dead\ Load + 20\%\ Live\ Load)$

The components of the seismic actions shall be combined as follows:

- Absolute sum of 100% of internal forces resulting from ground motion in one of three orthogonal directions and 30% of internal forces from ground motion in the two other directions

The detailed design should be based on a more rigorous approach as given in the Appendix 8.

8.3.2 Design Load Combinations

The design load combinations shall be as follows:

(1) Ultimate Limit State (ULS)

Case	1	1.40 D + 1.6 L
	2	1.2 (D + L + W)
	3	1.2 (D + L + T)
	4	1.2 (D + L + Br)
	5	1.2 D + 0.30 L + 1.0 EQ

(2) Serviceability Limit State (SLS)

Case	1	1.0 D + 1.0 L
	2	0.8 (D + L + W)
	3	0.8 (D + L + T)
	4	0.8 (D + L + Br)
	5	0.8 D + 0.20 L + 0.8 EQ

Where:

D = Total Dead Load including Superimposed Dead Load
L = Traffic Load (Live Load)
W = Wind Load
T = Temperature Change
Br = Braking Load
EQ = Earthquake Force

The detailed design should include additional load cases to include for differential settlement of foundations and shrinkage and creep in concrete.

8.3.3 Material Properties

(1) Structural Concrete

Following concrete characteristics determined by 28-days compressive strength (with 150mm x 150mm cubes) are adopted in the preliminary design:

- 300 kg/cm^2 (29 MPa), abutments, diaphragm walls, bored piles and pile caps
- 450 kg/cm^2 (44 MPa), pier columns and decks

(2) Reinforcing Bar

Reinforcing bar shall be Grade 52 with an ultimate tensile strength (UTS) of 52 ton/mm^2 (510 MPa). Yield strength of reinforcing bar shall be taken as 90% UTS = 460 MPa.

(3) Structural Steel

Structural steel shall be Grade 52 with an ultimate tensile strength (UTS) of 52 ton/mm^2 (510 MPa).

8.3.4 Clearances

(1) Vertical Roadway Clearance

The minimum operational vertical clearance over at-grade roadways, other than in tunnels and depressed structures, shall be 5.50 m as a desirable minimum standard. Absolute minimum shall be 4.5m. The minimum operational vertical clearance over elevated roadways shall be 4.8 m.

In tunnels and depressed structures the minimum operational vertical clearance over roadways shall be 4.50 m.

Along routes used by the Ministry of Defense (MoD), including Nasr Road, the minimum operational vertical clearance over at grade roadways shall be 6.0 m.

(2) Railway Clearances

The minimum operational and construction clearances at railways shall be as given below. The clearances were confirmed following a meeting with Egypt National Railways (ENR) officials.

- Horizontal operational clearance (above ground) from edge of rail to face of structure is 2.5m minimum.
- Vertical operational clearance above rail track to underside of structure is 5.5m. Vertical clearance shall be measured from the top of the highest rail.
- For foundations the construction clearance requirements are:
  - ✧ A minimum of 1.0m from the edge of the sleeper to the face of the permanent foundation structure is required
  - ✧ No specific requirements are specified by ENR for deep excavations adjacent to the track; the basic requirements are that methods of construction of foundations adjacent to the track should ensure the stability of the track
  - ✧ For piling rigs working adjacent to the track; a required working clearance of 1.5 times the height of the rig is required from the railway track to ensure that an overturned rig will not fall on the track. If temporary possession of the track can be arranged then this requirement is waived.

- For the double track in rock cut at the Citadel an overall width of 10.5m is required for the railway made up as follows:
  - ✧ 2 x 2.5m (edge clearance) + 2 x 1.5m track gauge (twin track) + 2.5m (inside rail – to – inside rail offset)

8.3.5 Geotechnical Design Criteria

(1) General

In consideration of the types of bridge foundations typically constructed in Cairo, deep foundations are proposed for the bridge structures for all expressway sections.

Deep foundations for bridge structures shall be cast-in-situ bored piles with diameters from 60cm up to 1.5m for the typical case. Notwithstanding that pile diameters larger than 1.8m have not been used on previous bridge construction in Cairo, consideration will be given in the preliminary design to the use of bored piles with diameters up to 3.0m for special cases where the use of such large diameter piles can be justified.

The center to center spacing of bored piles shall be equal or greater than diameters. Bearing capacity shall be determined using the groundwater level consistent with that used to calculate load effects. The effects of buoyancy shall be considered in the design

The ultimate factored axial bearing capacity design of the bored piles shall be based on the provisions of AASHTO LRFD Bridge Design Specifications, Third Edition, 2005.

The bearing capacity of the pile includes contribution from both shaft and base, with corresponding strength reduction factors applied. The strength reduction factors depend on the type of soil and whether shaft or base resistance is being calculated. The specific methods adopted to determine shaft and tip resistance are fully prescribed in AASHTO LRFD.

(2) Presumptive Allowable Bearing Capacity of Bored Piles

The following presumptive allowable bearing capacities are used in the preliminary design at this stage:

- 60cm diameter : allowable (working) capacity = 90 tons
- 80cm diameter : allowable (working) capacity = 200 tons
- 100cm diameter : allowable (working) capacity = 350 tons
- 150cm diameter : allowable (working) capacity = 780 tons

The above capacities are based on values obtained on previous projects in Cairo, according to the experience of the Arab Contractors.

(3) Diaphragm walls and Cut-and-Cover Construction

Two types of diaphragm wall equipment are available and have been used in Cairo for the construction of diaphragm walls: hydraulic grabs for normal soils conditions and Hydrophrase[1] units for cutting through hard soil/ rock layers.

Size of available grabs is 60 cm, 80 cm and 100 cm, and a panel width is 2.7 m.

Depths down to 42 m have been achieved with Hydrophrase. Greater depths can be achieved with the grab type equipment (cable suspended). Diaphragm wall construction can be applied to all types of soil/rock conditions encountered in Cairo according to the experience of The Arab Contractors.

For limited headroom locations, such as crossing points beneath 1st level flyovers, diaphragm wall equipment cannot be used. In this instance modified boring rigs can be used to construct temporary bored pile curtain walls in place of the diaphragm wall to support the open cut.

8.4 RAPID BRIDGE CONSTRUCTION

8.4.1 General

The benefits of accelerated bridge construction in a confined urban environment, with a focus on prefabrication of structural elements, are well known. They include minimized traffic disruption and congestion, improved work zone safety, and minimized environmental impact. Additionally, prefabrication can improve constructability, increase quality, and lower life-cycle costs. Rapid bridge construction is prevalent in urban areas of Europe and Japan.

Prefabricated bridges offer significant advantages over onsite cast-in-place construction in several key areas. The substantial reduction in onsite time required to construct a prefabricated bridge is an advantage that is critical in many cases. Offsite manufacturing minimizes the impact on bridge or roadway closure time. Reduction in onsite construction time leads to lower costs for contract administration, traffic control, environmental impact mitigation, and costs incurred by the public due to traffic congestion or detours, a concern that is continually taking on greater significance. Offsite manufacturing may have the lowest cost that can be achieved, e.g., if standardized components are used or if a large number of similar components are required in the project. A further benefit from decreased onsite construction time is the lower potential for traffic and construction accidents. Another advantage is that offsite fabrication allows greater quality control in the manufacturing of the bridge components, providing quality components for good long-term performance.

[1] Hydrophrase; name of machine specialized for trench excavating

Many job sites impose difficult constraints on the constructability of bridge designs: heavy traffic on existing city roads, difficult elevations, restricted work areas due to adjacent buildings or other facilities. Using prefabricated bridge elements and systems relieves such constructability pressures.

The significant advantages of prefabricated bridge construction do not come without challenges or concerns. Some of these are the lack of knowledge and experience in prefabricated bridge system design and detailing, including connections between components; durability of the connection details; ability of the prefabricated system to accommodate curvilinear geometry; details to develop negative moment continuity; availability of prefabricators capable of producing the components; limitations on component size; availability of equipment to erect the components; and knowledge/experience of local bridge contractors with techniques needed to construct bridges built of prefabricated components.

A success of a project is dependent on properly deciding whether a job should be fast track, the applicability of the design, abilities of contractors and suppliers and recognizing how the construction requirements affect cost and schedule.

The Study has therefore made use of suitable forms of construction in conceiving preliminary bridge designs at locations where rapid bridge construction is identified as a high priority.

8.4.2 Substructure Forms for Rapid Construction

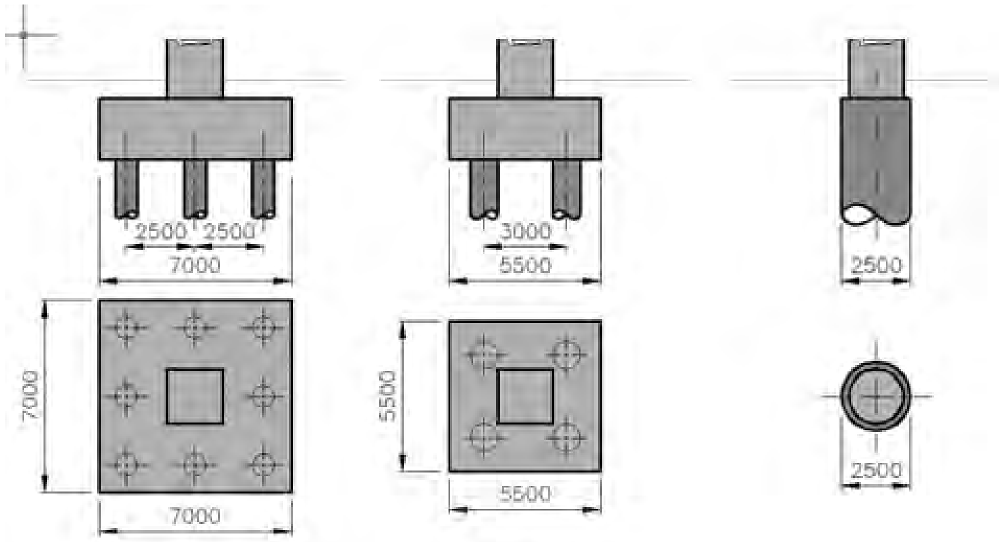
The following issues can be considered in assessing bridge foundations for rapid bridge construction:

- Rationalized Foundation and Substructure Layouts
- Prefabricated Piers
- Rotated Pier Caps

(1) Rationalized Foundation and Substructure Layouts

The layout of bridge foundations in crowded urban settings can have a significant impact on both construction time and impact on other facilities, particularly underground utilities. Minimizing both the number of piles and the pile cap size, in the case of pile foundations, is a key consideration in avoiding both conflicts on site and in reducing time for construction. Dispensing with pile caps altogether, such as with the use of single large diameter bored piles, will substantially promote rapid construction and minimize conflicts with exiting utilities.

Figure 8.4-1 illustrates the layouts of three different foundations with similar vertical bearing capacities.



- Footprint 7.0mx7.0m
- Requires Pile Cap
- Can be installed at limited headroom locations

8 No. 800mm Dia. Bored Piles

- Footprint 5.5mx5.5m
- Requires Pile Cap
- More difficult to install at limited headroom locations

4 No. 1000mm Dia. Bored Piles

- Footprint is the pile
- No Pile Cap
- Cannot be installed at limited headroom locations

Single 2500 mm Dia. Bored Pile

Figure 8.4-1 Alternative Foundation Layouts

Multiple column supports at pier locations are structurally the most efficient and provide substantial ductility and redundancy to the design, particularly beneficial in zones with high earthquake forces. However multiple column piers require not only greater space for construction, with increased risk of conflict with existing facilities, but also multiple construction stages, both of which adversely affect speed of construction. Single column piers centrally located in existing medians, though not the most structurally efficient, are best suited to rapid construction techniques in terms of their rationalized foundation design, occupy the least space that can otherwise be used to accommodate at-grade traffic lanes and sidewalks and entail fewer construction stages.

(2) Prefabricated Piers

- Concrete Bridge Piers

Prefabricated concrete piers are not common for small to medium span bridges and are typically used for major bridge projects where prefabrication brings clear benefits, such as at sites with extreme conditions where in-situ work is not possible (such as at the Second Severn Crossing in the UK which is sited in an estuary with a very large tidal range).

In the USA, however, since the 2001 AASHTO Technology Implementation Group’s focus on Prefabricated Bridge Elements and Systems, there have been uses of precast concrete for pier column bents (Waco, Texas –1170m long bridge with 64 repeating 18m long spans) and precast post-tensioned column segments (Dallas/Fort Worth International Airport – minimizing site possession time was critical) where pre-fabrication offered clear advantages.

Figure 8.4-2 illustrates some recent precast concrete pier bridges in the USA.

Lake Belton Bridge, Waco, Texas

Precast Concrete Hammerhead Bent

Forth Worth International Airport, Dallas, Texas

Precast Post-Tensioned Column Segments

Figure 8.4-2 Prefabricated Concrete Bridge Piers in the USA

- Steel Bridge Piers

Steel piers are not as commonly used as concrete piers. However, given the high strength/weight ratio and compact design, steel bridge piers can be found in urban settings where concrete piers would provide a too massive alternative (such as for tall/long span pier bents at multiple level road interchanges) or where at-grade lane configurations require slender solutions for bridge supports.

Examples of steel bridge piers used on the Nagoya Expressway in Japan are illustrated in Figure 8.4-3.

Figure 8.4-3 Steel Bridge Piers – Nagoya Expressway, Japan

Steel pipes or tubes filled with concrete known as composite columns offer the most efficient use of the two basic materials. Steel at the perimeter of the cross section provides stiffness and triaxial confinement, and the concrete core resists compression and prohibits local elastic buckling of the steel encasement. The toughness and ductility of composite columns makes them the preferred column type for earthquake-resistant structures in Japan. In China, composite columns were first used in Beijing subway stations as early as 1963. Over the years, composite columns have been used extensively in building structures as well as in bridges

Composite columns can be very effectively used with large diameter bored piles. Following research and testing in Japan conducted for Japan Railways, a construction methodology has been developed, approved for use by both Japan Railways and Japan Road and Highways, which ensures transfer of forces between the composite column and the pile. The method involves embedding a section of the composite column (in the order of 2-3 times the column diameter) within the top section of the bored pile. Over this embedded length an outer steel pile casing is installed, with the casing extending for a length below the base level of the column.

The arrangement of the column-pile joint is shown in Figure 8.4-4.

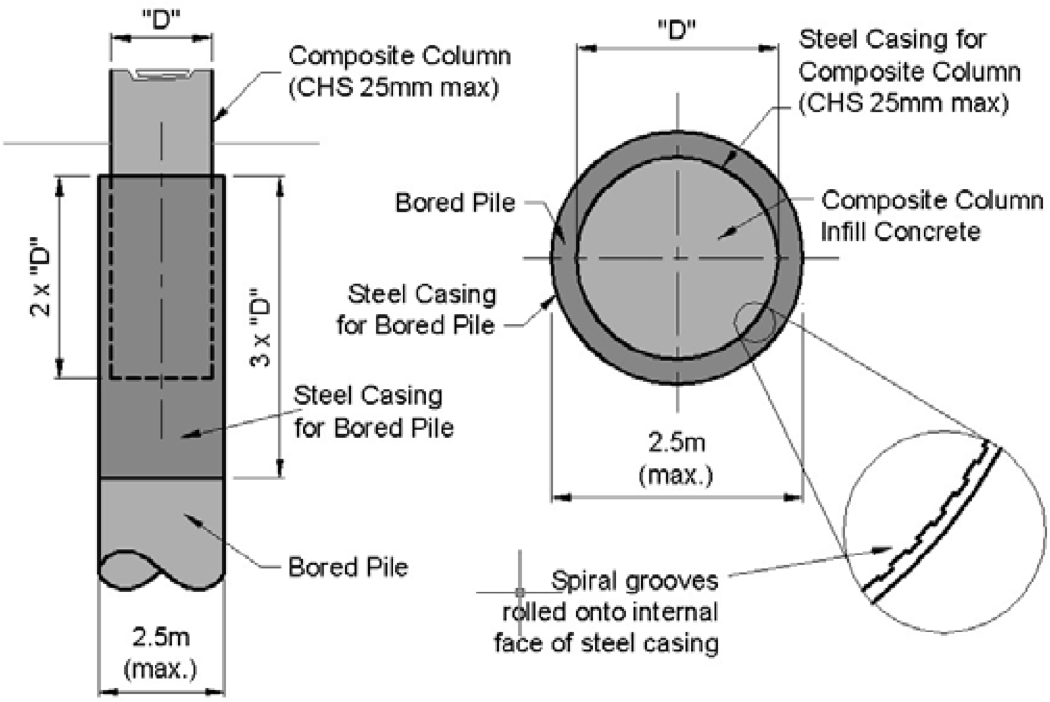


Figure 8.4-4 Composite Column-Pile Joint Made of Steel Pipes Filled with Concrete

In Japan large diameter steel pipes up to 2.5m in diameter can be fabricated using spiral pipe mills. Thickness of the pipe wall can range from 5mm to 25mm. In addition, the plates can be rolled with grooves that then form continuous spiral features on the inside face of the finished pipes, creating a very effective interface to carry shear stresses between the concrete core and the steel casing of the composite column.

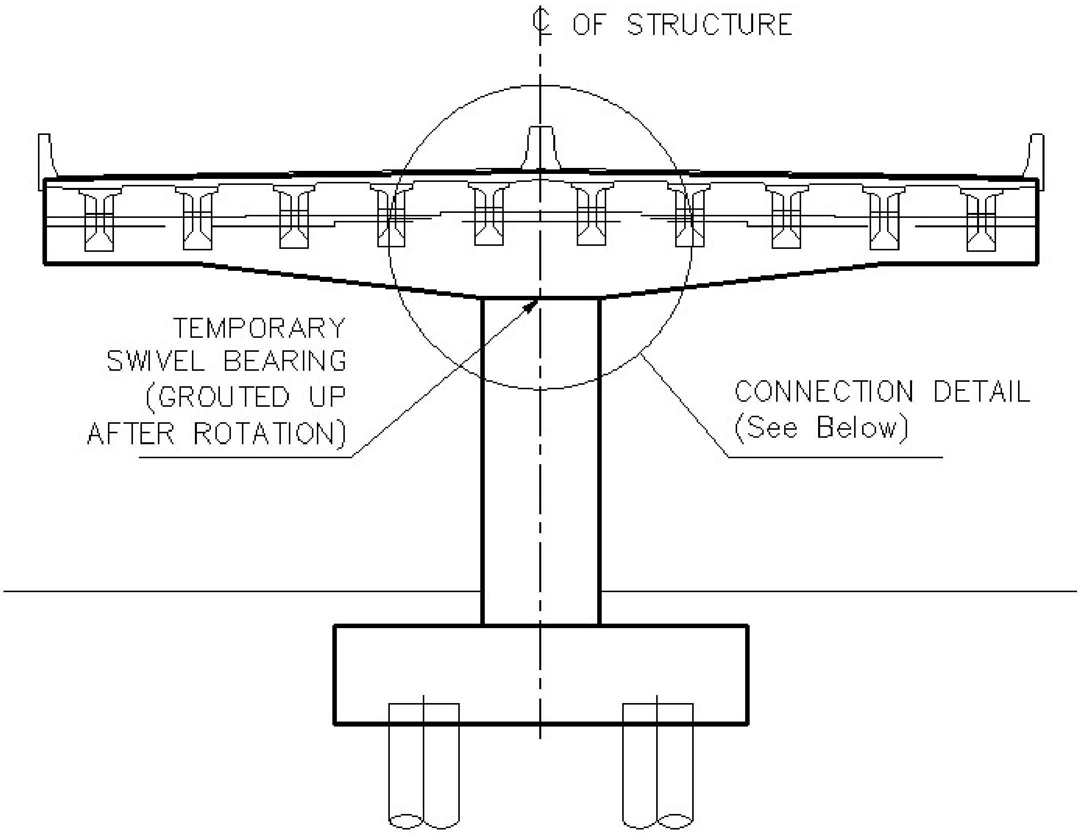
(3) Rotated (Swing) Pier Caps

Single column piers supporting wide dual three lane expressway decks above parallel at-grade local roads can be constructed using rotated (swing) pier caps.

The method involves casting the cap on the column in a longitudinal orientation and on temporary swivel bearing, occupying the central reserve and thereby minimizing impact on local traffic during its construction. Once the concrete has developed adequate strength the shoring can be struck and, during a night, the cap is rotated on its temporary bearing. Once in its final orthogonal position, cantilevered over the at-grade lanes, the cap is grouted into position and vertical pre-stress is installed to connect the cap with the pier column. This method was adopted during the construction of the South Luzon Expressway south of Manila, Philippines.

A typical detail of the vertical pre-stress is shown in Figure 8.4-5. The stages of construction are shown in Figure 8.4-6.

For more details of the rotated (swing) pier cap construction method refer to Section 14.1.2.



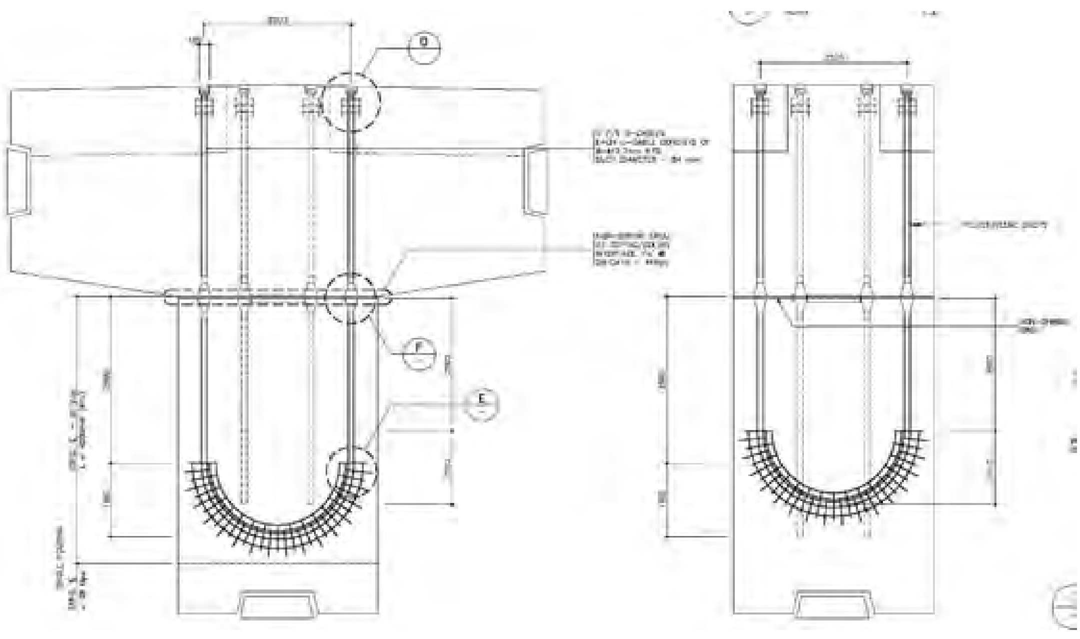


Figure 8.4-5 Vertical Pre-stress for Rotated Cap – Single Column Pier

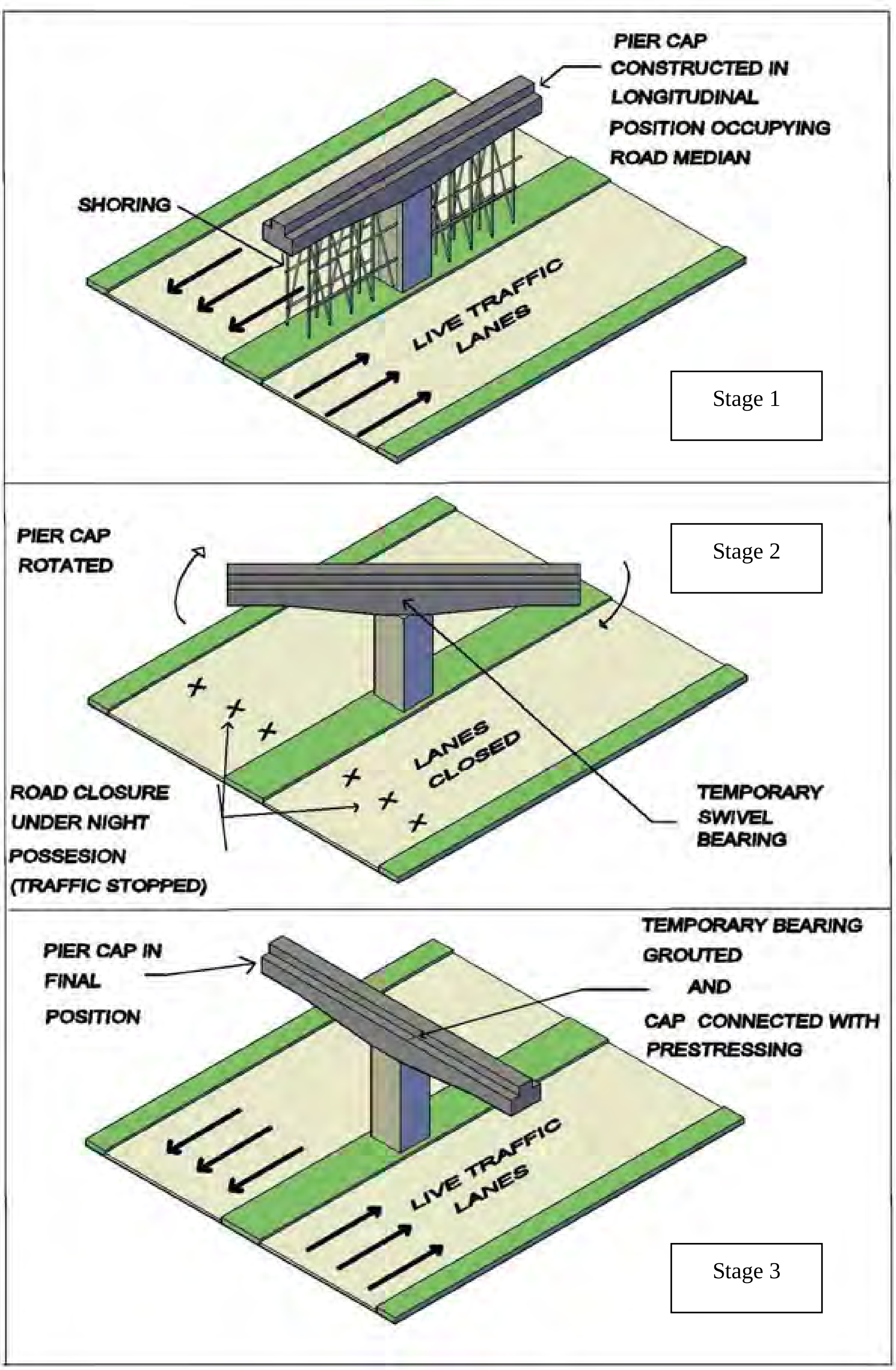


Figure 8.4-6 Rotated Pier Cap – Stages of Construction

8.4.3 Superstructure Forms for Rapid Construction

Prefabrication and standardization of structural elements is the key to rapid construction of bridge decks. Consideration can be given to the prefabrication of all major elements that go into the deck including:

- Prefabricated beams and deck segments
- Prefabricated deck slabs
- Prefabricated railings

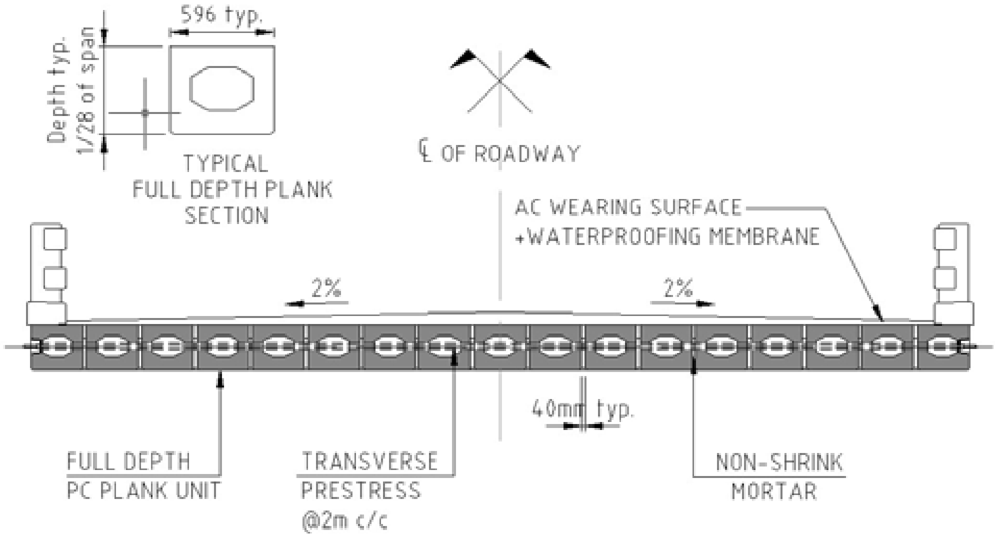
(1) Prefabricated beams and deck segments

- Concrete Bridge Decks

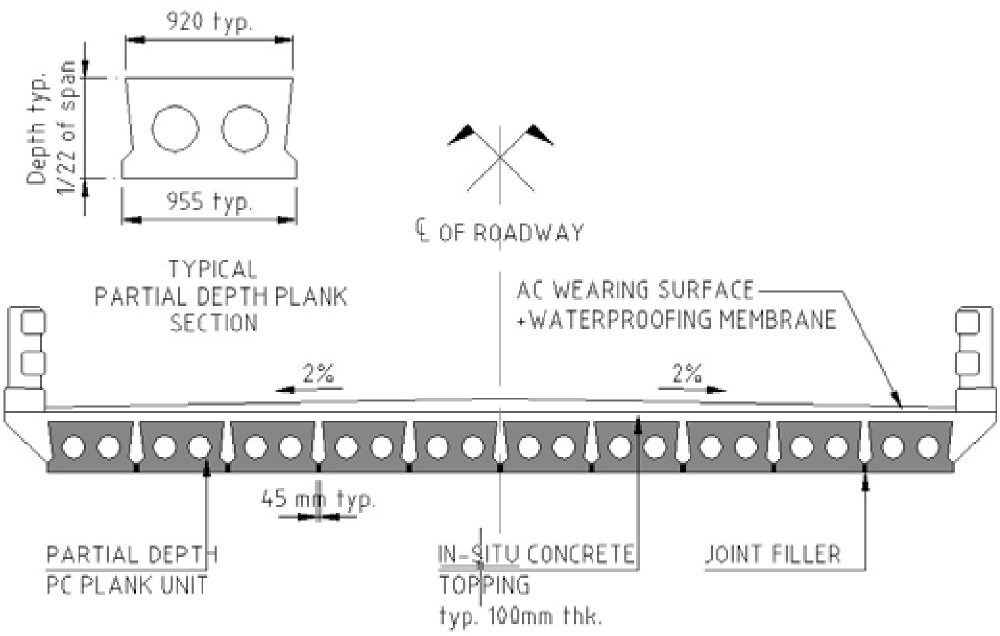
Prefabricated concrete bridge decks can either take the form of (1) several precast planks or girders placed longitudinally side by side, either with joints between planks/girders parallel to the longitudinal axis of the bridge or with an in-situ deck slab, or (2) segmental construction where the segments are slices of a structural element between joints which are perpendicular to the longitudinal axis of the structure.

Precast pre-tensioned planks can be economically applied for spans ranging from 12m up to 21m, although longer spans up to 28m can be achieved. Two systems are in use: (a) a full depth plank with no in-situ concrete topping and with the planks laterally post-tensioned and (b) partial depth planks with female-female longitudinal joints and an in-situ concrete topping and lateral mild reinforcement. The planks can be used on curved decks but, in this case, require specially shaped edge units.

An example of both deck plank systems is shown in Figure 8.4-7.



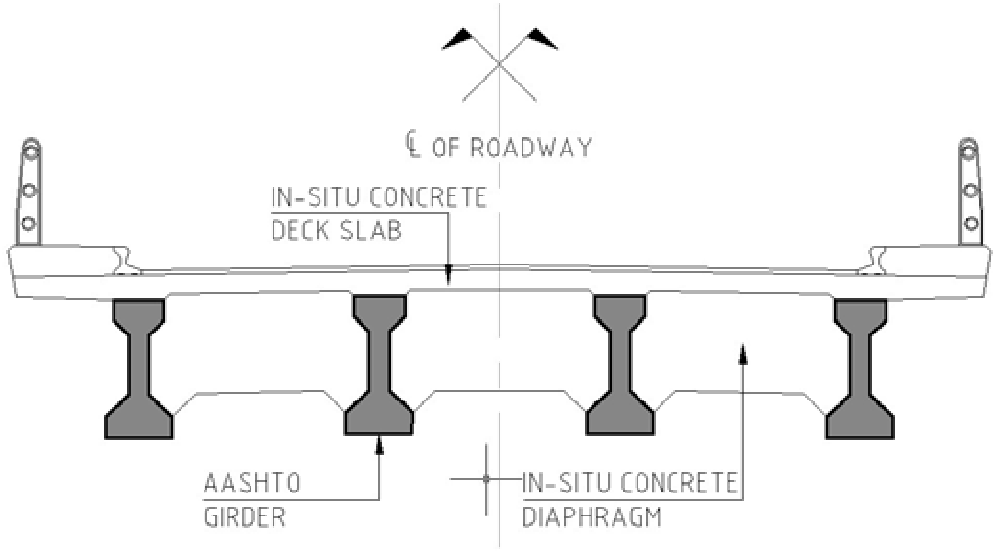
a. Full Depth Plank with Transverse Pre-stress



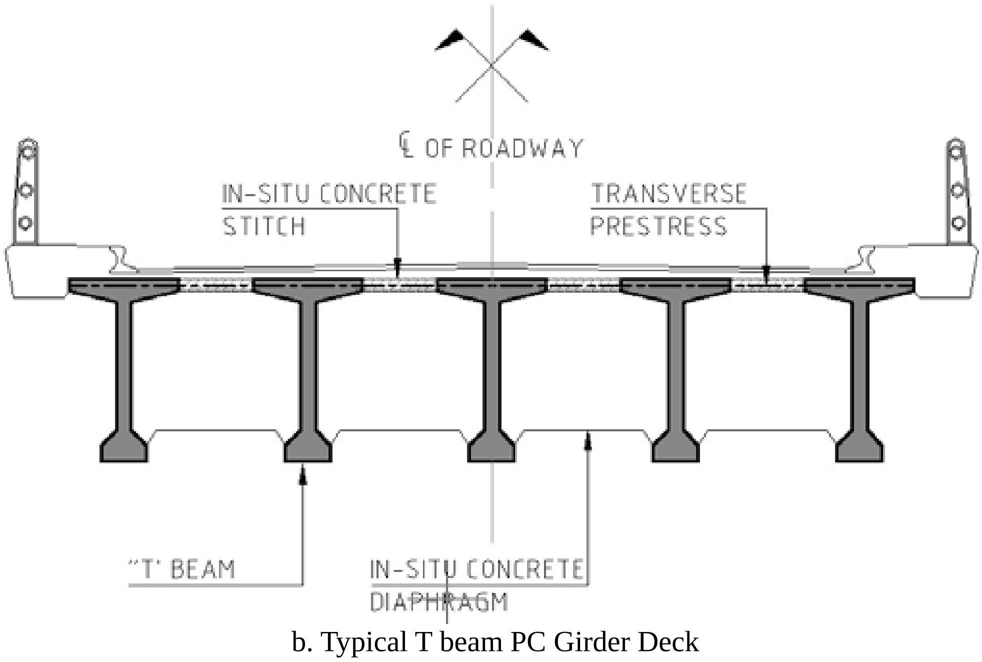
b. Partial Depth Plank with In-situ Concrete Topping

Figure 8.4-7 Precast Pre-Tensioned Concrete Plank Deck Types

Precast girders forming beam and slab decks are applicable for short to medium span applications (up to 40m) and can be constructed with conventional erection gantries and cranes. The girders are typically post-tensioned. This form of construction is common both in urban and rural locations. In order to minimize falsework stages and in-situ concrete work, the precast beams can be precast with top slab extensions, requiring only a narrow longitudinal in-situ concrete stitch, or can make use of stay-in forms, partial depth or full depth precast deck panels. The primary disadvantage of this form of deck is that the girders have little torsional stiffness and require intermediate transverse diaphragms in order to achieve adequate torsional strength and good transverse load distribution. Precast beam and slab bridges are therefore not well suited to highly curvilinear alignments. The diaphragms also increase the number of construction stages required, typically involving work above live traffic lanes (to maintain the benefits of the rapid precast construction of the girders), leading to increased risk of accidents and longer construction periods. Figure 8.4-8 illustrates typical PC Girder deck section.



a. Typical AASHTO type PC I-Girder Deck



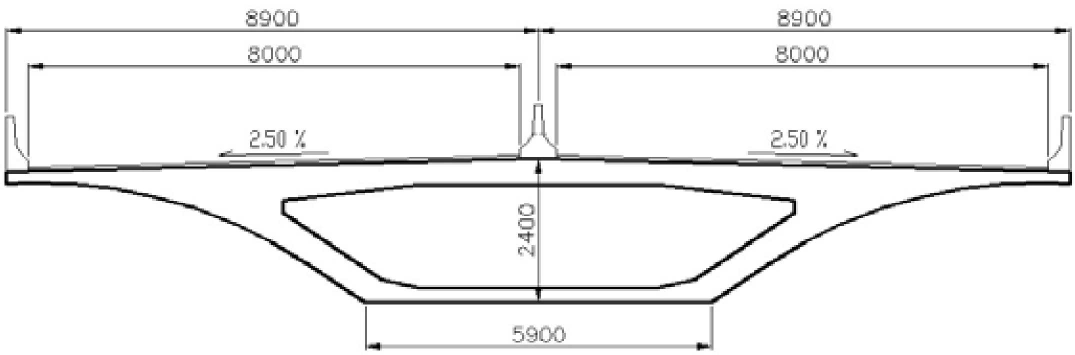
b. Typical T beam PC Girder Deck

Figure 8.4-8 PC I-Girder Beam and Slab Deck

Precast segmental construction is a fast construction method determined by the time required for the erection. The major part of the work is performed in the precasting yard, where it can be protected against inclement weather. Precasting can start simultaneously with the foundation work. The time-dependent deformations of the concrete become less important, as the concrete may have reached a higher age by the time the segments are placed in the structure. Typically, the industrialized production and fablication of the structure leads to higher quality of the finished products. This method requires relatively important investments in precasting yard, molds, lifting gear, transportation, and erection equipment. Therefore, this method requires a certain volume of work to become economically viable. A minimum length of bridge viaduct structure in the order of 2000m is required before this method becomes economically viable for urban expressway structures.

Precast segmental bridges are typically erected either by adopting progressive span-by-span construction methods or with the use of balanced cantilever techniques. A minimum economical span for this type of construction is 25 to 30m, with typical spans in the range of 40m to 60m for progressive span by span construction and up to 250m for balanced cantilever construction. Constant-depth girders deeper than 2.5 to 3.0m are unusual and therefore for spans greater than 50m consideration should be given to varying-depth girders through providing a curved soffit or haunches. The size and weight of precast segments are limited by the capacity of transportation and placing equipment. For most applications segment weights of 40 to 80 tons are the norm, and segments above 250 tons are seldom economical.

Figure 8.4-9 illustrates a Precast Segmental Bridge Deck section.



Baru Baru Flyover, a 2.4km dual two-lane viaduct located in Penang, Malaysia. Constructed 2004.

Span length : 47m
1082 match cast segments
Length segment : 1.7m (pier segment), 2.4m (in-span)
Weight of segments: 80t to 105t
Concrete characteristic cube strength : 55MPa
Erected with overhead gantry by balanced cantilever method.

Figure 8.4-9 Precast Segmental Bridge Deck

The incremental launching technique is an effective alternative for the bridge designer to consider for precast segmental bridges when the site meets its particular alignment requirements (either straight or constant curvature alignment). The method entails casting the superstructure, or a portion thereof, at a stationary location behind one of the abutments. The completed or partially completed structure is then jacked into place horizontally, i.e. pushed along the bridge alignment. Subsequent segments can then be cast onto the already completed portion and in turn pushed onto the piers. Because all of the casting operations are concentrated at a location easily accessible from the ground, concrete quality of the same level expected from a precasting yard can be achieved. The procedure has the advantage that, like the balanced cantilever technique, it obviates the need for falsework to cast the girder. Moreover, heavy erection equipment, cranes, gantries, and the like, are not necessary, nor is the use of epoxy at segment joints. Usually, the only special equipment required is light steel truss work for a launching nose to reduce the cantilever moments during launching. This method has been used in Cairo for construction of a section of 15th May Bridge.

- Steel Bridge Decks

In many cases steel beams, being lighter than concrete elements having the same weight-bearing capability, facilitate prefabrication.

Conventional small to medium span steel bridge decks feature multiple plate girders, with regularly spaced transverse bracing and steel diaphragms at piers, supporting in-situ concrete deck slabs. Multiple girders with the associated transverse bracing have in the past resulted in higher costs for fabrication, painting, erection and maintenance.

Figure 8.4-10 illustrates a typical Multiple Plate Steel Girder Bridge Deck.

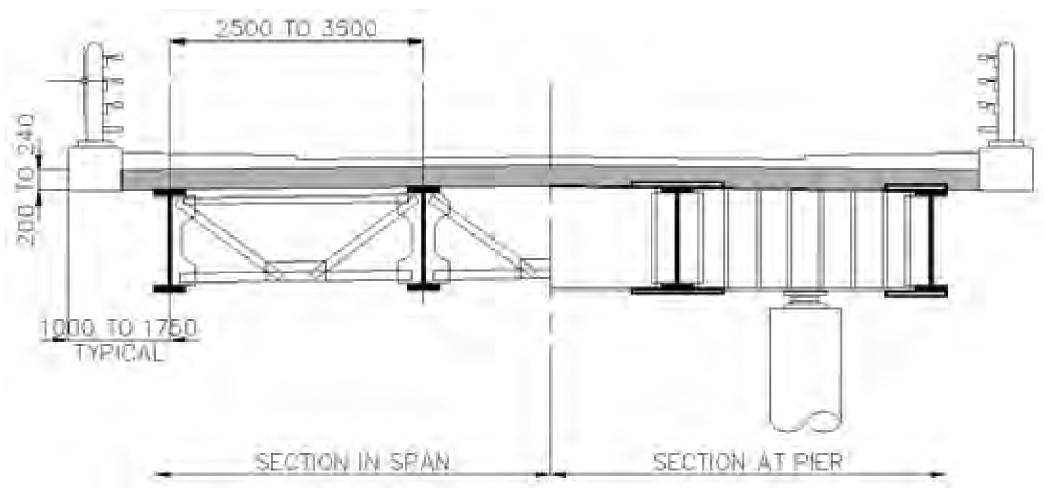


Figure 8.4-10 Typical Multiple Plate Steel Girder Bridge Deck
(Span 25m to 50m)

A modern trend in steel bridge construction is to simplify the design to facilitate savings in fabrication and painting costs, time for erection and to improve durability. New ways to prefabricate superstructures offer opportunities for bridge designers and contractors to significantly reduce construction time.

The twin girder deck type aims to reduce construction costs and construction duration by reducing the number of main girders in the deck. The deck depth/span ratios increase but this is accompanied with a corresponding reduction in the number of steel elements that require fabrication, welding and painting. Typically transverse bracing is either eliminated or reduced significantly. The reduction in the number of construction operations on site leads to savings in time and improvements in construction safety. The method relies upon the use of pre-stressed concrete deck slabs to span the relatively large distance between the reduced number of girders. Typically transverse spans up to 6.0m are accommodated by the deck slab, although this has been pushed out to 10.0m in Japan. The deck slab can either be cast in-situ and post-tensioned, or pre-cast in panels, pre-stressed and lifted into place. The pre-cast concrete deck panels are either in-situ stitched together with a specially developed RC loop joint or are longitudinally post-tensioned. The method is competitive and economic for spans between 30m and 60m, although in Japan the method has been used for spans up to 90m. In France the twin-girder steel deck has been used widely since 1975 with an average annual consumption of between 30,000 and 40,000 tonnes in recent years.

An example of a twin girder deck as developed in Japan is shown in Figure 8.4-11.

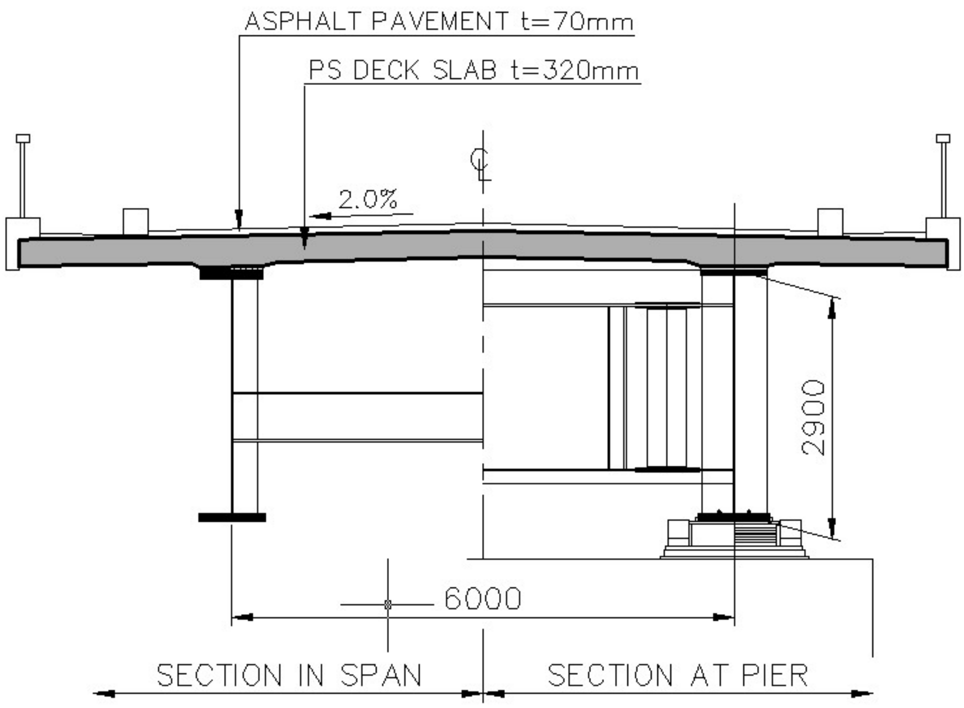


Figure 8.4-11 Twin Steel I-Girder Bridge Deck

Due to advances in fabrication technology, the use of steel trapezoidal box girders for bridge structures has become popular. The sloping webs of the trapezoidal box allow the bottom unsupported flanges to be reduced in width, thereby facilitating an improved compression flange design at the supports. The rapid erection, handling stability during erection, efficiency of load distribution, long span capability, suitability for curved decks, economics, and aesthetics of these girders make them more favorable than other structural systems. This form of construction has a substantially lower span/depth ratio the twin I-girder deck and is therefore more applicable to urban settings where controls on profile restrict girder depths. The major structural advantage of the trapezoidal box is its large torsional stiffness. A closed box has a torsional stiffness 100 to 1000 times greater than a comparable I-section.

A typical conventional box girder system consists of one or more U-shaped steel tub girders that act compositely with a cast-in-place concrete deck. The composite action between the steel girder and concrete deck is achieved through the use of shear studs welded to the top flanges of the girders. However, before hardening of the concrete deck, the conventional steel box is an open U-section with very low torsional stiffness and strength.

To stabilize the girders during construction and to increase the torsional stiffness prior to hardening of the deck, internal braces must be are provided. Internal braces consist of a permanent, top-lateral truss system used to provide a pseudo-closed section and K-braces that control stability and cross section distortion. In addition, external truss-type diaphragms may be provided. These intermediate external diaphragms are usually removed after the concrete deck hardens to prevent fatigue problems and improve aesthetics.

An example of a conventional U-shaped steel girder deck is shown in Figure 8.4-12.

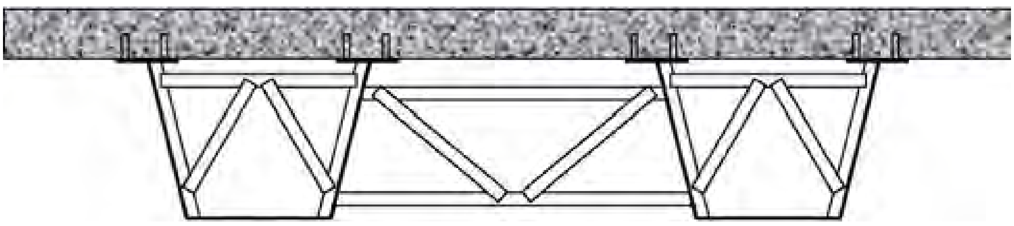


Figure 8.4-12 Conventional U-Shaped Steel Girder

Composite box girders with live loading and quasi-closed box girders during construction have to be considered during the design of these bridges. Considering both of these cases, the design for construction loading is the least understood and is the most important. Stresses coming from construction loading can reach up to 60-70 percent of the total stress on a cross section. In addition, the forces acting on the bracing members depend almost entirely on the construction loads. For all these reasons, great emphasis should be placed on this issue.

In order to avoid construction loading problems and dispense with both temporary and permanent bracing, a recent innovation in steel tub girder design is to use a full width steel top flange for each girder. The resulting closed tub girder is therefore inherently torsionally stiff and does not rely on the deck slab to complete the box section. This form of construction can therefore dispense with temporary and permanent bracing, substantially simplifying the steel fabrication and speeding the erection process. In addition the girder section can be designed to be non-composite with the concrete deck slab, thereby simplifying the shear connection design and allowing the use of precast concrete deck slabs with simpler shear connections (refer to section 8.4.3.b below).

An example of a closed steel tub girder deck is shown in Figure 8.4-13.

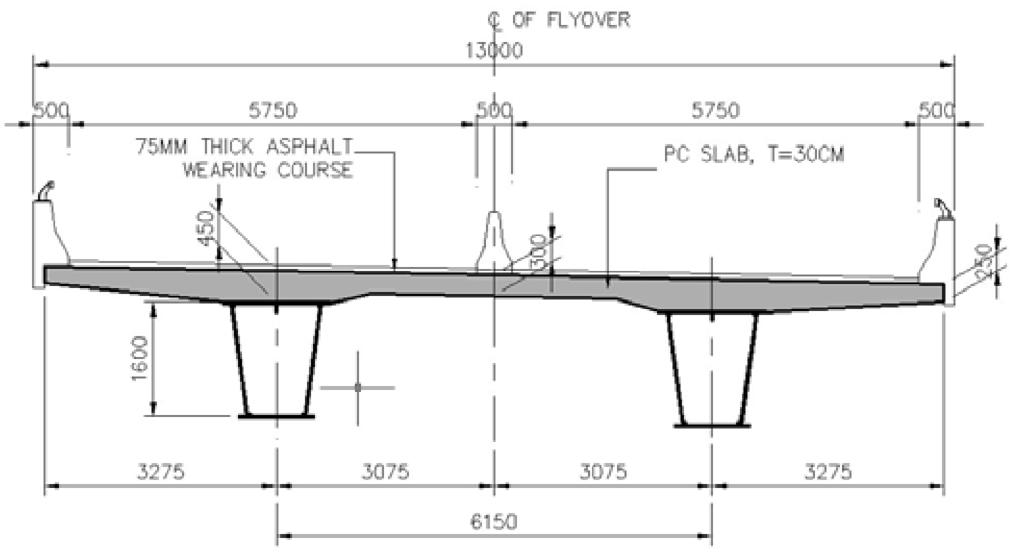


Figure 8.4-13 Twin Closed Steel Tub Girder
(Intermediate bracing typically not required)

The simplified construction of closed tub girder decks combined with reduced number of girders, in comparison with more conventional I-girder decks, also has the advantage of facilitating integration with prefabricated steel pier frames.

(2) Prefabricated deck slabs

Prefabricated concrete deck slabs can take the form of:

- Partial depth panels with cast-in-place concrete topping
- Full depth concrete panels with or without longitudinal pre-stress

The partial depth or full depth panels with longitudinal pre-stress are suitable for beam and slab bridges. Full depth panels without longitudinal prestress are best suited to bridge decks with fewer girder lines, such as twin tub girder decks, when the deck is designed as non-composite.

Partial depth panels typically do not require special jointing, during construction the panels serve the purpose of stay-in-place forms and are then designed to behave compositely with the cast-in-place topping in service for transverse load. Partial depth panels are typically 75mm thick. The top surface of the panels is roughened to prevent horizontal shear slip at the slab interface, thereby avoiding the need for additional reinforcement across the interface.

An illustration of a partial depth precast panel deck is shown in Figure 8.4-14.

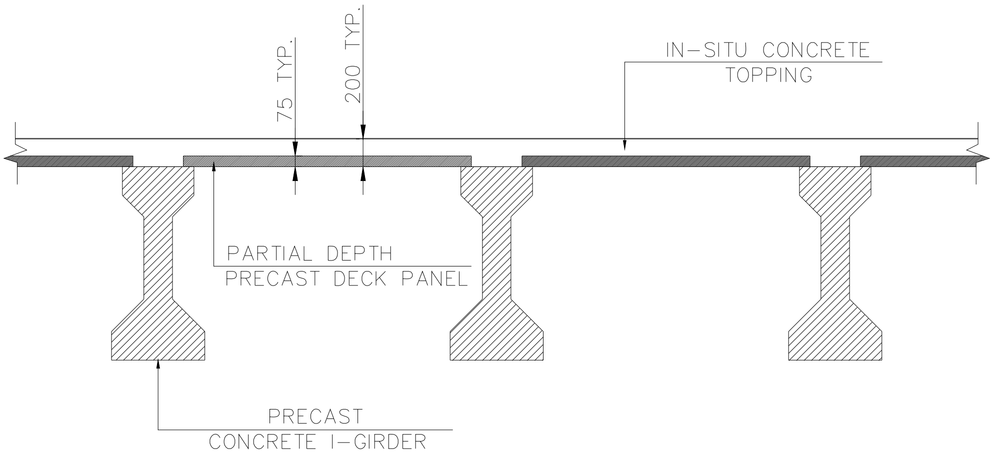


Figure 8.4-14 Partial Depth Precast Deck Slabs – PC I-Girder Decks

Full depth panels that are intended to act compositely with precast pre-stressed I-Girders typically take the form as shown in Figure 8.4-15. The self weight of the panels is transferred to the beams through leveling bolts. Leveling bolts are threaded through the depth of the panels and protrude through the bottom of the panels. The protrusion can be adjusted depending on the desired haunch height or desired top-of-deck elevation. The haunch is placed after the post-tensioning operation. Once the grout in the haunch has cured, the leveling bolts are removed and the panels and beams act as a composite system.

The most common type of joint between adjacent panels is a grouted female-female shear key; an epoxied male-female shear key is also an option. Either type of joint creates a mechanical interlock that provides continuity between the panels. The panels are post-tensioned together to add strength to the joint, provide distribution reinforcement, reduce the chance for cracking and water leakage at the joint, and improve the durability of the deck. However, if post-tensioning is not applied, mild reinforcing steel should be placed across the joint in order to properly reinforce the joint. The mild reinforcing steel must be properly developed on each side of the joint.

Composite action between the deck and beams is provided by shear connectors that extend out of the beam and into the shear pockets of the panels. The connectors typically consist of either hooked reinforcing bars or shear studs.

An illustration of a full depth composite precast panel deck is shown in Figure 8.4-15.

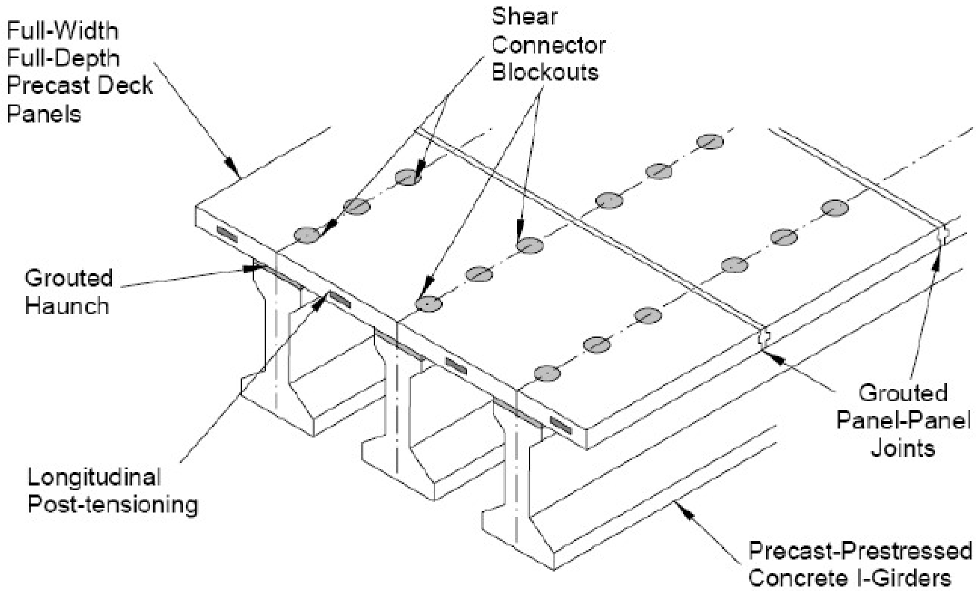
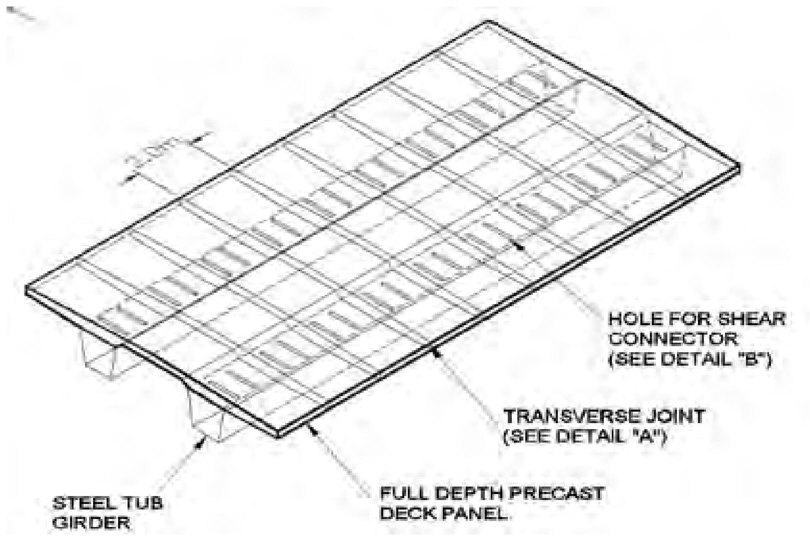


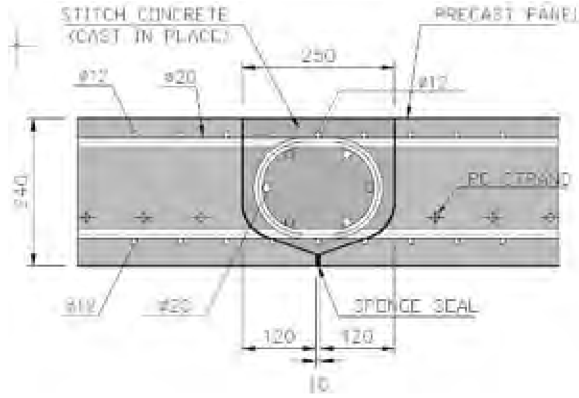
Figure 8.4-15 Composite Full Depth Precast Deck Slabs – PC I-Girder Decks

Full depth panels that are not intended to act compositely with the deck girders may take the form as shown in Figure 8.4-16. The principle is the same as for composite slabs except that the units do not need to be longitudinally pre-stressed. The panels however are pre-stressed transversely in order to control slab depth. For straight alignments the panels can be simply placed on preformed joint filler strips directly on top of the top flange of the girder. For curved alignments requiring varying super elevation the panel elevations would be adjusted with leveling bolts. A common type of joint between adjacent panels is a concreted female-female shear key. The joint features interlaced mild reinforcement to prevent cracking and leakage of the joint. The pre-cast panels are cast with shear pockets that fit over shear studs on the steel girders. These are then filled with non-shrink mortar to achieve fixity between girders and slab. The pre-casting of deck panels requires a sufficient length of deck girder construction in order to achieve the necessary economies of scale that would make it a cost effective option.

An illustration of a full depth precast panel twin girder deck is shown in Figure 8.4-16.

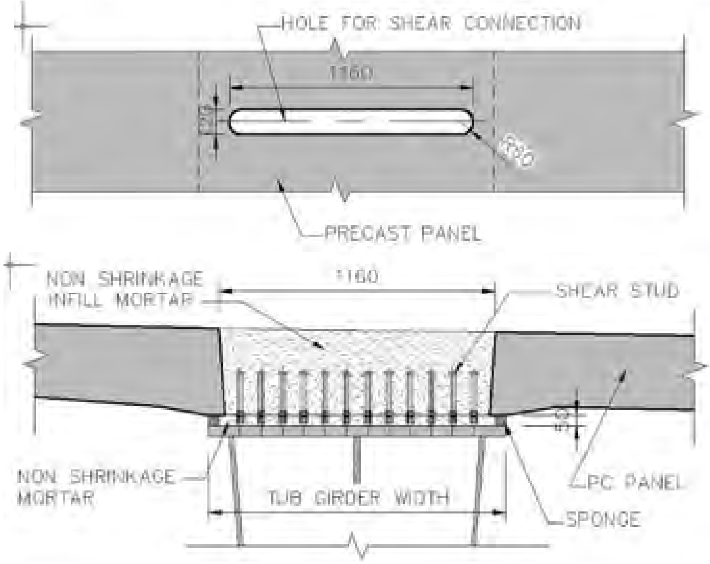


a) Isometric view of Full Deck Deck Panels



b) Transverse Joint Detail “A”

Plan



Section

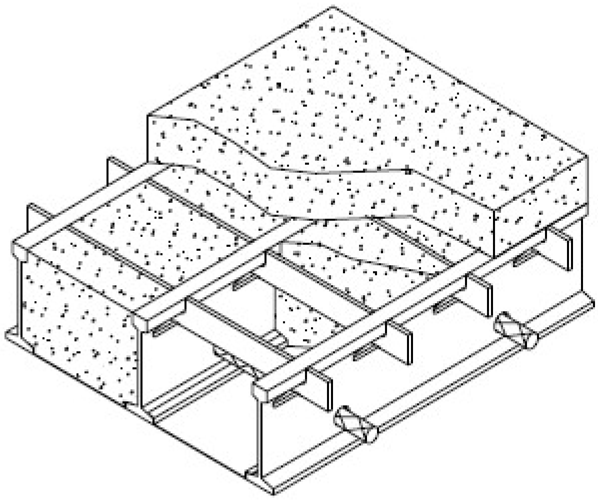
c) Hole for Shear Connector Detail “B”

Figure 8.4-16 Typical Full Depth Precast Deck Slabs – Twin Girder Deck

Prefabricated composite deck slabs can take the form of concrete filled steel grids. Since their inception in the early 1930's in the USA, concrete filled steel grid decks are unsurpassed in durability and longevity. In fact, there are bridges in the U.S.A that have their original filled grid deck systems still in place from the 1930's. It is common to see filled grid deck systems last 50-70 years and, more often than not, find that the filled grid deck outlasts other superstructure components.

The longevity of filled grid deck can be attributed to its fundamental design. In combining the tensile strength of steel with the compressive strength of concrete, filled grid deck utilizes the capabilities of both materials. In addition, filled grid deck is usually substantially lighter than reinforced concrete slabs. In 1999 the historic Brooklyn Bridge in New York, USA, under a deck replacement contract, received more than 7500 square meters of filled grid deck in a period of six months with work only taking place during night-time possessions.

Refer to Figure 8.4-17 for an illustration of a concrete filled steel grid deck.



Steel Yield Strength 350MPa

Concrete compressive strength 30MPa

Steel grid depth 110mm.
Total deck depth 160mm (including 50mm of concrete overfill)

Maximum clear span of 3.5m under AASHTO HS25 Load.

Figure 8.4-17 Typical Concrete Filled Steel Grid Deck

The initial construction cost of a bridge deck comprising prefabricated deck panels may be higher than a cast-in-place deck, but tremendous benefit can be found in the greatly reduced construction time and greatly enhanced durability of the deck. If designed and constructed properly, the deck panel system is an excellent option when rapid bridge deck construction or replacement is required.

8.4.4 Materials

(1) Rapid Curing Concrete

Another way to speed bridge construction is to use rapid curing concrete. Precasters in the USA have been able to attain strengths of 28-56 MPa overnight by using Type III cement, low water/cement ratios and steam curing. Some high performance concretes will attain high strengths in only a few days old. However, these materials are usually used for rapid repair of concrete or for pavements, rather than for casting entire concrete structures. No specific instances of rapid curing concrete for structural applications were found in the literature.

8.4.5 Applicability of Rapid Construction to the Study

The applicability of rapid bridge construction to each section of the Study is presented in Table 8.4-1 for E1-2, E2-2 and E3-1.

A detailed analysis of Pre-Feasibility study sections E3-2 and E3-3 will be made at the time of a future feasibility study. However, given that both sections run along busy traffic corridors (Nasr Road, Saleh Salem, Rawdah Street and Giza Square) and across the Nile River, the benefits of prefabrication are apparent. Pre-Feasibility Study of these sections has therefore focused on prefabrication for the preliminary design.

Table 8.4-1 Applicability of Rapid Construction Techniques to Feasibility Study Sections (E1-2, E2-2 & E3-1)

	Construction Issue	**SECTION E1-2** *(4.5km – Extension of 6^{th} October Bridge (E1) to connect with E11 AT Saft El Laban)*	**SECTION E2-2** *(1.8km – 15^{TH} May Bridge eastbound extension from Nile to 6^{TH} October Bridge)*	**SECTION E3-1** *(6.5km – Nasr Road from Suez Desert Road (E4) to 6^{TH} October Bridge (E1) junction)*
1.	Reduction in Construction Time	Tunnel is on the critical path for construction. Approach structures from existing 6^{th} October Bridge are therefore not a high priority for rapid construction. Demolition of the existing structure likely to have the greatest impact on construction period for this first section. Viaduct structures and tunnel approach structure along ENR corridor adjacent to Sudan Street are a high priority for rapid construction.	Prefabrication will bring substantial advantages in reducing construction time, particularly for the double deck sections along 26^{th} July Street (above 15^{th} May Bridge).	“Cut-and-Cover” depressed structures are substantially constructed using in-situ techniques. However benefits in reducing construction time can be found with the use of prefabricated top slabs. The Westbound viaduct sections are not on the critical path for construction. The viaduct is therefore not a high priority for the use of rapid construction methods.
2.	Reduction in Traffic Disruption	Road closed with traffic detour proposed during construction of approach structures from 6^{th} October Bridge. Demolition of existing structure likely to have the greatest impact on disruption to traffic. Rapid construction of approach structures therefore not the highest priority. Minimizing disruption to rail services during construction along ENR corridor will be a high priority. Rapid construction techniques are essential at this location.	26^{th} July Street will be closed in that section during construction. Minimizing possession of the street will bring great benefit in minimizing traffic disruption. Prefabrication of both piers and deck sections will therefore bring major benefits. Minimizing disruption to traffic along the very busy Ramsis Street, with the use of prefabricated pier/deck construction, will also be a priority.	Prefabricated top slabs will allow the contractor more flexibility in planning and executing the work. Night time possessions can be considered for this operation. This will bring about substantial reductions in traffic disruption if properly planned..
3.	Constructability and impact of Prefabrication	Prefabrication does not offer any substantial benefits for constructability for Approach structures from existing 6^{th} October Bridge to the shield tunnel. Prefabrication for the viaduct structures along ENR corridor is essential for constructability. Prefabrication will improve constructability along Saft El Laban road corridor.	Prefabrication of piers and deck will bring major constructability benefits particularly along double deck section. Large span lengths drive in prefabricated deck sections and pier copings at several locations. Prefabricated pier/deck will allow integration of pier/deck connections throughout giving improving constructability.	Constructability issues related to prefabrication are not prominent for this section. Without the pressures of reducing construction time and minimizing disruption to traffic, in-situ construction could be considered for the top slabs of the cut-and-cover sections.
4.	Impact on Construction Cost from Prefabrication	The proposed lengths of the viaduct sections are sufficient for prefabrication to have an impact on reducing construction costs.	The proposed length of viaduct at 1.8km is sufficient for prefabrication to have an impact on reducing construction costs.	The proposed lengths of depressed structure are sufficient for prefabrication to have a substantial impact on reducing construction costs of the cut-and-cover top slab.
5.	Improved Quality Control and Reduced Life Cycle Cost from Prefabrication	A priority for all sections. Particularly important for viaduct sections running along ENR corridor.	A priority for all sections. Particularly important for the double deck sections with potential for major traffic disruption in the future with defective in-situ concrete replacement operations	A priority for all sections. Particularly important for cut-and cover top slab construction; given traffic volumes along Nasr Road and potential for major traffic disruption in the future with defective in-situ slab replacement operations.
6.	Safety Issues a related to Prefabrication	Safety during construction along ENR corridor will be of utmost priority. Prefabrication will bring distinct benefits in improving safety during construction along this sensitive corridor.	Safety issues at this highly congested location will have high priority. Prefabrication will bring distinct benefits in improving safety during construction.	The “top-down” method for the depressed sections will benefit with the use of prefabricated top slab in minimizing risk of accident during construction.
7.	Environmental Impact	Major impact for approach structures from 6^{th} October Bridge will be the demolition of the existing structure. This will not be mitigated by rapid construction techniques. Reduced construction times will lessen environmental impact along ENR and Saft El Laban corridors.	The reduction in construction period will have substantial effect on reducing environmental impact in this very congested location. Prefabrication will bring major benefits.	The reduction in construction period will have substantial effect on reducing environmental impact in this very congested location. Prefabrication of the top slabs of the cut-and-cover sections will bring major benefits.
8.	Overall Assessment	Rapid Construction recommended for viaduct structures along ENR corridor adjacent to Sudan Street and for Saft El Laban road corridor.	Rapid Construction recommended for both pier and deck construction throughout the section.	Rapid Construction is recommended for the top slabs of the cut-and-cover sections.

8.4.6 Implementation for Rapid Construction

(1) Decision Making Process

Bridge owners should lead the rapid bridge construction change process, working with consultants and contractors. However the use of prefabrication should not be a policy or be mandated, but instead should be used when it is a good business decision.

Bridge owners need to support and encourage contractors' proposals to accelerate projects by committing to quick turnaround in the review and approval process. This commitment should be communicated in writing to the contractor during the bidding process.

Other factors that may be considered related to the decision-making process to further implement rapid bridge construction are summarized below.

- Develop a standard decision-making process that considers prefabrication on a project-by-project basis.
- Obtain contractor input early in the decision-making process. Review constructability of prefabrication at the detailed design stage, including contractors and suppliers in this review process.
- Capture all costs in evaluating whether prefabrication is the best choice for a project. Consider cost of day versus night work. Consider the maintenance-of-traffic costs.
- Consider limited work windows that may be driving the schedule. Work with those that are driving the limited work windows, e.g., local permitting agencies and government authorities.
- Conduct internal training for better understanding of the technologies and the possibilities that are available when using these technologies.
- Initiate public informational outreach at the preliminary planning stage. Show renderings of structure types and discuss construction timelines. Solicit input.

(2) Construction Strategies

Construction strategies to further implement rapid bridge construction include.

1. Use advance work contracts for major utility relocations to help ensure the critical path of the project is not affected.
2. Value Engineering: contractor may be allowed to propose alternatives of equal or better quality, and the cost savings will then be reviewed.

8.5 SUPERSTRUCTURE AND SUBSTRUCTURE TYPES : E1-2

8.5.1 General

Expressway section E1-2 will be an extension of Expressway E1-1 (6th October Bridge) commencing from a location before the Agricultural Museum in Giza and terminating at Saft El-Laban, also in Giza. The overall length of the expressway is 5.5km with approximately 3km of that length in tunnel.

The expressway descends from the existing 6th October Bridge on ramps to enter a tunnel along El-Mat haf al-Zira'i Street before crossing beneath the road in front of the Agricultural Museum. The expressway emerges from tunnel parallel with Sudan Street on separate Inbound and Outbound alignments each side of the existing ENR track. E1-2 then follows the alignment of the currently under construction Saft El Laban structure and ties in with it for both Inbound and Outbound alignments.

Refer to Figure 8.5-1 for an overall layout plan of Section E1-2 showing principal structure sections.

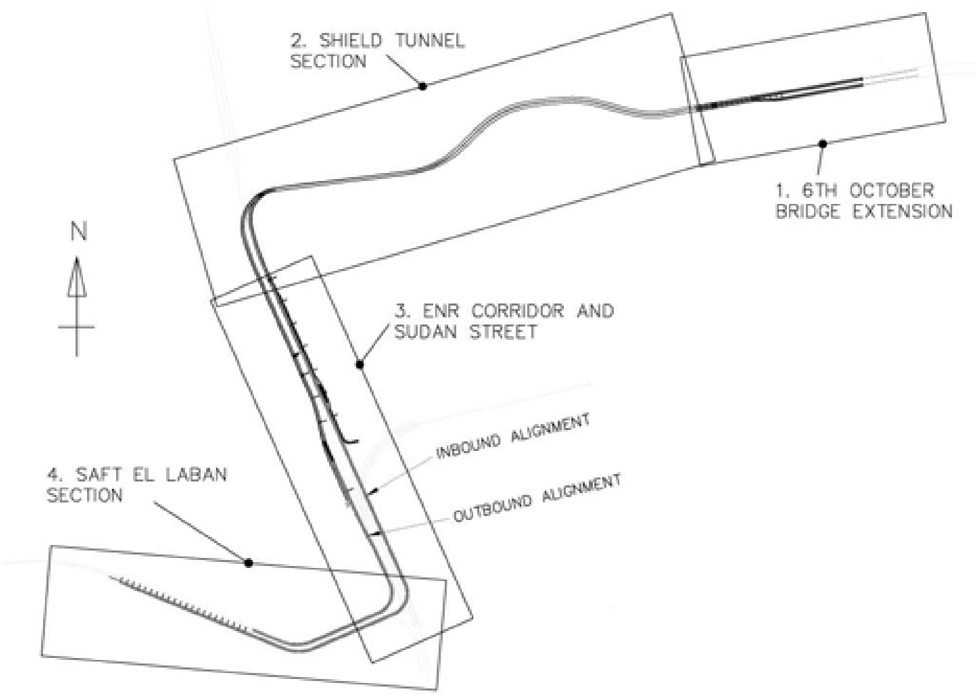


Figure 8.5-1 Overall Layout of Section E1-2

The expressway comprises the following structural components:

6^TH^ OCTOBER BRIDGE EXTENSION

- Demolition of section of existing bridge and ramps (refer Figure 8.5-2)
- New ramps from 6th October Bridge to replace the existing straight ramps onto El-Mat haf al-Zira'i Street. Ramps to remain elevated and connect with existing 6th October Bridge deck – length 311 m: Sta. 0+130 to 0+441
- New central deck and embankment section to connect expressway with tunnel transition structure – length 277 m: Sta 0+130 to 0+307
- Transition Structure (Open Cut) – length 134 m: Sta. 0+307 to Sta. 0+441
- Cut and Cover Tunnel – length 249 m: Sta. 0+ 441 to 0+690

SHIELD TUNNEL SECTION

- ESA ("Endless Self Advancing") Tunnel –length 70 m: Sta. 0+690 to 0+760
- Shield Tunnel (Twin Bore)
  - ✧ Left Tunnel (Inbound) – length 2140 m: Sta. 0+760 to Sta. 2+900
  - ✧ Right Tunnel (Outbound) – length 2540 m: Sta. 0+760 to Sta. 3+300

ENR CORRIDOR AND SUDAN STREET

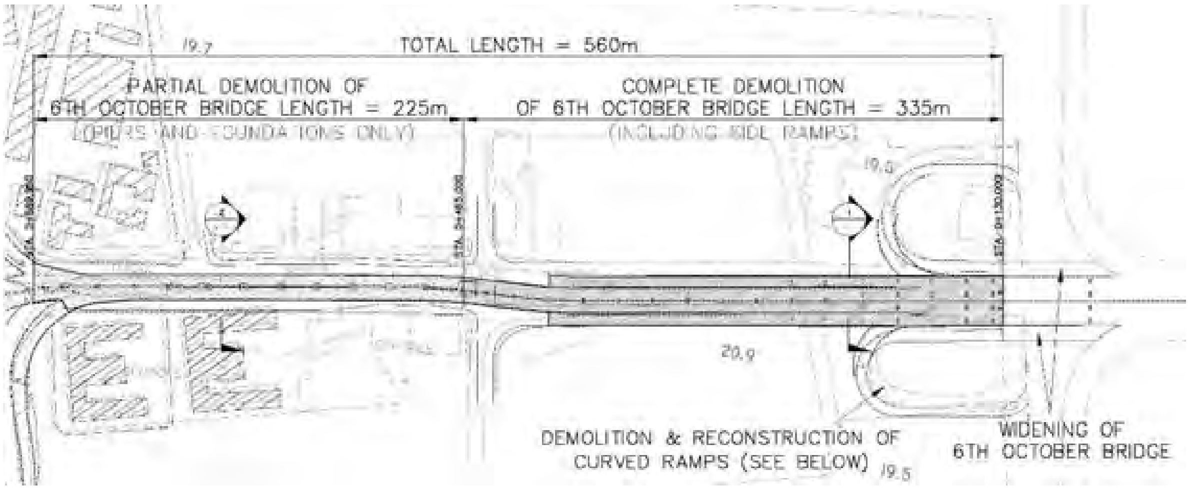
- Cut and Cover Tunnel
  - ✧ Left Side of ENR Track (Inbound) - length 75 m: Sta. 2+ 900 to 2+975
  - ✧ Right Side of ENR Track (Outbound) - length 125 m: Sta. 3+ 300 to 3+425
- Transition Structure (Open Cut)
  - ✧ Left Side of ENR Track (Inbound) - length 160 m: Sta. 2+975 to Sta. 3+130
  - ✧ Right Side of ENR Track (Outbound) - length 140 m: Sta. 3+425 to 3+580
- Elevated structures
  - ✧ Left Side of ENR Track (Inbound) - length 1288 m: Sta. 3+ 130 to 4+418
  - ✧ Right Side of ENR Track (Outbound) - length 777 m: Sta. 3+580 to 4+357

SAFT EL LABAN

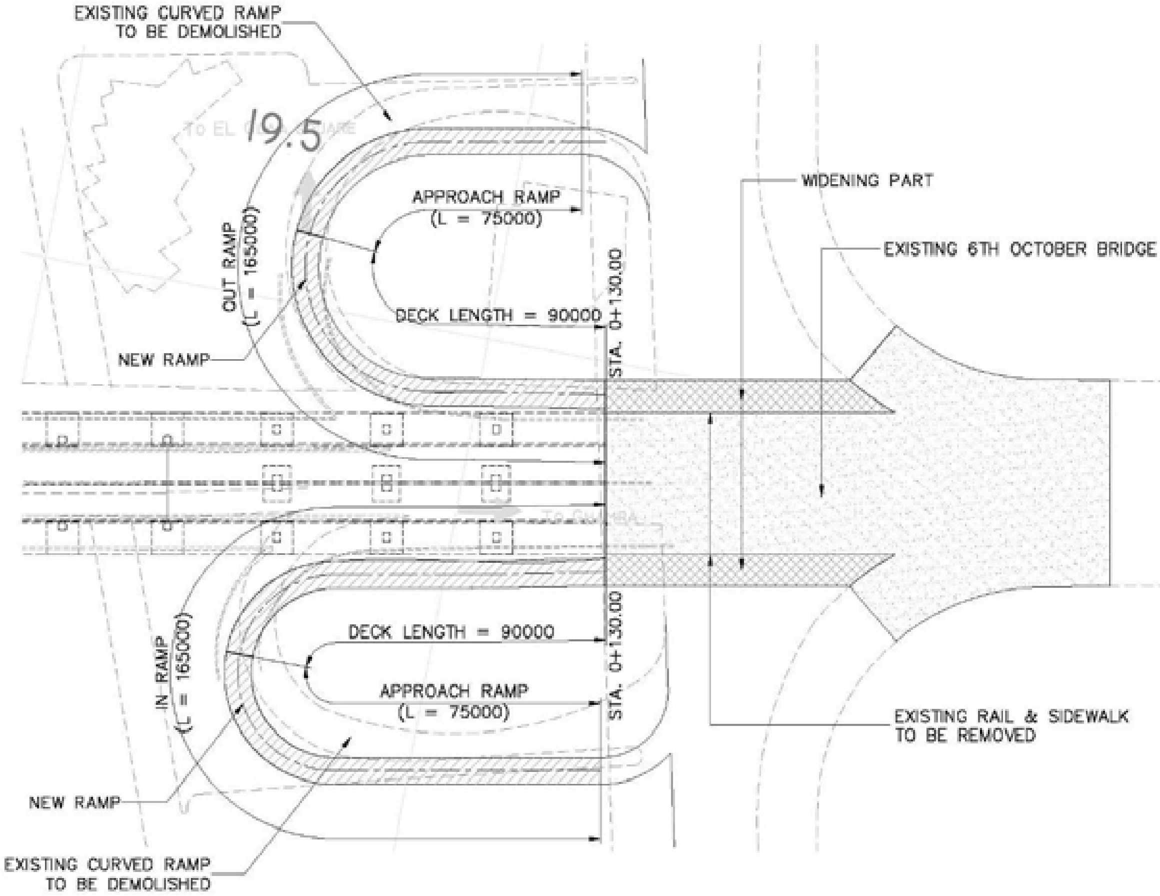
- Elevated structures
  - ✧ Inbound: length 1139 m: Sta. 4+ 418 to 5+557
  - ✧ Outbound : length 360 m: Sta. 3+ 060 to 4+717

The existing 6th October Bridge will require widening, demolition and reconstruction works in order to accommodate the new expressway structure. Refer to Figure 8.5-2 for a layout plan showing the extent and type of demolition work proposed at 6th October Bridge.

The Inbound Expressway (left side of ENR track) features a single on ramp facility that transitions to the cut-and-cover tunnel structure along Sudan Street. The Outbound Expressway (right side of ENR track) features a single off ramp facility that commences at the point that the expressway emerges above ground on the ENR corridor. Refer to Figure 8.5-3 for a layout plan of the on and off ramp structures.



a. Overall demolition plan on 6th October Bridge Extension



b. Plan on Curved Ramps at Start

Figure 8.5-2 Layout of Proposed Demolition Work 6th October Bridge

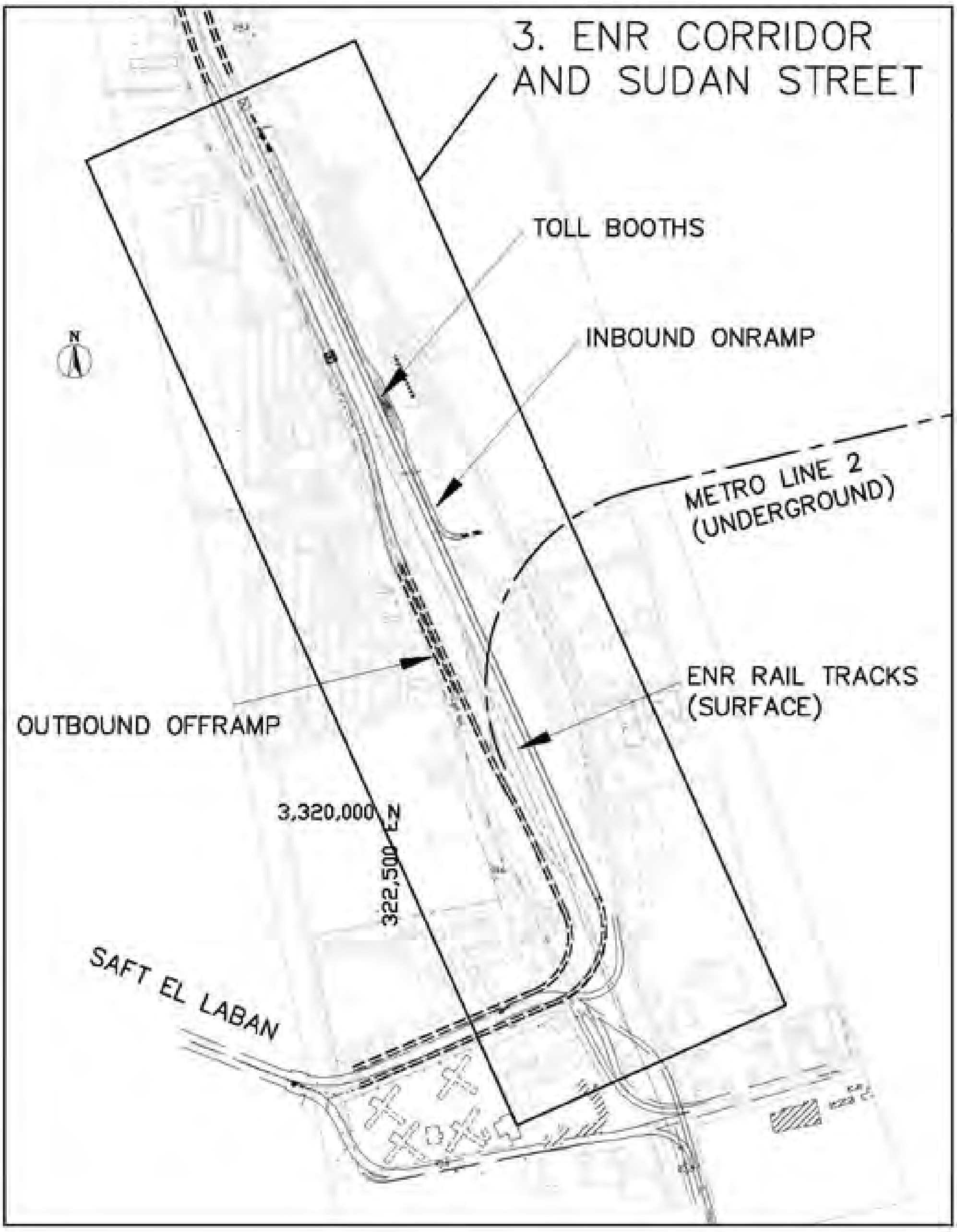


Figure 8.5-3 Layout of On Ramp and Off Ramp at ENR Corridor

The main expressway structures carry 2-lanes in each direction. The on and off ramps provide single lane access/egress.

Features along the route that have an impact on the preliminary structural design are:

- The existing 6th October Bridge will impact the method of construction both of the transition structures and the cut-and-cover tunnel. It is proposed that during construction the existing deck will remain in place on temporary supports to enable the existing bridge pier and foundations to be demolished making way for the cut-and cover tunnel. Once the cut-and-cover-tunnel top slab is completed, new bridge piers will be constructed to support the existing deck on the new tunnel structure.

- Crossing beneath the existing road in front of the Agricultural Museum will require the use of ESA (“Endless Self Advancing”) tunnel construction, a continuous self-advancing construction technique that will have minimal impact both on the existing road and on under ground utilities.
- The shield tunnel takes a route along Geddah Street before turning onto Sudan Street. Geddha Street is narrow, with approximately 15m between the facing buildings, and therefore requires the tunnel to adopt a tandem arrangement (one tunnel constructed above the other) to avoid adversely impacting the building foundations.
- The proposed MoH Elevated Busway will run along Sudan Street adjacent to the proposed Inbound Expressway before terminating at El Bohoos MRT Station. The proposed expressway structure will be required to occupy the available space between the proposed busway and the ENR track.
- The existing canal running adjacent to ENR property line is at present being lined and covered with reinforced concrete box culvert structures. Either the canal will require relocation or the expressway alignment will have to be modified to accommodate the existing condition.
- The proposed expressway route runs adjacent and above the existing ENR track, station, sidings and maintenance sheds. The ENR facilities therefore will have a substantial impact on the structural layout and design.
- The elevated structures under construction at Saft El-Laban dictate the layout and arrangement of the expressway structures at this proposed intersection location.

Features that may have any impact on the detailed structural design are:

- Underground utilities - these may need to be carried in service troughs or require some other modification to the top slab design of the cut-and-cover tunnel structures.

8.5.2 E1-2 Superstructure Types

(1) 6th October Bridge Extension

The existing 6th October Bridge superstructures, including the ramps to be demolished and replaced, feature reinforced concrete cellular bridge decks.

The proposed new expressway structure and the ramps for local traffic to access the remaining 6th October Bridge shall therefore also be in the form of reinforced concrete cellular decks in order to remain sympathetic with the existing structure. Refer to Table 8.4-1 for comments on rapid construction in this section.

(2) Structures along ENR Corridor and Sudan Street

The study on superstructure types is based on several considerations including:

- Determination of required span length along each critical, in particular taking into account the ENR facilities
- Construction cost and construction time considerations
- Restrictions imposed on method of construction for sections adjacent to and above the existing ENR track, sidings, platforms and maintenance sheds
- Required temporary works and construction equipment with corresponding impact on railway operations and road traffic
- Inspection and maintenance considerations
- Aesthetic considerations

This section is identified as a high priority for rapid construction (refer Table 8.4-1). The advantages provided by structural steel twin tub girder decks combined with precast concrete deck slabs (refer Section 8.4.3) in constructing the expressway viaducts along the ENR corridor were key in selecting this form of construction to illustrate the preliminary design. The form of construction adopted effectively reduces in-situ concrete construction to a minimum, a major priority for construction adjacent to (and above) the existing railway lines, and allows span lengths to be pushed out in order to cross the ENR tracks and the Metro Line 2 underground.

(3) Structures at Saft El Laban

The study on superstructure types is based on several considerations including:

- Layout and type of Saft El-Laban superstructures
- Construction cost and construction time considerations
- Inspection and maintenance considerations
- Aesthetic considerations

This section is identified as a candidate for rapid construction with regard to constructability issues (refer Table 8.4-1). Given that the Saft El-Laban viaduct currently under construction features post-tensioned precast beam and slab decks, the same form of construction was adopted in the preliminary design in order to achieve compatibility in form and to promote constructability. The simplified steel tub design can also be very effectively integrated with composite columns.

8.5.3 E1-2 Substructure Types

(1) 6th October Bridge Extension

- Pier Type

The existing 6th October Bridge substructures, including the ramps to be demolished and replaced, feature single and multiple column piers. The piers take the form of simple rectangular columns monolithic with the deck.

The substructure for the new expressway and the new ramps of 6th October Bridge shall therefore also be in the form of single column reinforced concrete piers monolithic with the deck to remain sympathetic with the existing structure. Refer to Table 8.4-1 for comments on rapid construction in this section.

- Foundation Type

Bored pile foundations have been selected as the foundation type to be applied to the preliminary design.

The final choice of bored pile diameter will be made during the detailed design. Minimizing pile cap sizes with larger bored piles is not identified as a high priority in this area. For the purposes of establishing quantities for the cost estimate in this Study, 800mm diameter bored piles have been selected to illustrate the preliminary design.

(2) Structures along ENR Corridor and Sudan Street

The study on substructure types has been made based on several considerations including:

- Study on soil conditions
- Construction cost and construction time considerations
- Operational and construction clearance requirements adjacent to ENR track
- Restrictions imposed on substructure location by the ENR facilities
- Operational clearance requirements of at-grade streets
- Impact of method of construction on railway operations and road traffic
- Aesthetic considerations

Refer to Table 8.4-1 for comments on rapid construction in this section.

Given the proximity of the proposed expressway piers to the existing ENR track, both minimizing in-situ work and controlling the size of the pier columns are high priorities. Single composite pier columns are therefore an excellent choice at this location. The steel casing of the column requires minimal temporary works to set in place and the composite design results in columns section size that is substantially smaller (up to 20%) than a conventional reinforced concrete column. In addition the composite column can be very effectively integrated into both the steel tub girder deck selected for the superstructure and large diameter bored pile foundations (see below)

Large footprint pile foundations are not recommended along the ENR corridor, both given the proximity of the adjacent railway tracks and because of the multiple stages required for pile and cap construction will result in longer construction times along this very sensitive section. Single large diameter bored pile foundations are an obvious choice in this section. In order to provide a suitable connection for the composite column piers proposed, a maximum diameter of 2.5m was selected in order to achieve a composite pile section.

(3) Structures at Saft El Laban (E11)

The study on substructure types has been made based on several considerations including:

- Study on soil conditions
- Construction cost and construction time considerations
- Operational clearance requirements of at-grade streets
- Impact of method of construction on road traffic
- Location of Saft El-Laban foundations

Conventional reinforced concrete columns with conventional bored pile foundations typically are selected to illustrate the preliminary design in this section. This substructure form matches with the ongoing Saft El Laban construction.

8.5.4 Cut and Cover Tunnel Sections

(1) Along El-Mat haf al-Zira’i Street

- General Considerations

El-Mat haf al-Zira’i Street will require to be closed to vehicular access during construction. Alternative routes for local traffic will therefore have to be ensured and proper attention given to traffic management during construction.

The relatively narrow space available during construction (down to 30m between building faces) and the multiple storey buildings lining the road require that cut and cover construction is adopted for the tunnel structure. Given that the road will be closed to traffic during construction, both “top down” and “bottom up” methods can be employed for the cut-and-cover construction. However the depth of excavation (more than 12m) required in order to make the transition to the shield tunnel and the proximity of the multiple storey buildings militate against simple un-propped cantilever wall solutions during construction. “Top/down” construction has therefore been adopted in which the top slab of the tunnel is constructed, in relatively shallow excavation supported by the vertical cantilever walls, before the excavation of the main tunnel works is commenced. In this way the top slab behaves like a prop to ensure the stability of the cantilever walls. This method will require the construction of a central contiguous bored pile wall to provide support to the top slab during the main excavation.

- Types of Cut-and-Cover Construction

The following types of cut-and –cover construction can be considered:

- ✧ Bored pile curtain wall construction
- ✧ Diaphragm wall construction

Refer to Section 8.7-4 for descriptions and illustrations of each type.

Given that the existing 6th October Bridge deck will be imposing a headroom restriction on construction equipment required to be located beneath the existing bridge deck to construct the supporting walls of the transition structures, bored pile curtain walls have been selected for the preliminary design. Borings rigs can be modified to work in limited headroom situations and so allow the construction of bored piles beneath existing bridge decks. The construction equipment for diaphragm walls cannot be so readily modified. Bored pile curtain wall construction will take more time than diaphragm wall construction. However the cut-and-cover tunnels are not on the critical path for construction programming for E1-2 since the twin bore shield tunnels will be the major component of the construction.

- Integration of the Existing 6th October Bridge with the Tunnel Structure

The deck of the existing 6th October Bridge above the cut-and-cover tunnel section of the expressway will rest on temporary supports during construction. This will allow the existing bridge piers and foundations to be demolished to enable the cut-and-cover tunnel to be constructed. Pier supports will then be reconstructed monolithic with the tunnel structure.

The following construction steps are envisaged (not including mechanical, electrical or drainage works):

1. Close the road to traffic and construct the bored pile curtain walls each side

2. Excavate for the top slab of the cut-and-cover tunnel between the existing bridge foundations
3. Construct the central contiguous bored pile support wall between the existing bridge foundations
4. Construct the top slab of the cut-and-cover tunnel between the existing bridge foundations
5. When the top slab has gained sufficient strength, place temporary column supports on the slabs at locally reinforced points (additional diaphragms constructed and top slab thickened as necessary) and span between these points with temporary steel stringers adjacent and parallel with the existing bridge piers.
6. Jack the existing deck onto bearing points on the stringers. The jacking forces are determined to be equivalent to the pier reactions, thereby effectively unloading the piers. The deck is now temporarily supported from the top slab of the tunnel through the temporary supports.
7. Demolish the existing bridge piers and foundations and excavate for the remaining top slab of the cut-and-cover tunnel.
8. Construct the remaining central bored pile wall and top slab of the cut-and-cover tunnel at the location of the now demolished bridge foundations. The new top slab will be designed to carry all loads from the bridge piers and will have all necessary reinforcement including starter bars for the new bridge piers.
9. Construct the new bridge piers to be monolithic with the new top slab of the tunnel and cement grout the pier column/deck joints.
10. Release and remove the temporary supports. The deck is now permanently supported on the new bridge piers from the top slab of the tunnel.
11. Fill above the tunnel slab, reinstate the pavement works and reopen the road to traffic.
12. Excavate the main tunnel space and construct the tunnel wall linings and base slabs to complete the cut-and cover tunnel civil works. This can be done at the same time as steps 9, 10 & 11 above.

(2) Along Sudan Street

The total lengths of open cut and cut-and cover structures, at 235m for Inbound and 265m for Outbound, are not on the critical path for construction; however given that these structures are located adjacent to ENR track, rapid methods of construction have been selected to illustrate the preliminary design. Diaphragm walls combined with precast pre-stressed planks for the cover slab are proposed.

Refer to Section 8.7-5 for descriptions and illustrations of alternative cut-and-cover walls and top slab construction.

8.5.5 ESA and Shield Tunnel Sections

For more details on the Endless Self Advancing (ESA) box tunnel and circular shield tunnel methods refer to Section 14.1.2.

8.5.6 Typical Sections

Typical sections illustrating the preliminary design are presented in Figure 8.5-4 to Figure 8.5-10.

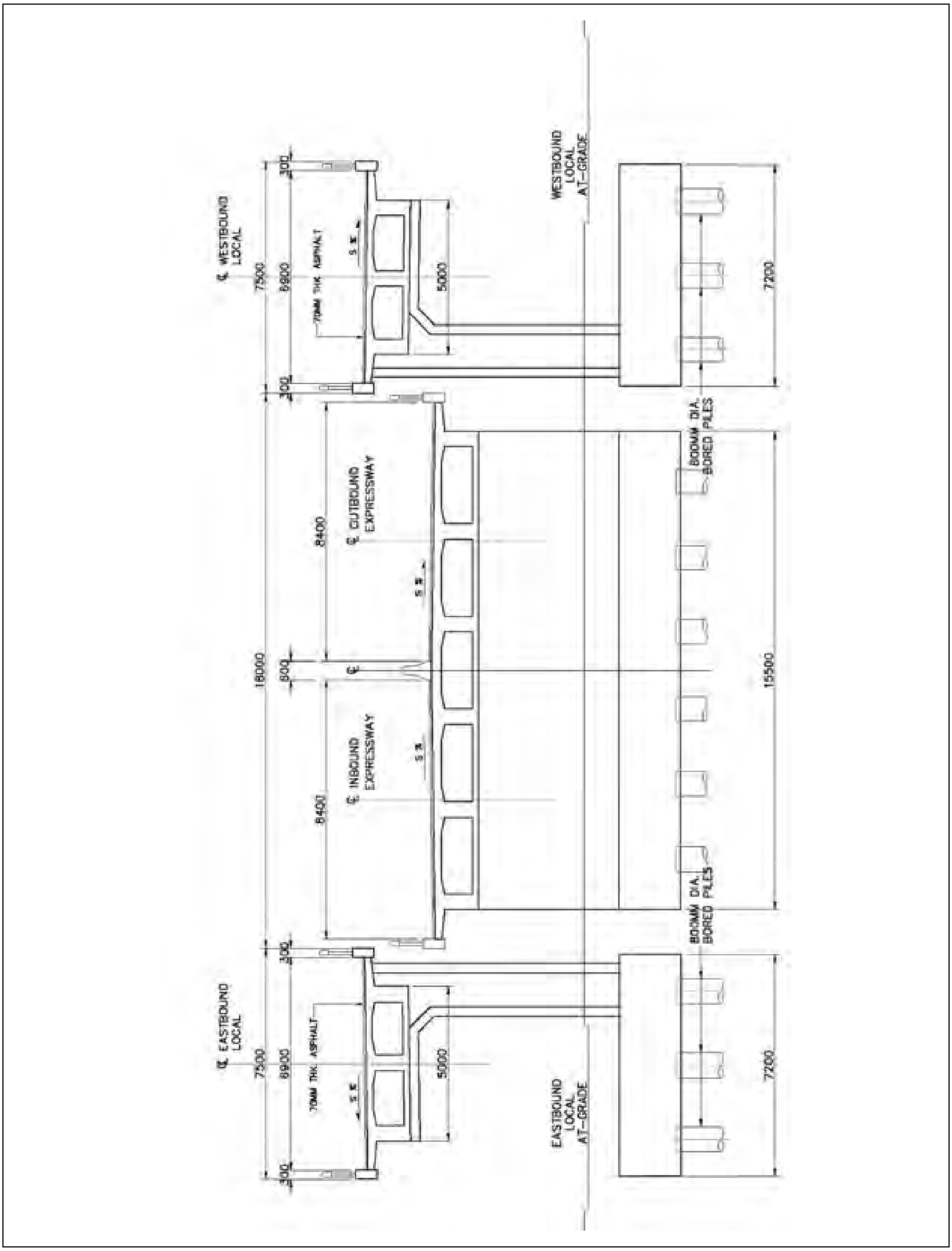


Figure 8.5-4 Section at Sta. 0+230: E1-2
(El-Mat haf al-Zira'i Street)

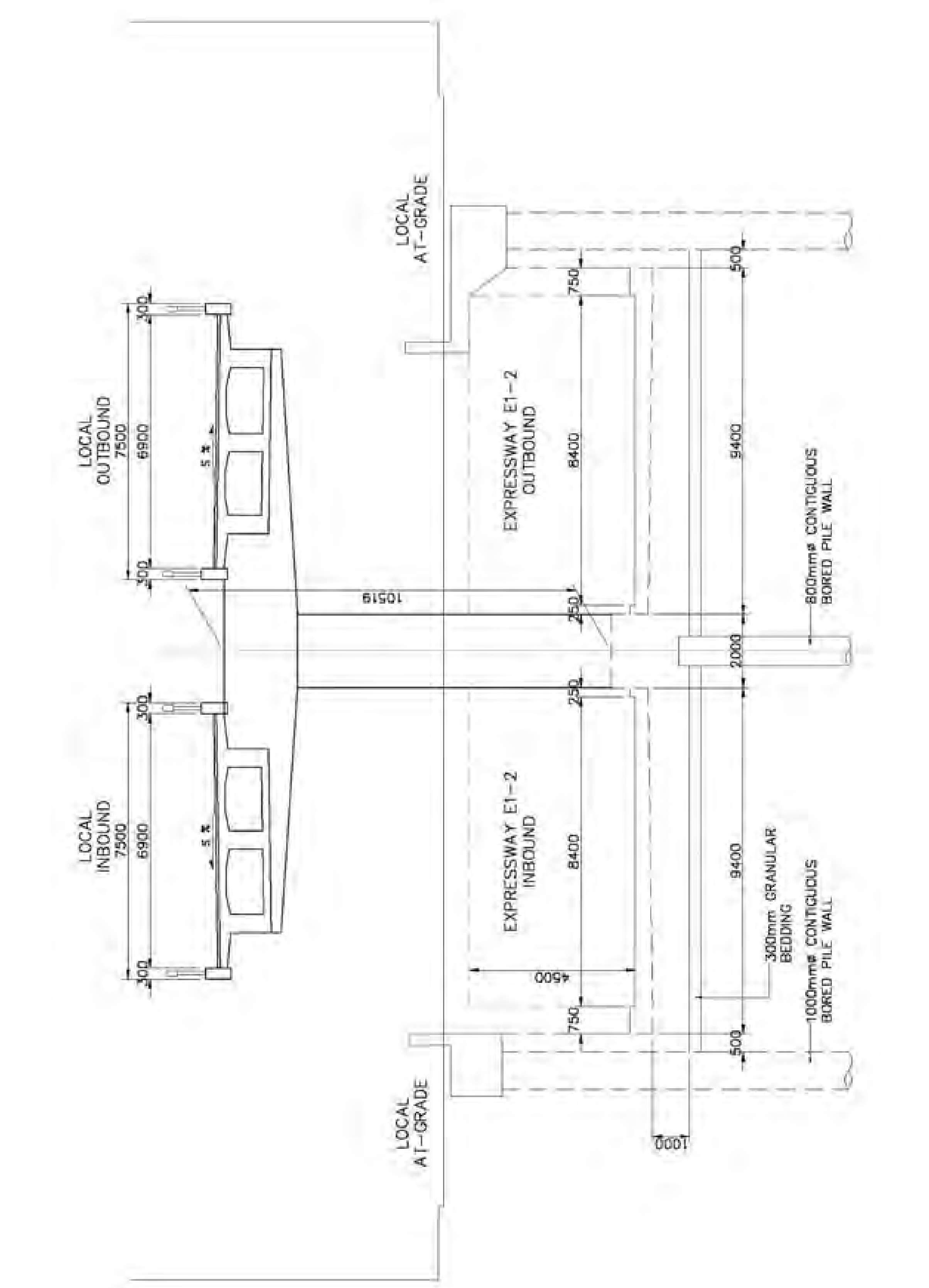


Figure 8.5-5 Section at Sta. 0+414: E1-2
(El-Mat haf al-Zira'i Street)

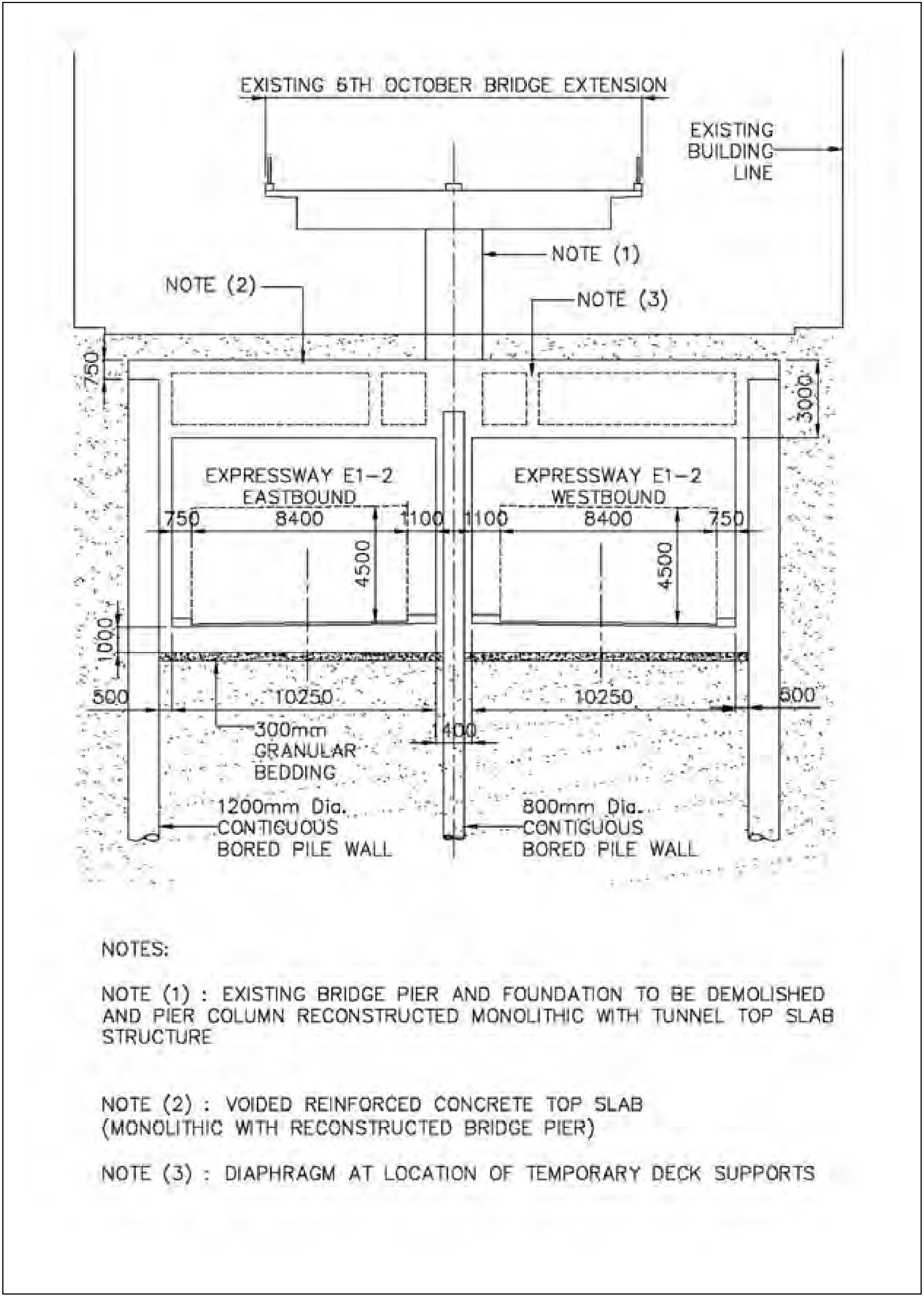


Figure 8.5-6 Section at Sta. 0+600: E1-2
(El-Mat haf al-Zira'i Street)

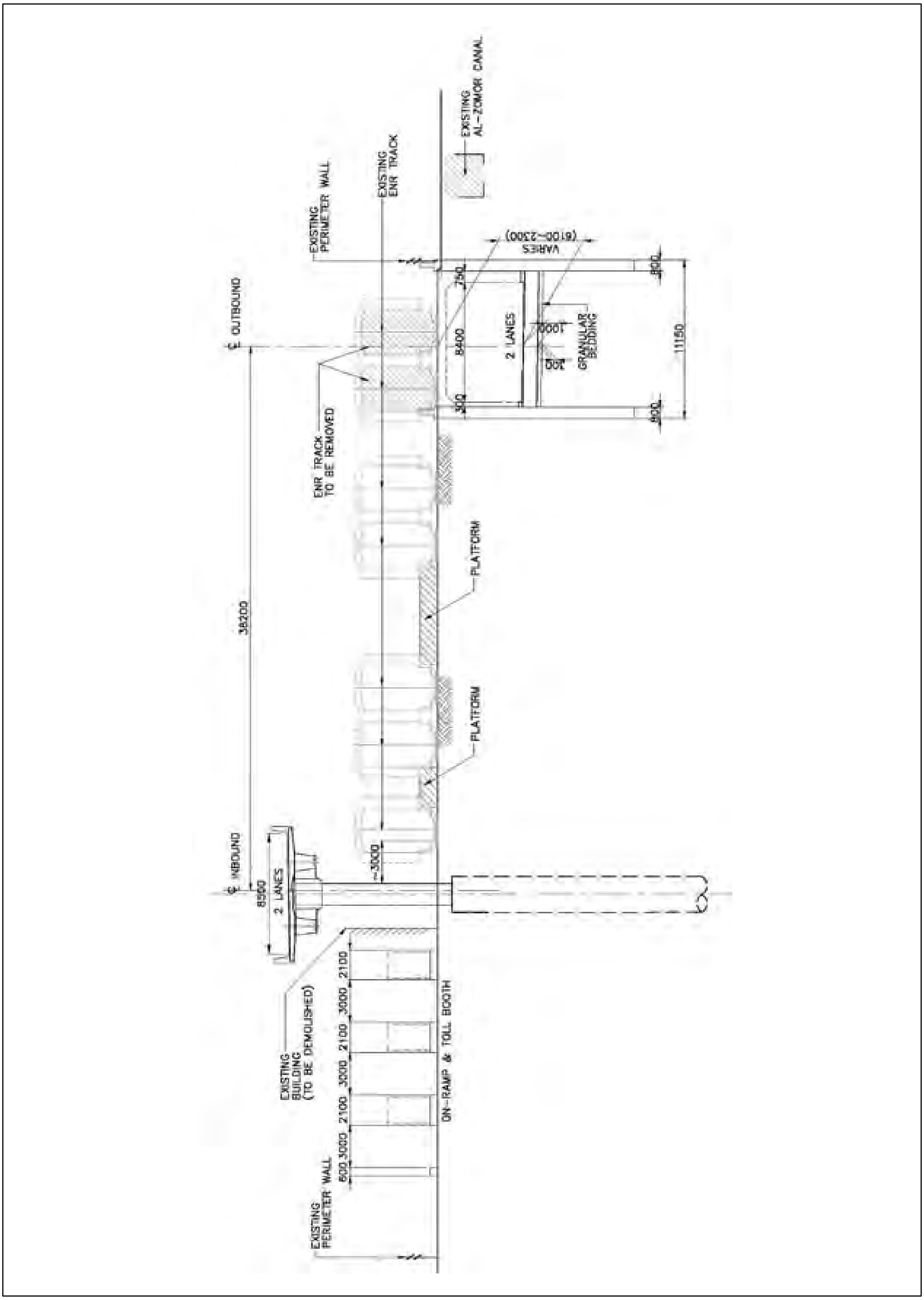


Figure 8.5-7 Section at Sta. 3+500: E1-2
(ENR Corridor)

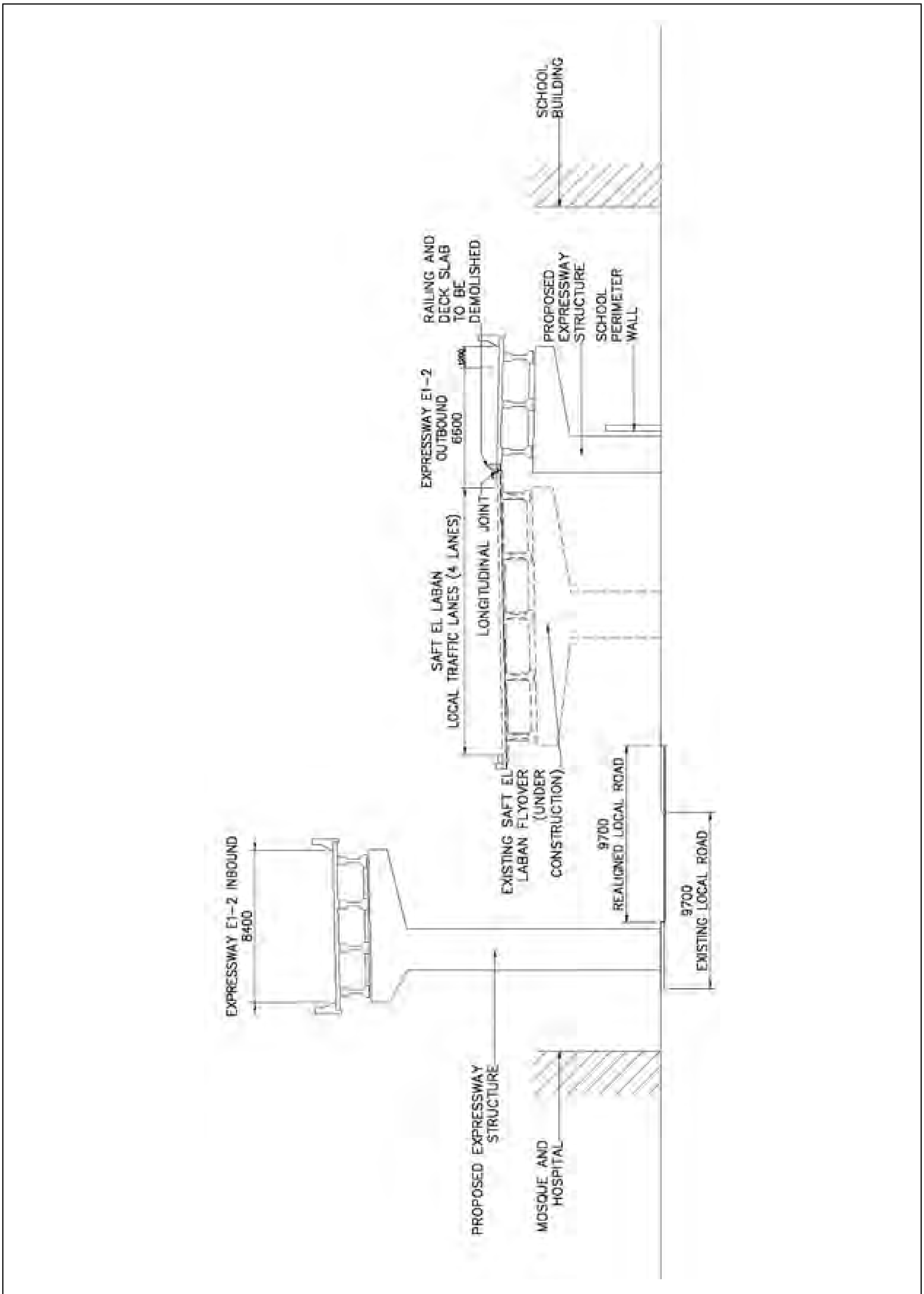


Figure 8.5-8 Section at Sta. 4+418 (Inbound): E1-2
(Saft El Laban)

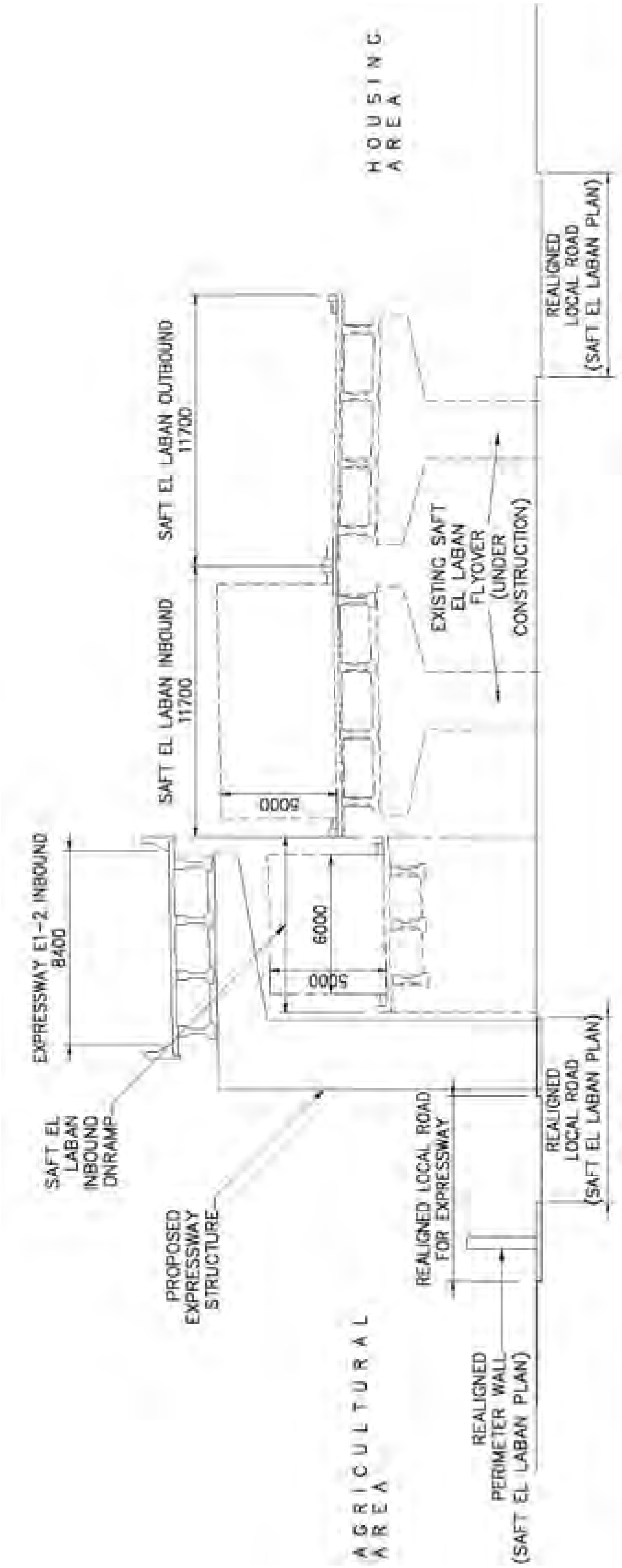


Figure 8.5-9 Section at Sta. 5+200 (Inbound): E1-2
(Saft El Laban)

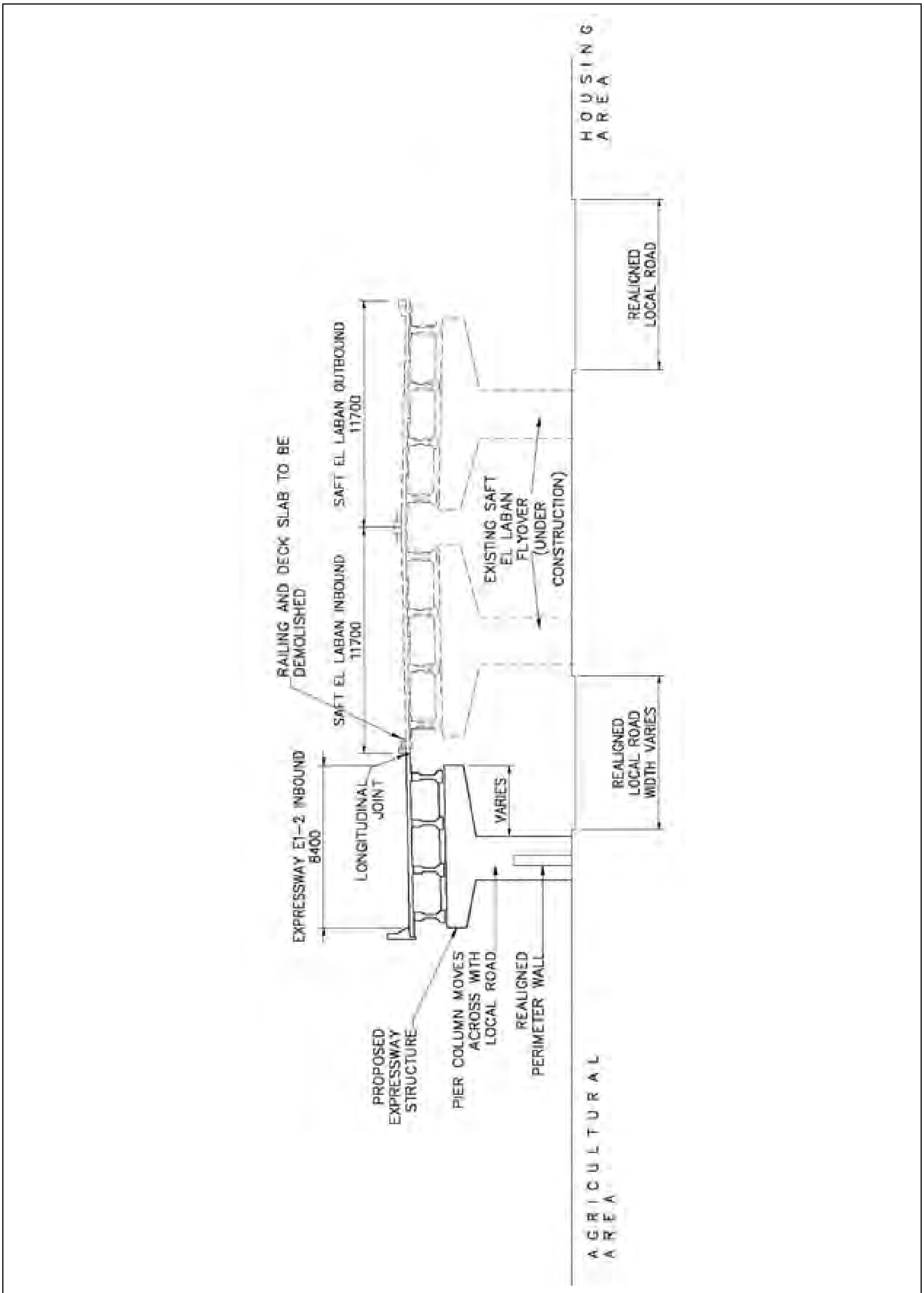


Figure 8.5-10 Section at Sta. 5+500 (Inbound): E1-2
(Saft El Laban)

8.6 SUPERSTRUCTURE AND SUBSTRUCTURES TYPES: E2-2

8.6.1 General

Expressway section E2-2 will provide the "missing link" for Eastbound traffic in the network, connecting E2 with E1. The expressway section will commence at the existing "jump off" point on 15th May Bridge (E2-1), at Abu El-Ela on 26th July Street. The expressway section will extend east along 26TH July Street above the existing 15th May Bridge (E2-1), will pass over 6th October Bridge (E1) and then will turn and extend north-east along Ramsis Street before again turning north onto Orabi Street for the connection with E1. The existing 15th May Bridge at this location provides only two lanes for Westbound traffic coming from E1. The proposed missing link will therefore provide two lanes for Eastbound traffic, passing directly above the Westbound traffic along 26th July Street. Refer to Figure 8.6-1 for a layout plan of E2-2.

The expressway section will be elevated for a total length of 1.8km; including a 1.0km double deck section along 26TH July Street. No on ramp/off ramp facilities are proposed in this relatively short section.

Features along the route that have an impact on the preliminary structural design are:

- The existing 15th May Bridge.
- The limited width of 26th July Street (19m minimum between buildings).
- The proposed underground Metro Line 3 and Maspero underground station along 26TH July Street
- The existing underground Metro Line 1 along Ramsis Street
- Nasser underground station at El-Esaaf intersection and Orabi underground station on Metro Line 1 at Orabi Street.
- The proposed E1-3 Expressway 1-3 along Ramsis Street.
- The layout of 6th October Bridge at the proposed E1/E2-2 intersection.

Features that may have any impact on the detailed structural design are:

- Existing foundation layouts for 15th May and 6th October Bridges (details unavailable for the Study)
- Final detailed design layouts of Maspero underground station. These may lead to changes in spanning and/or pier column locations from those established in the preliminary design
- Underground utilities – these may require modification to the pier pile caps in the form of block-outs or duct boxes (relocation of utilities typically is not feasible in central Cairo)

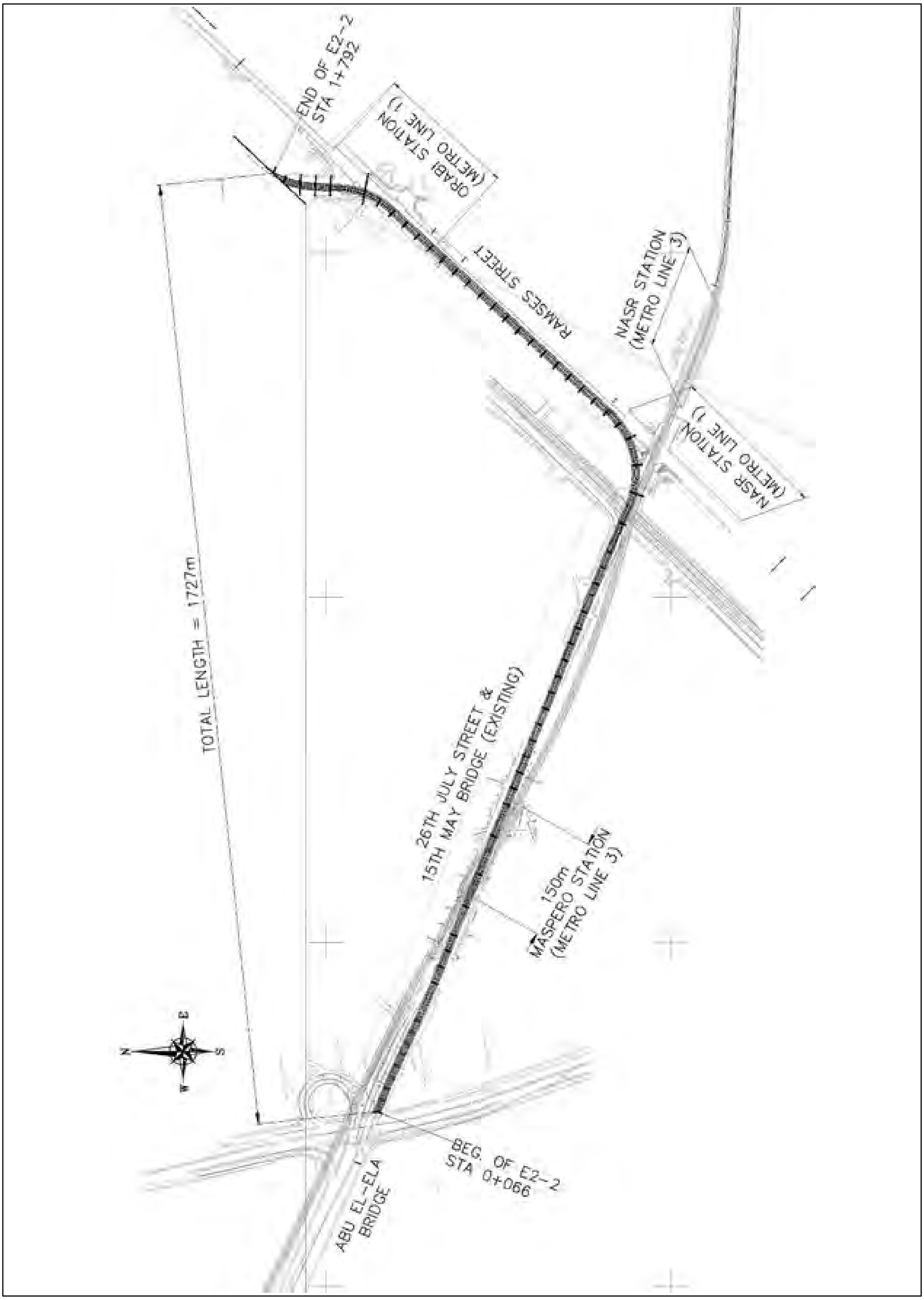


Figure 8.6-1 Layout of Expressway Section E2-2

8.6.2 Existing 15th May Bridge

The existing 15th Bridge along the proposed section of expressway E2-2 comprises the following components (starting from skewed pier at Corniche El-Nil - the "jump off" point of E2-2 Expressway):

- 7x25m span Concrete Box Girder (total length 175 m)
- 1x60m span curved Steel I Girder : spanning the access ramps to Aboul Ela Bridge over the Nile (part of 15TH May Bridge)
- 4x25m span Concrete Box Girder (total length 100 m)
- 3x60m span Steel I Girder (total length 180m) : location of proposed Metro Line 3 Maspero underground station (see Section 8.5.3)
- 2x25m span Concrete Box Girder (total length 50m)
- 11x25m span PC Girder (total length 275 m)
- 5 span RC Girder (total length 120m), varying span lengths: these spans include a breakdown bay and progress to a curved alignment on the approach to the connection with 6th October Bridge. The concrete piers are in alternative cantilever configurations in order to provide support to the wider deck for the breakdown bay and the curved deck departing from the tangential central reserve.
- 2x22m span curved Steel I Girder (total length 44m): These two spans provide the connection to 6th October Bridge and are supported on alternative configuration cantilever piers. The last pier is in structural steel and features pre-stressing bars located on the top flange of the cantilever beam. The pre-stress is required in order to minimize the cantilever beam depth and to provide a minimum of 4.5m operational headroom clearance over the at-grade roads under 6th October Bridge.
- Total length 1.0km.

26th July Street is very narrow for most of the length along this section of 15th May Bridge, being 19m between building lines at its narrowest. At this location 26th July Street provides two (2) lanes of traffic at-grade in each direction. These constraints have dictated the types of structure adopted for 15th May Bridge, including emphasis on pre-cast/prefabricated deck construction and the use of single column piers. Refer to Figure 8.6-2 for photographs of the existing bridge.

This section of 15th Bridge was constructed by the MoH and ownership transferred to Cairo Governorate. Construction of this section of the 15TH May Bridge was completed in 2002.

a. Proposed "jump off" point at skewed pier adjacent to Corniche El-Nil at Aboul Ela

b. Long span curved Steel I girder spanning access ramps onto Aboul Ela Bridge

c. Long Span I Girders at location of proposed Maspero Station of Metro Line 3

d. Limited width between buildings along 26th July Street (19m)

e. PC girders spanning to cantilever pier at location of breakdown bay

f. Cantilever steel pier at connection with 6th October Bridge

Figure 8.6-2 Existing 15th May Bridge

8.6.3 Metro Line 3 and Maspero Station

The National Authority for Tunnels (NAT) are currently undertaking the construction of Metro Line 3. Metro Line 3 is planned to provide an east-west underground rail facility in Greater Cairo that will be integrated with the existing Metro Line 1 and Line 2. Metro Line 3 will be 34.2km in length and will be constructed in 4 phases. Phase 1 and 2 are estimated together to require 6 years to construct with Phase 3 requiring 4 years (no information available for Phase 4). Construction of Phase 1 of Metro Line 3 commenced on 7thJuly 2007. The construction cost of Phase 1 & 2 is estimated at 280 million Euros and is to be financed through a French ODA soft loan. The consultant for the design of Phase 1 & 2 is SYSTRA. There is to date no basic design for Phase 3 and financing for Phase 3 is still not yet in place. Refer to Figure 8.6-3 for a plan of Metro Line 3.

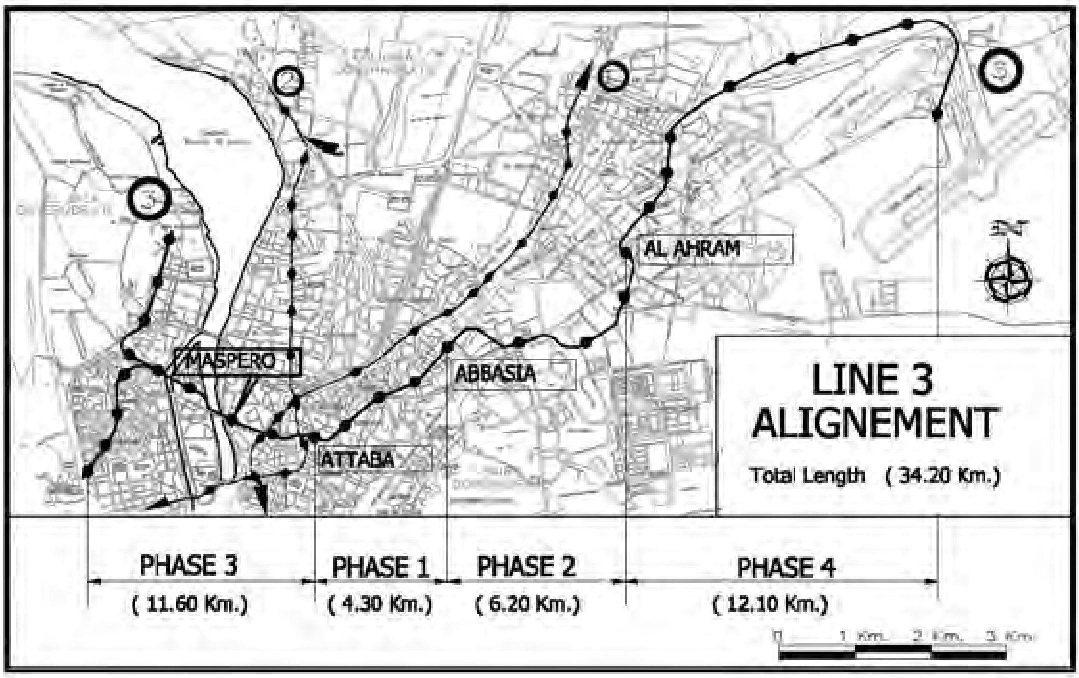


Figure 8.6-3 Plan of Metro Line 3

Phase 3 of Metro Line 3 will pass beneath the E2-2 expressway on 26th July Street. At this location the Maspero underground station is proposed to be constructed. Refer to Figure 8.6-4 for an approximate location plan of Maspero station. Although no basic design has yet been undertaken, NAT propose that Maspero station will be of a standard construction established for other underground stations on the Metro.

It is understood, following meetings by the JICA Study Team with NAT regarding Metro Line 3, that when 15thMay Bridge was under detailed design, the NAT provided MoH with requirements regarding foundation location for the new bridge at the position of Maspero Station. As a result the spans of the bridge were pushed out to 60m at the future station location in order to minimize the future integration works with the station. The intention is that the existing bridge deck will be supported on temporary piers during the construction of Maspero station so that the existing bridge piers and foundations can be demolished and new pier columns constructed integral with the station structure.

Figure 8.6-4 Approximate Location of Maspero Station: Metro Line 3 on 26TH July Street

8.6.4 E2-2 Superstructure Types

The strategy for construction of the elevated expressway along 26th July Street will have an influence on the type of superstructure selected. There are several alternative strategies for the construction along 26th July Street including the following:

1. Retain the existing 15th May Bridge and construct a single new deck above it on separate independent foundations.
2. Demolish the existing 15th May Bridge and construct new double-deck structure on new foundations along 26th July Street
3. Partially demolish the existing bridge at the location of the proposed Maspero MRT underground station. Construct new double-deck structure on new foundations, giving an optimum arrangement for integration with the future underground station, along

this demolished section. Construct a single new deck, connected with the double deck section, above the remaining existing 15th May Bridge on separate independent foundations.

The extent of the partial demolition will have to be addressed at the detailed design stage. At least two options are available:

a) Demolish both the pier foundations and the existing deck and replace with new structure.
b) Jack the existing deck onto temporary supports and demolish the existing piers. Reconstruct new piers to support both the existing deck and the new deck above it. Remove temporary supports leaving existing deck supported on new pier structure.

Refer to Table 8.6-1 for a comparison of the construction strategies options for E2-2 along 26TH July Street.

For the purposes of illustrating the preliminary design, Strategy 3 has been selected given that it combines the advantages of the other two strategies. For the purposes of establishing quantities and costs of the preliminary design the option of completely demolishing the section of existing bridge at the proposed site of Maspero Station has been assumed i.e both the existing bridge piers and the existing steel deck are demolished.

The study on superstructure types has been made based on several considerations including:

- Determination of required span length along each critical section, particular attention was paid to the location of the proposed Metro Line 3 and Maspero station, the existing Metro Line 1, Nasser and Orabi underground stations in determining span arrangement
- Restrictions imposed on method of construction and span arrangement by the existing 15TH May Bridge and 6TH October Bridge
- Construction cost and construction time considerations
- Required temporary works and construction equipment with corresponding impact on traffic management
- Inspection and maintenance considerations
- Aesthetic considerations

Table 8.6-1 Comparison of Construction Strategies along 26th July Street : E2-2

Option	Advantages	Disadvantages
1. Retain existing bridge. Construct new single deck above existing on new foundations.	Construction cost less than other alternatives. Less impact on traffic during construction. Less time to construct than other alternatives.	Existing Westbound carriageway width below expressway standard (7m existing; required 8.4m). Foundation locations of new construction controlled by existing structure. Additional bridge foundations required to be integrated with proposed Maspero station. Overhead restrictions on construction of new bridge foundations. Operational clearance for at-grade traffic below standard. Additional pier construction is more visually intrusive.
2 Complete demolition of existing bridge and construction of new double deck bridge on single foundations	Expressway standards can be imposed for lower Westbound deck. Optimum arrangement to integrate bridge foundations with Maspero Station Operational clearance can be maintained at standard 5.5m for at-grade traffic. Asymmetrical deck layouts, with single upper deck cantilever pier, can be considered in order to minimize overall structure width.	Highest cost of all options. Greatest impact on traffic during construction. Longest construction period of all options. Re-connection of Westbound lower deck to 6TH October Bridge will remain below expressway standard (50m radius bend and restricted width of carriageway).
3 Partial demolition of existing bridge at location of Maspero Station. Combination of Option 1 and 2 above.	Optimum arrangement to integrate bridge foundations with Maspero Station. Less costly than Option 2. Shorter construction period than Option 2.	Higher cost than Option 1. Substantial impact on traffic during construction. Longer construction period than Option 1. Other related disadvantages of Options 1 and 2 apply.

Deck types that have been adopted in the construction of double deck bridges in and around Cairo are (1) pre-cast PC beam and slab (Weli Bridge), (2) steel I-girder (Weli Bridge) and (3) in-situ concrete box girders (Saft El Laban).

Rapid construction methods are recommended at E2-2 (refer Table 8.4-1). The form of superstructure therefore should be selected to maximize the use of prefabrication. The in-situ concrete girder option has therefore been discounted both for the double deck section and construction along Ramsis Street.

PC girders in beam and slab decks are a cost competitive option to consider during the detailed design. However, this form of construction has the following characteristics:

- It is not well suited to integration with double deck frame piers, particularly if the piers are in structural steel, or with composite columns
- May involve the installation of numerous bridge bearings with subsequent inspection and maintenance issues, particularly disadvantages for the upper deck of double deck bridges during the expressway operation
- Bearings can be avoided if methods are adopted to effect continuity – however this is a full depth treatment in the deck located above the pier support, an option that will not be available in the restricted headroom situation encountered at E2-2
- Expansion joints at each support can be avoided with flexible slabs, but this involves an additional construction stage
- The alignment includes sections with relatively tight curves with limited locations for pier supports (and therefore longer spans) making the selection of long straight PC girders impractical at these locations
- The PC girders require to be lifted in full length, therefore offering less flexibility in erection than structural steel, particularly in the restricted working area afforded by the very narrow 26th July Street
- the PC girders bring a rather utilitarian visual impact which will be accentuated in a double deck bridge and for the rather elevated deck section descending onto Ramsis Street

Given the above considerations, the PC girder beam and slab deck have not been selected to illustrate the preliminary design.

Given the advantages brought with the use of steel closed box tub girders combined with pre-cast deck slabs (refer Section 8.4.3), this form of deck construction has been selected to illustrate the preliminary design, rather than a multiple steel I-girder deck.

For the short section of deck leading off from Abu El-Ela Bridge, a form of construction sympathetic with the existing structure has been selected. Although thus is an in-situ option, given that it will be constructed over a relatively short section, in a single deck configuration, and that the road will be closed to traffic anyway, it is considered not to have a detrimental impact on the period of construction of E2-2.

Similarly for the short section of deck that will connect with the existing 6th October Bridge at Orabi Street, the form of construction has been selected to match the existing deck i.e. a PC girder beam and slab solution. The multiple girder arrangement will also be advantageous in “stepping down” the deck width as it connects with the existing bridge.

8.6.5 E2-2 Substructure Types

The study on substructure types has been made based on several considerations including:

- Study on soil conditions
- Construction cost and construction time considerations
- Operational clearance requirements of at-grade lanes on 26TH July Street, 6TH October and Ramsis Street
- Restrictions imposed on method of construction for foundation locations beneath 15th May Bridge
- Restrictions imposed on method of construction for foundation locations above or adjacent to the proposed Metro Line 3 and the Maspero underground station, the existing Metro Line 1, Nasser and Orabi underground stations
- Impact of method of construction on traffic management
- Aesthetic considerations

Rapid construction methods are recommended at E2-2 (refer Table 8.4-1).

The study on foundations basically can be divided into two main sections, (1) 26th July Street section in the vicinity of the existing 15th May Bridge and (2) Ramsis Street section. The factors involved in determining type of foundation and pier column layouts for these two main sections are summarized in Table 8.6-2.

It is noted that bored piles are the typical foundation of choice in Cairo given the prevailing soil conditions and the need to prevent undue noise and vibrations from occurring during construction.

Table 8.6-2 Foundation Considerations and Pier Column Layouts : E2-2

		26th July Street	**Ramsis Street**
a.	Soil Conditions (Nasser Station to Maspero Station)	• Top fill layer to a depth of 2.5m to 3.0m. • A layer of stiff to very stiff silty sandy clay below the fill extending to a depth ranging between 7.5 to 14.5m. • A layer of dense to very dense fine to medium sand with traces of silt extending below the above layers. *Conditions are therefore very conducive to the use of bored piles with pile tips finding footing in the dense sand. Pile lengths have conservatively been set to a length of 20m to ensure founding in the sand. (Terminating large diameter bored piles in clay should be avoided if possible. Piles founded in dense sand effectively will not fail due to bearing capacity limitation.)*	
b.	Construction time	The closure of 26th July Street to traffic and the impact of both the demolition works and the Maspero Station works (if concurrent) means that period of construction is not a vital consideration for the bridge foundations.	A key issue along Ramsis Street given that this street will not be fully closed to traffic during construction. The selection of foundation types that minimizes construction time is a priority. Large diameter bored piles, without the need to construct pile caps, offer distinct advantages.
c.	Traffic Disruption	Given that 26th July Street will be closed to traffic, traffic disruption is not a direct factor in determining foundation type. However a well conceived traffic diversion plan will be essential to avoid disruption to overall traffic circulation during construction.	Avoiding traffic disruption along Ramsis Street is vital. Any foundation proposal that involves a large footprint occupying a portion of the street cannot be considered.
d.	Operational clearance requirements of at-grade lanes	Space is only available in the central reserve for the bridge piers. Therefore single column piers are the only alternative.	The location of the proposed deck above the at-grade lanes militates for the use of single column piers with cantilever pier caps to support the deck.
e.	Constructability	The headroom restrictions imposed by the existing 15th May Bridge prevent the use of large diameter bored pile rigs.	No headroom restrictions are in place. All types of bored piles can be constructed.
f.	Proximity of existing foundations and underground structures	Existing bridge foundations require that new foundations are sited such as not to conflict. (for integration with Maspero Station see Section 8.6.6)	Both Metro Line 1 (mainline and Orabi Station) and Metro Line 3 impose severe restrictions on the location and available footprint for new foundations. Large diameter bored piles offer distinct advantages.
g.	Utilities	No data available to the Study on underground utilities.	
Proposed Foundation Type		**800 mm Dia. Bored Piles and Pile Caps supporting Single Column Piers**	**2.5 m Single Bored Piles supporting Single Column Piers**

The study on pier column type has been driven primarily by considerations of rapid bridge construction. The following factors have led to the adoption of structural steel and composite columns in the preliminary design:

- Minimizing period of construction, fabricated steel brings obvious advantages
- Minimizing column size, both along 26th July Street (given the narrow width) and along Ramsis Street (given the relatively large pier cantilevers imposing high demand on the columns). Structural steel and composite columns bring substantial advantages in allowing compact section sizes.
- Constructability issues, particularly for the construction of a pier frame around the existing deck of 15th May Bridge.
- Integration of pier column with large diameter bored piles. Composite columns allow a very direct solution without the need for anchor frames.
- Integration of pier and deck structure to avoid bearings/joints
- Span length of the pier portals demand a structural steel solution
- Aesthetics: a sculpted pier form with elegant proportions is more easily attainable in structural steel (refer Figure 8.4-3)

Twin column pier portals are necessary both where the alignment transitions from 26th July Street to Ramsis Street (no roadway space available for direct support) and in the vicinity of Orabi Station on Metro Line 1.

Orabi Station in particular is very problematic given that it effectively extends across the complete width of Ramsis Street with extensions into Orabi Street. As a result there are few locations available to site pier columns for E2-2 at this location and the only available option for this Study is to make use of very long span portals combined with single large diameter bored pile foundations. It is acknowledged that the span length of at least one of the portals at Orabi is on the limit for practical design; 38m span at Pier P56 requiring a limiting 3m deep portal beam. Alternative configurations can be investigated at the time of the detailed design; locating pier supports at suitable locations directly above Orabi underground station may be an option subject to the agreement of NAT and provided that detailed drawings of the existing station structure are made available.

8.6.6 Integration with Proposed Metro Line 3 Maspero Underground Station

The study on integration with the future Metro Line 3 Maspero underground station is substantially dependent on NAT plans of the proposed station works and their requirements regarding bridge pier foundation construction works above the station.

NAT made the following information available to the Study:

- Layout plan of Metro Line 3
- Proposed location of Maspero station
- Layout of standard station (proposed for Maspero)
- Preliminary profile from Nasser Station to Maspero Station and beyond
- Borehole logs for borings along 26th July Street

The NAT requirements regarding integration of bridge pier foundations with Maspero Station were not forthcoming during the period of the Study. The construction schedule for the proposed work has also not been fixed at the time of preparation of this report.

There are several scenarios to consider for the preliminary design:

Scenario 1. E2-2 is constructed in advance of Metro Line 3 Phase 3 with:
 a. EITHER conventional bridge foundations (piles and pile caps), or
 b. Maspero Station included wholly are partly in the E2-2 contract so that the structures can be fully integrated

Scenario 2. Metro Line 3 Phase 3 is constructed in advance of E2-2 with provision made in the Maspero station design to support the future bridge piers of E2-2

Scenario 3. Both constructions occur more or less concurrently with provision made in the Maspero station design to support the E2-2 bridge piers

Most of the above scenarios involve a non-conventional foundation for the E2-2 structure, with the pier column integrated into the station roof. This arrangement has therefore been incorporated into the preliminary design. Refer to Chapter 13 for the impact of this arrangement on cost.

The rail level at Maspero station is approximately 22m below street level. This is due to the proposal to pass Line 3 below Line 1 at the location of nearby Nasser station. As a result, there is approximately 4m between the top of the station roof slab and street level. There is therefore sufficient depth to accommodate an integral cross-beam to support E2-2 pier columns and transfer loads into the diaphragm walls of the station structure. This layout has been adopted in illustrating the preliminary design. Full co-ordination with NAT is required in the detailed design in order to establish a final concept for the structure and to ensure that all requirements of NAT are met regarding the integration.

8.6.7 Demolition Option at 26th July Street

An alternative to the double deck configuration proposed for the preliminary design is an option that involves the demolition of buildings along 26th July Street. This option will create space on the north side of 26th July Street to allow the construction of an additional deck adjacent to the existing structure, thereby avoiding the need to construct a double deck bridge.

The concept requires westbound traffic, currently using the existing 15th May Bridge, to be transferred onto the new adjacent structure, so that Eastbound traffic can then be carried on structure using the space currently occupied by the existing bridge.

Two basic options are available:

Option 1: Demolish the entire length of the existing 15th May Bridge between Abu El-Ela and 6th October Bridge. Construct a new viaduct structure to carry both Eastbound and Westbound traffic along this section of 26th July Street.

Option 2: Retain a section of the existing 15th May Bridge to be used by E2-2 Eastbound traffic. Construct new adjacent structure for Westbound traffic.

<u>*Option 1*</u>

The main advantage of Option 1 is that it offers a free hand in conceiving new optimal substructure layouts to support the two new adjacent decks; this will allow the maximum use of the available space either to create car parking facilities or a commercial space beneath the bridge decks. The option also has the following additional advantages:

- It provides some flexibility in choice of horizontal alignment; shifting the alignment of the combined structure to the south side would result in a correspondingly reduced right of way width to be acquired on the north side of 26th July Street
- It allows full expressway width structures to be provided for both directions (the existing bridge width is sub-standard)
- It allows full freedom in choice of deck for both directions, affording some degree of optimization where the decks are directly adjacent
- It also offers the best arrangement for integrating bridge substructures into the proposed Maspero Station underground structure.

<u>*Option 2*</u>

Option 2 has the advantage of retaining a section of the existing structure in order to minimize cost and construction period. However, given that new substructure will be constructed adjacent to the existing bridge piers, this option will result in a substantial loss of space and may also prove not to offer significant savings in cost or time for construction.

Of the total existing 15th May Bridge length in this section, only about 400m can be retained to serve E2-2 Eastbound traffic. Up to about 200m of the existing 15th May

Bridge, connecting to Abu EL-Ela Bridge, will have to be demolished to allow construction of new structures, and similarly about 300m of the existing bridge will have to be demolished from a point near the connection with 6th October Bridge.

Note also that the 400m length of the existing 15th May Bridge to be retained for Eastbound traffic includes the 180m long section of steel I-girder deck (three 60m spans) at the location of the proposed Metro Line 3 Maspero Station. Part or all of this existing bridge structure section will therefore require demolition for the purpose of integration with the future station structure. The rationale behind retaining lengths of existing bridge under these constraints will therefore have to be carefully addressed during the detailed design, should the demolition option be seriously pursued.

Refer to Chapter 6 for a layout plan of the proposed demolition scheme. Refer to Figure 8.6-5 and Figure 8.6-6 for typical draft cross-sections of Option 1 and Option 2.

8.6.8 Typical Sections

Typical sections illustrating the preliminary design are presented in Figure 8.6-7 to Figure 8.6-15.

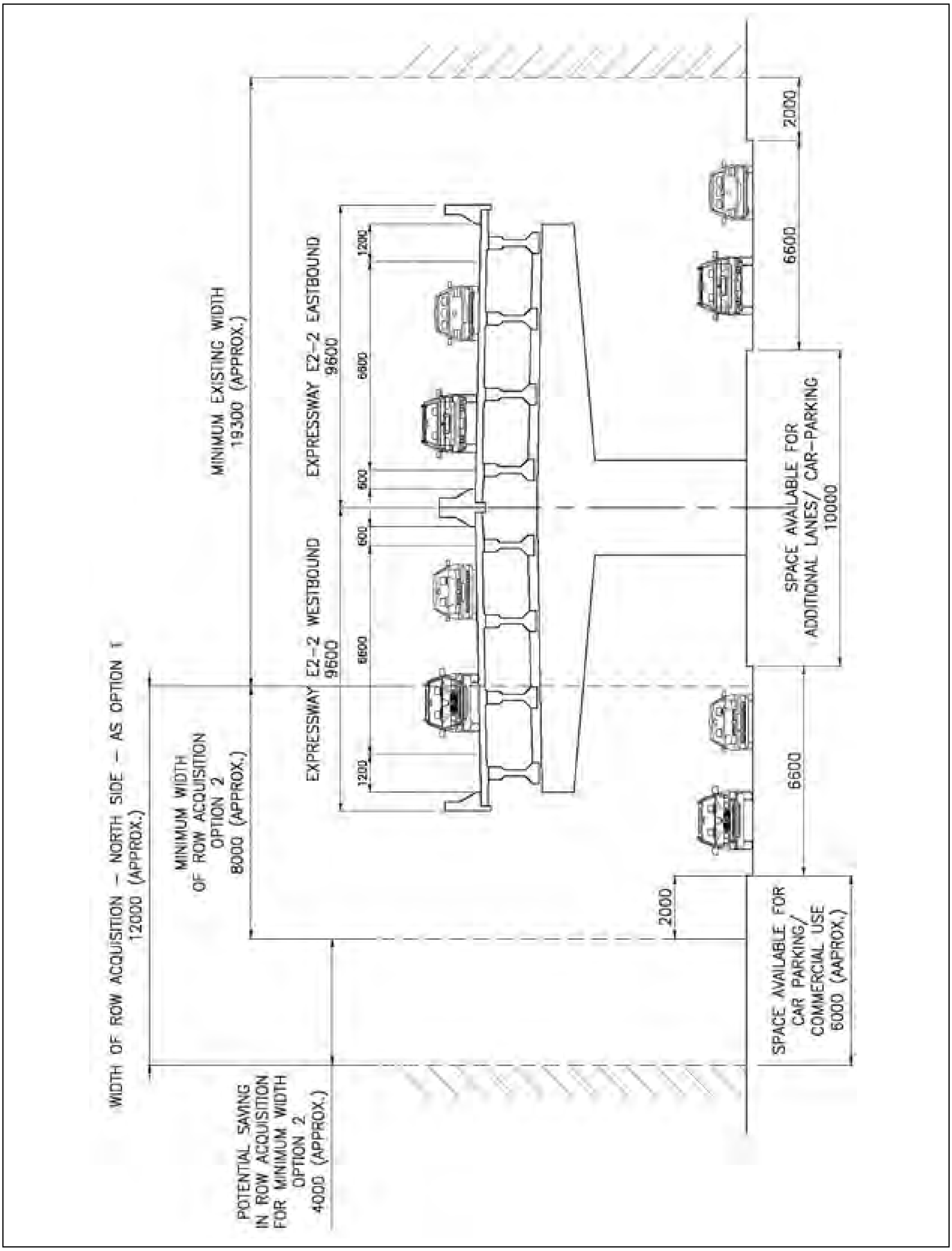


Figure 8.6-5 DRAFT Section
(DEMOLITION OPTION 1: 26th July Street)

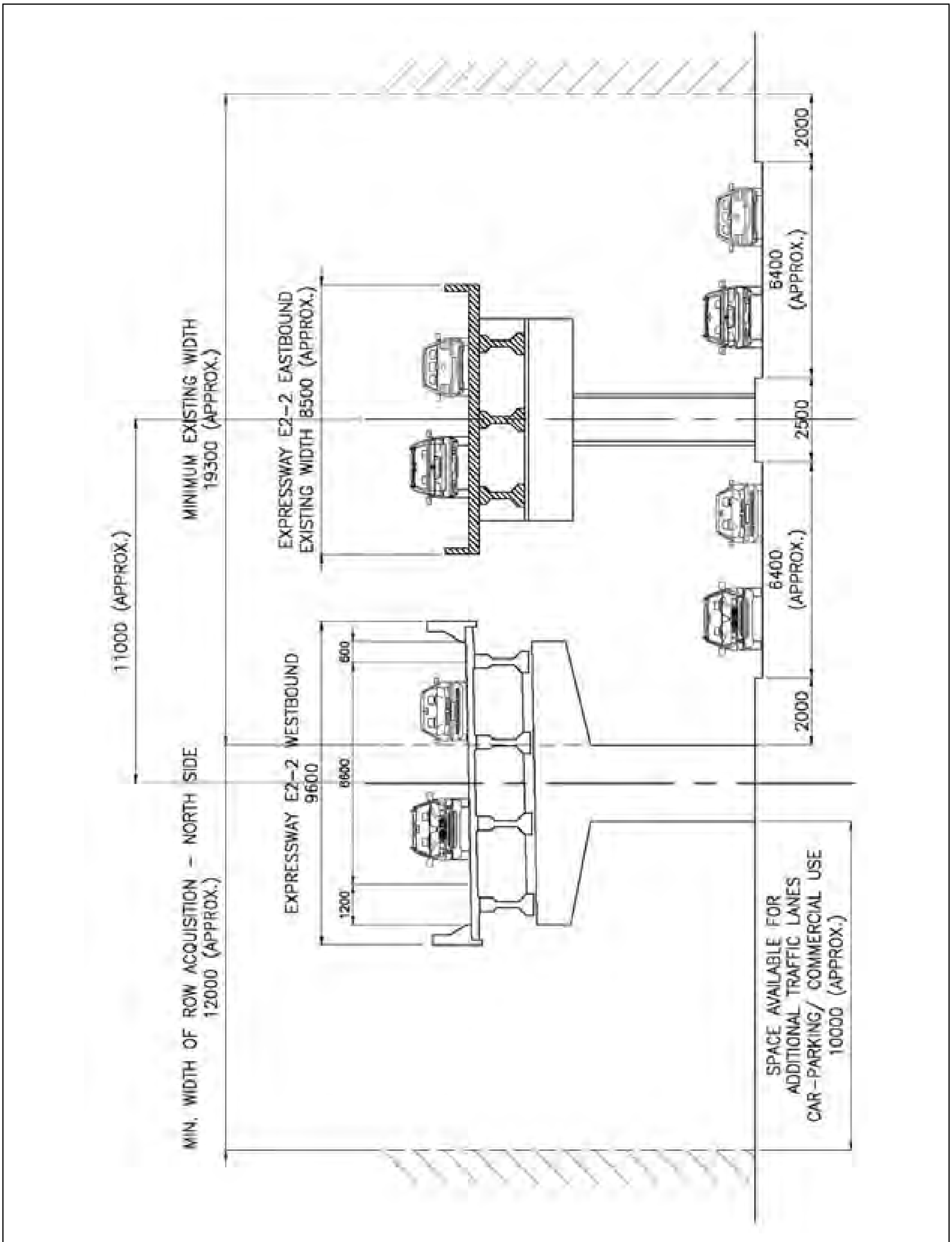


Figure 8.6-6 DRAFT Section
(DEMOLITION OPTION 2: 26th July Street)

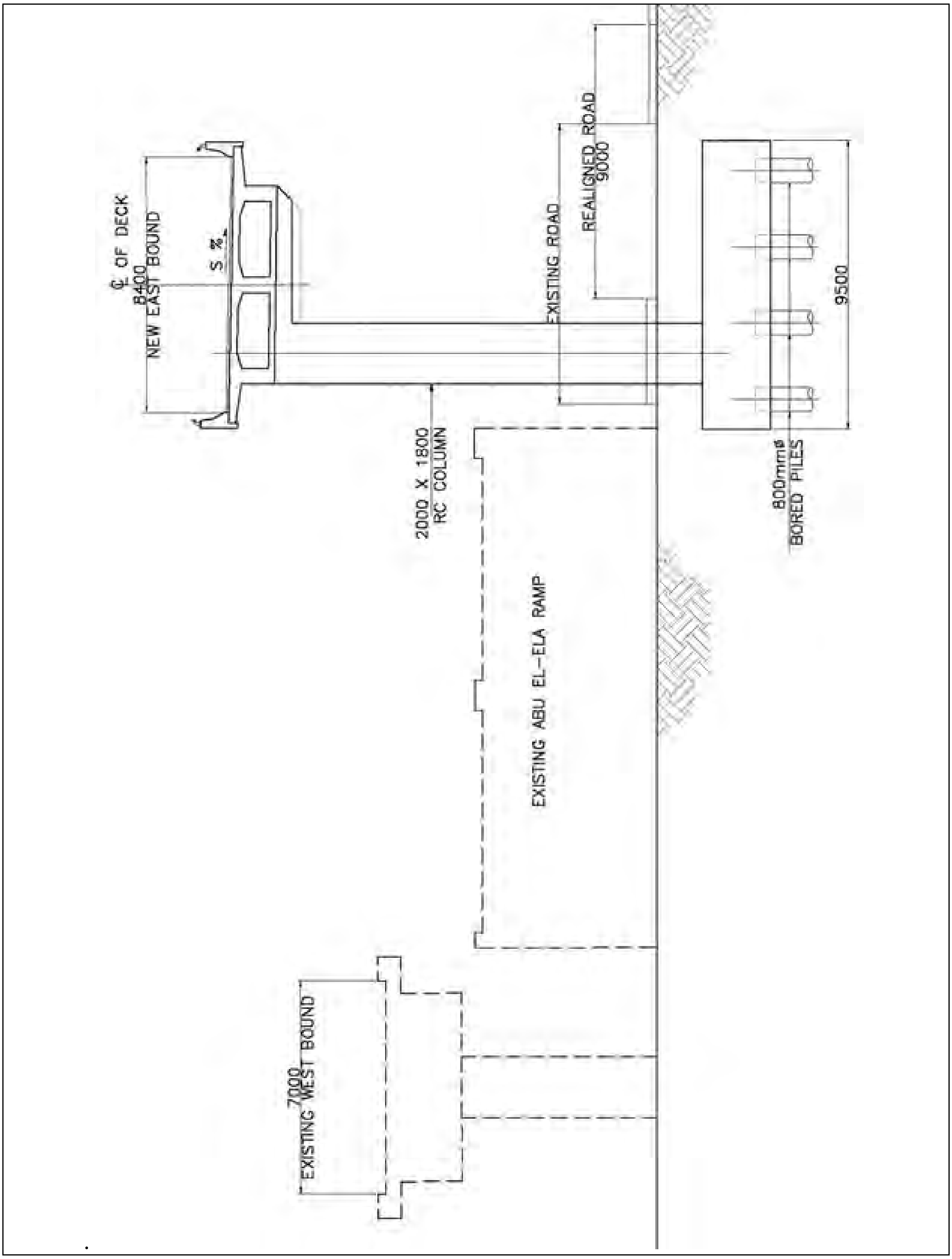


Figure 8.6-7 Section at Sta. 0+200: E2-2

(26th July Street)

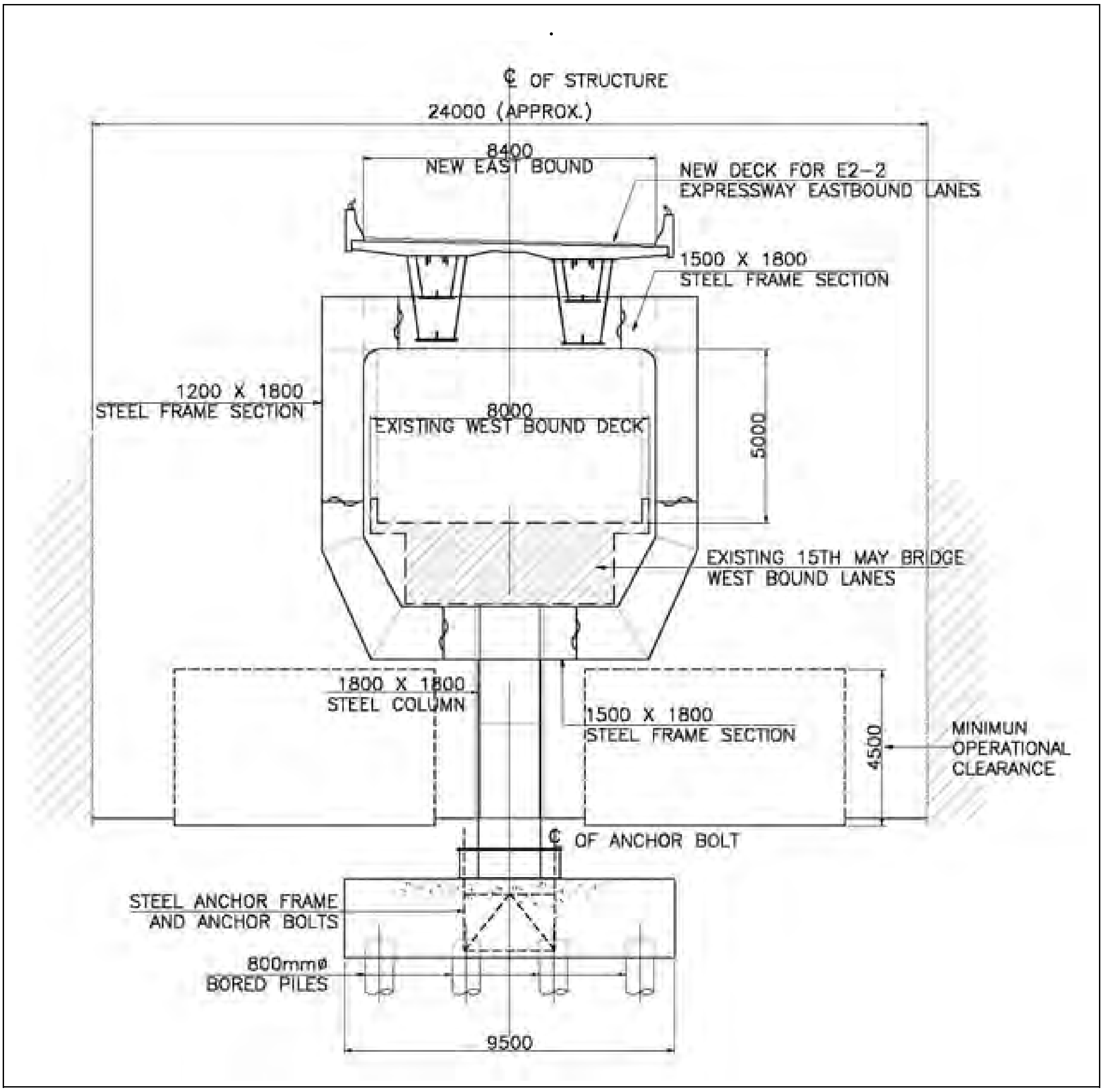


Figure 8.6-8 Section at Sta. 0+270: E2-2
(26th July Street)

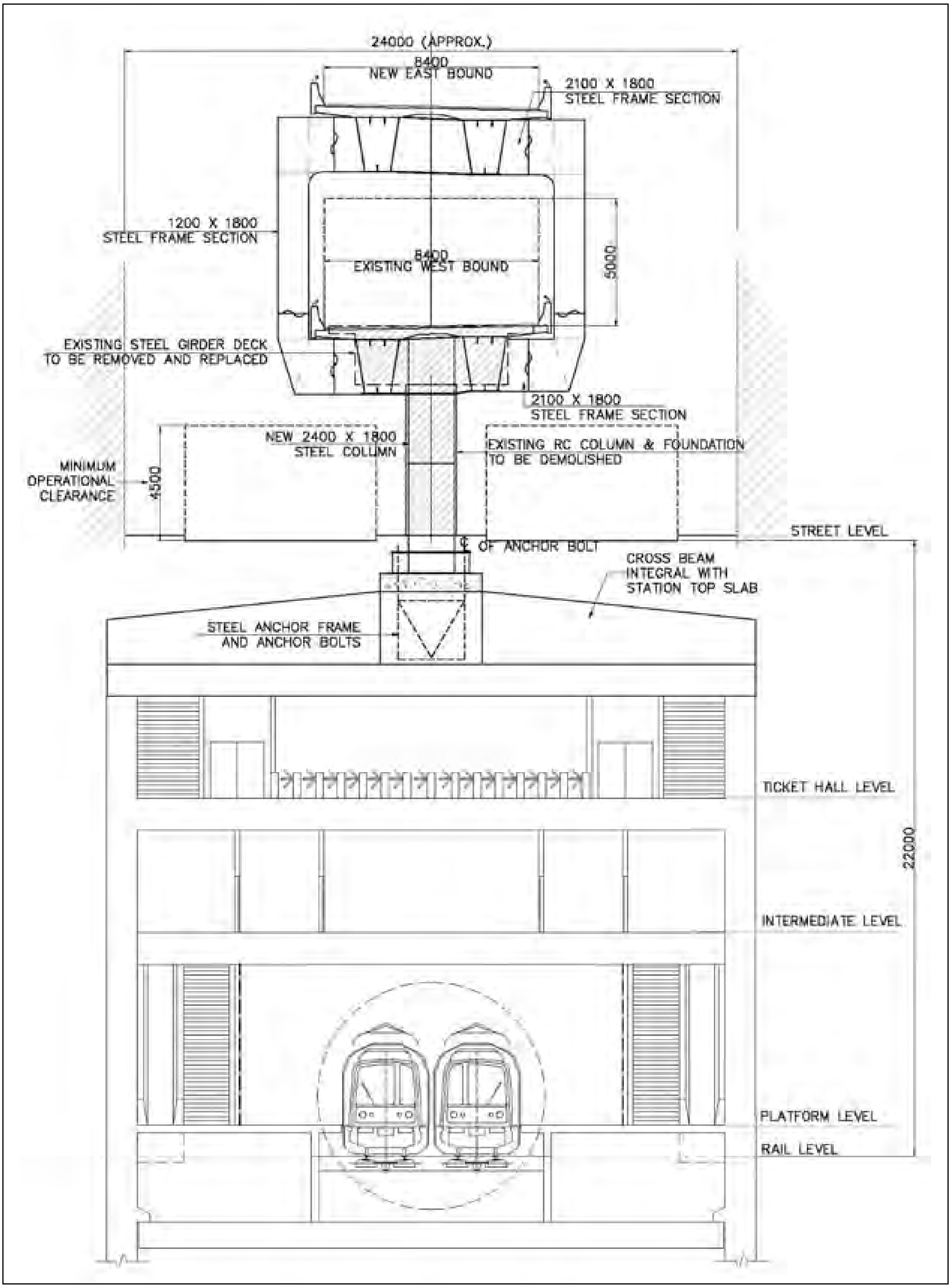


Figure 8.6-9 Section at Sta. 0+440: E2-2
(26th July Street)

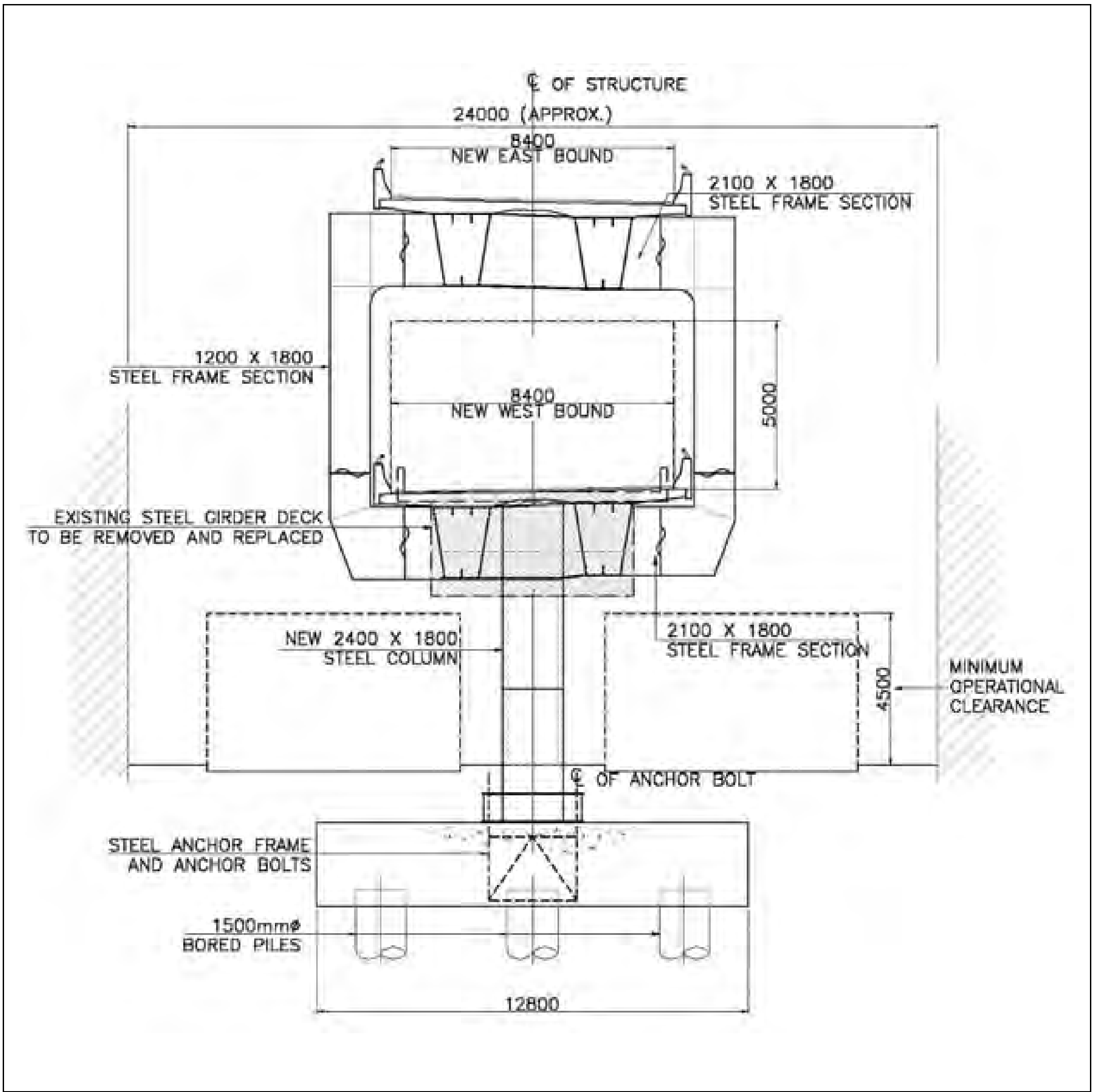


Figure 8.6-10 Section at Sta. 0+550: E2-2
(26th July Street)

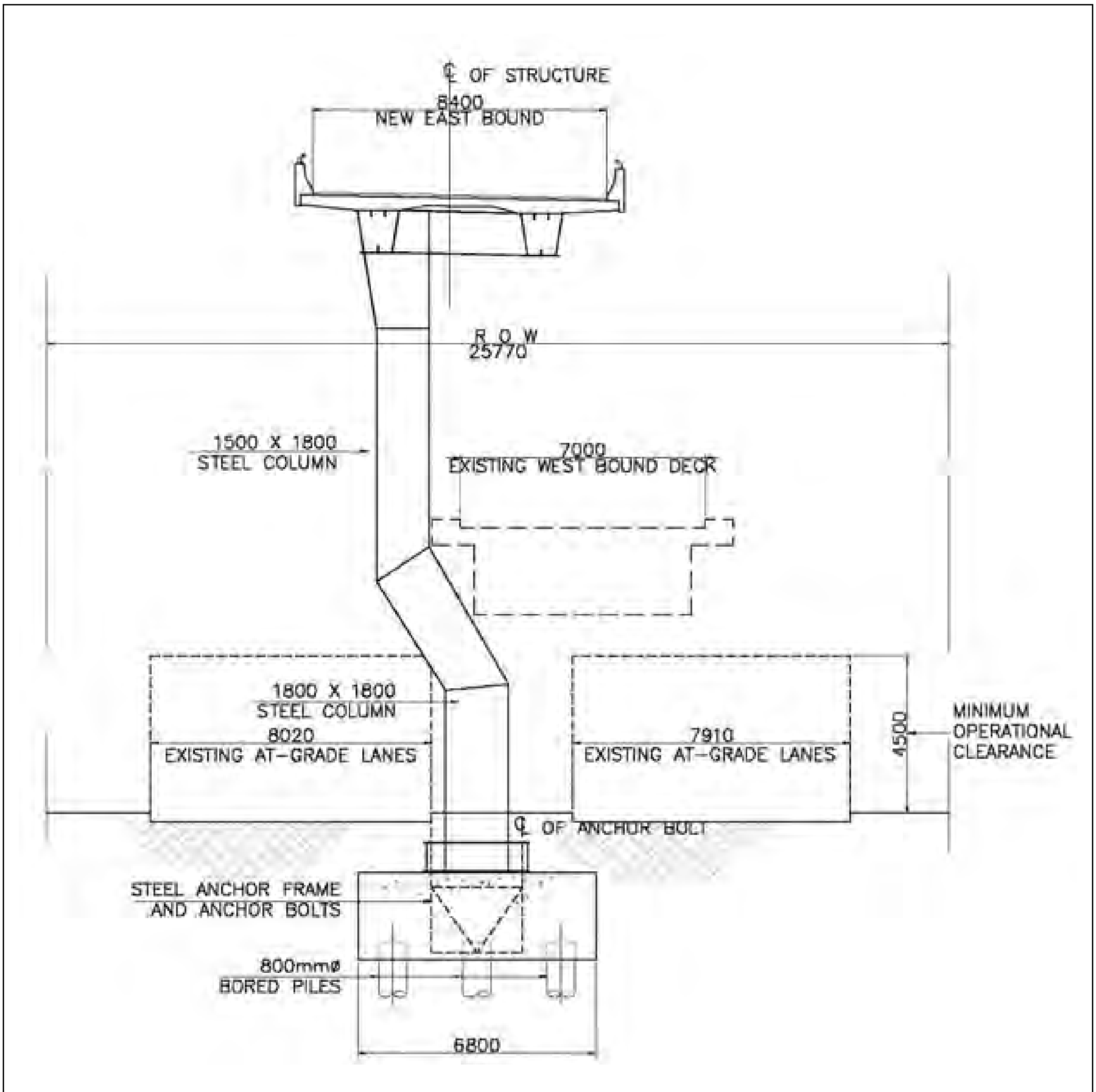


Figure 8.6-11 Section at Sta. 0+910: E2-2
(26th July Street)

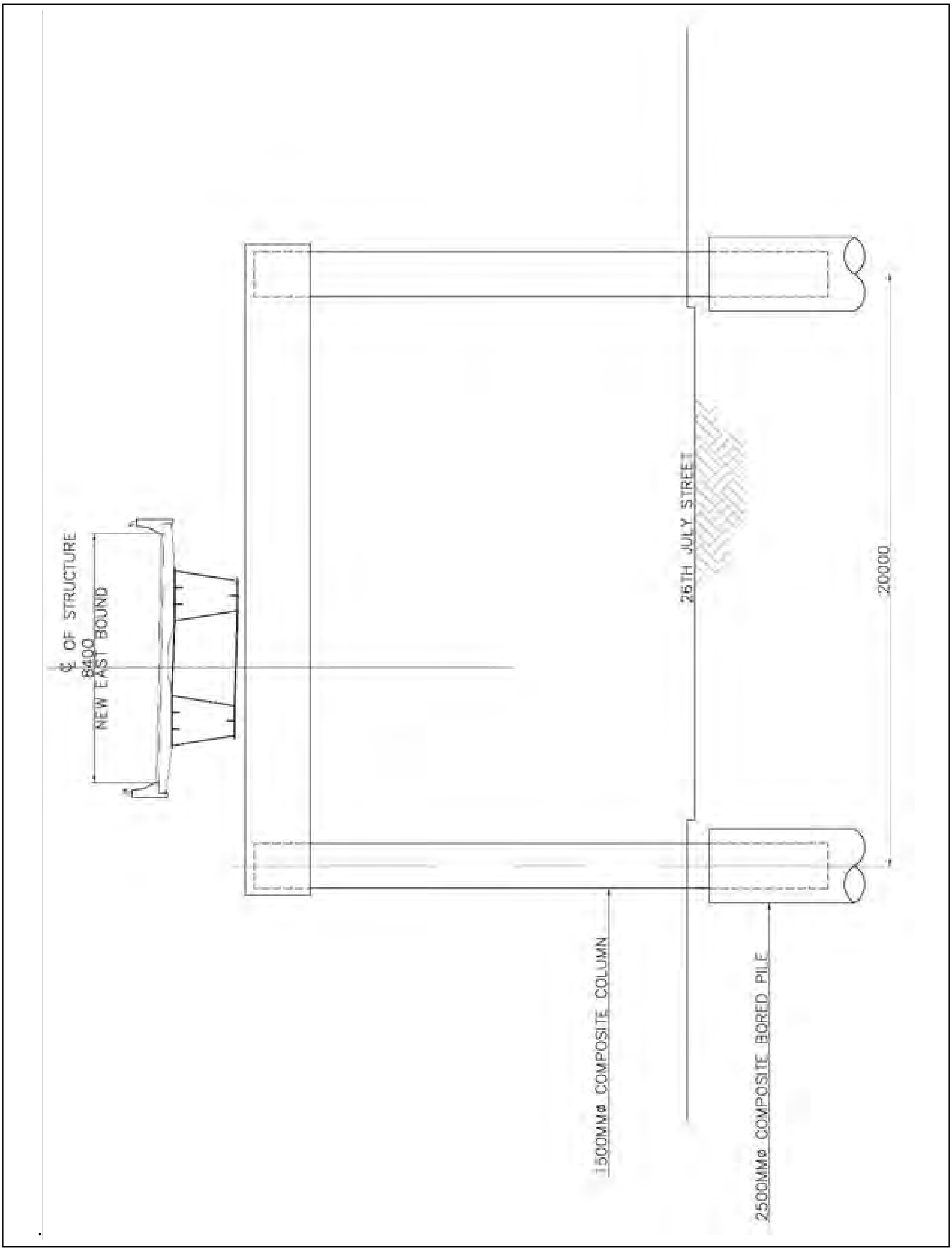


Figure 8.6-12 Section at Sta. 1+040: E2-2
(26th July Street)

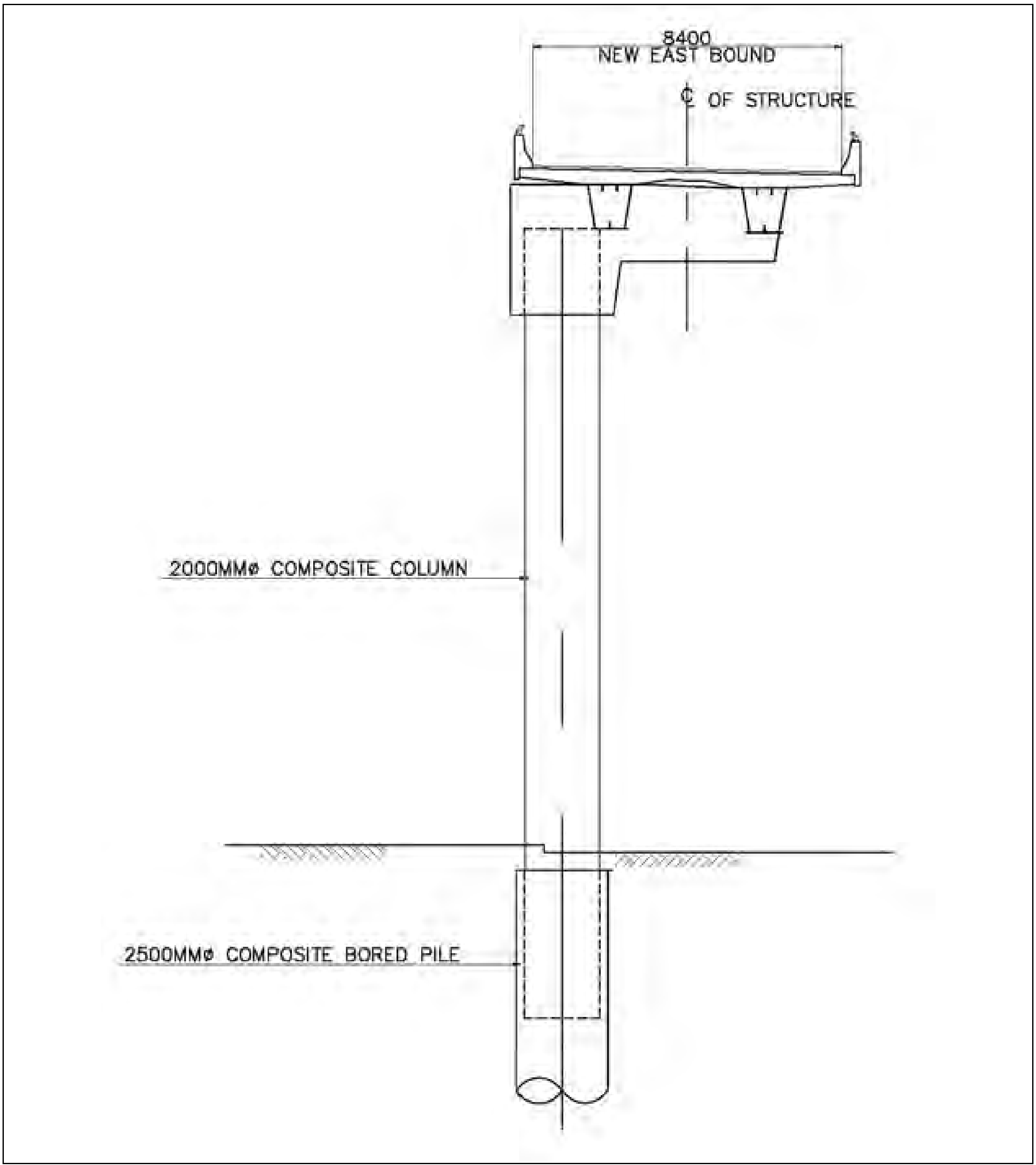


Figure 8.6-13 Section at Sta. 1+140: E2-2
(Ramsis Street)

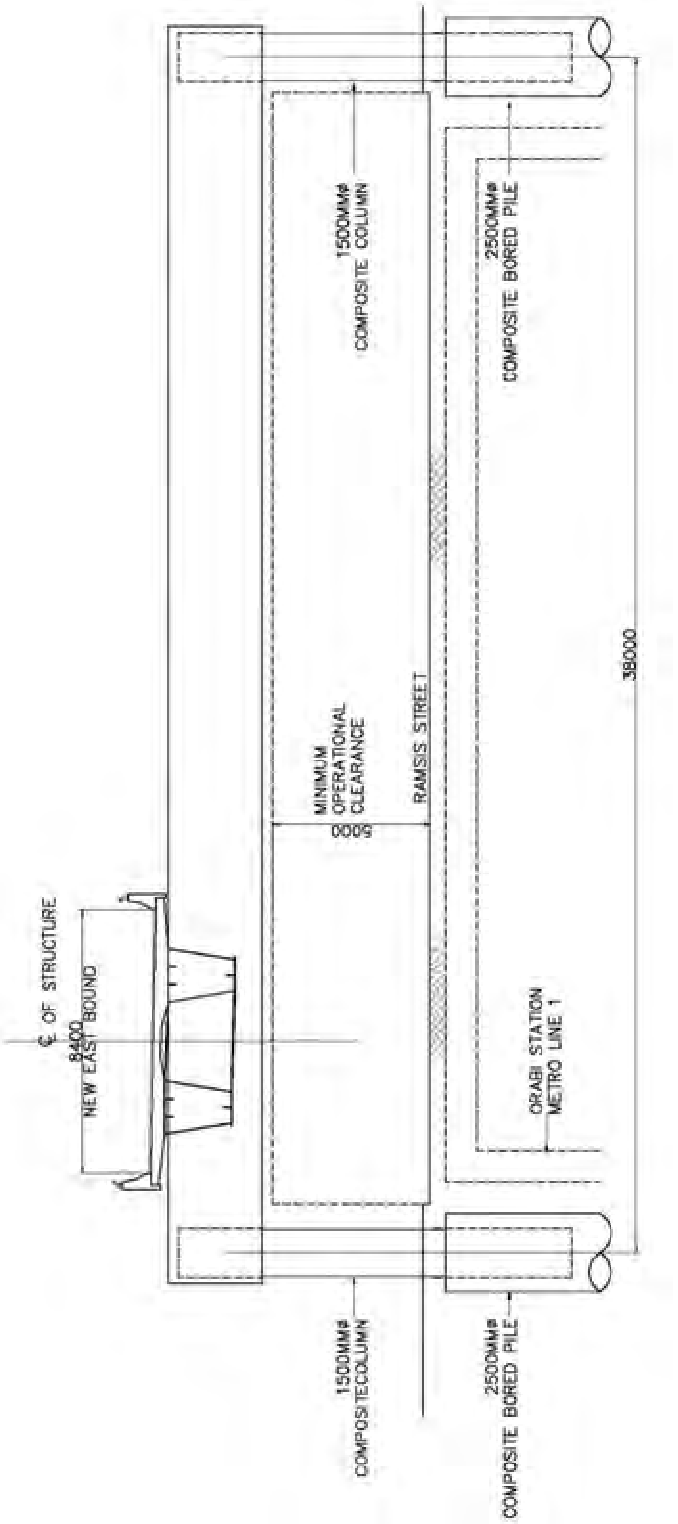


Figure 8.6-14 Section at Sta. 1+640: E2-2
(Ramsis Street)

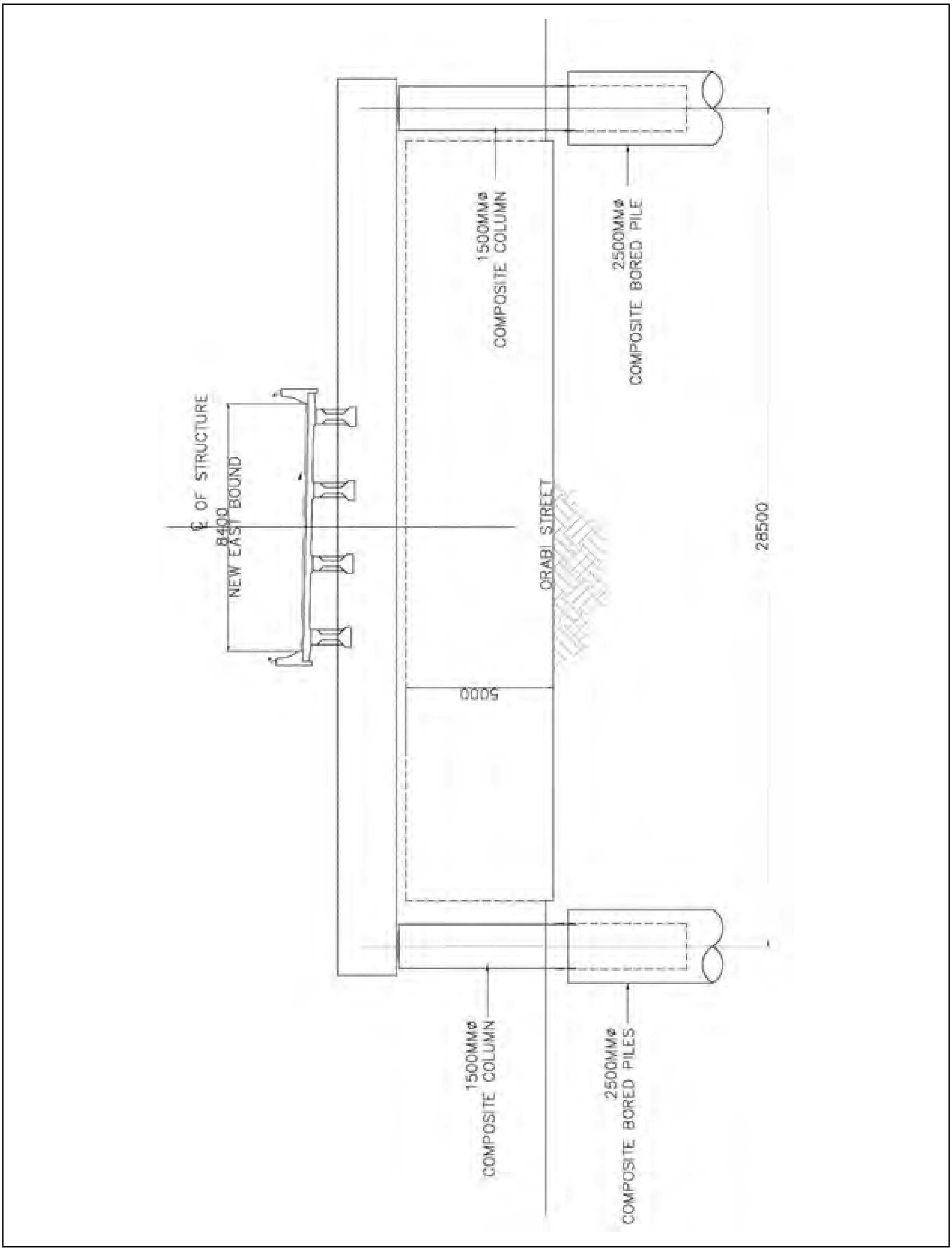


Figure 8.6-15 Section at Sta. 1+720: E2-2
(Orabi Street)

8.7 SUPERSTRUCTURE AND SUBSTRUCTURE TYPES: E3-1

8.7.1 General

Expressway section E3-1 on El-Nasr Road will be constructed in two stages in order to connect with the proposed E4/E6 components of the expressway system on the Suez Desert Road. The initial stage will commence on El-Nasr Road 700m from the intersection of E3-1 (El-Nasr Road) with E4/E6. The ultimate stage will complete the connection once the E4 /E6 sections of the expressway are constructed.

The initial stage of the expressway comprises the following structural components:

Westbound Mainline (to 6th October Bridge and E3-2)

- Elevated Structure (Viaduct) – total length 1.5km (including 80m of approach embankment): Sta. 0+ 700 to 2+200
- Depressed Structure (Cut & Cover Tunnel) – length 4.3km: Sta. 2+200 to Sta. 6+500

Westbound Onramp and Off ramp facilities

- Elevated On-Ramp (Ramp 1) – length 310m (including 135m of approach embankment): Sta. 1+210 to 1+520
- The Westbound depressed structure features single on and off ramps located at the right (north) side of El-Nasr Road (Ramp 2).

Eastbound Mainline (to Suez Desert Road)

- Depressed Structure (Cut & Cover Tunnel) – length 5.8km: Sta. 0+700 to 6+500

Eastbound Onramp and Off ramp facilities

- The Eastbound depressed structure features single on and off ramps located at the median of El-Nasr Road (Ramp 3), requiring separation of the Eastbound and Westbound depressed structures at Station 3+530 (3.53km from junction of El-Nasr Road with Suez Desert Road).

Connection to E1 (6TH October Bridge)

The Eastbound and Westbound depressed structures remain separated in order to pass each side of the existing centrally located ramps of 6th October bridge at the junction of El-Nasr Road with Ramsis Extension. At this location a connection is provided between E3-1 and 6TH October Bridge (E1) in the form of a 4-lane centrally located ramp to the depressed expressway.

The main expressway structures carry 3-lanes in each direction. The on and off ramps provide single lane access/egress.

Features along the route that have an impact on the preliminary structural design are:

- The tramline crossing El-Nasr Road at Sta. 4+650 – the depressed structure will pass underneath this operational double track
- The 1st level up-ramp from EL-Nasr Road to 6th October Bridge – this existing structure crosses over the line of the proposed Westbound depressed expressway near the junction of El-Nasr Road and Ramsis Extension and will influence the method of construction of the depressed structure

Features that may have any impact on the detailed structural design are:

- The proposed MoH road underpass along El-Fangary beneath El-Nasr Road at Sta. 5+930 – it will be required to push through the underpass itself beneath the expressway at depressed level (-2) and modification of the cut-and-cover design of the expressway
- Underground utilities crossing El-Nasr Road - these may need to be carried in service troughs or require some other modification to the top slab design of the depressed structure.

Refer to Figure 8.7-1 for a schematic plan showing the disposition of the E3-1 components and Figure 8.7-2 for a more detailed layout plan.

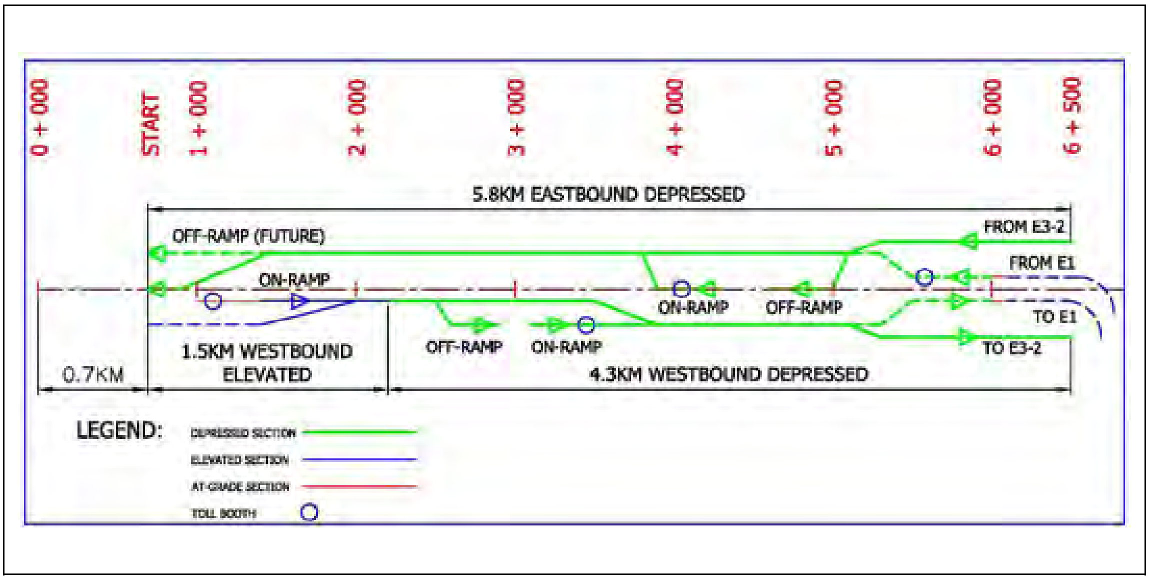


Figure 8.7-1 Schematic Plan of E3-1

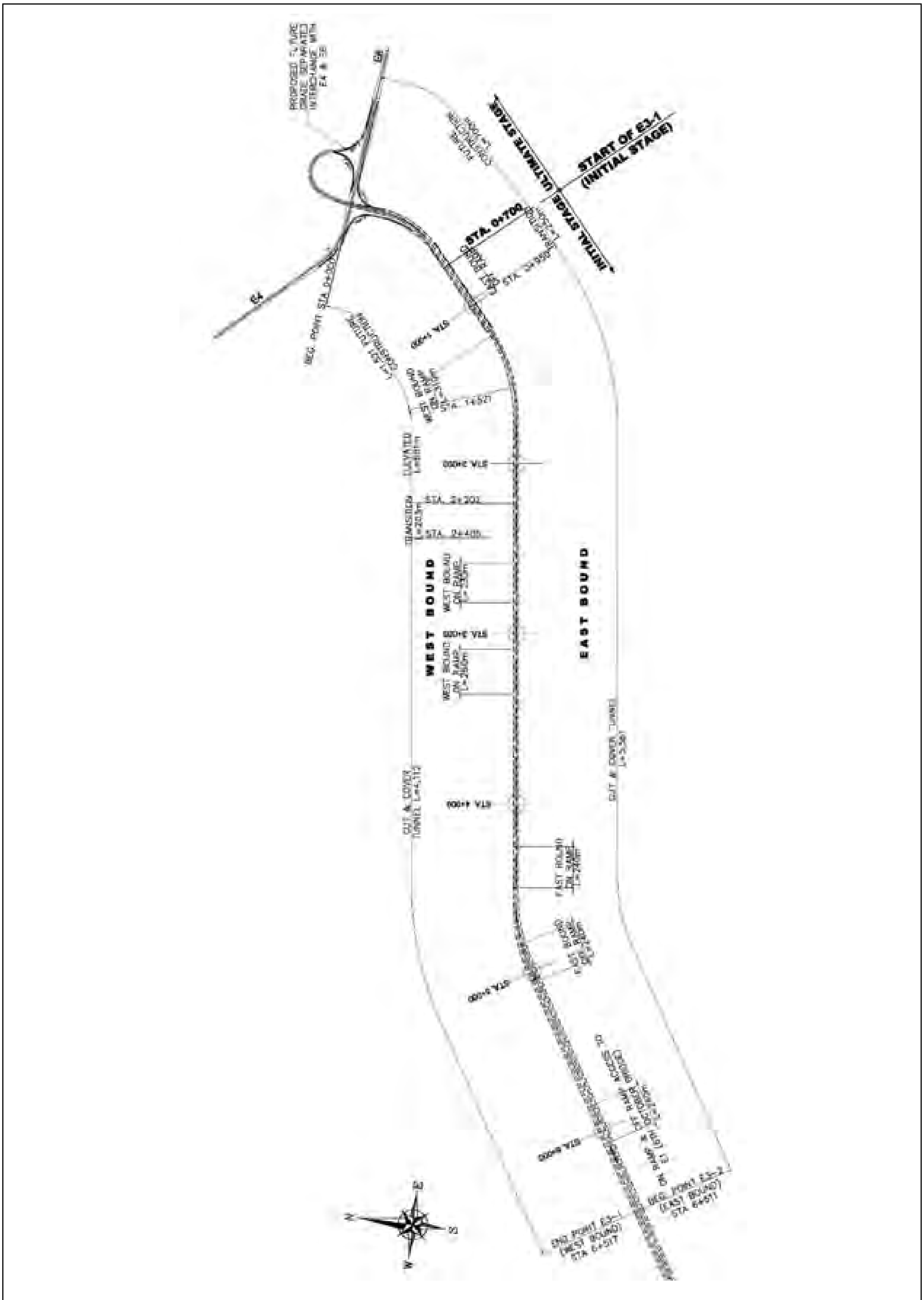


Figure 8.7-2 Detailed Plan of E3-1

8.7.2 Initial Stage and Ultimate Stage

The expressway is proposed to be constructed in two stages, (1) the initial stage and (2) the ultimate stage.

Initial Stage

The initial stage will comprise the major part of E3-1 and will form an operational expressway component prior to the construction of the proposed E4 and E6 expressway structures along the Suez Desert Road.

In order for the initial stage of the expressway to be made operational, the Westbound elevated structure need only constructing from the nosing point with the Westbound Elevated On-Ramp (Ramp 1) at Sta. 1+520. The elevated Westbound section from the start of the initial stage at Sta. 0+700, to the nosing at Sta. 1+520, need not be fully constructed at the initial stage. Only the foundations that are integral with the depressed structure require to be constructed in the initial stage.

Similarly the Eastbound depressed structure, at its eastern end, need only transition to the existing at-grade road at the initial stage; Eastbound traffic will simply exit onto the existing Nasr Road at Sta. 0+700.

Ultimate Stage

The ultimate stage will make a fully grade separated intersection with the future E4 and E6 expressway structures along Suez Desert Road.

The superstructure of the ultimate stage, and the substructure supporting it that is independent from the initial stage construction, do not need constructing at the initial stage as it will have no function at this stage; there will be no traffic running on the deck at the initial stage and the grade separated structures at the intersection with Suez Desert road will have no connecting structures to link with.

Refer to Figure 8.7-3 for a layout showing the extent of the Initial and Ultimate Stages of E3-1. Refer to Section 8.7.6 for a description of the structure types envisaged for the connection to E4/E6 on Suez Desert Road.

8.7.3 Subsoil Conditions

Refer to Figure 8.7-4 for a profile along El. Nasr Road giving subsoil conditions.

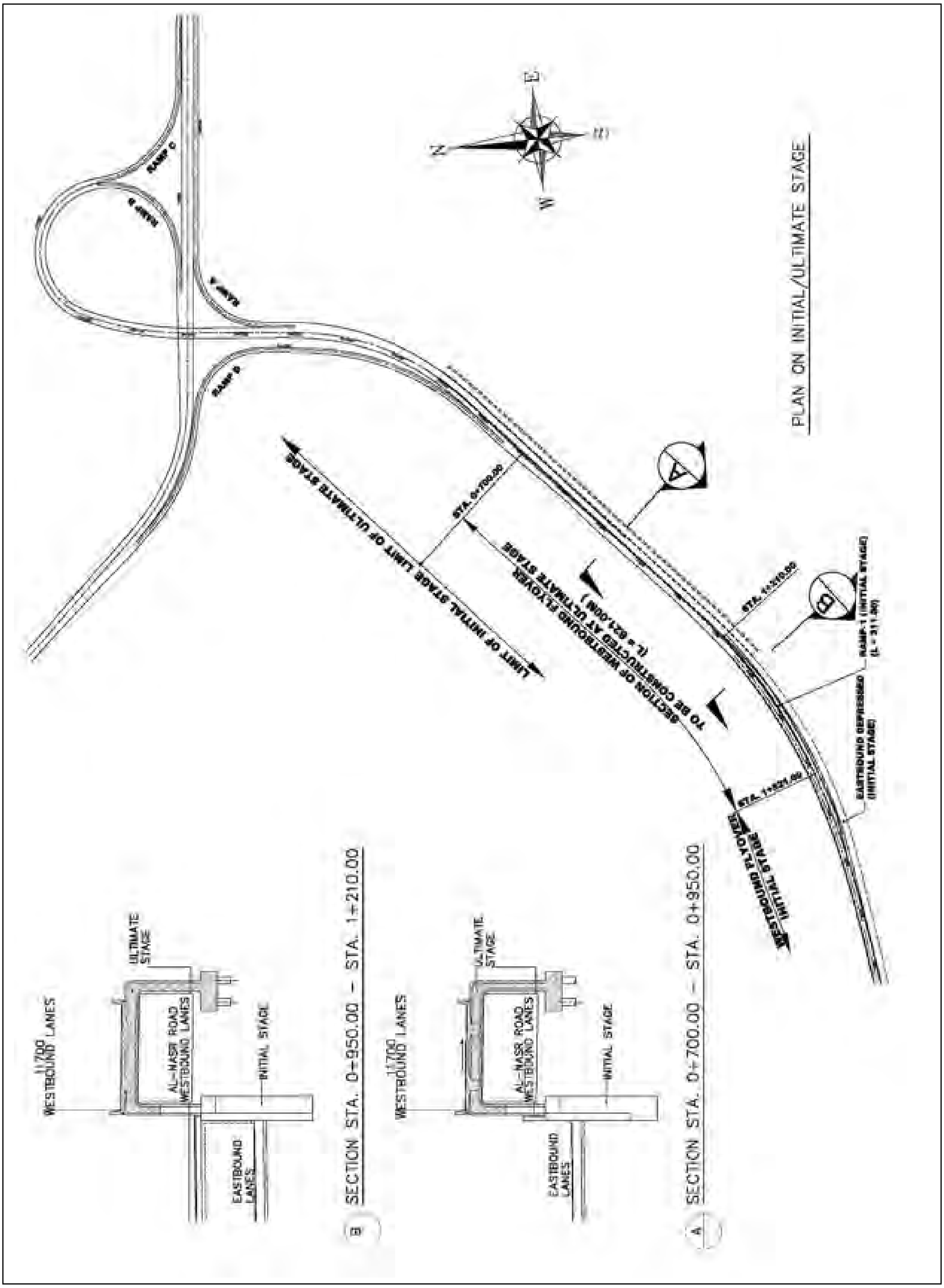


Figure 8.7-3 Layout of Initial and Ultimate Stages – E3-1

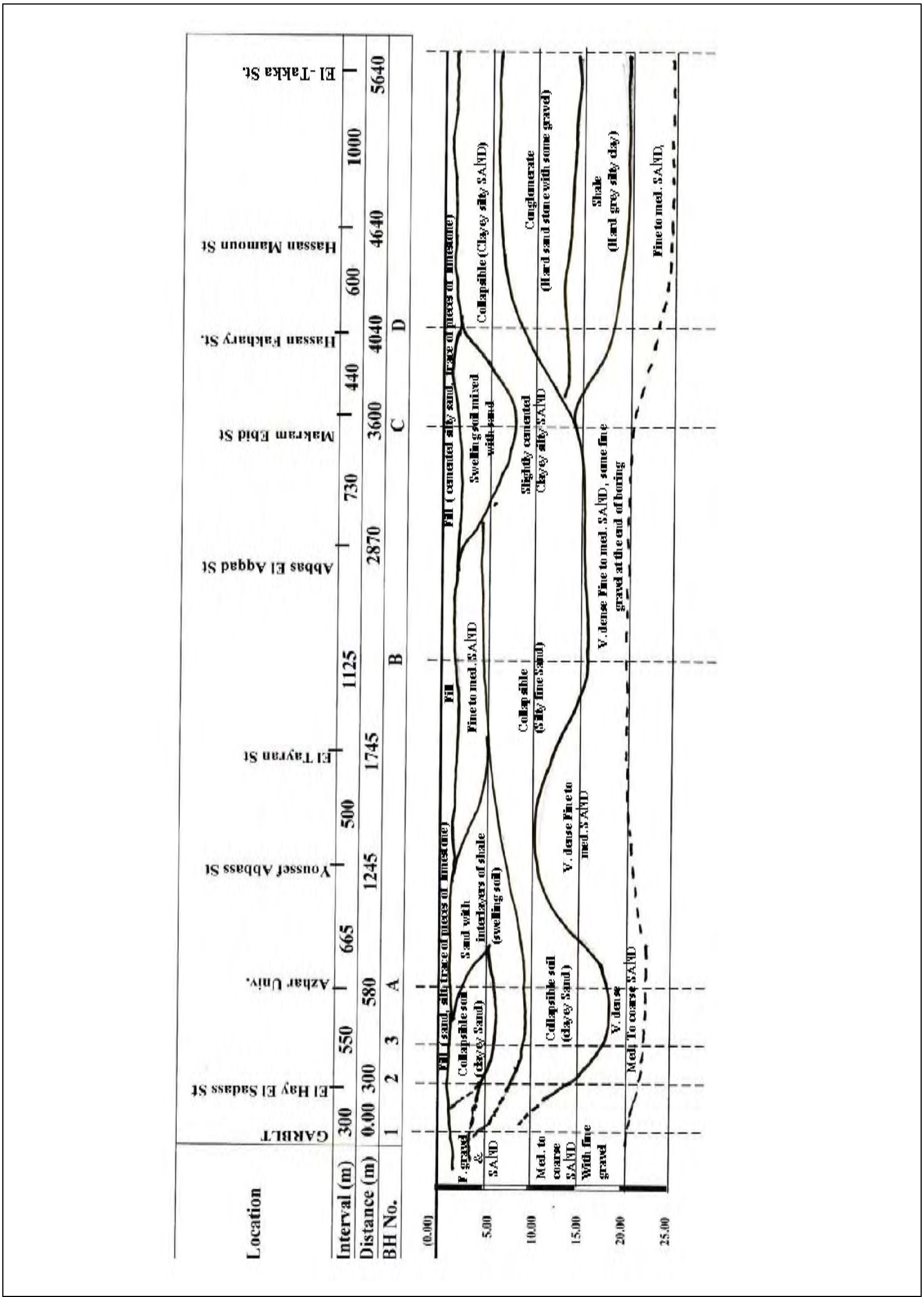


Figure 8.7-4 Soil Profile along El Nasr Road – E3-1

The soil profile shows a depth of collapsible clayey and silty sand down to a depth of typically 15m, with one section of length 300-400m where the collapsible soil extends to a depth of 18m. Below this horizon are stiff clay and dense sands to depth.

Piled foundations therefore should extend at least 5 pile diameters below this horizon to avoid major problems settlement and loss of capacity. The preliminary design will therefore adopt a minimum pile length of 20m, giving a depth to the pile toe of at least 23m.

With regard to the cut and cover tunnel sections, depth of diaphragm walls should extend below the collapsible soil horizon; however given that vertical bearing capacity from the wall base is not a crucial aspect of the design of the diaphragm walls, an embedment depth below the collapsible soil horizon of 3 times the wall thickness (i.e at least 2.4m) has been adopted. An overall depth of 18m has therefore been assumed to determine quantities. Depth of wall may be shallower or deeper than this over certain sections, however for the purpose of determining quantities this depth is considered acceptable.

The method of construction of the diaphragm walls (i.e. excavation under bentonite slurry) should be sufficient to support the excavation, with a build up of a bentonite cake in the sand face to prevent water penetration. However if soil conditions are found to be problematic for the excavation, advance soil stabilization works can be done, involving a curtain of consolidation grouting along the line of the wall. This will have to be investigated during detailed design and with construction trials before the main works commence if necessary.

8.7.4 E3-1 Superstructure Types

(1) General

The proposed Westbound elevated structure constructed in the initial stage, including the on-ramp, will be located either (i) above the existing wide median on El-Nasr Road, (ii) in the proposed future median or (iii) above existing central lanes reserved for U-turn traffic movements.

There are therefore no locations where long span structures or prefabricated construction requiring structural steel are required. The study on superstructure types has therefore not included steel girder decks, given that steel decks will have a higher construction cost.

The study on superstructure types is made for a span length of 25m. This is a typical span length for flyover and viaduct structures found in Cairo and will provide a good basis for cost estimation purposes. A detailed span length study will be undertaken in the detailed design in order to optimize the structure layout in accordance with encountered soil conditions and the required at-grade channelization.

The study on superstructure types has included the following forms of construction:

- Type 1 Concrete beam and slab with pre-cast post-tensioned beams
- Type 2 In-situ reinforced concrete box
- Type 3 In-situ pre-stressed concrete voided slab
- Type 4 In-situ pre-stressed concrete spine girder

(2) Description of Superstructure Types

The superstructure types included in the study are briefly described below. Refer to Figure 8.7-5 for illustrations of each type.

- Type 1 - Concrete beam and slab with pre-cast post-tensioned beams

  The concrete beam and slab deck, with pre-cast post-tensioned beams, is a type found at several locations in Cairo (including sections of 6th October Bridge and the Autostrad flyover). The pre-cast beams may either follow a standard design, such as AASHTO girder types, or may be designed case by case which is the usual practice in Egypt. Beams are spaced at up to 2.5m centers and for a span of 25m a typical deck depth is 1.7 m, including slab. The beams receive an in-situ concrete top slab and require transverse diaphragms to distribute live load. The deck is not torsionally stiff and the straight pre-cast beams are not well suited to highly curved bridges. The principal advantage of this form of construction is that the beams can be cast on simple casting beds without the need for major shoring works. The beams can be cast and post-tensioned in advance and lifted in by crane, during night possessions if necessary, to promote fast construction with a minimum impact on traffic at grade. This form of deck is also advantageous in construction over railway lines, where in-situ construction is not possible. The major disadvantage of this form of deck is the need to provide bearings and expansion joints at each pier and the very utilitarian appearance of the deck. The girders can be made monolithic with the piers, thereby negating the need for bearings and expansion joints; however this requires a flexible pier design, requires the girders to be landed fully on top of the pier coping without “dapped” ends and involves an additional stage of concrete pouring during construction. Alternatively the expansion joints can be replaced with “flexible slabs” that not only remove the joint, and the future maintenance obligations, but also improve ride quality over the pier.

- Type 2 - In-situ reinforced concrete box

  The cast in-situ reinforced concrete box type deck is widely used in Cairo. The box section gives a highly torsionally stiff deck well suited to curved ramps, the deck does not require transverse diaphragms for distribution of traffic loads, and the form is well suited to varying deck widths. The deck typically comprises multiple cells with top slab thickness of 250 mm and bottom slab 200 mm. Webs can vary in thickness from 400 cm to 600 mm and are spaced at 3m to 4m centers. A 25m span bridge will typically have a 2.0m deck depth. The deck is built on shoring and requires at least two construction stages in order to complete; the base slab concrete is poured first, void forms are then fixed and the webs and top slab can then be poured. The decks are constructed to be continuous over typically three or four spans. In Cairo the decks are also typically constructed monolithic with wall type piers; at expansion joints the piers are constructed as a split pair with each side constructed monolithically with the deck. Access holes are formed in the bottom slabs in each span to facilitate access to the internal voids for maintenance.

- Type 3 - In-situ pre-stressed concrete voided slab

  The cast in-situ post-tensioned concrete voided slab deck is highly suited to locations where deck width is varying (such as at ramp nosing points) and where span-to-depth ratio must be maximized. The deck slab is formed with cylindrical voids held in place with proprietary tie-downs. The void formers can either be made from expanded polystyrene or from winding pipe. The “webs” formed between the voids are typically 300 mm to 500 mm wide and can accommodate groups of tendons draped across the deck. A 25 m span bridge will typically have a 1.25 m deck depth in span, increasing to 1.5 m at the pier supports. The deck is built on shoring and typically requires only one construction stage (concreting + prestressing) to complete. The decks are constructed to be continuous over typically two or three spans with pre-stressing tendons jacked from both ends. The slabs can be made monolithic with the piers, thereby negating the need for bearings and expansion joints.

- Type 4 - In-situ pre-stressed concrete spine girder

  The cast in-situ pre-stressed concrete spine girder provides an elegant and efficient form of deck with the structure offering a clean and attractive visual impact in urban settings. The spine girders are torsionally stiff and can be adopted to curved bridges. The deck does not require transverse diaphragms for distribution of traffic loads, and the form can accommodate varying deck widths. The spine girders are spaced at centers of 6.0 m or so and accommodate groups of pre-stressing tendons draped along the deck. The deck slab is typically 300 mm in thickness between the spine girders and is pre-stressed transversely. A 25 m span bridge will typically have a 1.5 m constant deck depth with a horizontal thickening of the spine beams at the supports. The deck is built on shoring and requires only one construction stage in order to complete (concreting + prestressing). The dimensions of the spine girders allow the use of couplers on the longitudinal pre-stressing tendons. The decks can therefore be constructed progressively span by span, from contra-flexure point to contra-flexure point, and can be made continuous over multiple spans. The decks are also typically constructed monolithic with the piers.

(3) Comparative Study

The comparative study is presented in Table 8.7-1. All of the above deck types can be considered for construction. Both Type 1 and Type 2 are common forms of construction in Cairo and the remaining types have been used for viaducts and flyovers in similar busy urban settings in other countries. The central location of the structure above the median and the width of El-Nasr Road will allow the use of shoring, required for in-situ decks, without a substantial impact on at-grade traffic during construction. In-situ forms will allow the integration of pier columns with the deck, thereby minimizing the use of bearings and expansion joints. Where the focus is on construction with a reduced future maintenance demand, which will be the case for an expressway with a planned total length in excess of 100km, forms of construction requiring expansion joints and bearings at each support, as required for Type 1, cannot be recommended.

The final choice of deck will be made during the detailed design when all necessary data and surveys have been undertaken. For the purposes of establishing quantities for the cost estimate in this Study, Type 4 has been selected to illustrate the preliminary design.

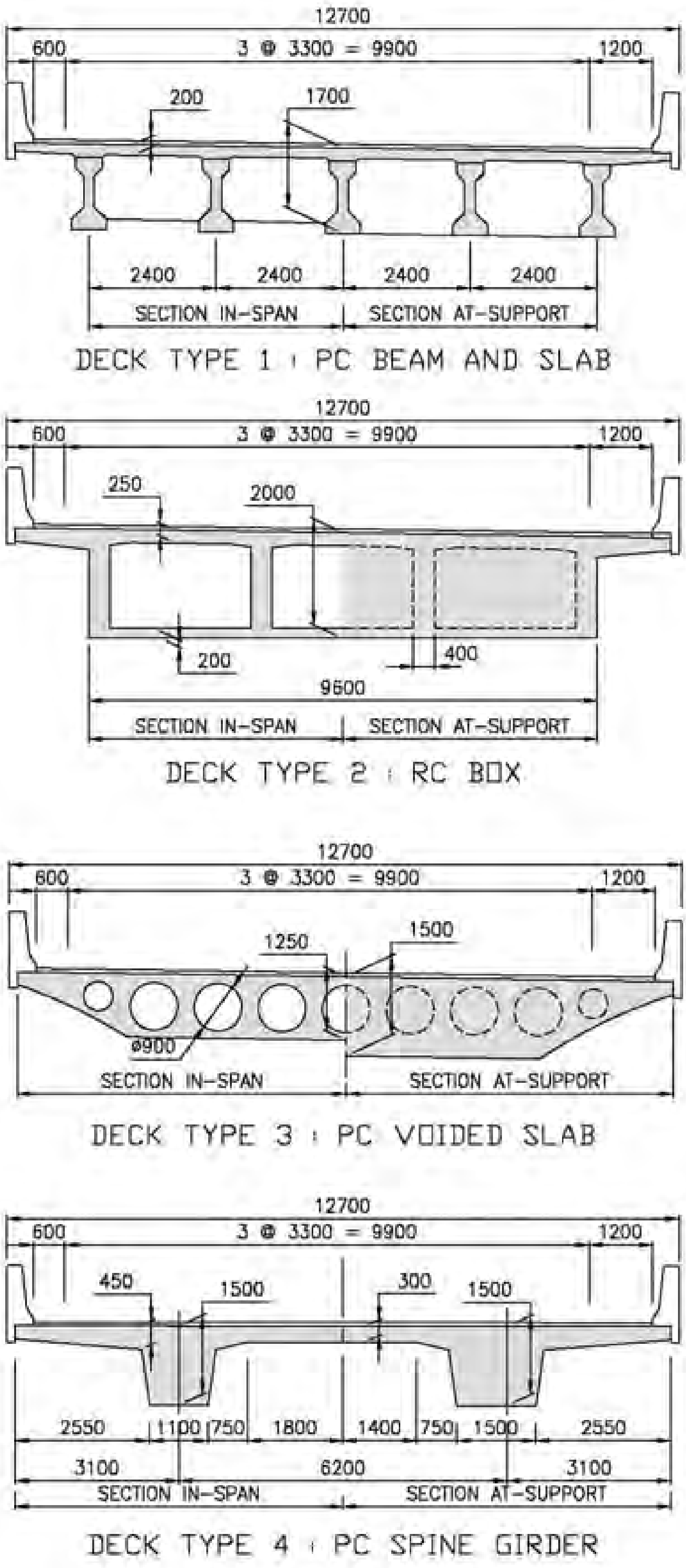
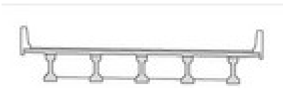
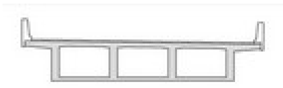
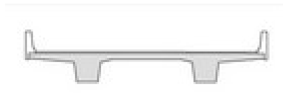


Figure 8.7-5 Deck Types for Superstructure Study: E3-1

Table 8.7-1 Comparison of Deck Types for Superstructure Study: E3-1

Deck Type	 **Type 1 – PC Beam and Slab**	 **Type 2 – RC Box**	 **Type 3 – PC Voided Slab**	 **Type 4 – PC Spine Girder**
Construction Cost	More economical than Type 3 & Type 4	Most economical	Least economical	Reasonably economical. Efficient construction offsets pre-stressing costs.
Constructability	Girders cast in advance. Shoring not required for girders. Easy to construct span by span. PC cables imported.	Shoring required. Two stage construction. Cannot be constructed span by span. All materials available locally.	Shoring required. Single stage construction using proprietary tie-downs for void forms. Variable deck depth required. Cannot be constructed span by span. PC cables imported.	Shoring required. Single stage construction. Span by span construction possible. Requires transverse pre-stress.. PC cables imported
Construction Time	Shortest construction period.	Longest construction period.	Longer construction period than Type 4.	Shorter construction period than Type 2 & Type 3.
Structural Aspect	Cannot easily be made monolithic with piers. Typically requires bearings and joints. Requires diaphragms to distribute loads transversely. Not suitable for highly curved alignments	Can be made monolithic with piers. Torsionally stiff without diaphragms. Suitable for curved alignment.	Heaviest deck with impact on foundation design /costs. Can be made monolithic with piers. Torsionally stiff without diaphragms. Suitable for curved alignment.	Can be made monolithic with piers. Torsionally stiff without diaphragms. Suitable for curved alignment.
Maintenance	Bearings and expansion joints give large maintenance obligation.	Least maintenance obligation if deck made monolithic with piers.	Lower maintenance obligation if deck made monolithic with piers.	Lower maintenance obligation if deck made monolithic with piers.
Aesthetics	Very poor utilitarian visual impact.	More attractive than Type 1 but still rather uninteresting visually.	More enhanced visual impact with sculpted form	Best aesthetic impact with highly sculpted appearance.
Remarks	Least recommendable.	Recommendable	Less recommendable.	Recommendable

8.7.5 E3-1 Substructure Types

(1) Pier Type

With regard to seismic design, Egyptian bridge design standards require that adequate structural details shall be provided to ensure appropriate ductility for connections and end of piers and girders. The philosophy of the design is therefore close to that given in AASHTO for Seismic Zone 2, although no explicit references to requirements for transverse confinement reinforcement in pier columns have been found.

The design requirements do not require that foundations are to be designed for plastic hinging effects in the pier columns. The columns can therefore take several forms, including the more rigid wall type structures, without undue impact on foundation design.

Pier forms considered in the study on substructure type therefore are:

- Wall Type
- Single Column
- Twin Column
- Architectural

The pier forms are illustrated in Figure 8.7-6 and a simple comparison is presented in Table 8.7-2.

The final choice of pier column will be made during the detailed design. For the purposes of establishing quantities for the cost estimation in this Study, the architectural pier has been selected to illustrate the preliminary design.

(2) Foundation Type

Bored pile foundations have been selected as the foundation type to be applied to the preliminary design. Selection of number and diameter of bored pile is a function of soil conditions, the arrangement of the pile cap required to support the pier columns and design demand from the columns.

Larger diameter bored piles offer the advantage of reducing the number of piles required at each pile cap, with corresponding advantages in speed of construction, but require larger boring rigs to be mobilized. Smaller diameter bored piles allow for the use of smaller piling rigs but require more construction time to install the larger number of piles required.

The final choice of bored pile diameter will be made during the detailed design. For the purposes of establishing quantities for the cost estimate in this Study, 800 mm diameter bored piles have been selected to illustrate the preliminary design.

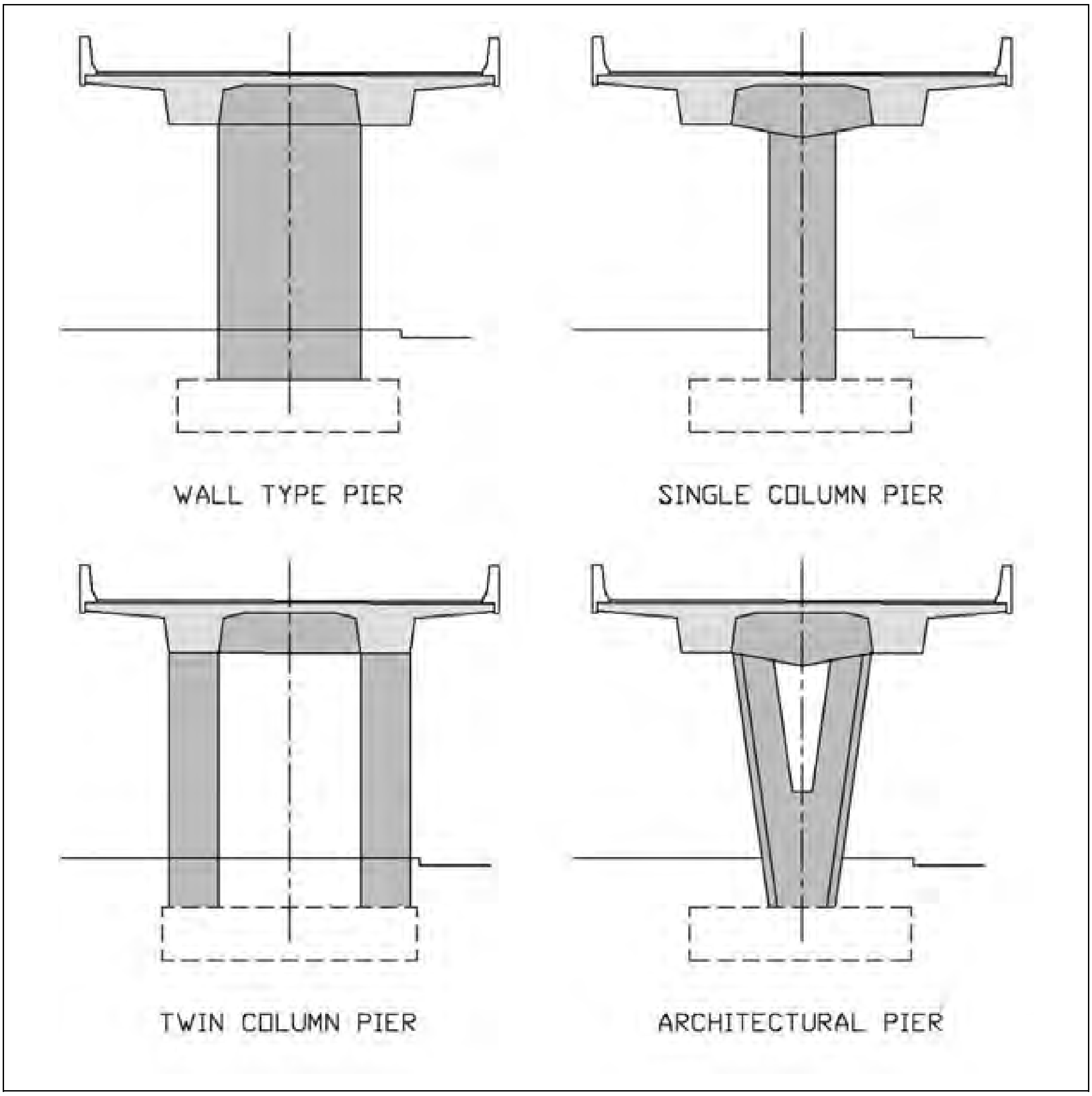


Figure 8.7-6 Pier Types for Substructure Study: E3-1

Table 8.7-2 Comparison of Pier Types for Superstructure Study: E3-1

Pier Type	Advantages	Disadvantages
Wall Type	• Intrinsically stable • Simple to construct • Lowest density of reinforcement	• Poor visual impact • Provides reduced space beneath deck for at-grade roads
Single Column	• Simple to construct • Provides greatest space beneath deck for at-grade roads	• Highest density of reinforcement
Twin Column	• Intrinsically stable • Relatively simple to construct	• Provides least space beneath deck for at-grade roads
Architectural	• Best aesthetic impact • Reinforcement density less than single column pier • Provides space beneath deck for at-grade roads	• Most complicated to construct

8.7.6 Depressed Structure Form

(1) General Considerations

El-Nasr Road is a major traffic thoroughfare. Construction of a depressed expressway for a length of 5.8 km along El-Nasr Road will require careful planning and proper attention to traffic management.

In order to maintain traffic along El-Nasr Road “top down” cut-and-cover construction will be required for the depressed structure. In this form of construction, the necessary support for the excavation is put in place first, either as temporary works or as part of the permanent work, working in stages with a minimum number of traffic lanes closed or traffic diverted onto temporary lanes occupying the road median. The top slab of the tunnel section can then be constructed in stages, with sufficient traffic lanes maintained in operation and maximum use made of night-time possession of road space. Once the top slab is in place, traffic can be diverted as necessary to use the at-grade lanes above the tunnel works; the main excavation works can then proceed from below and the remaining structural works completed from inside the excavated space.

(2) Types of Cut-and-Cover Construction

The following types of cut-and-cover construction can be considered:

- Bored pile curtain wall construction

- Diaphragm wall construction

Both types of construction have been used in Cairo and are briefly described below. Refer to Figure 8.7-7 for illustrations of each type.

- Bored pile curtain wall construction

  This type of construction is common in the region and is proposed for the MoH El-Fangary Tunnel. Contiguous bored pile walls are constructed first and, after the concrete piles have gained sufficient strength, the top slab is then constructed on a prepared excavated surface. Once the top slab is sufficiently strong, the main tunnel excavation can then be made and the tunnel section constructed. The tunnel section comprises a reinforced concrete box, structurally connected to the bored pile walls. The size of the bored piles depends on soil conditions and excavated height; 1200mm piles are specified for Fangary Tunnel. If soil conditions allow intermittent bentonite piles can be used alternatively with bored piles. The advantage of the method is that it makes use of standard construction techniques and procedures without the need for the specialist diaphragm wall grabs. A disadvantage of the method is that it is relatively slow and requires additional construction phases than the diaphragm wall method. The method is therefore more suited to relatively short tunnel sections and where fast construction is not a main priority.

- Diaphragm wall construction

  This type of construction is appropriate for longer tunnel sections where speed of construction is a priority. The method makes use of specialist grabs that can excavate, under bentonite, sections of the diaphragm wall in panels; grabs lengths are typically 2.5m to 2.7m and wall sections typically from 60 cm to 1.0 m can be excavated, although wall thickness of up 1.5 m can also be made. Two methods are used to progress the panels, (i) alternative construction with primary panels excavated and concreted first followed by construction of secondary panels and (ii) intermittent panels constructed between temporary tubular steel stop-ends that can then be removed to allow the remaining wall section to be excavated and concreted. The first method has just recently been used on the South Dashour Bridge Project for the main bridge pier foundations in the Nile and requires the use of specialist grabs with counter-rotating cutter wheels given that the secondary panels cut back part of the concrete in the primary panels. The second method can make use of conventional cable operated grabs, allows longer panel lengths to be excavated (with wider reinforcing cages) and provides more flexibility in the planning of the wall excavation. The advantage of the diaphragm wall technique, irrespective of method of construction, is that the diaphragm wall requires no additional construction works for the vertical faces of the tunnel; the excavated face of the wall requires either only facia panels to be mounted or a shotcrete finish. For this reason the method was used in Cairo to construct the underpass on El-Urubah at El-Marghani, a location near the President's residence where speed of construction was a major priority, and the approach works on Salah Salem Road for the El-Azhar Tunnels.

Given that 5.8 km of depressed expressway structure is to be constructed, the bored pile curtain wall type method is considered too slow to be recommended for the main tunnel works. For the purposes of establishing quantities for the cost estimate in this Study, diaphragm walls have therefore been selected to illustrate the preliminary design.

Where sections of the expressway cross locations where diaphragm wall construction may not be possible, such as the tramline crossing and at locations where headroom is restricted, the bored pile curtain wall technique may be adopted.

(3) Top slab construction

The top slab for the depressed structure will be constructed prior to the main tunnel excavation, thereby avoiding the use of shoring works and minimizing impact on at-grade traffic lanes during construction. There are several methods of construction that can be used for the top slab including:

- Reinforced concrete solid or voided slab, cast in-situ
- Pre-stressed concrete voided slab, cast in-situ
- Precast pre-stressed planks with or without in-situ reinforced concrete topping

Reinforced concrete slabs are applicable for relatively small spans. Given that, at nosing points for on/off ramps of the proposed depressed expressway, the span of the top slab will be in excess of 25m, reinforced concrete slabs are not applicable.

The study on the top slab is therefore limited to pre-stressed concrete construction. Refer to Figure 8.7-8 for illustrations of both cast-insitu and pre-cast pre-stressed concrete slab construction.

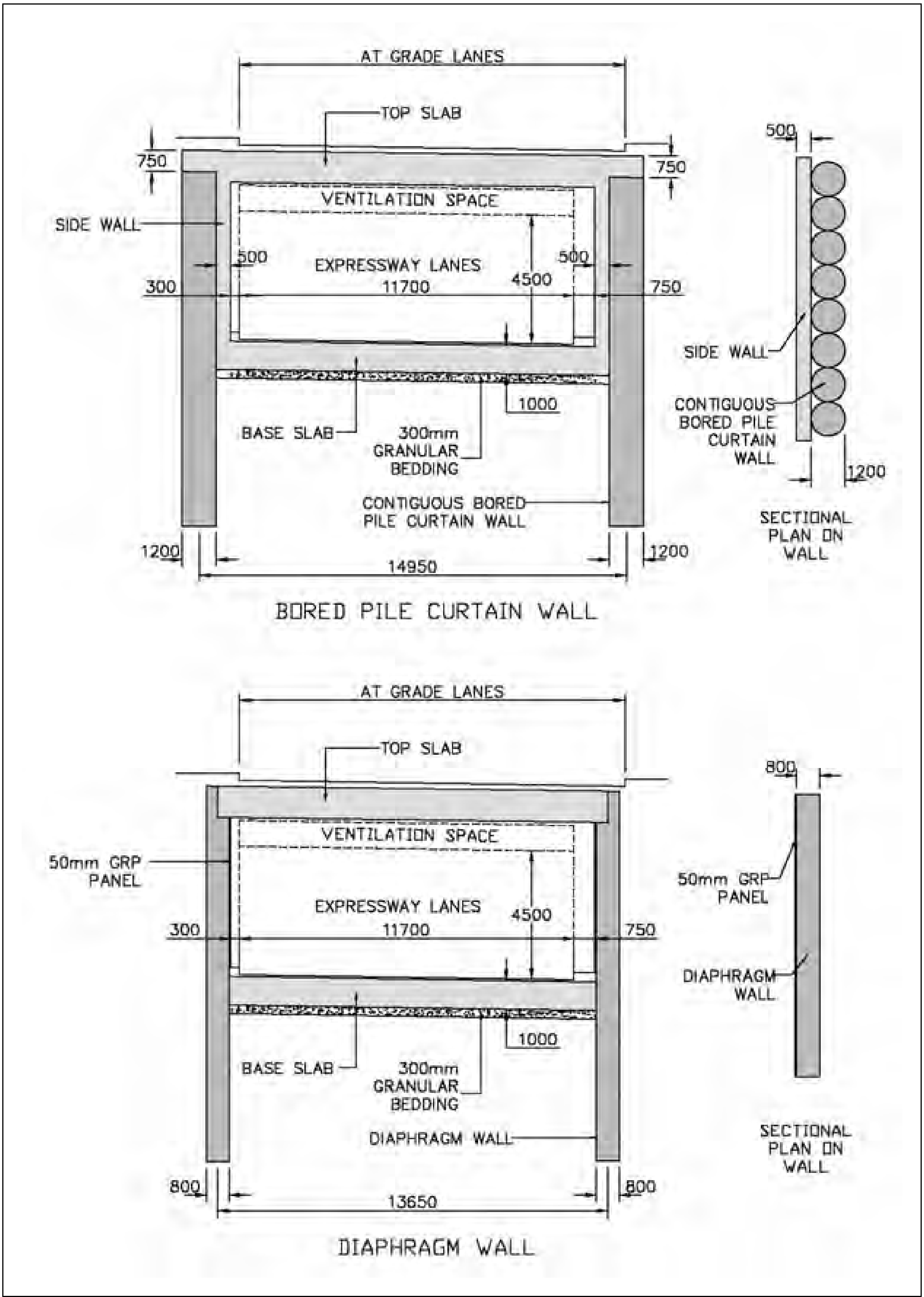


Figure 8.7-7 Cut-and-Cover Tunnel Construction: E3-1

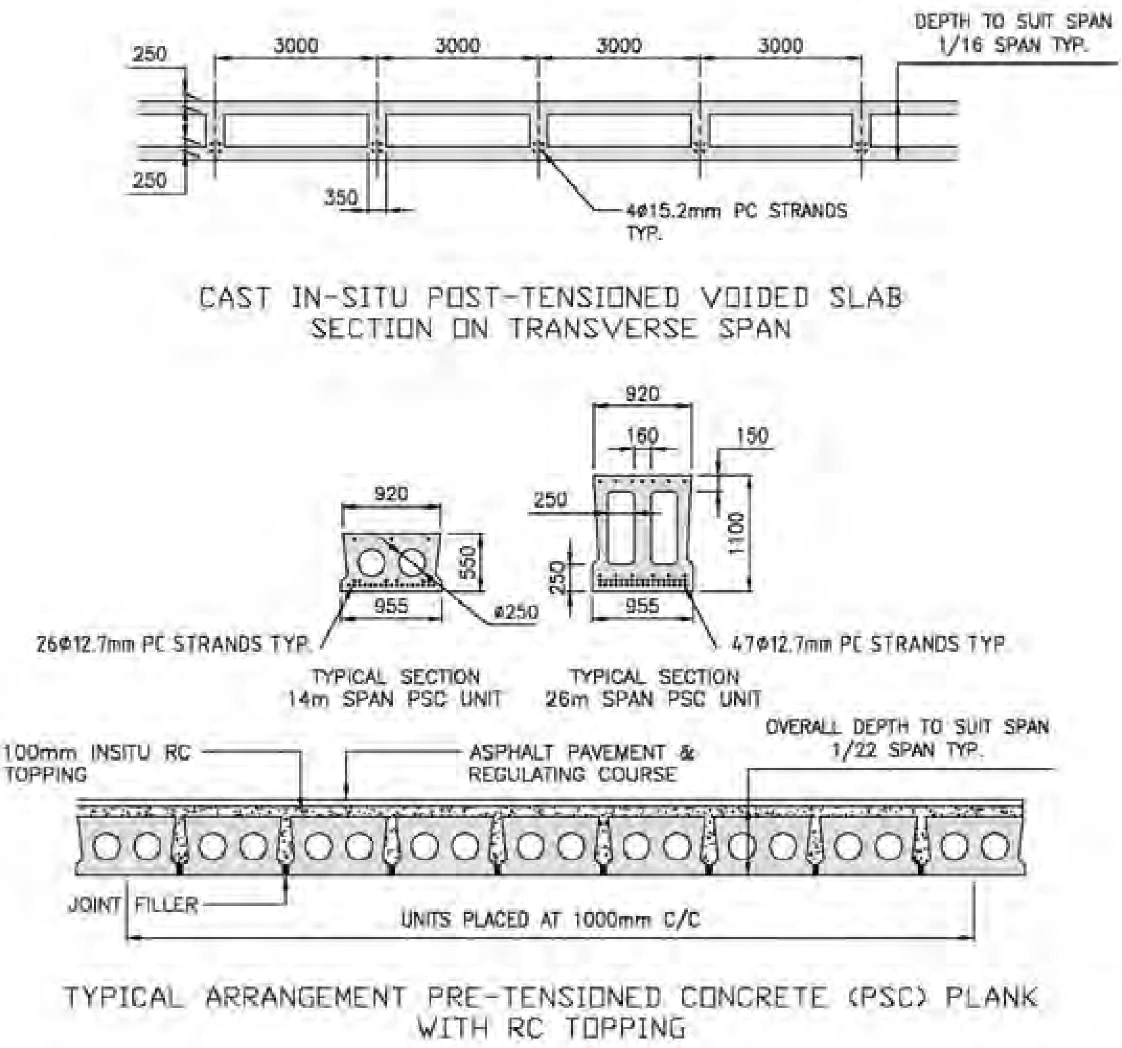


Figure 8.7-8 Top Slab Construction, Cut-and-Cover Tunnel: E3-1

In-situ slabs will require relatively large spaces on El-Nasr Road to be occupied for long periods for the construction. In addition, the longer spans will require to be supported on bearings in order to accommodate the large dead load rotations that occur when the slab support is removed. The pre-cast planks can be pre-cast in advance and will require less time for erection and placement of concrete topping than in-situ construction. The segmental nature of the pre-cast construction will also afford additional flexibility in managing excavation works and consequently traffic management during construction. In addition the pre-cast planks can be landed directly onto prepared mortar beds and do not require bearings.

For the purposes of establishing quantities for the cost estimate in this Study, pre-cast planks therefore have been selected to illustrate the preliminary design.

8.7.7 Integration with Expressway E4/E6 on Suez Desert Road

The preliminary design has established a trumpet type grade separated interchange between expressway E3 and the proposed E4 expressway along Suez Desert Road.

It is understood that the proposed E4 expressway will transition from elevated structure on the east side of the interchange with E3 to depressed structure on the west side of the interchange, given the restrictions imposed by the adjacent airport landing strip.

The following structures types will be therefore required on the respective legs of the interchange:

- E3 mainline passing over Suez Desert Road:
  - ✧ 4-Lane single deck viaduct structure carrying E3 traffic on 2-lanes in each direction over the existing Suez Desert Road
  - ✧ 2+2 Lane split level structures onto ramps (ramps B & C) connecting to the westbound lanes of the proposed E4
  - ✧ Length of structure is approximately 530m, from intersection of E3 with Suez Desert Road to bifurcation point of ramps B & C.
- Ramp A (E3 eastbound to E4 eastbound – located south side of Suez Desert Road)
  - ✧ 2-Lane viaduct structure : length 750m
- Ramp B (E3 eastbound to E4 westbound – located north side of Suez Desert Road)
  - ✧ 2-Lane depressed structure: length 290m from bifurcation point.
- Ramp C (E4 westbound to E3 westbound – located north side of Suez Desert Road)
  - ✧ 2-Lane viaduct structure: length 420m from bifurcation point.
- Ramp D (E4 eastbound to E3 westbound – located south side of Suez Desert Road)
  - ✧ 2-Lane depressed structure connecting from depressed section of E4: length of depressed structure 500m .
  - ✧ 2-lane transition structure to connect with elevated E3 comprising retained embankment and viaduct : length of embankment 100m, length of viaduct 370m

Refer to Figure 8.7-9 for a schematic plan layout at connection with E4 on Suez Desert Road.

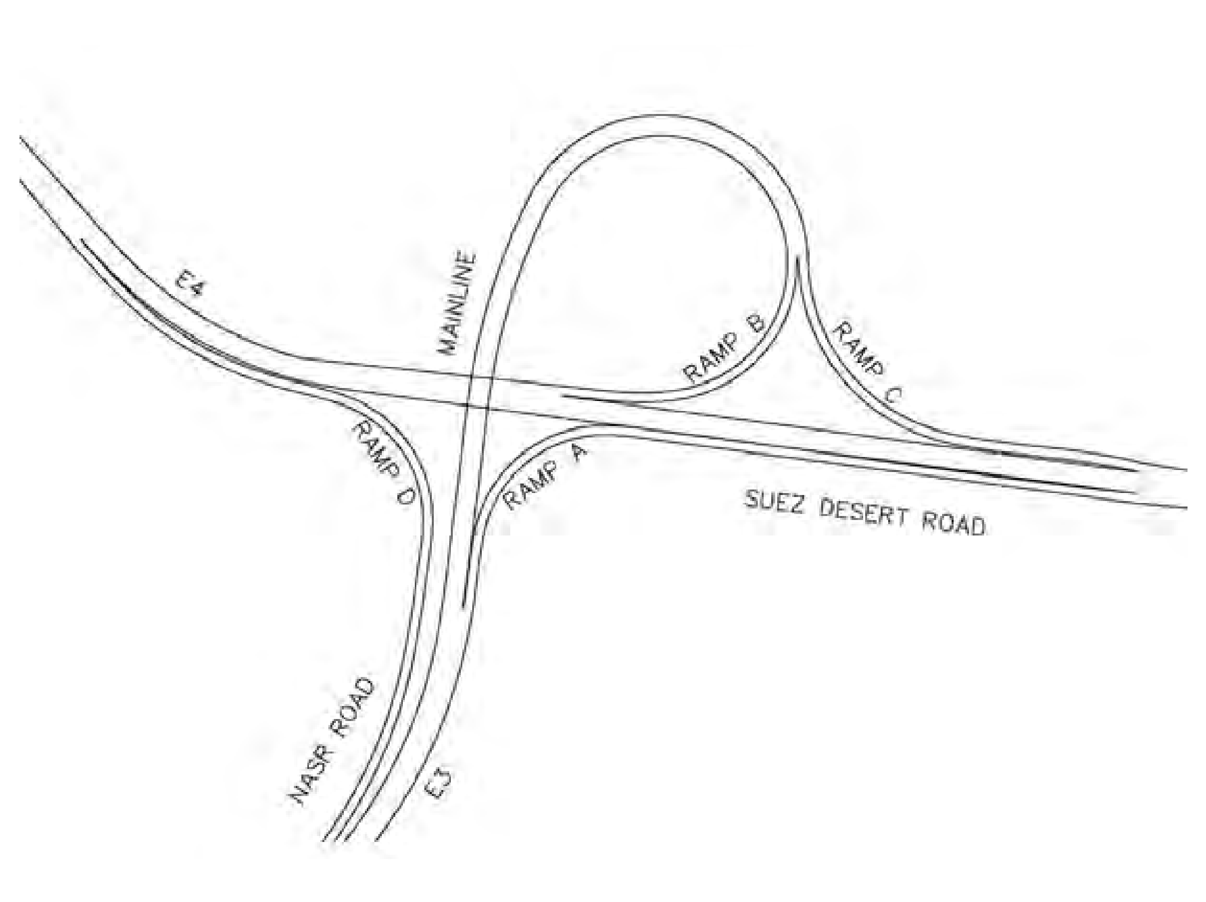


Figure 8.7-9 Schematic Plan Layout at Connection with E4 on Suez Desert Road: E3-1

8.7.8 Typical Sections

Typical sections illustrating the preliminary design are presented in Figure 8.7-10 to Figure 8.7-15.

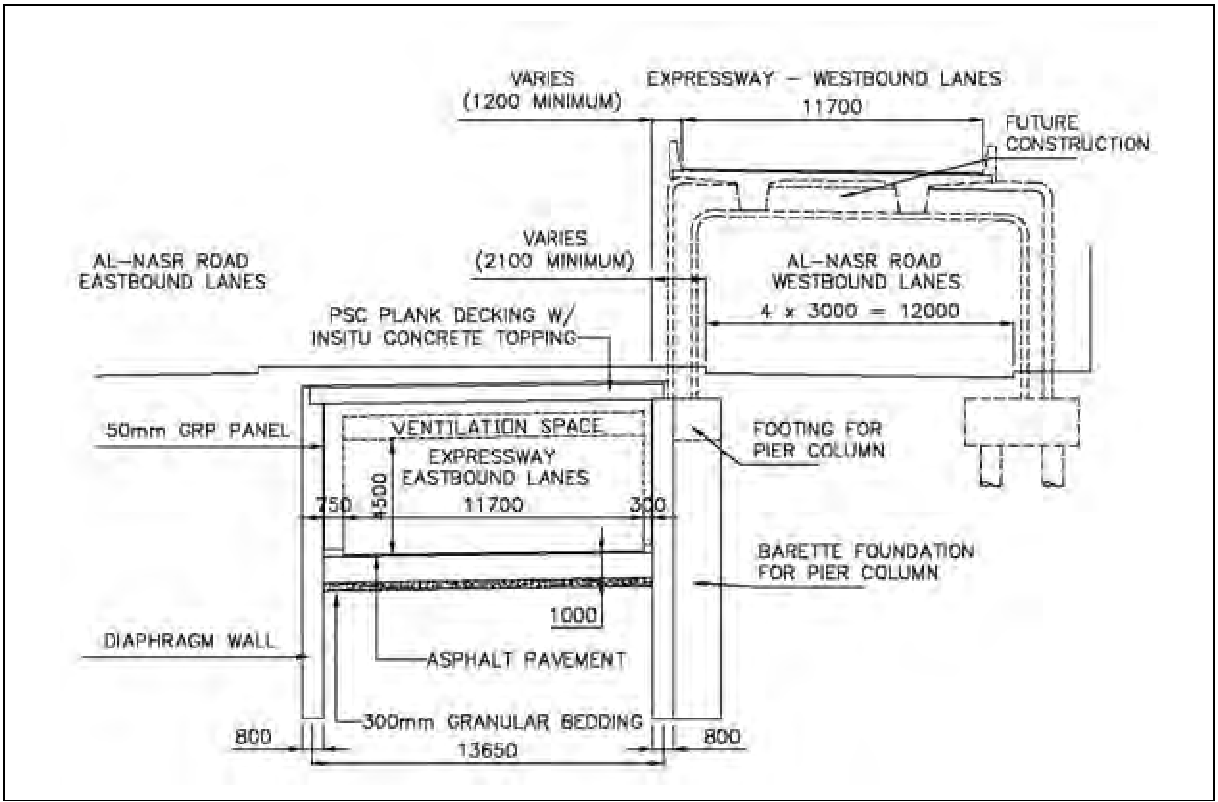


Figure 8.7-10 Section at Sta. 1+000: E3-1

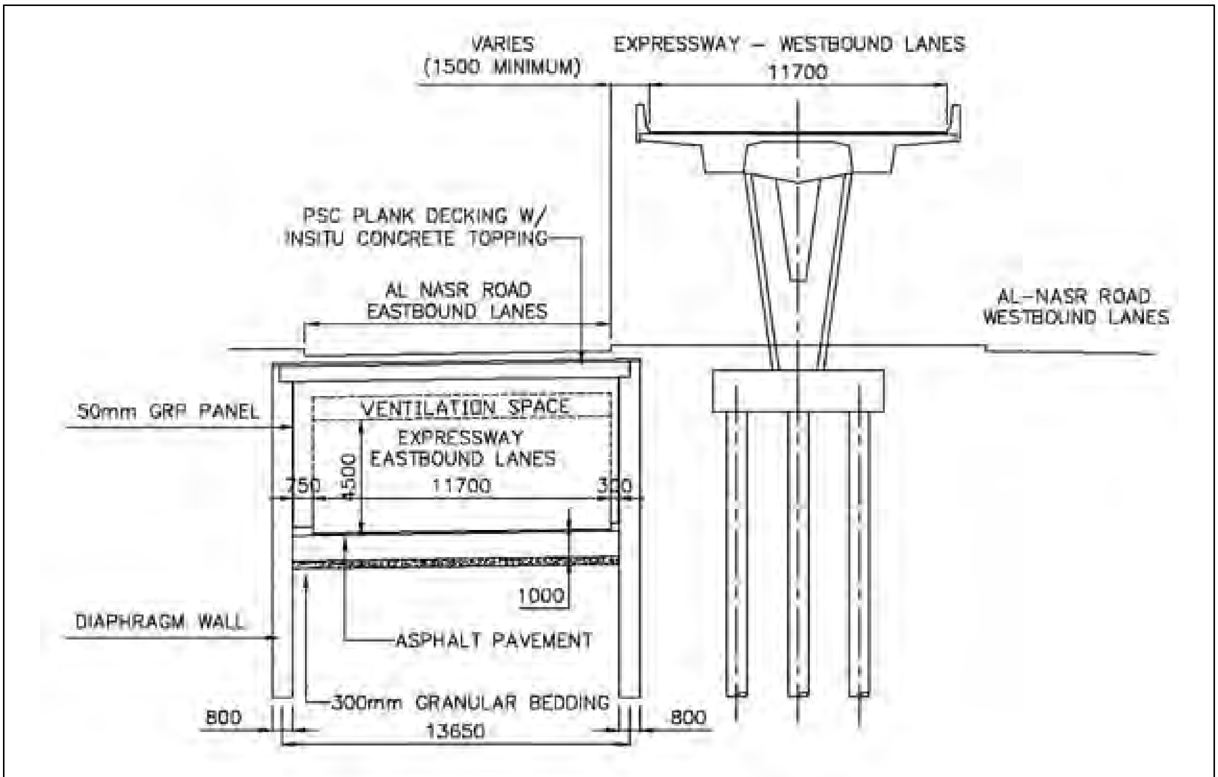


Figure 8.7-11 Section at Sta. 2+000: E3-1

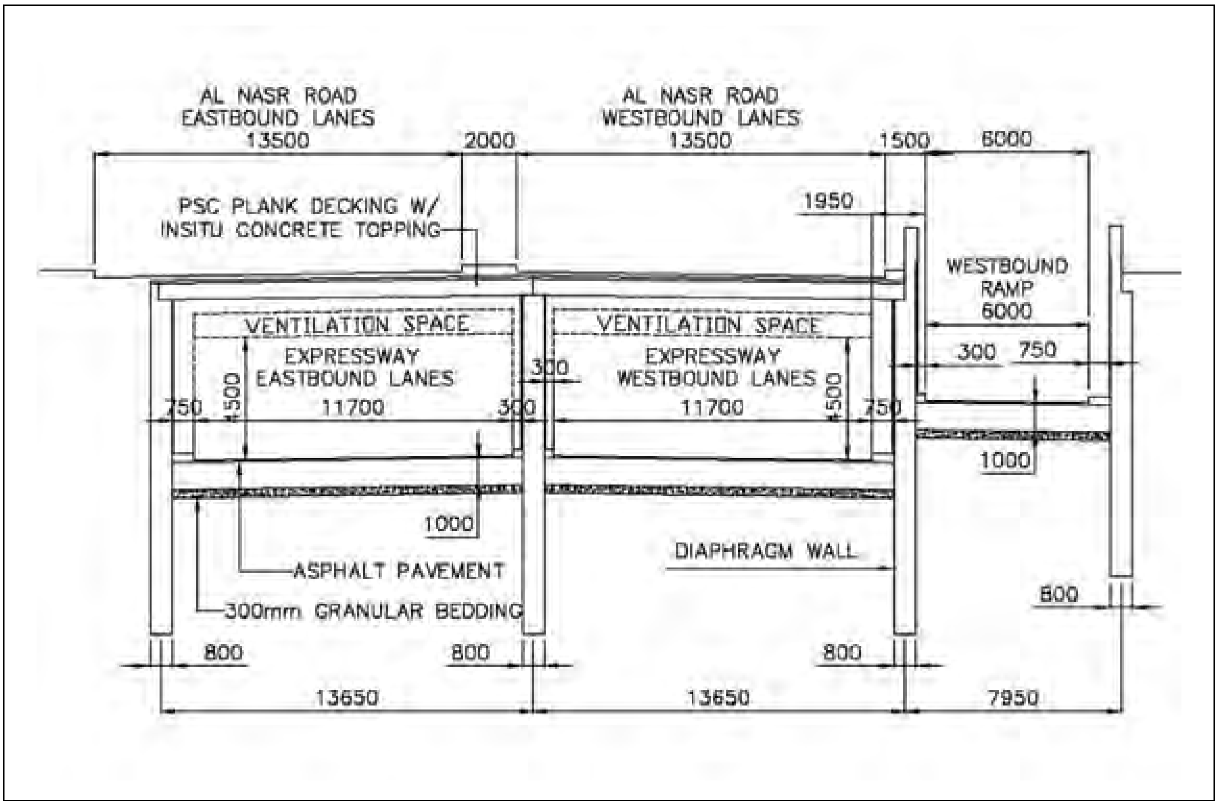


Figure 8.7-12 Section at Sta. 3+200: E3-1

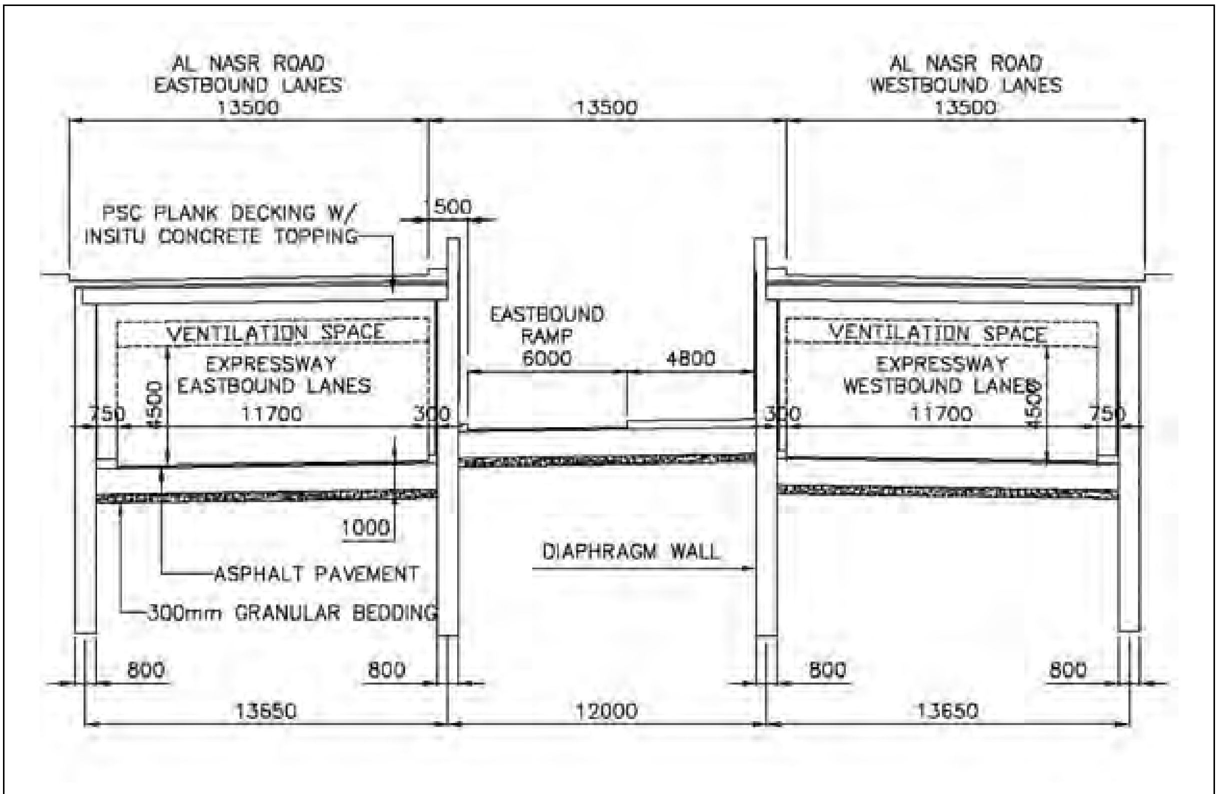


Figure 8.7-13 Section at Sta. 4+300: E3-1

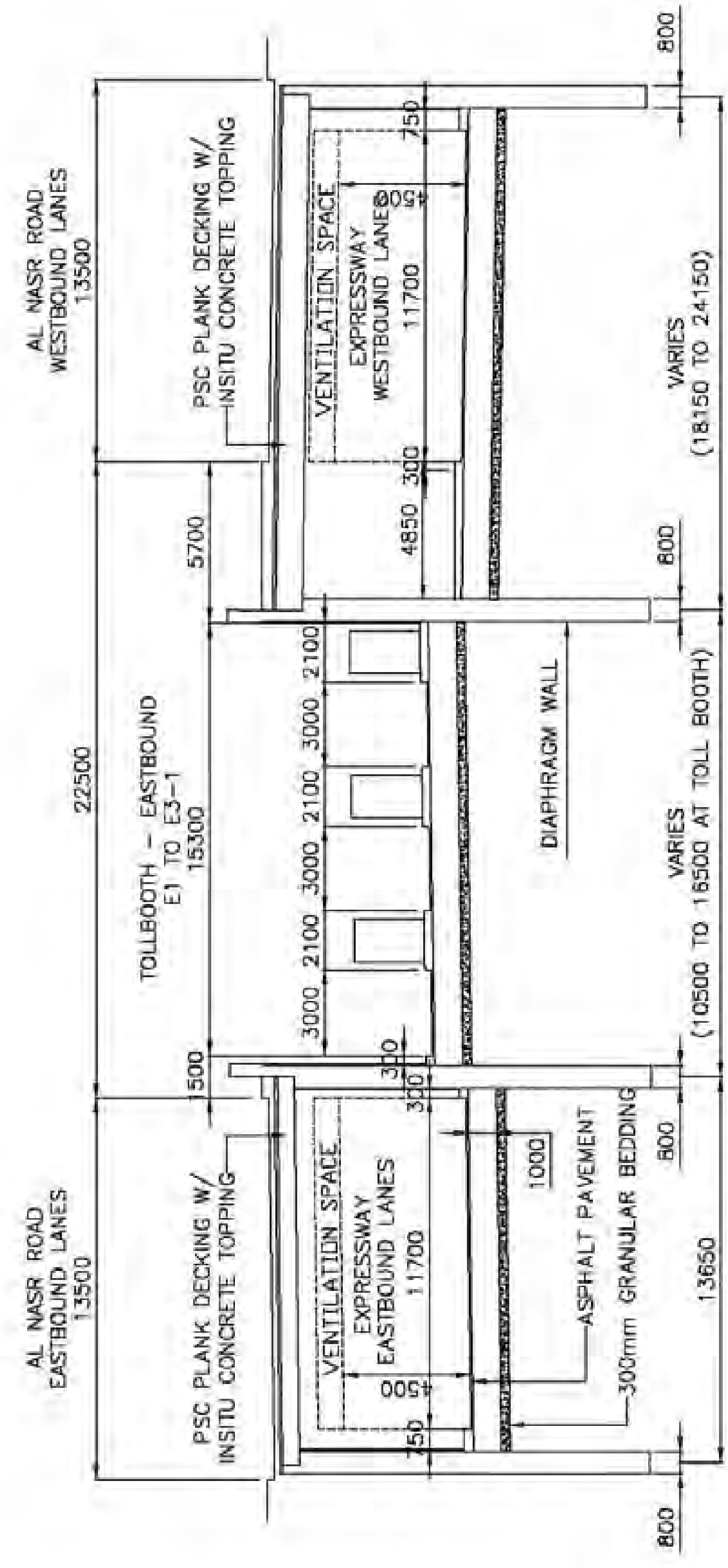


Figure 8.7-14 Section at Sta. 5+600: E3-1

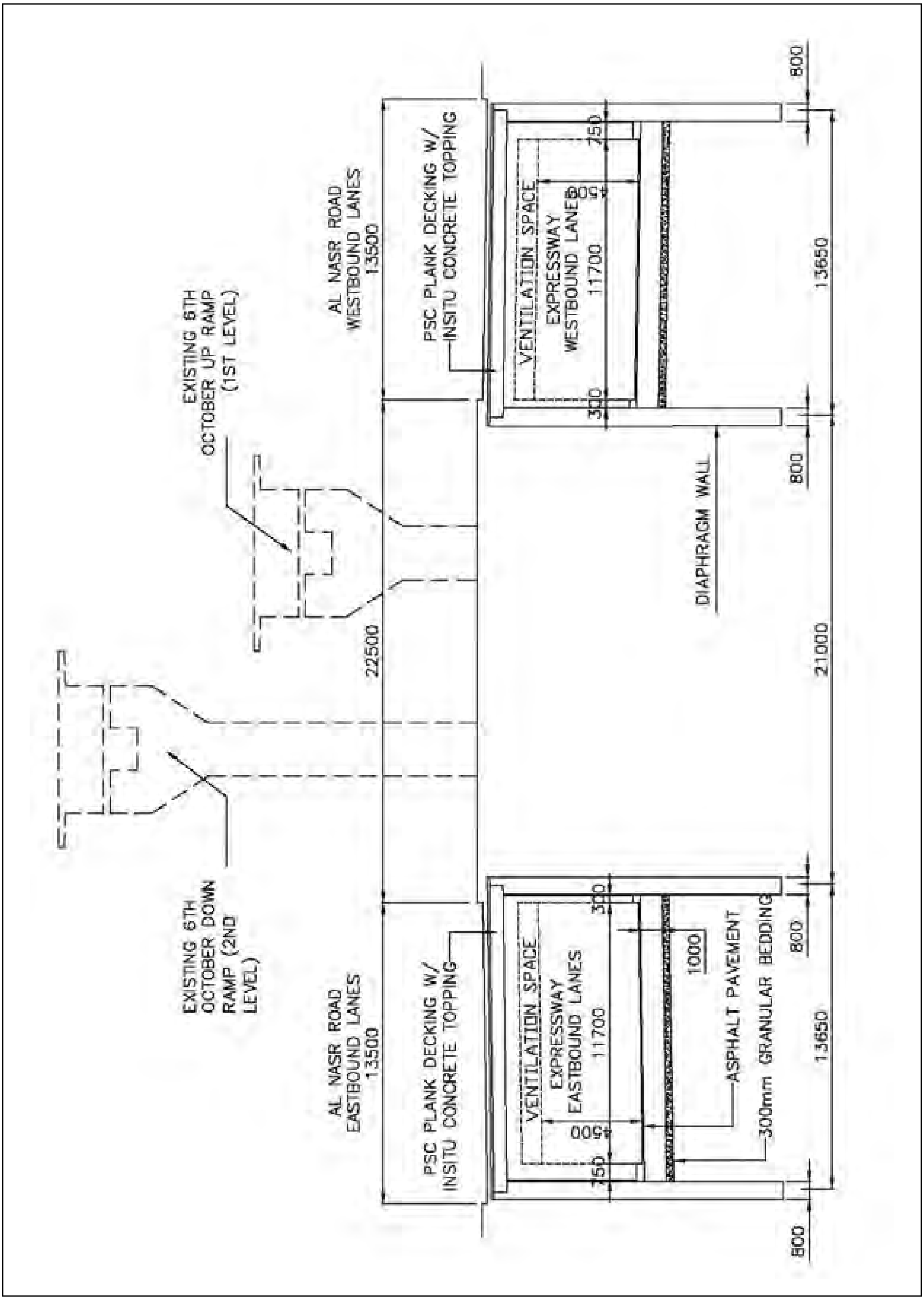


Figure 8.7-15 Section at Sta. 6+400: E3-1

8.8 SUPERSTRUCTURE AND SUBSTRUCTURE TYPES: E3-2 (Pre-F/S)

8.8.1 General

Section E3-2 will extend the proposed E3 expressway south-west along El-Nasr Road from the junction with 6TH October Bridge and will transition onto Salah Salim Street infront of the Citadel. E3-2 terminates at the junction with E3-3 in the Southern Cemetery beyond the Citadel.

The expressway section will commence as depressed structure, continuing the depressed E3-1 structure, will emerge above ground after approximately 1.3 km and will then continue as elevated structure for the remaining length. The total length of the E3-2 section is 6.88 km with approximately 5.2 km of that on viaduct.

Along the transition length from depressed to viaduct structure, the expressway will run on embankment adjacent to El-Nasr Road, requiring ROW acquisition. The expressway will pass onto viaduct structure just before crossing above the existing single line ENR track and will then run along the south side of Nasr Road, adjacent to the railway track. Before reaching Salah Salim Street the expressway will cross over El-Nasr Road, at the location of the Autostrad flyover, and will pass adjacent to the Northern Cemetery. The expressway will then follow the Autostrad as it passes in front of the Citadel to connect with Salah Salim Road. Along the section in front of the Citadel the expressway will occupy the space above the ENR track between the Autostrad and Salah Salim Street. South of the Citadel the expressway will take a new alignment through the Southern Cemetery to join with E3-3.

Proposed onramp and off ramp facilities comprise:

- A single Westbound on-ramp located where the alignment crosses El-Nasr road at the Northern Cemetery, Sta. 10+100
- A single Eastbound off-ramp where the alignment descends onto El Nasr Road, Sta. 7+900
- Single Eastbound on-ramp and Westbound off-ramp facilities at the proposed termination point of E3-2 within the Southern Cemetery.

The proposed expressway structures will provide 3-lanes in each direction. The on and off ramps provide single lane access/egress.

Refer to Figure 8.8-1 for a layout plan of E3-2 showing main structural components.

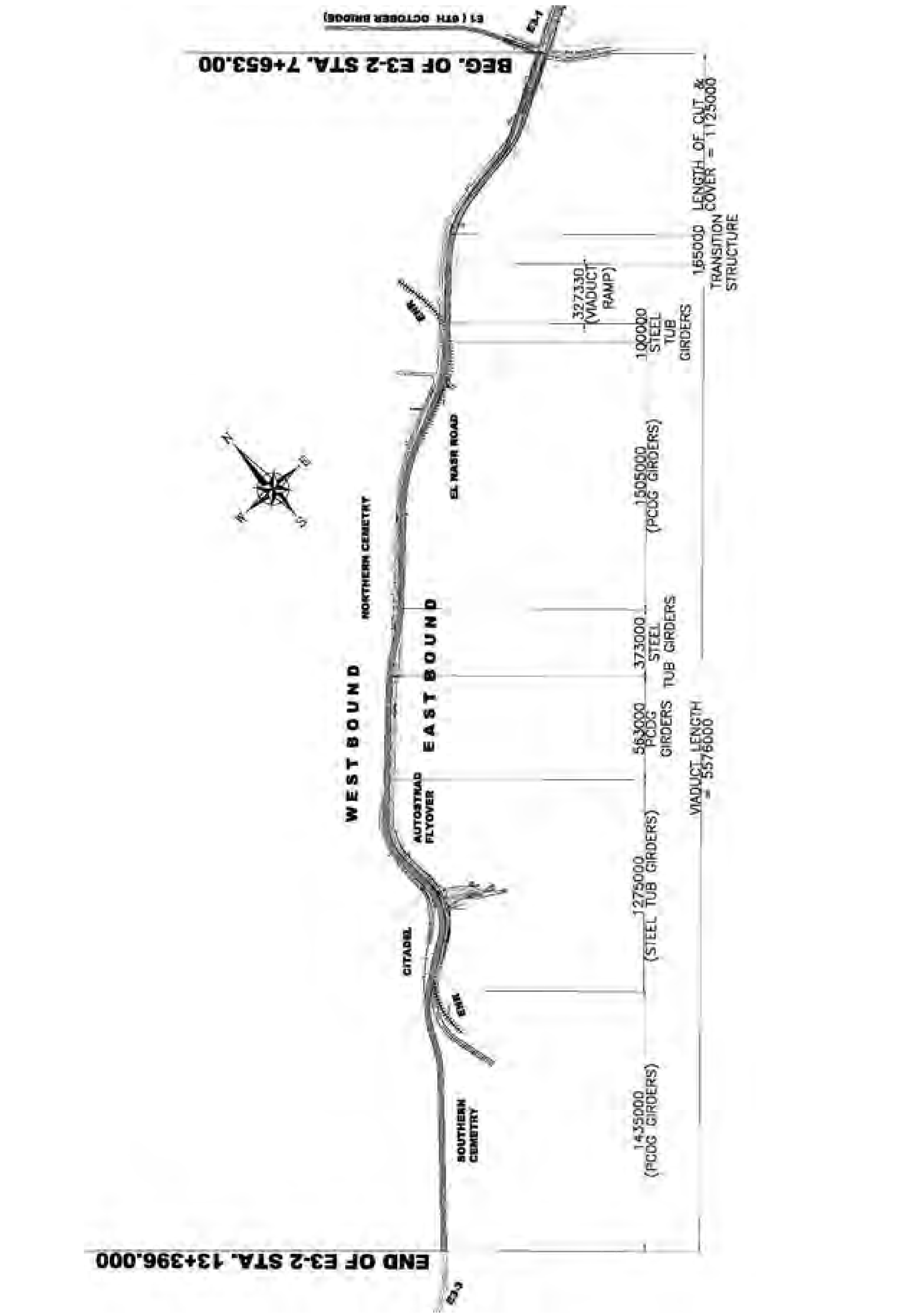


Figure 8.8-1 Layout Plan E3-2

8.8.2 E3-2 Superstructure Types

The study on superstructure types has been based on several considerations including:

- Preliminary determination of required span length along each critical section and at access ramp locations
- Construction cost and construction time considerations
- Restrictions imposed on method of construction for sections adjacent to and above the existing ENR track
- Restrictions imposed on method of construction above the existing Autostrad
- Required temporary works and construction equipment with corresponding impact on traffic management
- Inspection and maintenance considerations
- Aesthetic considerations

The preliminary design considerations for superstructure types for Pre-Feasibility Study are presented in Table 8.8-1.

Given that there are no critical controls on the vertical alignment, the final elevation of the deck can be selected to allow full depth continuity joints, if PC girder construction is adopted. Such joints can be used to dispense with bearings and expansion joints that are a usual feature of PC beams and slab decks, thereby greatly reducing the inspection and maintenance obligations for the expressway structure. The joint can be designed to carry full live load effect assuming full continuity over the support. An alternative arrangement is to use flexible slabs to connect decks at the piers. These slabs are flexible enough, compared to the much stiffer deck, that little bending moment is induced in the slab from live load. These effects can easily be designed out in the slab reinforcing. The advantage of these slabs is that expansion joints can be eliminated except at required intermittent locations. Bearings are still required but they are fully protected (no leaking joints) and can still be removed if required (flexible slab is demolished and reconstructed after bearing replacement).

Details of both a full continuity joint and flexible slab construction are presented in Figure 8.8-2.

For the purposes of illustrating the preliminary design for the Pre-Feasibility Study, the following deck sections have been adopted:

- PC Girders with flexible slab option for sections adjacent to El Nasr Road and ENR track (similar construction to Autostrad Flyover)
- Twin steel tub girder and pre-cast slabs for sections directly above ENR track or sections crossing El Nasr Road.

Table 8.8-1 Preliminary Design Considerations – Superstructure

	Issue	Comment	Preliminary Design Impact
1.	Preliminary determination of required span length	Single column piers supporting wide six (3+3) lane decks will be more economic at medium spans rather than short spans, particularly if swivel type piers are adopted. Twin column portals will be governed more by span of the portal, with shorter spans leading to more economic designs.	35m span adopted for single column 25m span adopted for twin column pier bents
2.	Construction cost and construction time issues	Rapid construction techniques are recommended for E2-2 (refer Section 8.4.5). Prefabrication of major elements to be a priority. E3-2 is of sufficient length for prefabrication of elements to have an impact on reducing costs.	Pre-cast beam and prefabricated girder decks proposed. Prefabricated deck slabs to be used where justified.
3.	Restrictions imposed on construction adjacent to and above the existing ENR track	ENR prefers that in-situ construction is not used above their tracks.	Reduced numbers of girders and use of prefabricated deck slabs are a priority above ENR tracks.
4.	Restrictions imposed on construction above the existing Autostrad	Construction methods to minimize impact on traffic are a priority.	Reduced numbers of girders and use of prefabricated deck slabs are a priority above traffic lanes.
5.	Required temporary works and construction equipment with corresponding impact on traffic management	Minimize use of temporary works. Adopt span lengths to allow conventional construction equipment to be used where possible.	Pre-cast beam and prefabricated girder decks proposed to minimize temporary works. 25m to 35m span lengths allow use of conventional equipment for erection.
6.	Inspection and maintenance	Minimize bearings and expansion joints.	Continuity joints and flexible slabs to be considered for PC beam decks. Full continuity can be achieved by structural steel decks to avoid joints.
7.	Aesthetics	A major consideration in front of Citadel.	No specific measures addressed at Pre-FS. To be reviewed during FS stage.

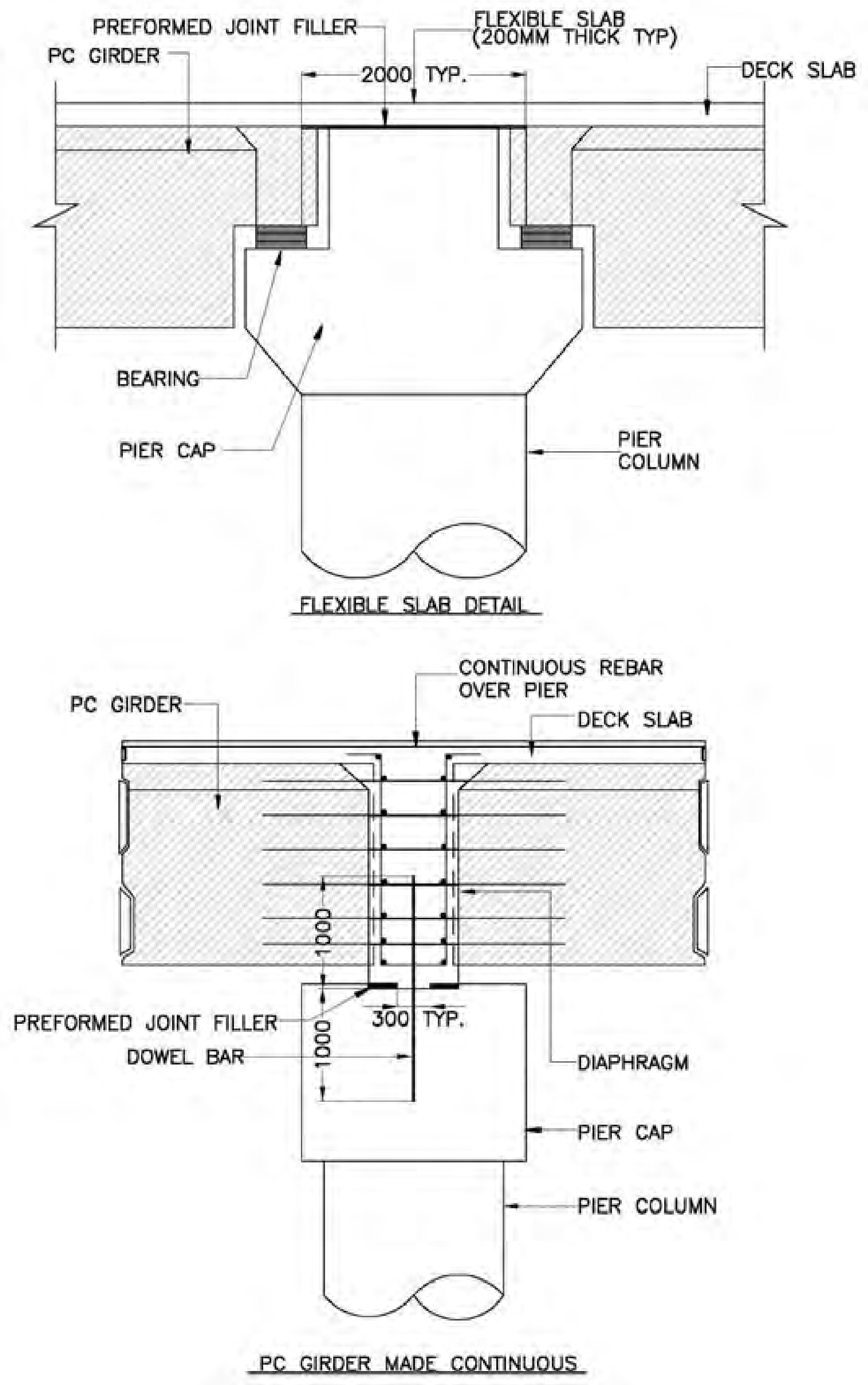


Figure 8.8-2 PC Girder Decks: Alternative Details at Pier Supports.

8.8.3 E3-2 Substructure Types

The study on substructure types has been made based on several considerations including:

- Desk study on soil conditions
- Construction cost and construction time considerations
- Operational and construction clearance requirements adjacent to ENR track
- Restrictions imposed on substructure location by the existing Autostrad flyover
- Restrictions imposed by the rock cutting in front of the Citadel
- Operational clearance requirements of at-grade lanes on El-Nasr Road and Salah Salim Street.
- Impact of method of construction on traffic management
- Aesthetic considerations

The preliminary design considerations for substructure types for Pre-Feasibility Study are presented in Table 8.8-2.

For the purposes of illustrating the preliminary design for the Pre-Feasibility Study, the following substructure types have been adopted:

Foundations

- Larger diameter bore piles and conventional pile caps, numbers of piles minimized to promote rapid construction
- Barrettes or single large diameter bored piles at the rock cut in-front of the Citadel. The Study Team does not have any detailed soil data for the exposed rock at this location. The preliminary design is therefore based upon a conservative approach regarding bearing capacity of the rock; deep foundations are proposed to find bearing below the invert of the cutting.

Piers

- Single reinforced column piers (rotated type) to support the deck where possible
- Twin column reinforced concrete pier bents with structural steel copings across ENR track, El Nasr Road and Salah Salim Street.

Table 8.8-2 Preliminary Design Considerations – Substructure

	Issue	Comment	Preliminary Design Impact
1.	Desk study on soil conditions	Soil conditions assumed similar to E3-1 at start of expressway. Bedrock outcropping at several locations. Conservative approach made regarding bearing capacity of rock.	Bored piles proposed for conventional pile caps. Pile lengths same as assumed for E3-1. Barrettes or large diameter bored piles at location of ENR rock cut near citadel.
2.	Construction cost and construction time issues	A study on optimal span lengths required to minimize costs at detailed design stage. Foundation layouts required to minimize construction time.	Span lengths adopted for Pre-FS Study based on previous experience for similar structures. Number of bored piles at each foundation to be minimized to promote rapid construction.
3.	Operational and construction clearance requirements adjacent to ENR track	Horizontal operational clearance (above ground) from edge of rail to face of structure is 2.5m minimum. For foundations the construction clearance requirement is a minimum of 1.0m from the edge of the sleeper to the face of the permanent foundation structure.	Single column piers to be adopted where possible in order to locate foundations to be well set back from tracks. Rotated pier copings to be used to avoid shoring over the tracks.
4.	Restrictions imposed on substructure location by the existing Autostrad flyover	Width of existing flyover and existing foundation locations will severely impact the expressway substructure design in the case that the expressway passes over the flyover.	Expressway aligned to avoid crossing the Autostrad flyover.
5.	Restrictions imposed by the rock cutting in front of the Citadel	Co-ordination required with MOD regarding methods of construction.	Foundations are to be well set back from rock cut to aid constructability and minimize disturbance to ENR tracks.
6.	Operational clearance requirements of at-grade lanes on El-Nasr Road and Salah Salim Street	Existing lane configurations restrict location of pier supports.	Single column piers to be adopted where possible. Twin column portals used where necessary to avoid conflicts with traffic lanes.
7.	Impact of method of construction on traffic management	Methods of construction adopted to minimize impact on traffic.	Rotated pier copings to be used to avoid shoring over traffic lanes. Pier copings in structural steel to facilitate erection methods that will least impact traffic.
8.	Aesthetics	A major consideration in front of Citadel.	No specific measures addressed at Pre-FS. To be reviewed during FS stage.

8.8.4 Structure above Egypt National Railways at Citadel

Based on a meeting of the JICA Study Team with ENR officials, the following points are to be taken into account in the preliminary design:

- The ENR track is used for military purposes; therefore proper co-ordination with Ministry of Defense (MoD) will be essential.
- The ENR do not have any objections in principle to the proposed location of the expressway adjacent to and above ENR track. However the following concerns were raised:
  - ✧ Construction methods to ensure safety and avoid debris/rocks falling on the track are essential.
  - ✧ Temporary possession of the railway track space during construction must be coordinated with MOD
  - ✧ Space provision to accommodate an additional track (existing is only single track) should be included in the design
  - ✧ Utilities adjacent to existing track may require to be relocated. Co-ordination with utility companies will be necessary
  - ✧ For sections where existing track is located in deep rock cutting (adjacent to Citadel), construction adjacent to the track above the cutting will require special co-ordination with MOD

For operational and construction clearance requirements refer to Section 8.3.4

8.8.5 Typical Sections

Typical sections illustrating the preliminary design are presented in Figure 8.8-3 and Figure 8.8-4.

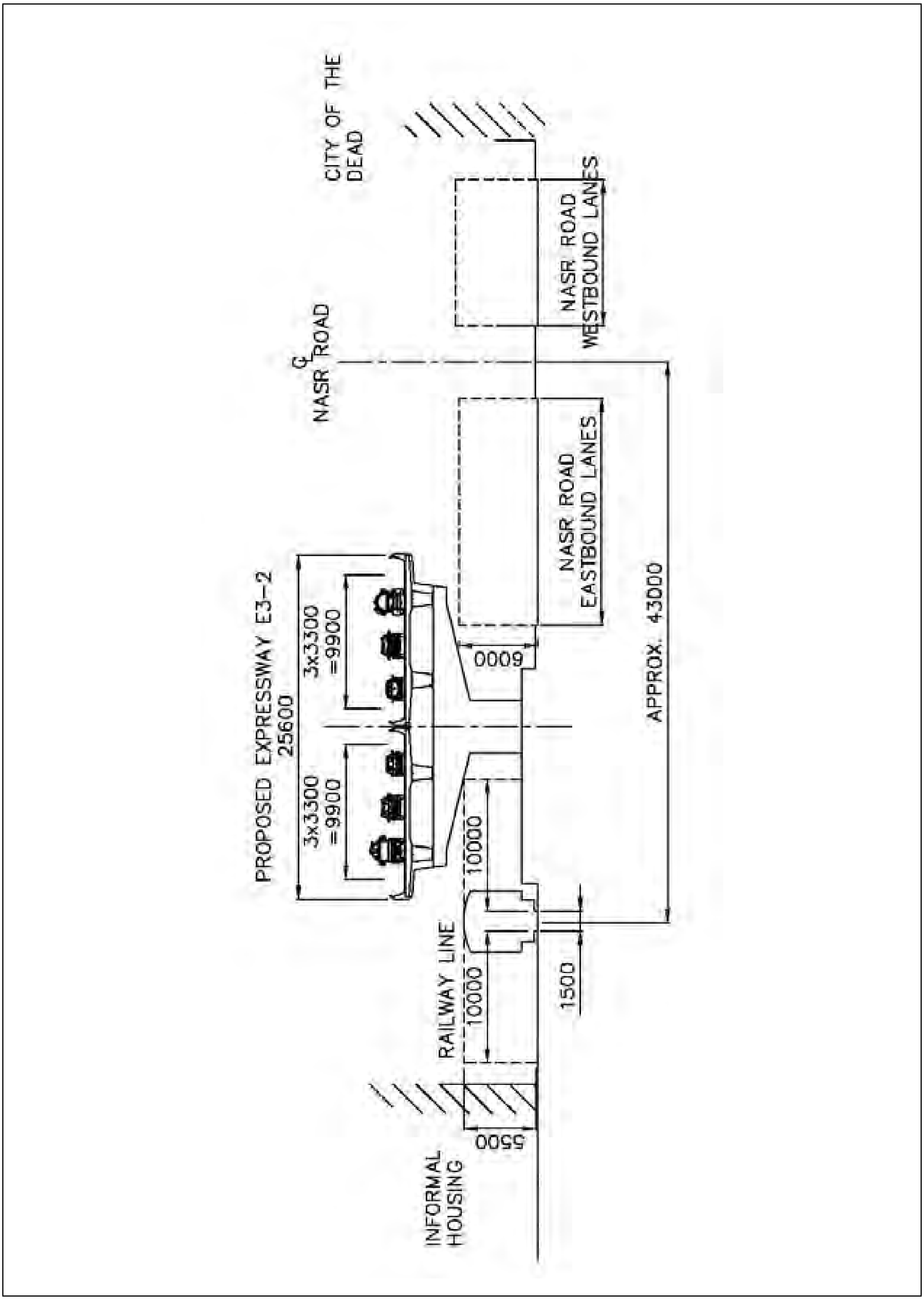


Figure 8.8-3 Section at Sta. 9+000: E3-2

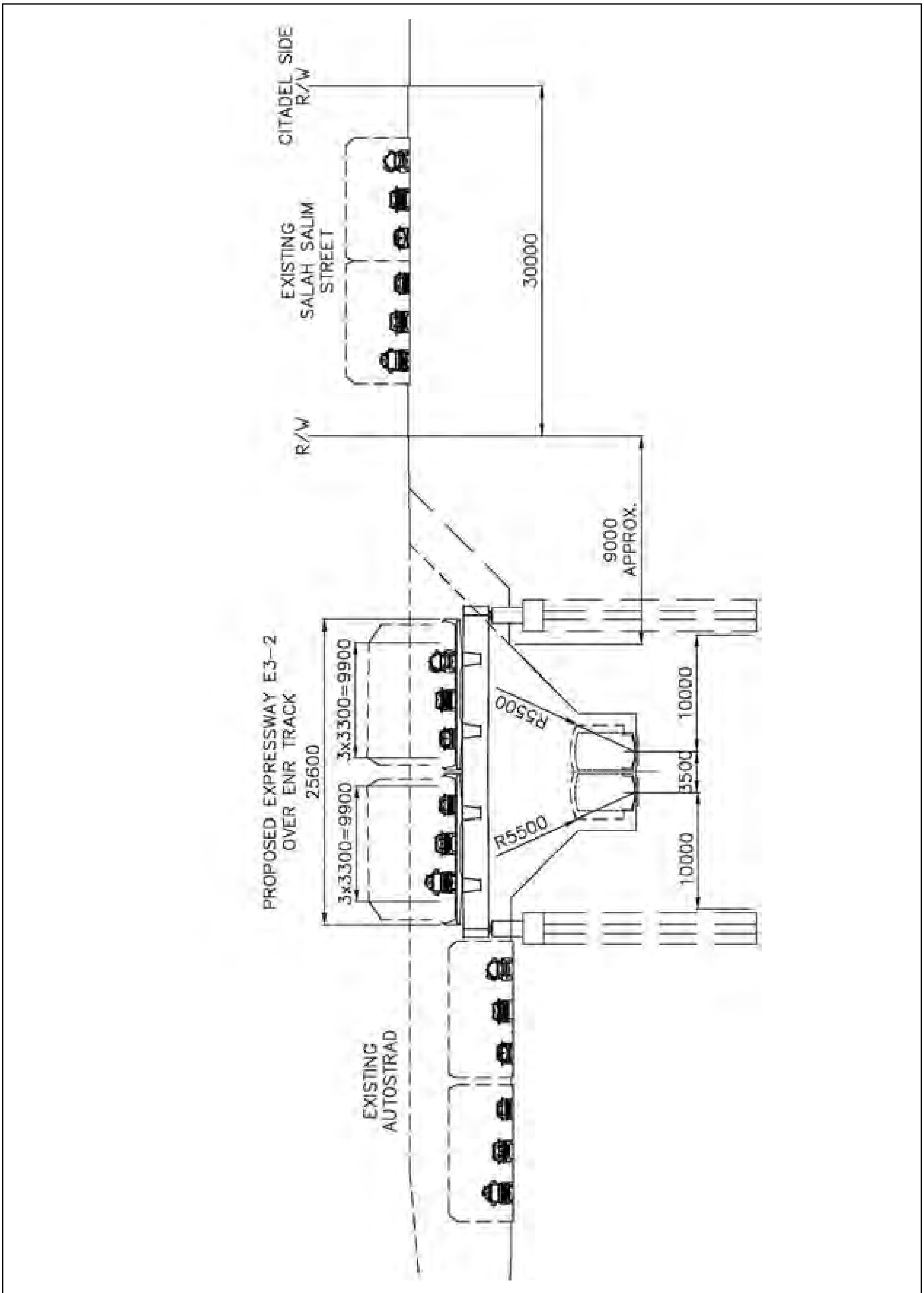


Figure 8.8-4 Section at Sta. 11+200: E3-2

8.9 SUPERSTRUCTURE AND SUBSTRUCTURE TYPES: E3-3 (Pre-F/S)

8.9.1 General

Section E3-3 will be the last section of the proposed E3 expressway, connecting from the continuation point with E3-2 in Cairo Governorate to the proposed E8 expressway in Giza and will require a crossing of the Nile River.

E3-3 will commence on new alignment at a location within the Southern Cemetery just south of the Citadel before rejoining Salah Salim Street again as the alignment turns west. E3-3 will continue along El-Malik as-Salih Street crossing above the existing Metro Line 1 track and the depressed roadway at this location. The existing flyover on El-Kumaysh Street will have to be replaced with a depressed structure in order to provide a clearway for the proposed expressway as it continues onto Manyal along El-Rawdah Street. Expressway E3-3 will then continue with a crossing of the Nile River directly above Giza Bridge. After crossing the Nile, the expressway will continue along El-Ahram Street and will cross above existing flyover structures in Giza Square. The expressway will then bifurcate with separate legs connecting into the proposed E8 expressway both along El-Ahram Street (passing along the depressed roadway and crossing both the ENR and the Metro Line 1 elevated track) and along Jamal ad-Din’Afifi Street (crossing Faysal Bridge).

Expressway section E3-3 will be elevated throughout its length, carried both on shorter span viaduct over existing roads and on longer span bridge structures over the Nile. The expressway will run on conventional single deck structure for the initial 3.7km and then will transition onto double deck structure on Salah Salim in order to pass through the narrower confines of El-Malik as-Salih Street and El-Rawdah Street (24m minimum between building lines). Eastbound lanes will be carried on the 1ST Level deck and Westbound lanes will be carried on the 2ND level deck. The expressway will cross the Nile with a double deck configuration above Abbas Bridge. The Eastbound and Westbound lanes will remain on separated decks structures through Giza Square until the connection with both local roads and the future Expressway E8.

The Study investigated two options at Giza Square:

Option 1: Expressway structures to pass over the existing 1ST Level Eastbound flyover, requiring a 2ND level deck for Eastbound expressway traffic and a 3RD Level deck for the Westbound expressway traffic.

Option 2: The existing 1st level Eastbound flyover structures in Giza Square to be incorporated into the proposed expressway to the maximum extent possible. This will allow the Eastbound expressway deck to descend to 1^{ST} Level and the Westbound expressway deck to descend to 2^{ND} level.

The advantages of Option 2 are directly related both to reduced plan extent required of the Eastbound expressway structure and to avoiding the use of 3^{RD} level Westbound structures; both will entail reduced costs and shorter construction periods.

However, the major disadvantage of Option 2 is that local and expressway traffic are merged together on the Eastbound deck; such an arrangement will be detrimental both to the serviceability of the expressway and to the location of toll booths to intercept Eastbound expressway traffic. In addition the future connection to Expressway E8 will require further modification and widening of the existing Eastbound flyover to accommodate the new Eastbound link from E8.

Taking the above factors into consideration, the preliminary design is therefore based upon the Option 1 arrangement at Giza Square.

The total length of the E3-3 section is about 5.5 km with approximately 3.5km of that length on double deck or separated Eastbound and Westbound structures.

Eastbound and Westbound on-ramp and off-ramp facilities are proposed where the expressway commences in the Southern Cemetery. The on and off ramps provide single lane access/egress.

The proposed expressway structures will provide 3-lanes in each direction extending across the Nile River until Giza Square. At Giza Square the expressway structure will bifurcate to connect with the local road network both at Faysal Bridge and at Al-hram Street (Pyramid Street). Provisions are also made for future connection with the proposed E8 expressway.

Refer to Figure 8.9-1 for a layout of E3-3 until the Nile. Refer to Figure 8.9-6 and Figure 8.9-7 for the layout West of the Nile until the connection with the proposed E8.

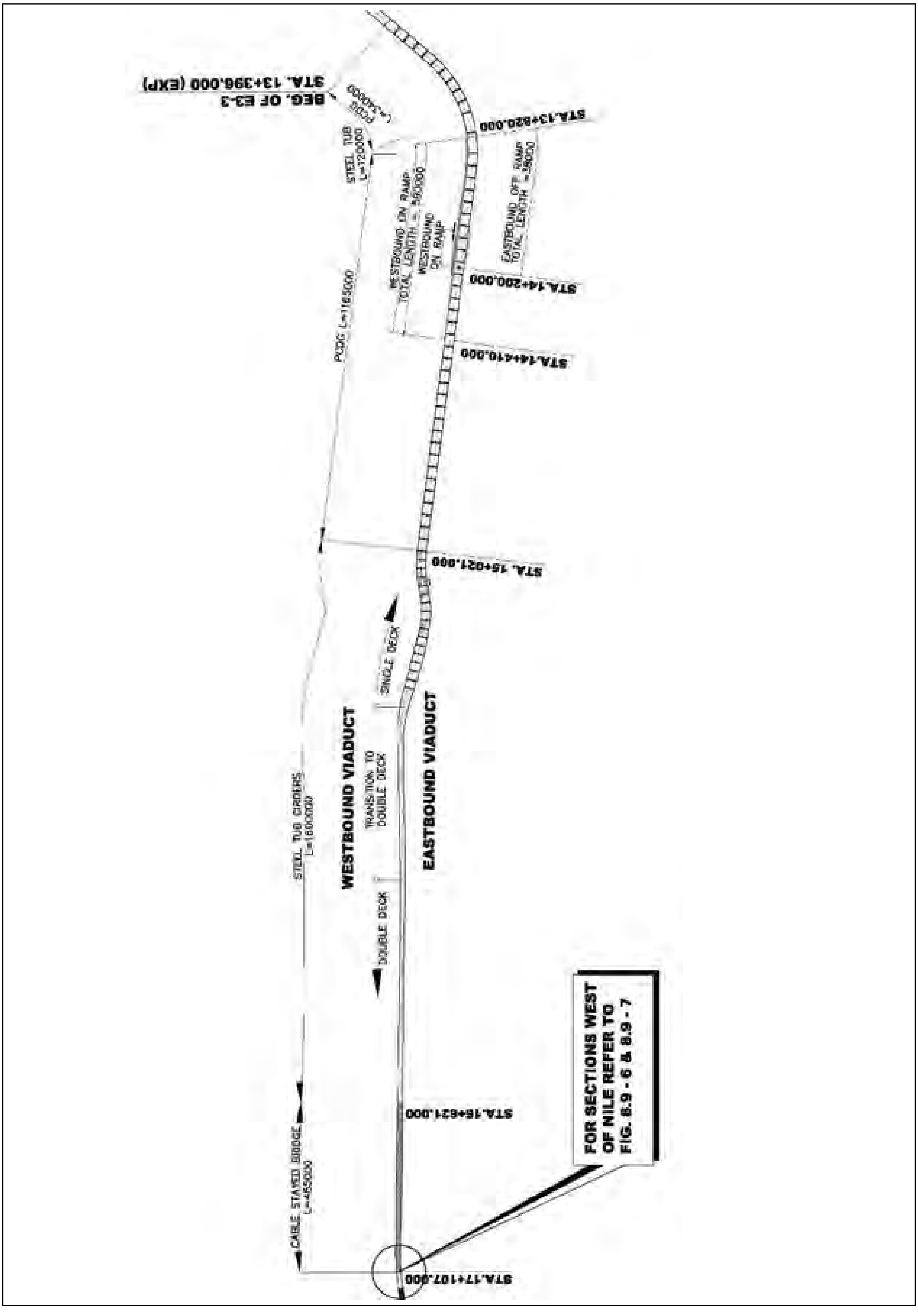


Figure 8.9-1 Layout Plan E3-3 (to the Nile)

8.9.2 E3-3 Superstructure Types (Viaduct Sections)

The study on superstructure types has been based on several considerations including:

- Preliminary determination of required span length along each of the critical sections and at access ramp locations (Southern Cemetery, Salah Salim Street, El-Malik El-Salih Street, Rawdah Street and Giza Square)
- Construction cost and construction time considerations
- Restrictions imposed on method of construction for sections above the existing Metro Line 1 track
- Required temporary works and construction equipment with corresponding impact on traffic management
- Inspection and maintenance considerations
- Aesthetic considerations

The preliminary design considerations for superstructure types for Pre-Feasibility Study are both similar to Expressway Section E3-2 (for the regular single deck sections) and also to Expressway E2-2 (restricted widths driving double-deck layouts in some sections).

For the purposes of illustrating the preliminary design for Pre-Feasibility Study, the following deck sections have been adopted:

- PC Girders with flexible slab option for the section extending from the Southern Cemetery to the transition onto double deck.
- Twin steel tub girder and pre-cast slabs for long span sections, double deck sections and where the expressway is elevated along and above existing roads and structures.

8.9.3 E3-3 Substructure Types (Viaduct Sections)

The study on substructure types has been based on several considerations including:

- Desk study on soil conditions
- Construction cost and construction time considerations
- Restrictions imposed on substructure locations in the Southern Cemetery
- Operational and construction clearance requirements adjacent to Metro Line 1 track
- Restrictions imposed on substructure locations by the El-Malik El-Salih Underpass
- Restrictions imposed on substructure locations by the existing flyovers structures in Giza Square and the structures along the streets beyond Giza Square to the proposed connection points with Expressway E8

- Operational clearance requirements of at-grade lanes on Salah Salim Street, El-Malik El-Salih Street, Rawdah Street, Giza Square and the streets beyond Giza Square to the proposed connection points with Expressway E8.
- Impact of method of construction on traffic management
- Aesthetic considerations

The preliminary design considerations for substructure types for Pre-Feasibility Study are both similar to Expressway Section E3-2 (for the regular single deck sections) and also to Expressway E2-2 (restricted space driving in non-conventional foundations).

For the purposes of illustrating the preliminary design for Pre-Feasibility Study, the following substructure types have been adopted:

Foundations

- Larger diameter bore piles and conventional pile caps, numbers of piles minimized to promote rapid construction
- Barrettes or single large diameter bored piles at locations where available space restricts the footprint of the foundation.

Piers

- Single reinforced column piers (rotated type) to support the single deck sections where possible
- Structural steel frames to support the double deck sections.
- Composite columns at locations where the deck is cantilevered off single column supports (locations West of the Nile)

8.9.4 Nile River Crossing

The Nile River crossing of E3-3 is a major engineering challenge that will require a unique structural solution, given the challenges imposed by the horizontal and vertical alignments of the proposed expressway and the substantial footprint of the existing Giza Bridge at this location.

The highway alignment established for the preliminary design requires a double deck structure to carry the proposed expressway directly above the existing bridge. The choice of deck type suitable to carry the expressway above the existing bridge is therefore constrained. A steel truss deck has been selected for the preliminary design.

Pre-Feasibility Study undertook a basic cost comparison of alternative bridge solutions, incorporating a double deck steel truss, to include:

- Standard truss supported on outrigger piers above the existing deck
- Asymmetrical cable-stayed bridge with a single tower located in the Nile.
- Steel tied arch

Refer to Chapters 13 and 14 for cost comparisons. From this analysis, it can be seen that a cable-stayed bridge option is reasonably competitive to other schemes with respect to construction cost.

An illustrative design of an asymmetrical cable-stayed bridge is shown in Figure 8.9-3 and Figure 8.9-4. The design features one tower located in the river adjacent to an existing central pier and two side span piers located on each side of the river but out of the water.

The tower will support the steel truss superstructure with twin cable planes and will have inclined lower legs, to straddle the existing deck and find foundation each side of the existing bridge. The steel truss will carry the Eastbound expressway lanes on deck structure spanning across the transverse floor beams through the truss and the Westbound lanes on deck structure spanning across the transverse floor beams above the truss.

A key feature in the arrangement of the tower is the transverse tie beams that will be required to carry the large horizontal component of the diagonal thrust carried by the lower legs, thereby relieving the foundations of a large permanent horizontal reaction.

Foundations shown in the illustration are steel-pipe-sheet-piles; this type of foundation features interlocking steel pipe piles (interlock grouted to achieve a watertight joint and provide shear capacity) and a deep in-situ concrete transfer slab, with substantial structural steel shear connectors, to facilitate transfer of forces between the tower legs and the foundations. The advantage of this method is that the sheet pile wall can be extended above design high water level during construction, effectively serving the purpose of a cofferdam to allow the construction of all in-situ concrete work in the dry. Once all concrete work is completed above water level, the sheet piles can be cut off below the design scour level.

The truss will be suspended by cables alone above the existing deck given that there is no space for a traditional crossbeam to sit the deck on. Transverse and longitudinal bearings can be fitted to bumper frames located on the inside faces of the inclined tower legs each side of the top chord to restrain the truss accordingly. The preliminary design includes an additional simply supported truss span located on the Giza side, given that this is the location of the existing El-Nil road underpass (passing beneath Giza Bridge between the abutment and the first pier) and a small landscaped park.

The depth of the truss will be more or less the same for all the alternatives; the depth is driven in the first instance by the need to provide necessary headroom clearance for the through traffic and by the need to provide an adequate span/depth ratio for the self supporting spans. The truss will have to be designed to have a moment resisting transverse frame (formed by the transverse floor beams and the truss diagonals) in order to avoid the use of sway frame bracing for the top chord (such bracing would increase the required chord to chord distance). For the self supporting truss spans, the top and bottom chords and the compression diagonals of the truss will be box sections and the tension diagonals will likely be I-section girders. The cable stayed truss will likely feature box sections for all truss elements, given that the tension diagonals will also support outriggers for the cable stay anchorages and therefore will need to be stiff out-of-plane.

In order to undertake a full assessment of alternatives established for the preliminary design the following data is required for a full feasibility study:

1. Desk study and site investigation on soil profile in the Nile River.
2. Topographic and hydrographic data for the Nile at this location
3. Published data or data obtained from previous projects as is available together with site surveys.
4. Scour depth data for the existing bridges over the Nile in this stretch of the river
5. Navigation requirements and maximum weight (displacement) of vessels plying the river
6. Data on the existing bridge - spanning, profile, foundation details as is available (the impact of the scour depth of the new construction on the existing pier foundations will likely be a crucial consideration)

8.9.5 Integration with Expressway E8

The study on integration with Expressway E8 will be made based on several considerations including:

- Final layout of Expressway E8 (not yet established)
- Full access for traffic movements between E3 and E8 will be a priority
- Connecting structures will be located both in the vicinity of Faysal Bridge and at the intersection of E8 with EL-Ahram street in order to avoid 4^{TH} level structures (3^{RD} Level structures are unavoidable)
- The demolition of existing buildings will be avoided if at all possible.

Figure 8.9-6 and Figure 8.9-7 show the location of “jump off points” where future connections can be made to link E3 and E8 both in the vicinity of Faysal Bridge and at Pyramid Street.

8.9.6 Typical Sections

Typical sections and plan layouts illustrating the preliminary design are presented in Figure 8.9-2 to Figure 8.9-11.

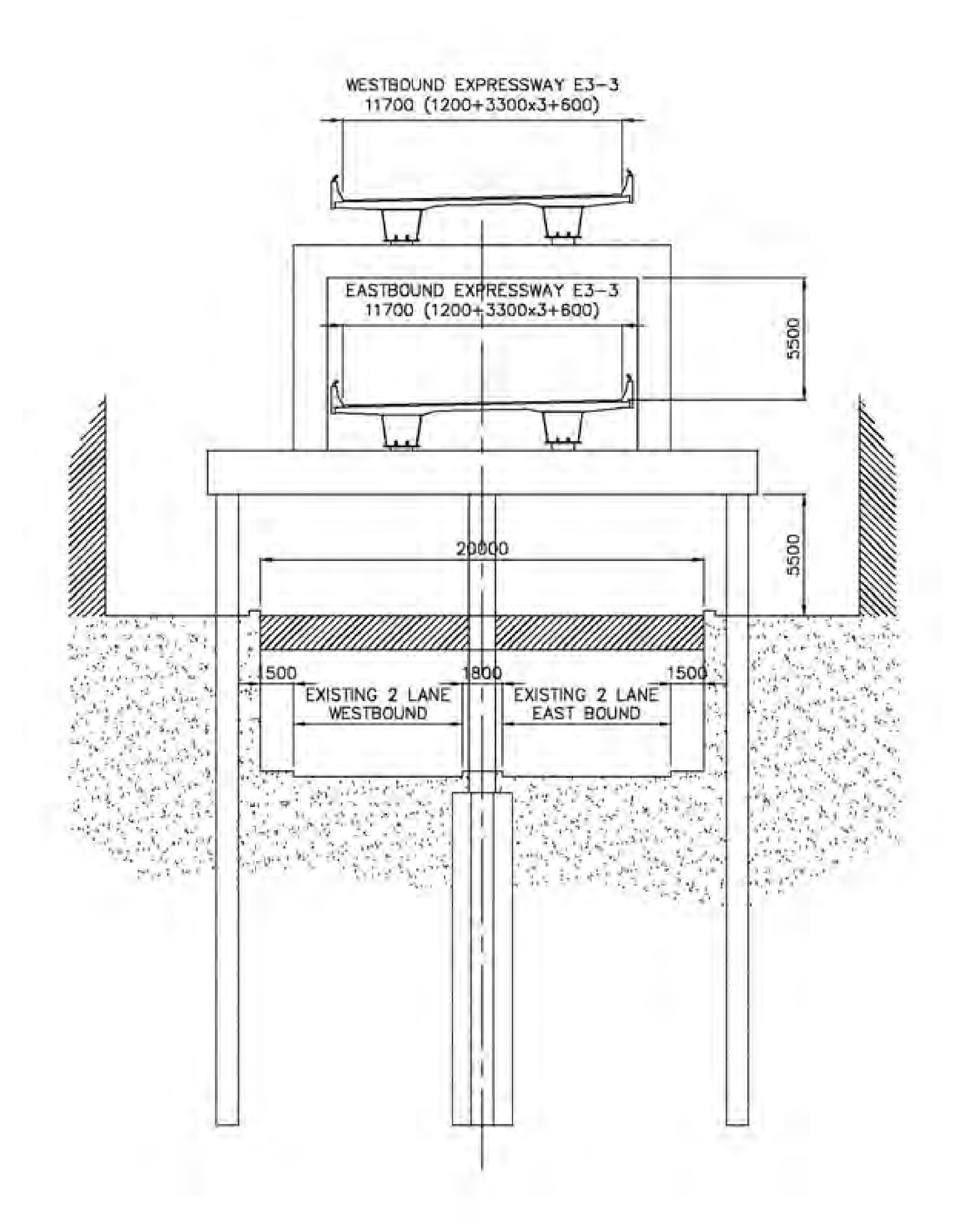


Figure 8.9-2 DRAFT Section at Sta. 15+800: E3-3
(El-Malik El-Salih Underpass)

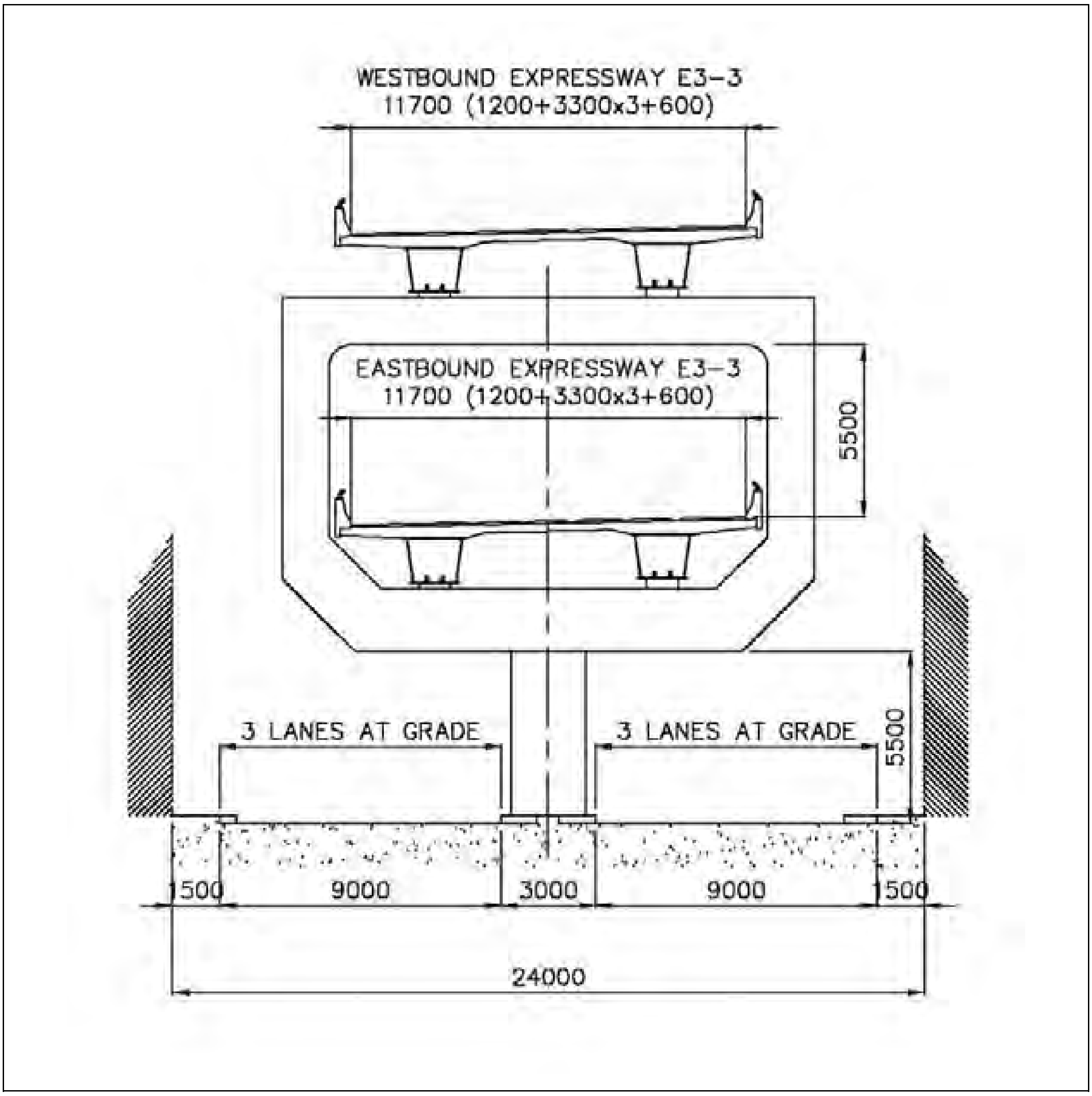


Figure 8.9-3 DRAFT Section at Sta. 16+200: E3-3
(El-Rawdah Street – Manyal)

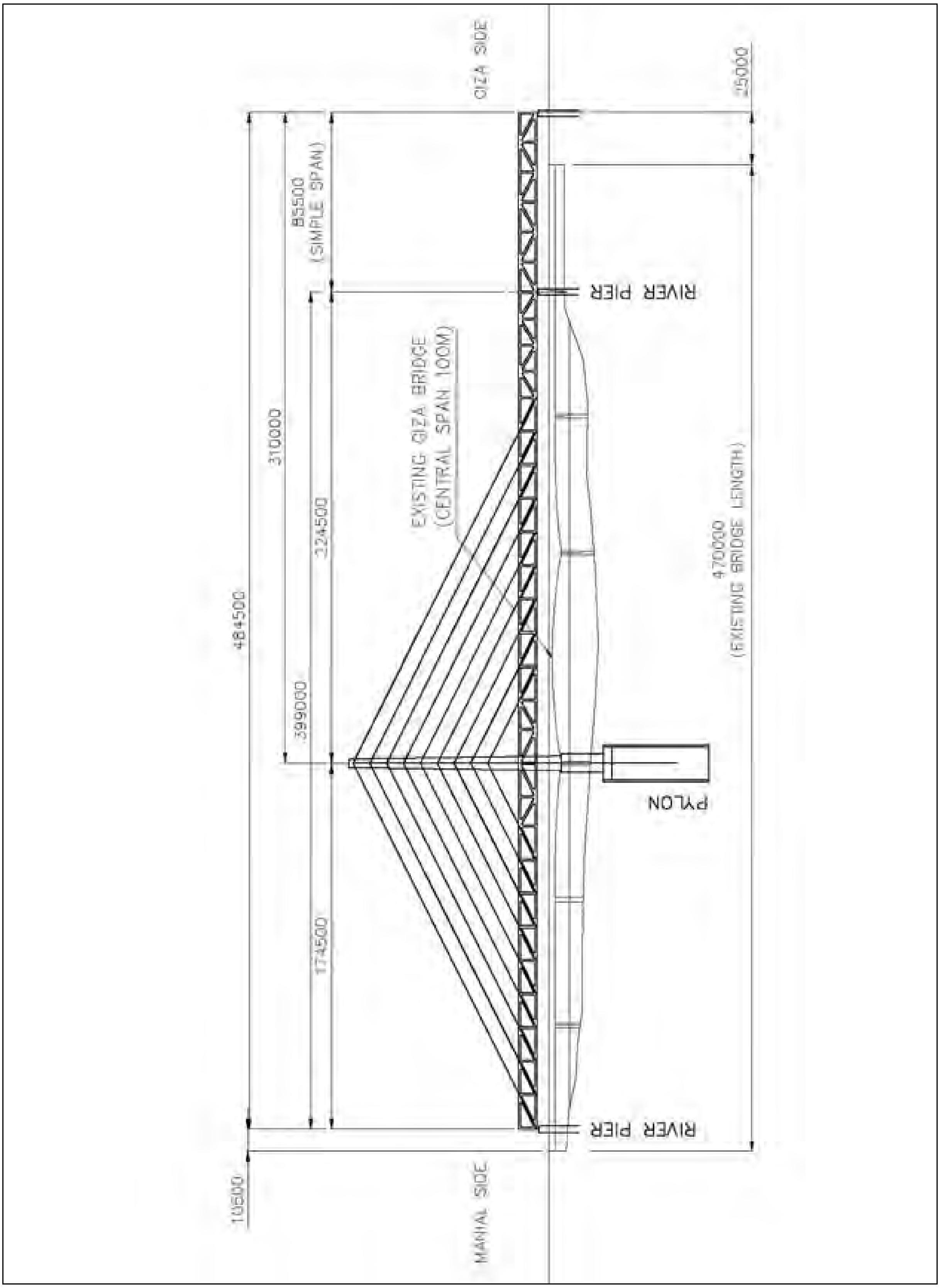


Figure 8.9-4 DRAFT Elevation on Nile River Bridge: E3-3
Cable-Stayed Bridge Alternative
(Crown of existing Giza Bridge at Sta. 16+850)

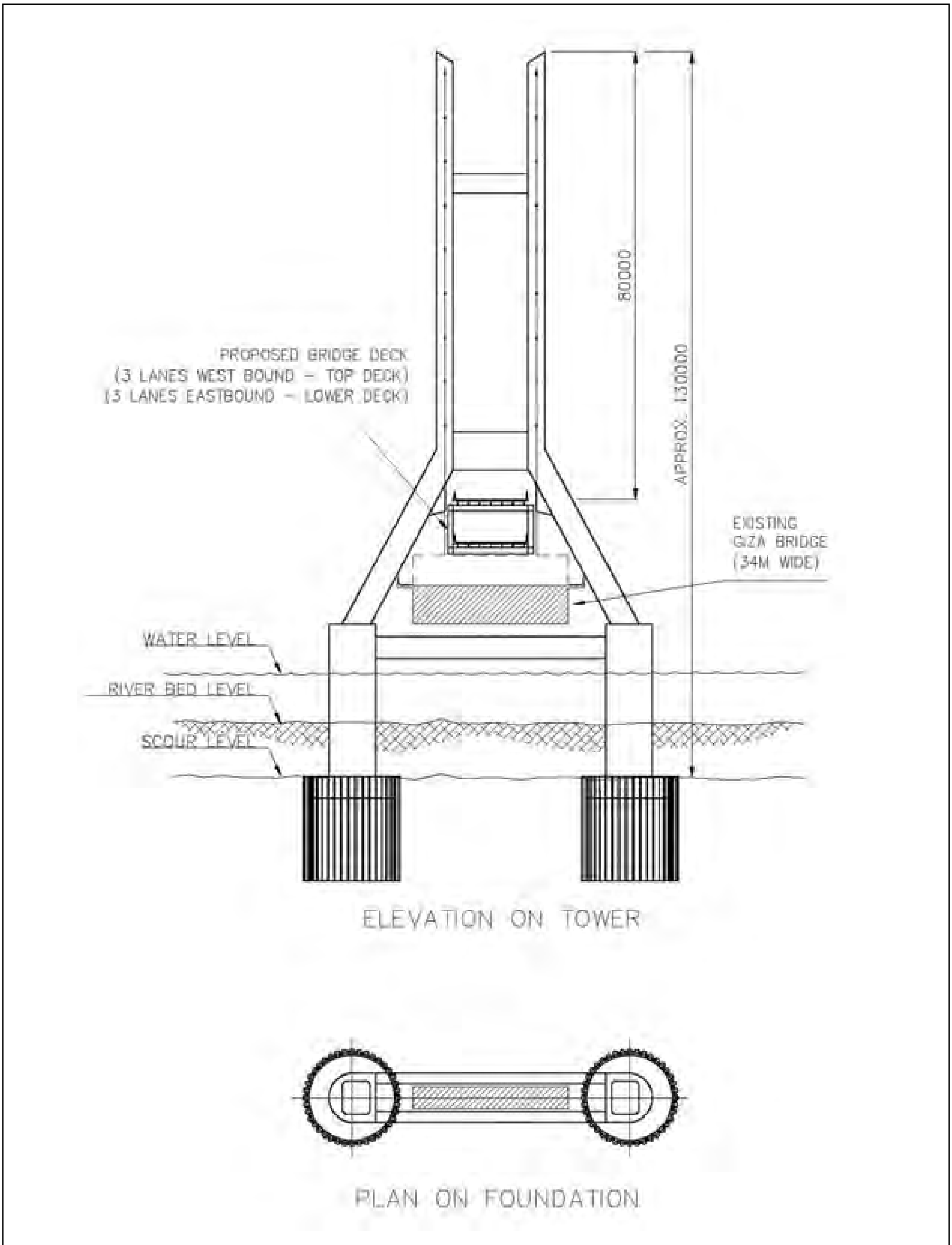


Figure 8.9-5 DRAFT Section at Nile River Bridge: E3-3
Cable-Stayed Bridge Alternative
(Tower at Sta. 16+800)

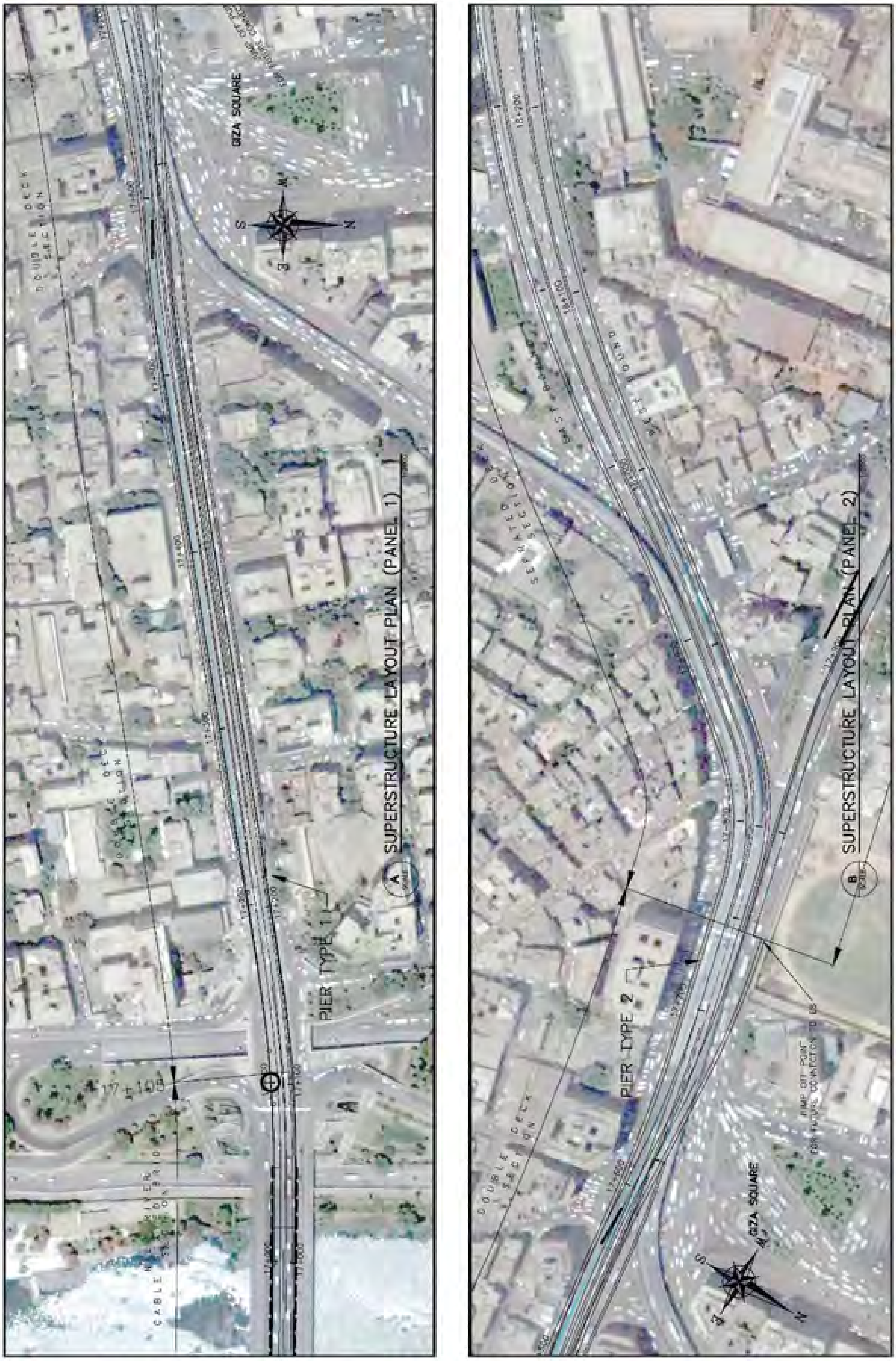


Figure 8.9-6 Plan of Structure Layout – West of Nile – E3-3 (1/2)

Figure 8.9-7 Plan of Structure Layout – West of Nile – E3-3 (2/2)

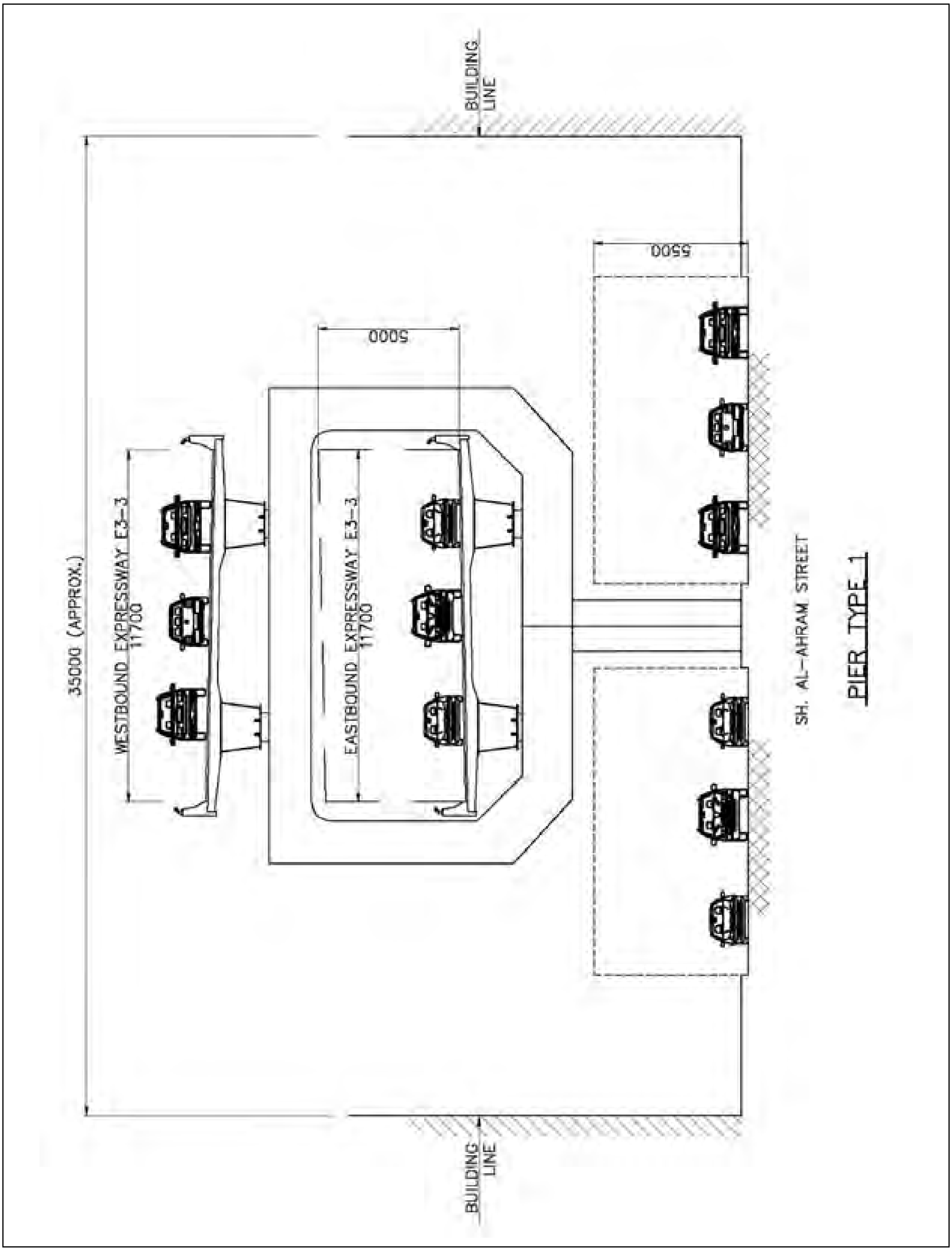


Figure 8.9-8 Section West of Nile – Pier Type 1

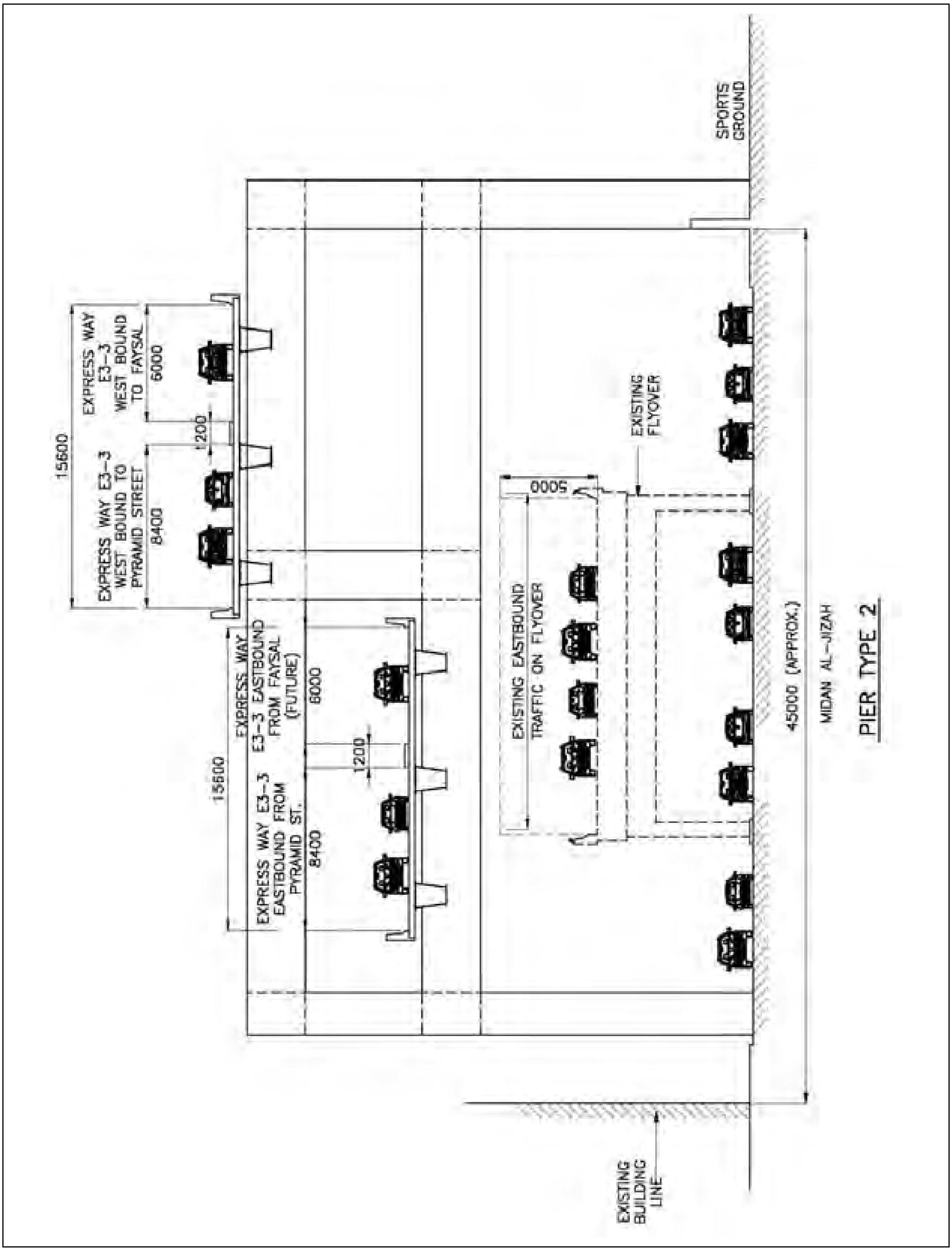


Figure 8.9-9 Section West of Nile – Pier Type 2

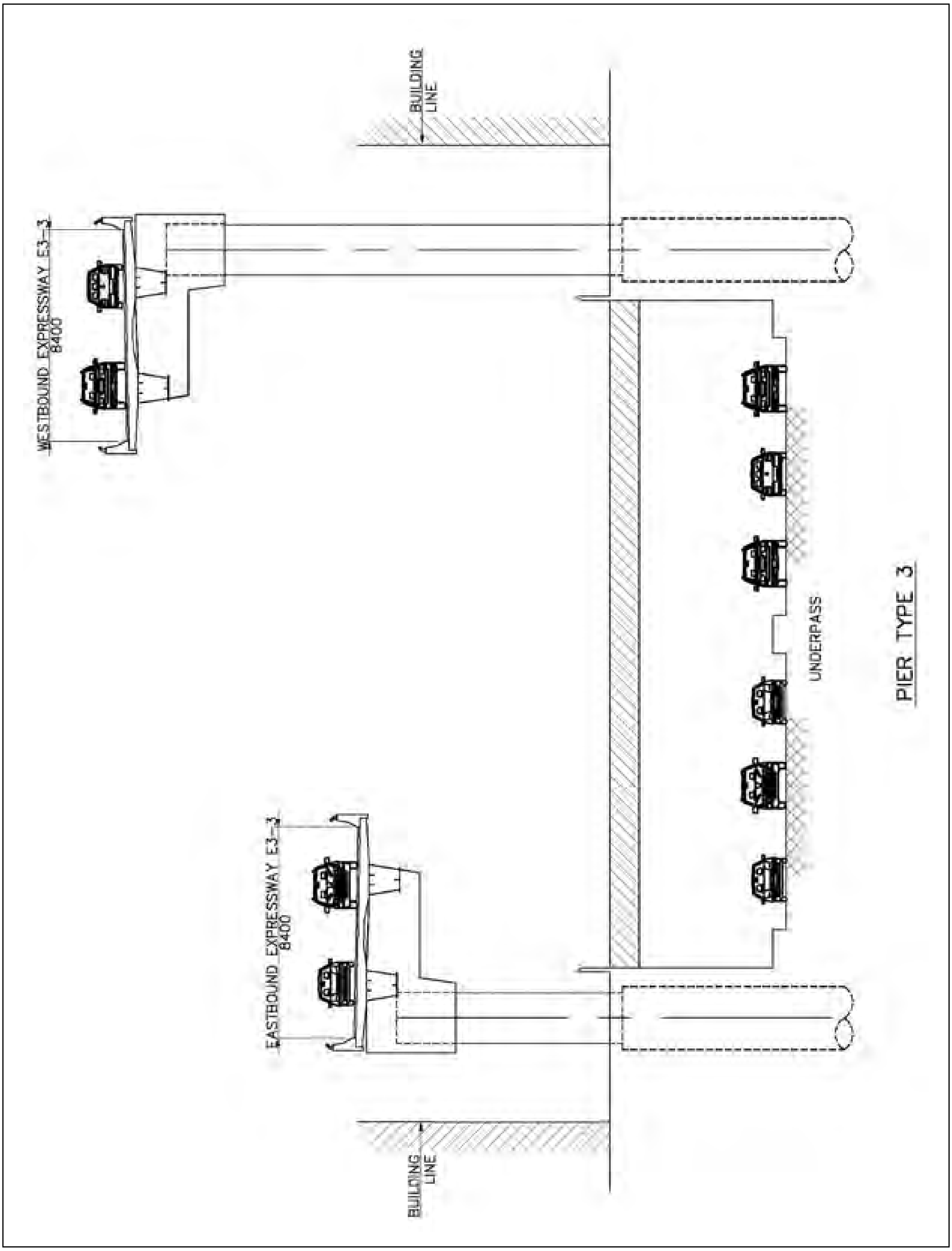


Figure 8.9-10 Section West of Nile – Pier Type 3

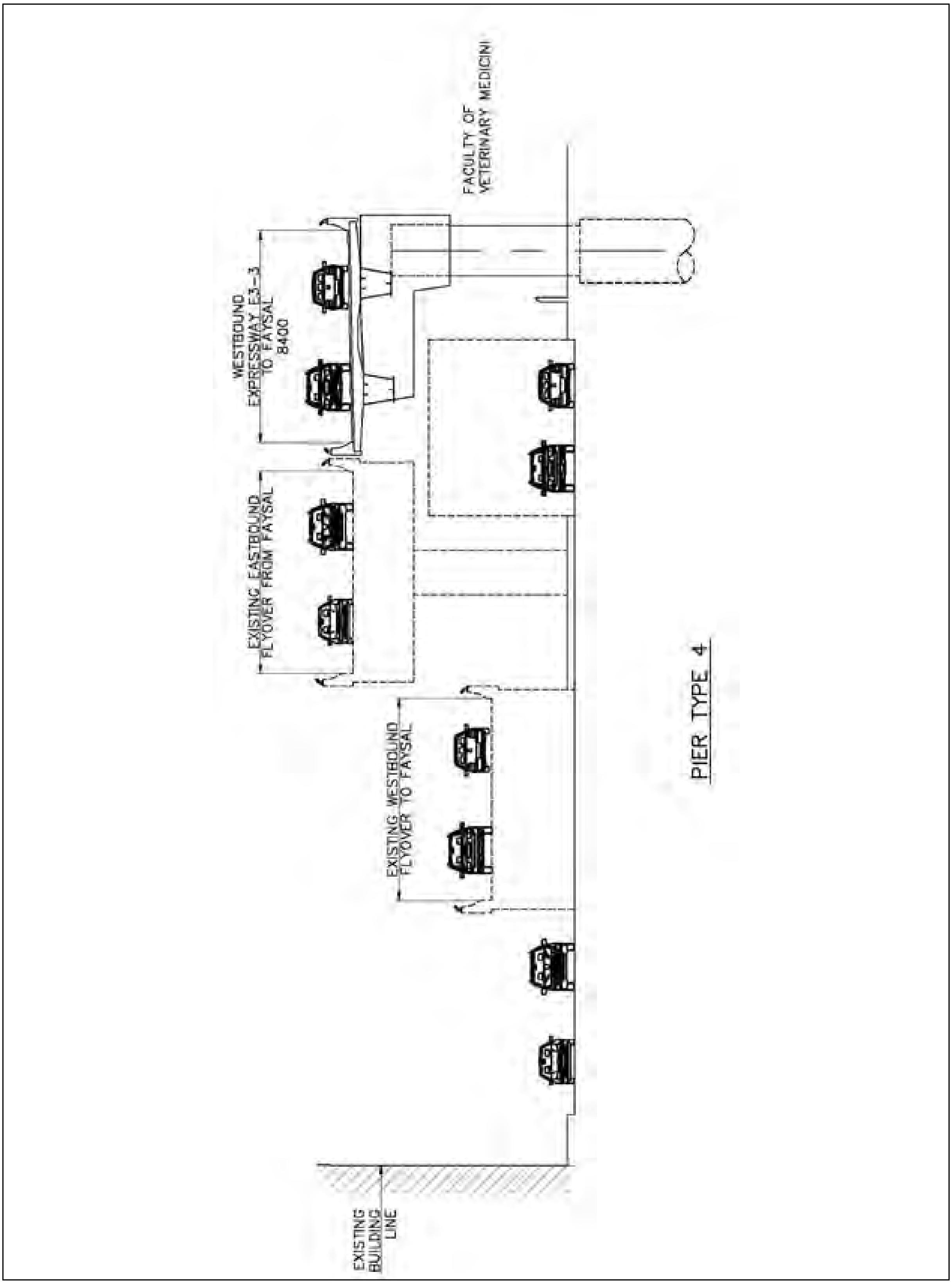


Figure 8.9-11 Section West of Nile – Pier Type 4

8.10 IMPACT ON EXISTING SECTIONS: E1-1 and E2-1

8.10.1 General

A further major task of the Study for E1-1 and E2-1 is to analyze methods of applying toll and to evaluate the impact of both existing and newly operated sections when extensions and/or new route are opened.

The following aspects are to be addressed:

- Widening of 6TH October Bridge to accommodate E1-1
- Widening of 15TH May Bridge to accommodate E2-1
- Toll plaza layout on E1-1 on 6TH October Bridge
- Toll plaza layout on E2-1 above 26TH July Street

8.10.2 Widening of Existing Bridges

(1) 6TH October Bridge: E1-1

The section of 6TH October bridge that runs across the Nile and Gazirah Island before connecting to the proposed E1-2 Expressway will require widening. The widening is required in order to accommodate both the proposed expressway traffic lanes (4-lanes) and local traffic lanes (6 lanes) i.e. 5 lanes in each direction.

The current capacity of 6th October Bridge at this location is 4-lanes per direction, therefore an additional lane is required on each side. In addition, the expressway lanes have to be separated from the local traffic with longitudinal barriers and also the Eastbound and Westbound expressway traffic lanes must be divided with a barrier.

Options available are:

Option 1: Strengthen the existing bridge to carry the additional traffic lanes and provide separate provision for pedestrians.
Provision for pedestrians can either take the form of:

- EITHER an independent structure or structures located adjacent to the existing bridge
- OR structures supported by the existing bridge with additional strengthening incorporated to carry the extra loading.

Option 2: Construct an additional bridge or bridges to carry the additional lanes.

The philosophy adopted in the Study is that additional bridges for highway traffic across the Nile should not, in the first instance, be constructed at the E1 or E2 corridors. Additional highway bridges across the Nile should be constructed as necessary at sites that would better serve other parts of the network once this is constructed in the future. The Study has therefore focused on Option 1; strengthen the existing bridge to carry more traffic lanes. The Option 1 lane arrangements, together with additional structures to carry pedestrian traffic, are presented in Figure 8.10-1.

Strengthening of the existing bridge will entail:

- External longitudinal pre-stress cables installed on the main bridge box girders
- Transverse strengthening of the bridge deck cantilevers, possibly using epoxy bonded carbon fiber sheets applied transversely to the top surface of the concrete
- Bearing replacements

It is assumed that the existing bridge substructure has sufficient redundant capacity not to require any strengthening works. Should the substructure require strengthening, then it is likely that Option 1 would not be a cost effective solution.

(2) 15th May Bridge

The local traffic weaving movements on the existing 15th May Bridge at both Abu El Ela (Big Nile) and El-Bahr El-Aazam (Little Nile) preclude any options that would separate local and expressway traffic with barriers. Local traffic and expressway traffic have to share the same facility at and between these locations.

The requirements regarding widening of the existing bridge are therefore dependant on final lane provisions over the existing bridge. Again the existing bridges have a capacity of 4-lanes in each direction. Given the requirements to provide access for local traffic to these bridges from side ramps, it is envisaged that an additional 2-lanes in each direction are likely to be required.

Widening options are in principle the same as for 6th October Bridge (E1-2) if only one (1) additional lane is required in each direction. However, the required width of trafficable deck, for an additional 2 lanes in each direction, will be wider than that provided by the existing bridges between railings. Therefore, additional structures to carry traffic loading at these locations are unavoidable if an additional 2 lanes are required in each direction.

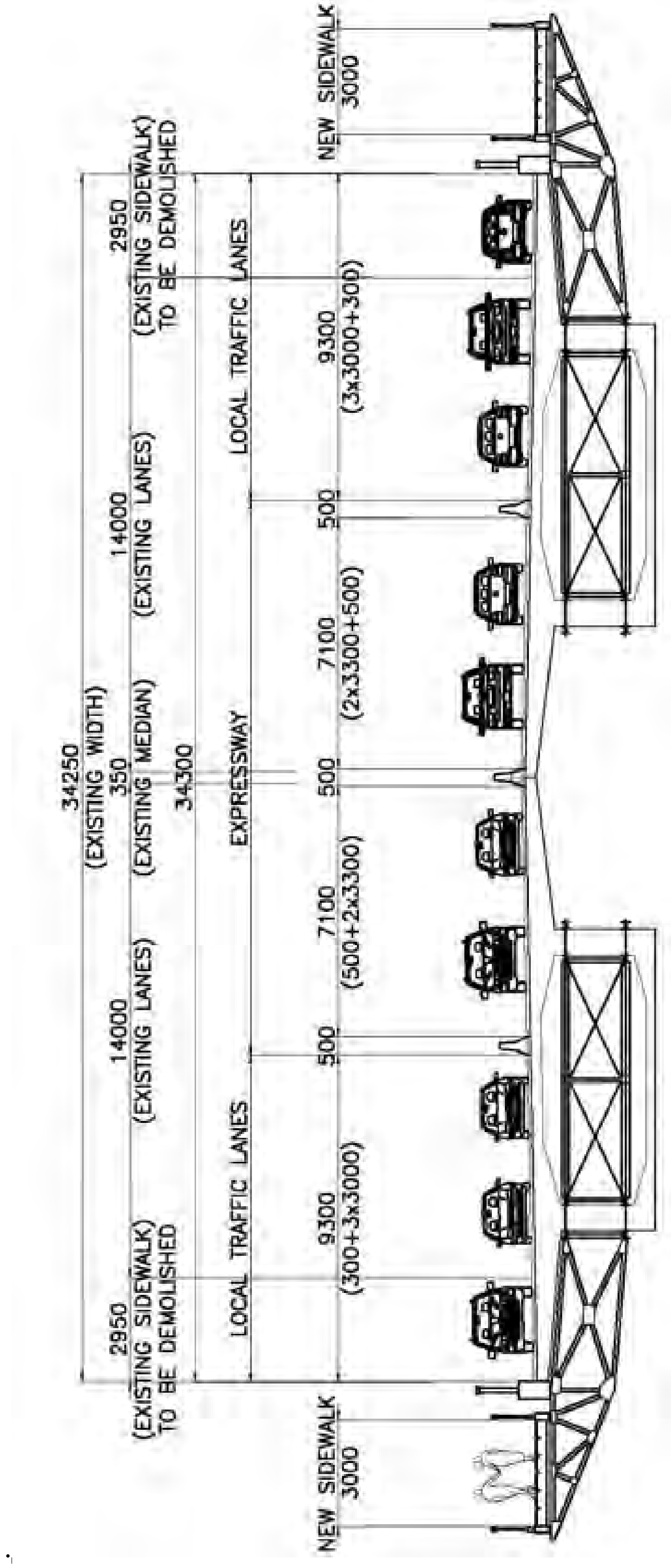


Figure 8.10-1 Widening of 6th October Bridge – Concept Scheme:E1-1

8.10.3 Toll Plaza Layouts

Toll plazas are proposed at the following locations:

- 15th May Bridge (E2-1) : At a location above the existing car park near Sphinx Square
- 6th October Bridge (E1-1) : At a location above the existing car park at Midan Abd Al-Munim Riyad.

The toll plazas are envisaged to accommodate five (5) toll booths in each direction and therefore will extend each side of the existing decks across the entire car parking area.

Given that the propose toll plaza structures are located above car parking facilities, there will be some flexibility with regard to form of construction. However, it is anticipated that prefabricated beam and slab decks will be the preferred solution at each location.

Refer to Figure 8.10-2 for a location plan of the proposed toll booth near Sphinx Square for E2-1.

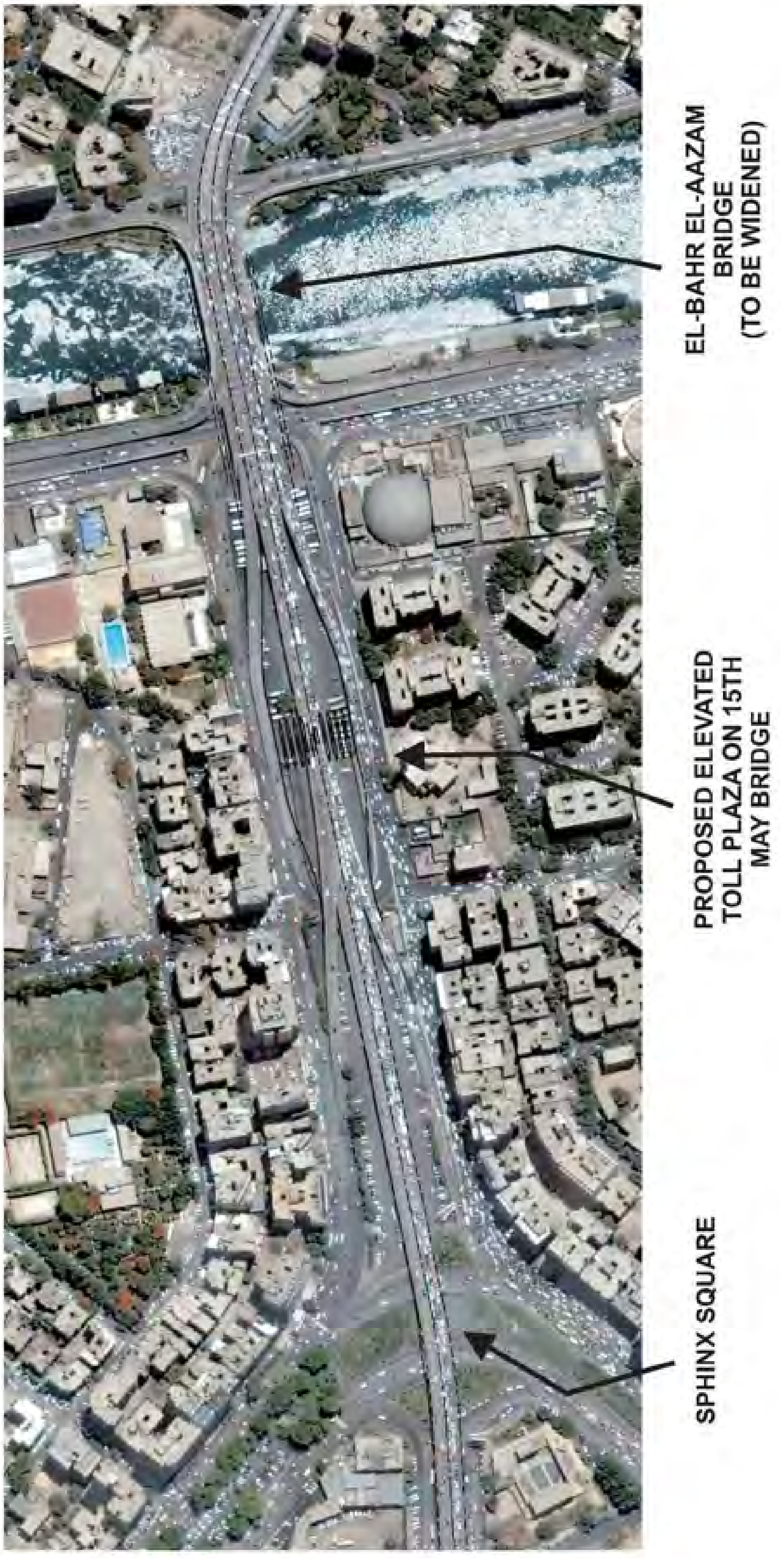


Figure 8.10-2 Proposed Location of the Elevated Toll Plaza on 15th May Bridge near Sphinx Square: E2-1

8.11 APPLICABILITY OF STEP LOAN CONDITIONS TO FS STUDY SECTIONS

(1) STEP Loans

The Government of Japan introduced "Special Terms for Economic Partnership" (STEP) in July 2002. Refer to Appendix for the Terms and Conditions of STEP Loans.

(2) Satisfying STEP Loan Requirements

STEP Loan requirements are as follows:

STEP Loan Requirements

(1) Procurement Conditions

- Prime contractors must be Japanese firms. Joint ventures (JV) with the firms incorporated and registered in recipient countries are also allowed to be a prime contractor under condition that a Japanese firm is a lead partner.
- Sub-contractors are untied and may be from any country.

(2) Country of Origin of Goods and Services Procured under STEP (Japan Origin)

- Not less than 30% of the total amount of contract(s) (except consulting services) financed by STEP loan must be accounted for by either (a) goods from Japan and services provided by Japanese firms, or (b) goods from Japan only, according to nature of the project.
- Goods procured from a manufacturing firm of the recipient country invested in by one or more Japanese firms will be regarded as goods procured from Japan, if they meet the following:
  a) Not less than 10% of the shares of the manufacturing firm of the recipient country are held by a Japanese firm; and
  b) The proportion of the shares held by the Japanese firm is the same as or greater than that of the shares held by any firm of a third country or region.
- Goods procured from a manufacturing firm in an approved developing country other than the recipient country invested in by one or more Japanese firms will be regarded as goods procured from Japan, if they meet the following:
  a) Not less than one third of the shares of the manufacturing firm are held by a Japanese firm; and
  b) The proportion of the shares held by the Japanese firm is the same as or greater than that of the shares held by any firm of a country or region other than Japan and the country or territory where the manufacturer is located.

In the case of projects which advanced technologies and/or know-how of Japanese firms can be identified in Services (e.g. construction methods) not only Goods but also Services may be included in the ratio of contract amount mentioned.

(3) F/S Sections and eligible projects suitable for STEP Loans

Sections E1-2 and E2-2 are clear candidates for satisfying STEP Loan requirements given that advanced (shield-tunnel) construction technology is involved and extensive steel structures, including double deck viaducts, are envisaged.

Section E3-1 is not recommended for STEP Loan given that materials can substantially be procured locally and local contractors are capable of executing the work.

The following components of the F/S Sections are identified as suitable to satisfy the requirements of a STEP Loan Scheme:

Table 8.11-1 Prospective Japan Origin – Goods and Services – F/S Section E1-2

FS Section	Item	Component	Condition
E1-2	Shield Tunnel	Advanced construction methodology	From Japan
		Tunnel Lining	Technology from Japan and Product of EGYPT – Japan JV Firm
		Services	From Japan
	ESA Tunnel	Advanced construction methodology	From Japan
		Tunnel Lining	Technology from Japan and Product of EGYPT – Japan JV Firm
		Services	From Japan
	Steel Girder	Material	From Japan
		Fabrication	By EGYPT – Japan JV Firm
		Services	From Japan
	Concrete Bridge	PC tendons and Anchor	From Japan
		Cement with admixture	Admixture may be blended with locally produced cement by EGYPT – Japan JV Firm
		Services	From Japan
	Precast PC Deck Slab	Unit	Produced by EGYPT – Japan JV Firm
	Composite Pier Column	Steel Casing	From Japan or Product of EGYPT – Japan JV Firm
	Large Diameter Bored Pile	Construction Equipment	From Japan
		Pile top steel casing (5~6m)	From Japan or Product of EGYPT – Japan JV Firm
		Services	From Japan
	Miscellaneous Bridge Components	Bearings	From Japan or Product of EGYPT – Japan JV Firm
		Expansion Joint	From Japan or Product of EGYPT – Japan JV Firm
	Mechanically Stabilized Earth Wall	Strip	From Japan
		Concrete Panel	Product of EGYPT – Japan JV Firm

Table 8.11-2 Prospective Japan Origin – Goods and Services – F/S Section E2-2

FS Section	Item	Component	Condition
E2-2	Steel Girder	Material	From Japan
		Fabrication	By EGYPT – Japan JV Firm
		Services	From Japan
	Steel Pier	Material	From Japan
		Fabrication	By EGYPT – Japan JV Firm
		Services	From Japan
	Precast PC Deck Slab	Unit	Produced by EGYPT – Japan JV Firm
	Composite Pier Column	Steel Casing	From Japan or Product of EGYPT – Japan JV Firm
	Integration of Viaduct Foundations with Metro Line 3	Construction Methodology	From Japan
		Services	From Japan
	Large Diameter Bored Pile	Construction Equipment	From Japan
		Pile top steel casing (5~6m)	From Japan or Product of EGYPT – Japan JV Firm
		Services	From Japan
	Miscellaneous Bridge Components	Bearings	From Japan or Product of EGYPT – Japan JV Firm
		Expansion Joint	From Japan or Product of EGYPT – Japan JV Firm

In addition to the physical components shown above, Japanese traffic related systems such as “Traffic Control and Information System” and “Electronic Toll Collection System” can be effectively considered under STEP scheme as well.

(4) Pre-F/S Sections and eligible projects suitable for STEP Loans

Notwithstanding the more limited depth of study undertaken for the Pre-F/S Sections E3-2 and E3-3, the following general observations are made regarding suitability of applying STEP Loan Schemes to these sections.

Section E3-2 makes use of relatively standard construction technology given the structural concepts envisaged at this stage i.e. cut and cover tunnels, approach embankments and short to medium span conventional viaduct sections. Several sections of the viaduct are envisaged to be in structural steel, both at transition sections where the viaduct crosses above El Nasr Road and where the expressway occupies the space above the existing ENR railway. However, the amount of structural steel envisaged (1.75km of steel deck viaduct out of a total construction length of 6.9km) may not be sufficient to justify the use of a STEP Loan for this section.

Section E3-3 is a good candidate for satisfying STEP Loan requirements given that much of the length of the section is envisaged to take the form of either double deck steel structures or multi-level steel deck flyovers supported on long span steel portals or composite columns. In addition the major structure proposed to span the Nile above the existing Giza Bridge, illustrated in this Study with a cable-stayed bridge concept, is a prime candidate for the application of advanced Japanese construction technology and materials procured from Japan.

More detailed study however is required before firm recommendations can be made regarding the suitability of applying STEP Loan Schemes to Section E3-2 and Section E3-3.