Elevated viaduct on Delhi MRTS phase I - Salient design features

M. Kumar, A. Srivastava and N. Gupta

In phase I of the Delhi MRTS project, the stretch between Yamuna west bank to Rithala is on elevated viaduct. The paper describes the salient design features of the viaduct including innovative planning and standardisation measures adopted. Aesthetically pleasing appearance and minimum disturbance to traffic were important criteria during the design.

In Phase I of the Delhi MRTS project, the stretch between Yamuna west bank to Rithala runs on elevated viaduct. This is a densely populated area with heavy traffic. The alignment is characterised by sharp curves at many locations as it passes through some of the most congested and crowded areas of the city.

Project requirement.

An important objective of the project included fast track construction utilising as little space as possible on the road with minimal inconvenience to traffic and people. As the project is situated amidst urban area, the aesthetics of the structure was given highest consideration. The construction was carried out in an environmental-friendly manner.

Superstructure

Standard spans

To achieve faster construction with superior quality control, standardisation was attempted to the maximum extent possible. A major part of the stretch was conceived as simply supported post-tensioned box girders with spans varying between 21.6 m to 29.1 m and construction carried out by precast segmental technique using internal cables with epoxybonded joints. These spans are termed as standard spans, Figure 1. The running segments (S2 to S6) were 2.5 m long while diaphragm segment (S1) was 2.025 m long. By taking out some of the segments from the middle section of 29.1 m span, the span lengths of 26.6 in, 24.1 m and 21.6 m were obtained. The box girder section was adopted due to its superior torsional properties. Its depth and profile was decided from aesthetic considerations.

Cross-section of the box.

All the segments have the same constant depth with identical external profile. Variation in the structural thickness was effected on the inside portion of the box. This helped in standardisation of external formwork. A shaped box girder having external profile with curves was provided, vastly improving the aesthetics by imparting 'sleekness', Figure 2. Fluted texture was provided on the external face of parapet to break the monotony of the large plain surface.

For drainage purposes a cross slope of 2.5 percent was provided towards the inner side of deck. This was -provided to ensure that the drainage system was not visible on the outside. Inlet boxes along with removable grating were provided to collect the run-off from the deck. The removable grating was provided to screen out larger debris. The runner pipe provided inside the box would collect the run-off and drain it to the down-take pipe embedded in pier to be drained at the ground level.

Shear keys were provided in segments at end face, Figure 3. The shear keys are unreinforced small size projections having a trapezoidal shape in longitudinal view. They were limited to the internal part of each web for aesthetic reasons. The keys in the deck slab helped in the positioning of each new segment and transferring of local shear forces, thus ensuring continuous behaviour across the joints under concentrated traffic loads.

Prestressing cables

All permanent prestressing tendons were of 19K15 cables. For facilitating standardisation of segments, the cables were so profiled that they draped down/up only in the last four segments but followed a horizontal profile in the middle four

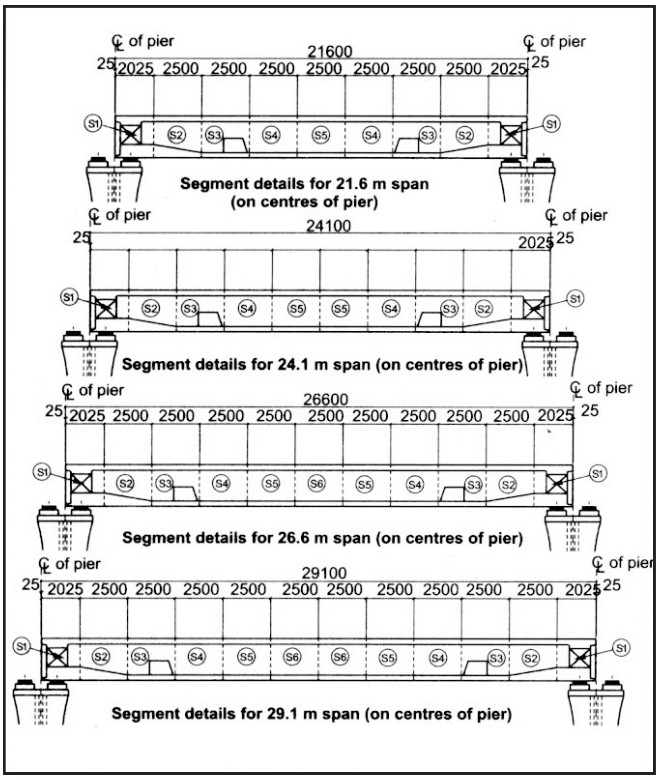

Figure 1. Segment layout of different standard spans

segments, Figure 4. Cable profiling was done in a way that no blister block was required anywhere. However, blister block was provided in segment S3 for future (external) prestressing.

Construction methodology

The segments were precast using short-line match casting for straight alignment and long-line match casting for curved alignment. The precast segments were manufactured in a centralised large casting yard located close to the viaduct site. The maximum weight of each segment was about 50 t and these were transported from casting yard to their respective location by means of multi-axle low-bedded trailers.

The construction was done sequentially, span-by-span, starting at one end of a continuous stretch and finishing at the other end. All the segments of the span were temporarily suspended from the overhead launching girder or rested on top of under-slung truss depending on the type of launching system until about 2/3 of permanent prestressing was mobilised and thereafter the assembled span was gradually lowered onto the temporary bearings and the balance stressing work was completed.

Temporary prestressing was required at the erection stage during assembly of the glued match-cast segment at site, Figure 4. Macalloy high tensile steel prestressing bars were used for the purpose. Generally, they were anchored on temporary steel frame over the deck slab and temporary concrete bracket from soffit slab. The forces and the number of Macalloy bars were adjusted to have a minimum of 0.3 MPa axial prestress across the joint.

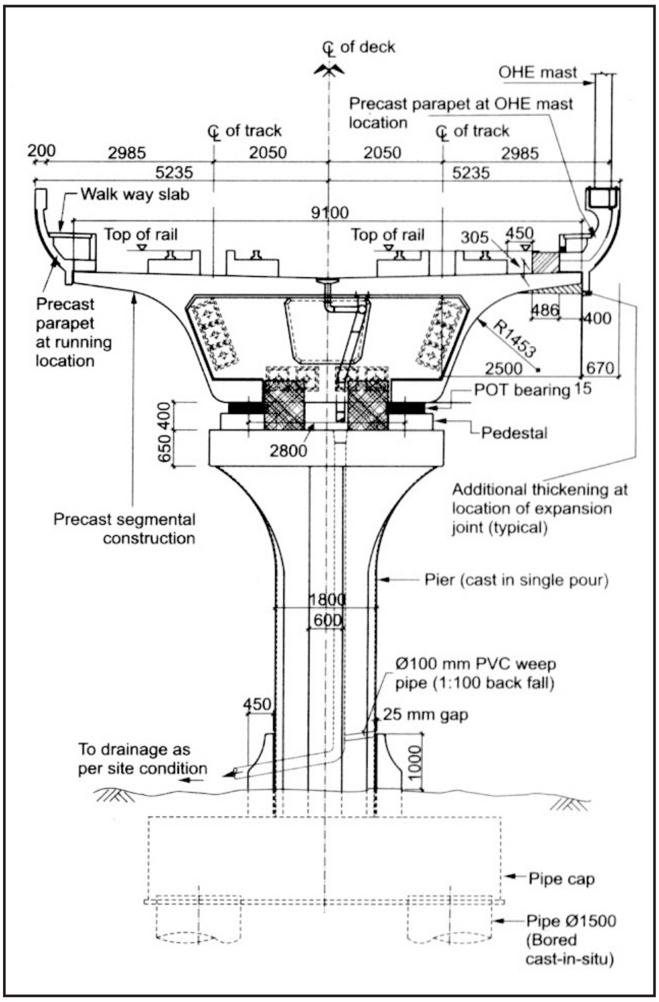

Figure 2. Typical cross section of a viaduct

The major steps in erection were as given below:

1. The individual segment at the site was lifted and rotated by 90^0 using erection gantry. From the erection gantry, the segment was lowered on the trolley placed over launching truss. The segment supported over the trolley was then taken to its final position.

2. Longitudinal, transverse and vertical alignment of all the segments of span was accomplished including dry matching.

3. After this, the segments were separated and epoxy glue was applied before temporary prestressing.

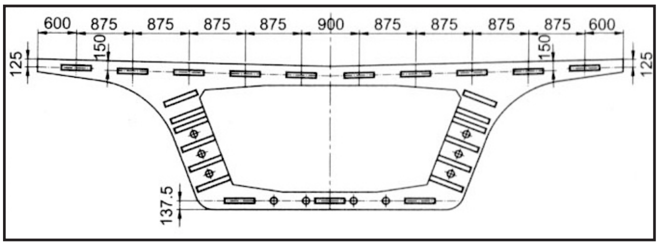

Figure 3. Shear key arrangement in the box girder

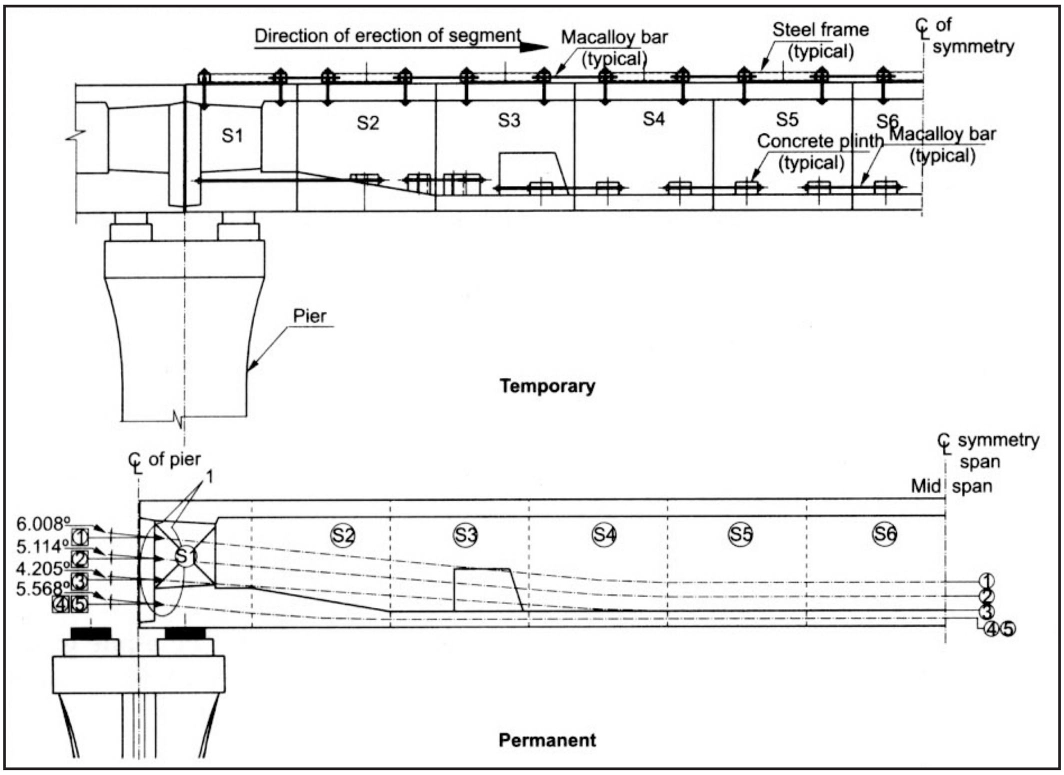

Figure 4. Prestressing arrangement for standard span

Portals

At certain locations, the superstructure alignment is such that the girder could not be supported over pier at the median because the centre line of the girder was far from it. At these locations, superstructure was supported over a simply supported girder provided transverse to it. This supporting structure, termed as portal, was in turn supported over two piers provided at either side of road, Figure 7.

The portals were constructed using precast segmental technique in almost the same way as the standard girders.

4. Temporary bearings were installed.
5. Post-tensioned cables were threaded in the assembled unit of segments.
6. The first stage prestressing was done from one end and then the unit was lowered over temporary bearings.
7. The second stage prestressing was then applied and the launching girder was shifted forward.
8. Temporary bearings were replaced with permanent bearings.

Non-standard spans

At road/railway crossings, generally three-span prestressed concrete continuous units were provided with the central span ranging from 30 m to 60 m, Figure 5, suiting the site conditions. Continuous units were cast on ground-supported staging as well as using cantilever construction equipment. The equipment moved till the tip of cantilever for casting of insitu successive segments. Prestressing cables were introduced with the erection of each segment and finally when the two cantilevers from neighbouring piers met, the stitch segment was cast and integration cables were stressed, Figure 6.

The construction of these units was planned such that the traffic below would remain undisturbed. Keeping in mind the importance of aesthetics, the box depth for the end span was kept in consonance with the standard spans, Figure 5, at their intersection.

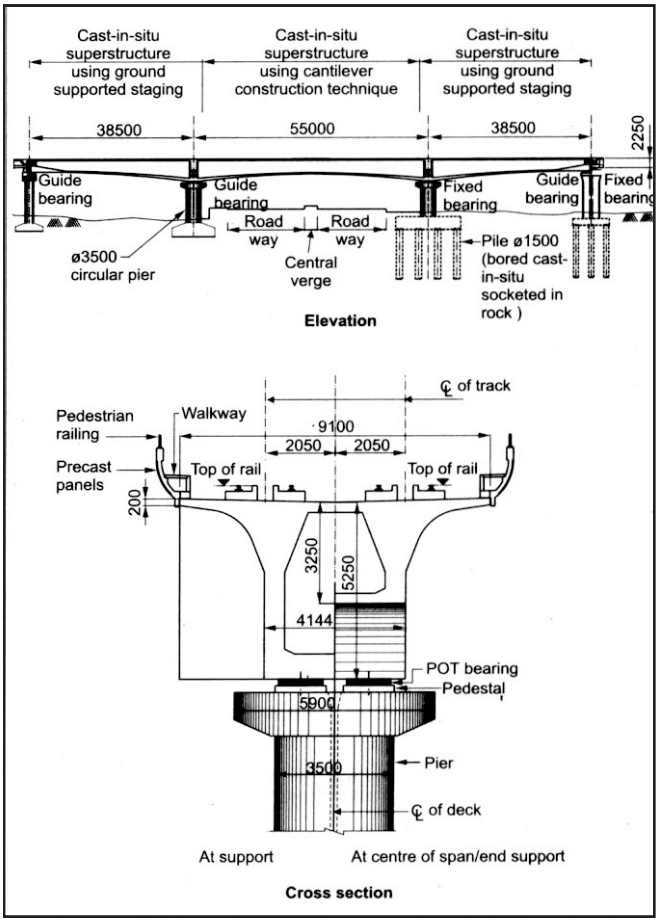

Figure 5. Typical details of a continuous unit

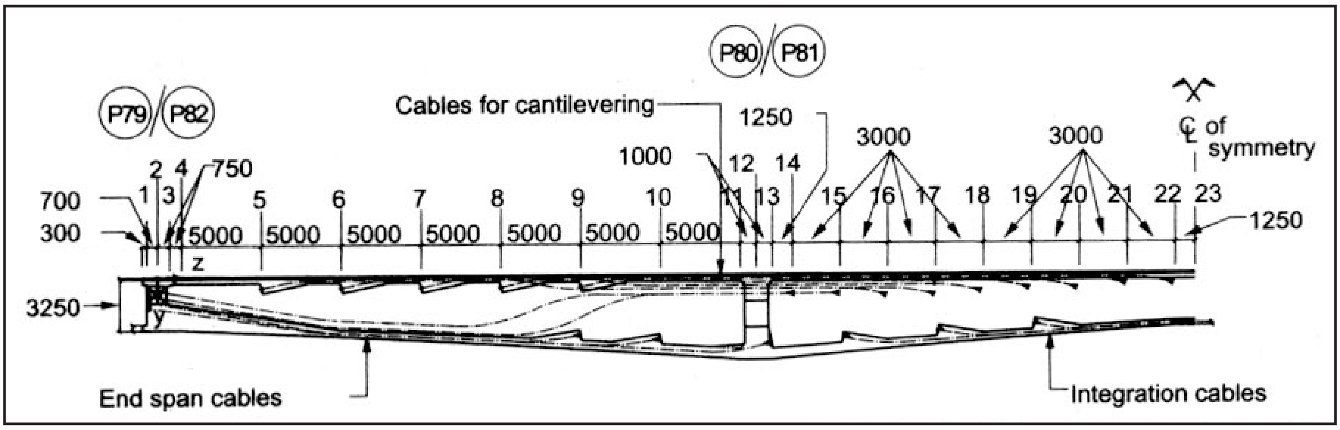

Figure 6. Typical prestressing profile for a continuous unit

Joints, substructure and seismic restraint devices

Bearings and expansion joints

POT-PTFE bearings were used for the project taking into account the seismic forces they may be subjected to. The bearings were designed to cater for seismic forces in the direction of traffic as well as transverse to it. In addition, longitudinal forces due to braking/traction and centrifugal force in the transverse direction were catered to. For standard spans, the vertical load on bearing was in the range of 300 t to 450 t while the horizontal force on fixed bearing was in the range of 45 t to 75 t. For continuous units, the loads were much higher for bearings at central pier. For one of such unit, vertical load was 2850 t with associated horizontal force of 430 t.

Generally, single strip seal expansion joints were provided for the project.

Pile foundations

The substructure was supported on castin-situ bored piles. The pile layout was decided keeping in view the utilities at ground level and below (such as water and sewerage pipe lines, electric cables, telecommunication lines, etc) which required close co-ordination with site personnel and improvements/changes in designs while the work at site was in progress. For standard spans, usually a nine-pile group having 1.2 m diameter or a six-pile group having 1.5 diameter was provided, Figure 8. The 6-pile group was aligned along the median to minimise the obstruction at road.

Well foundations

For intermediate piers of some continuous units, well foundations were adopted. This was necessitated by space constraints at these locations for the required pile group or due to liquefaction potential of soil during earthquake.

Open foundations

At some locations, a firm rocky stratum was available at shallow depth. Taking advantage of this, open foundation was provided at these locations. It was ensured that the area in contact under the worst load combination is not less than 80 percent of the total area. Thickness of open footing was in the range of 2.1 m to 2.5 m.

Piers

The pier cross-section was designed to be rectangular, flaring at top to support the bearings under the web of box. As a

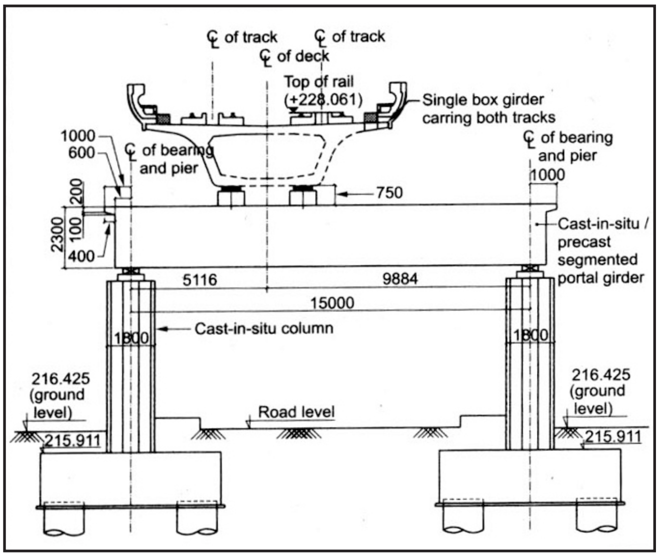

Figure 7. Elevation showing details of a portal

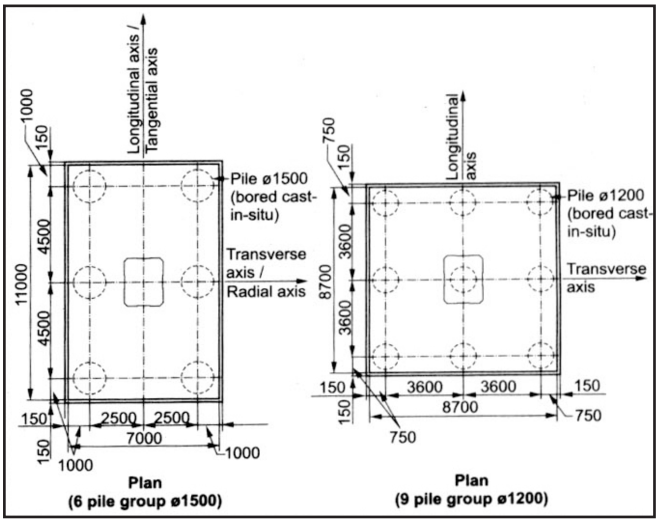

Figure 8. Typical standard pile layout

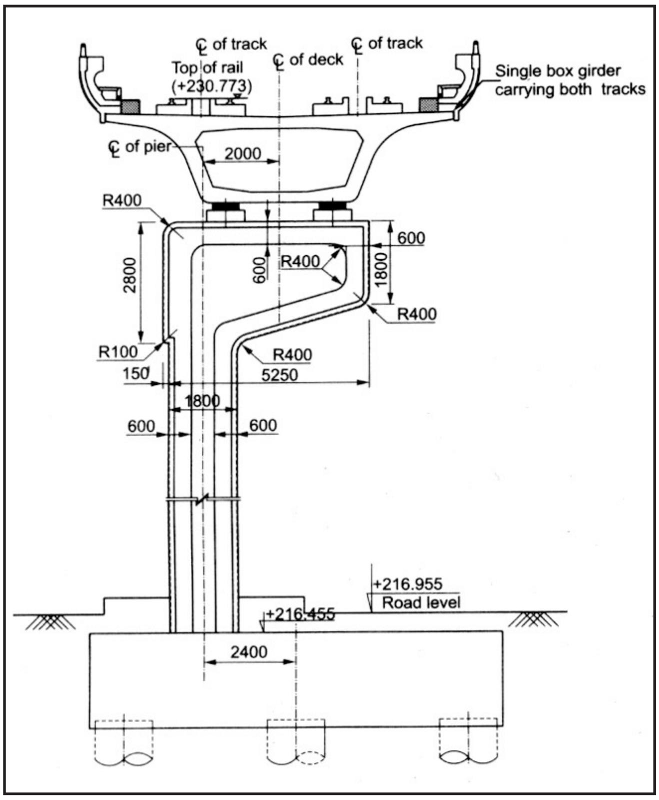

Figure 9. Typical arrangement of L-shaped pier

result, the piers have a sleek cross-section with a rounded bell-shaped profile at top. This matched with the profile of the girder at the top giving a good aesthetic appearance. The pier size was finalised by estimating the minimum dimensions required to satisfy structural requirements. The size of the cap in the transverse direction was finalised to match the soffit width of superstructure. A recess of 20 mm was provided at all the surfaces in the central zone to break the monotony of plain concrete surface.

This shape of the pier was retained throughout the alignment except where unavoidable. The longer dimension of the pier was aligned along the median to minimise the obstruction at road after the structure would be put into service.

To avoid construction joints, the piers along with the cap were cast in-place using rigid steel formwork in a single pour. Tie bolts through concrete were not permitted.

At locations, where the superstructure is eccentric to pier position at the median by not more than 2.0 in, L-shaped piers were provided in place of portals, Figure 9.

Seismic restraint devices

Although the bearings were designed to cater to the seismic forces, concrete stoppers were provided which would act as seismic restraints over column as a second line of defence.

These are intended to prevent dislodgement of superstructure from column during severe earthquake, Figure 10.

Design parameters

Geometric parameters

The geometric parameters of the structure are given below.

1. Overall width of deck: 9.10 m

2. Single box girder with inclined webs for supporting two tracks

3. Superstructure gradient: variable

4. Horizontal alignment of superstructure comprises of straight, curved and transition stretches. The minimum radius of curvature is 300 m.

5. Centre to centre of track: 4.10 in

6. Ballastless track

7. Differential settlement: 12 mm with long-term value of E for concrete

Vertical loads

Carriageway live load

After studying the expected loading from outline design criteria of rolling stock to be adopted on Delhi Metro, an equivalent loading as 70 percent of Indian Railway modified broad gauge (MBG) loading (as per IRS: Bridge Rules) was adopted.

This has now been proposed to be modified to lighter loading namely modified rolling stock loading, as the design of rolling stock has been firmed up by the manufacturer. Design checks were performed for one track loaded case as well as both the tracks loaded case. The appropriate impact factors were applied as per IRS Bridge Rules.

Superimposed dead load

It is advantageous to have as little superimposed dead load (SIDL) as possible. This was achieved by adopting ballastless track on viaducts. Total SIDI. on viaduct worked out to be 7.2 t/m including concrete plinth under the rails, rail and rail fastening features, parapet, services and walkways.

Longitudinal loads

Braking and tractive forces

Braking and tractive effort from loco were adopted based on the Calcutta Metro design criteria.

Force due to long-welded rails

The load effects due to the difference in thermal response of the superstructure and the track that consists of continuous long-welded rails were adequately considered in the design.

Transverse load

For spans in curved alignment, centrifugal forces were taken in to account. The radius of curvature is as sharp as 300 m at many locations. Centrifugal force was assumed to be acting at the centre of gravity level of live load. It therefore has significant effect on the substructure and bearing designs.

Seismic force

The seismic forces were evaluated based on the seismic coefficient method. The equivalent seismic force, F_{eq}, is given below:

$$F_{eq} = \alpha\beta y G$$

$$= 0.126\,G$$

where,

α = 0.07 (Equivalent seismic coefficient as per Indian meteorological department recommendations for MRTS)

β = 1.2 (Foundation factor)

y = 1.5 (Importance factor)

G = mass

Temperature effect

For estimation of movement for bearings and expansion joints, the temperature is considered ± 25^0 C For longitudinal flexure, a linear gradient between top and bottom fibre of girder was considered. This effect is significant only for the continuous unit. In addition, the temperature difference between the exterior and interior of the box girder was taken into account while carrying out transverse analysis.

Load combinations

Structural design for various elements was carried out for ultimate limit state combinations as well as serviceability limit state combinations as defined in Indian Railway Standard: Concrete Bridge Code-1997.

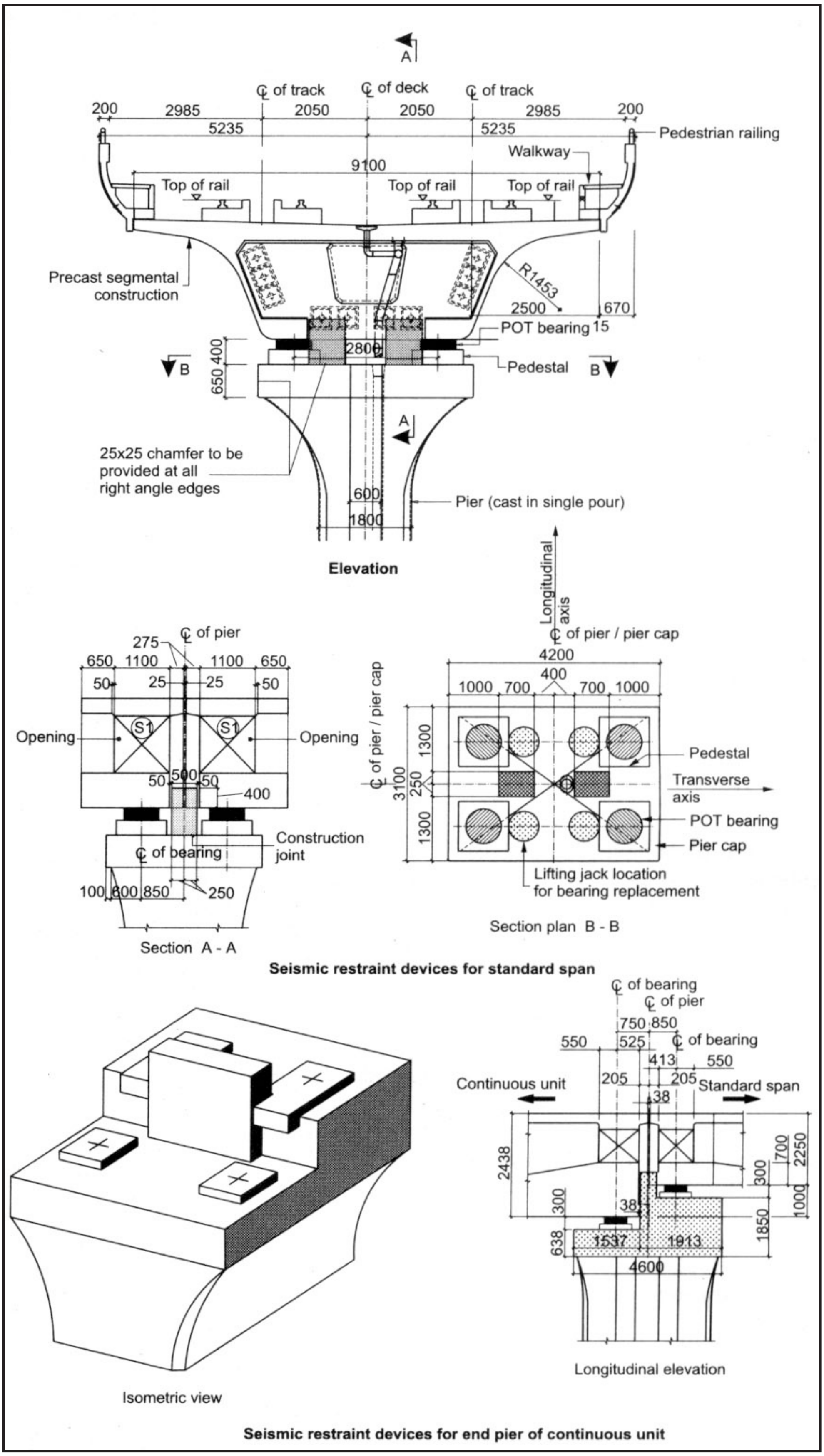

Figure 10. Seismic restraint devices

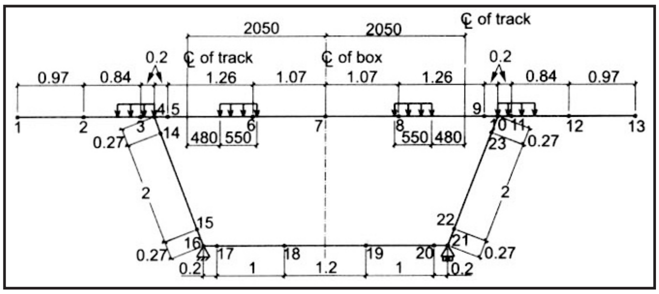

Figure 11. Model for transverse analysis of box girder

Salient design features

Superstructure

Longitudinal analysis

Prestressing was done longitudinally whereas transversely the superstructure was designed as a reinforced concrete element. For continuous units, design checks were conducted for each construction stage with appropriate prestress.

Strength check for flexure as well as shear in combination with torsion was done for ultimate limit state in accordance with IRS Concrete Bridge Code -1997 provisions.

Transverse analysis

The box girder section was analysed as close frame supported below web points at soffit level, Figure 11. Since IRS Bridge Rules do not specify any methodology for dispersion of live load through unballasted deck, the concentrated load due to axle load was assumed to be distributed on an equivalent width in accordance with IRC: 21.

At OHE mast location, thickness of parapet at running location was increased inside without disturbing the visible external profile for supporting OHE mast, Figure 2. Cantilever flange of deck slab at these locations was strengthened by providing higher reinforcement.

The diaphragm section was designed as a member with indirect support, that is, the support is outside the plane of loading because the web centreline is eccentric to bearing location. The diaphragm reinforcement for top tension was calculated using strut-tie model. The reinforcement so obtained was validated using finite element technique. Suspension reinforcement in the form of vertical stirrups was provided to transfer the shear force from the web to diaphragm, as the webs are not directly supported over the bearings. The design was carried out for service stage as well as bearing replacement condition.

The intermediate diaphragm (not at the bearing location) was not required to resist transverse bending as the box cross-section has considerable stiffness. However, they were provided to resist the deviation forces from soffit slab at locations wherever there was a kink in the soffit profile.

Diaphragms were also designed as corbels for prestressing forces wherever cables -permanent as well as future — are anchored to them.

Blister blocks were provided in standard spans for future prestressing only whereas they were provided for anchoring permanent cables too in continuous units. The local splitting tensile stresses behind prestressing anchorages were catered to by providing untensioned steel behind it. The blister location was chosen to be at the intersection of flange and web where the transverse section has the largest rigidity. At locations, where it was not possible to do so, full width blisters spanning from web to web were provided. In addition to tie back reinforcement, longitudinal local flexural and transverse bending in the flange was estimated using FEM modelling and additional reinforcement provided for these effects. To prevent separation of blister from superstructure due to shear and deviation forces, closely spaced vertical stirrups were provided considering shear friction mechanism of force transfer.

Deviator blocks have been provided in the soffit to effect change in angle of future prestressing cables. Reinforcement has been provided to stitch them with the soffit of the superstructure due to deviation force. In addition, the soffit slab has been designed for transverse bending due to this effect.

Substructure

Pier

The piers were grouped based on design loads obtained at design sections depending on geometric parameters and span configuration. Standardisation was achieved for pier design by providing same reinforcement arrangement in the pier groups with modification in diameter of reinforcing bars. Detailing was done allowing for the space required for a

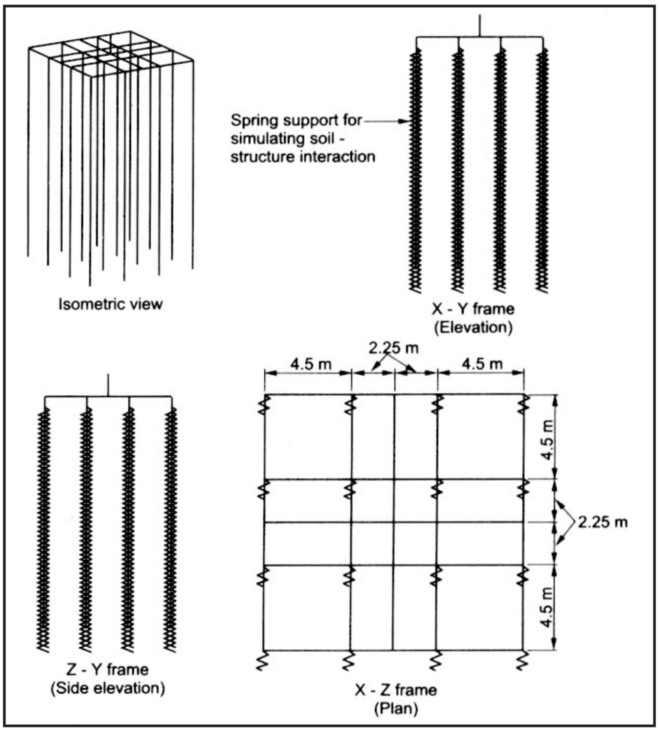

Figure 12. Pile group model for the (4 x 4) pile layout

workman to go inside the pier for compaction of concrete by a vibrator. As the superstructure is eccentric to the pier centre-line for L-shaped piers, these were designed for additional eccentricity of loads. For L-shaped piers, M 60 grade of concrete was used to control pier dimensions.

The pier caps were designed as corbels for the maximum bearing loads considering various load combinations. The pier caps were also designed for lifting of the superstructure during bearing replacement. For pier caps of continuous units, additional checks were done for construction stage loadings from superstructure. Seismic restraint devices were designed using shear friction theory.

Pile and pile cap

The pile foundation system was analysed considering the soil-structure interaction. The pile and pile group system was modelled as a space frame, Figure 12. The piles were connected to pile cap by rigid connection. Springs for horizontal forces have been provided at pile nodes with stiffness corresponding to soil property derived from bore hole data. At pile base, flexible supports were provided by springs corresponding to soil stiffness with respect to vertical loading. This value was checked against the values obtained from pile load test data at site. Liquefaction potential of soil was accounted for in the structural design of piles. The pile cap members were designed as flexural elements for forces arrived by 3-D frame analysis.

Pile foundations were also grouped as some standard types. At some locations, non-standard pile groups were provided due to site requirements. Pile cap thickness was kept in the range of 2000 mm to 3000 mm, based on pile configuration.

Concluding remarks

The Delhi Metro Rail project entailed large-scale construction with severe constraints, of both time as well as space. Being located in a highly urbanised and congested area, construction was required to be done in a swift and least obtrusive manner. Meticulous planning was required to meet all these objectives. Standardisation in designs and construction was attempted to the maximum extent possible. However, due to site constraints modifications or improvisation were required in substructure and foundation designs at various locations. Aesthetically pleasing external profiles were provided to the superstructure and substructure without compromising on functional and structural requirements. It is hoped that mass transit systems can be developed on similar lines in other cities with high density of population.

Mr Madhuresh Kumar is the chief engineer (planning) of the Delhi Metro Rail Corporation. He is an engineer of the 1977 batch of the Indian Railway Service. He has vast experience in construction of bridges, structures and maintenance of high-speed and heavy-density track on the Railways and is associated with execution of many construction projects of the Railways.

Mr Ashish Srivastava is a senior design engineer in Tandon Consultants Pvt Ltd, New Delhi. His area of interest is design of bridges and special structures. He did his masters in civil engineering with specialisation in computer aided design from the Indian Institute of Technology (IIT), Roorkee.

Mr Navneet Gupta is a senior design engineer in Tandon Consultants Pvt Ltd, New Delhi. His area of interest is in design of bridges and special structures. He did his masters in civil engineering with specialisation in building science and technology from IIT, Roorkee.

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