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Design of the Dubai Metro light rail viaducts—substructure

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The Dubai Metro light rail scheme is a flagship project in the United Arab Emirates which is currently one of the largest civil engineering projects under construction and, when completed, will be the longest fully automated rail system in the world. The first section of the rail system is due to be opened in September 2009. This paper describes the scheme outline and contractual set-up for the viaduct design and discusses the design and construction of the viaduct substructure. In particular, the design methodologies used for the piled foundations, single reinforced concrete columns and prestressed concrete pier heads are discussed as well as the design of elastomeric bearings that were used extensively for most of the viaduct spans. Seismic loading governed the design of many of the foundations and the seismic analysis and design methodology adopted is discussed, together with specific reinforcement detailing requirements. Rail–structure interaction analysis and design is also covered. Other critical design issues resolved included the fatigue performance of cranked reinforcement and the treatment of the onerous construction loading from overhead gantries used to erect the precast deck segments.

1. INTRODUCTION AND PROJECT BACKGROUND

In July 2005, the Dubai Road and Transport Authority (RTA) awarded a design-and-build contract to the Dubai Rapid Link (DURL) consortium for the construction of the first and second stages of the Dubai Metro Red and Green Lines. The DURL consortium comprised the Japanese companies Mitsubishi Heavy Industries, Mitsubishi Corporation, Obayashi Corporation and Kajima Corporation together with Yapi Merkezi of Turkey. Construction of the infrastructure and stations was the responsibility of a joint venture between Kajima, Obayashi and Yapi Merkesi (Japan–Turkey–Metro joint venture or JTMjv). JTMjv appointed Atkins as their designer in 2006. This project organisation is illustrated in Figure 1.

The Dubai Metro will be a driverless, fully automated metro network and will be the longest fully automated rail system in the world. Completion of the first section of the Red line is planned for September 2009, followed in 2010 with the first section of Green line. A further Blue line (along Emirates Road) and Purple line (an airport express route) are planned for subsequent years. The metro route map is illustrated in Figure 2, with a more detailed Red line route map shown in Figure 3.

The £1·5 billion, 52 km long Red line connecting Rashidiya to Jebel Ali port comprises 42 km of elevated viaduct with 22 overground stations, 5·5 km of tunnels with four underground stations, 2·5 km of at-grade section and two depots. The £800 million, 24 km long Green line runs around the city centre connecting Festival City to the airport free zone and comprises 16 km of elevated viaduct with 12 overground stations and 7 km of tunnels with eight underground stations (of which two are shared with the Red line).

This paper discusses the design and construction of the viaduct substructure. A further paper[1] covers the design and construction of the viaduct superstructure.

2. VIADUCT FORM

The proposed form of the viaduct was architecturally-led in appearance. Figure 4 gives an artist's impression of the proposed viaduct at the conceptual design stage and much of this form was retained in the final detailed design.

The viaduct superstructures were typically formed from U-shaped cross-sections as illustrated in Figures 5 and 6. The post-tensioned precast segmental deck segments were cast using either long-line or short-line moulds.

The following superstructure forms, all of post-tensioned segmental construction, comprised the majority of the scheme.

(*a*) One-span (single span) decks: simply-supported U-section decks constructed by the span-by-span method from an overhead gantry.
(*b*) Two-span (twin span) continuous decks: U-section decks constructed by the span-by-span method from an overhead gantry and made continuous over internal supports by subsequent in situ concrete stitching of adjacent decks.
(*c*) Three-span continuous decks: comprising a combination of U-section and box-section decks, erected by crane using the balanced cantilever method.
(*d*) Station spans: three- or four-span continuous U-section decks constructed by the span-by-span method from an overhead gantry, subsequently made continuous over internal supports by stitching adjacent spans together.
(*e*) Single-track decks: simply-supported U-section decks constructed by the span-by-span method from an overhead gantry (similar to one-span decks). These decks are used at

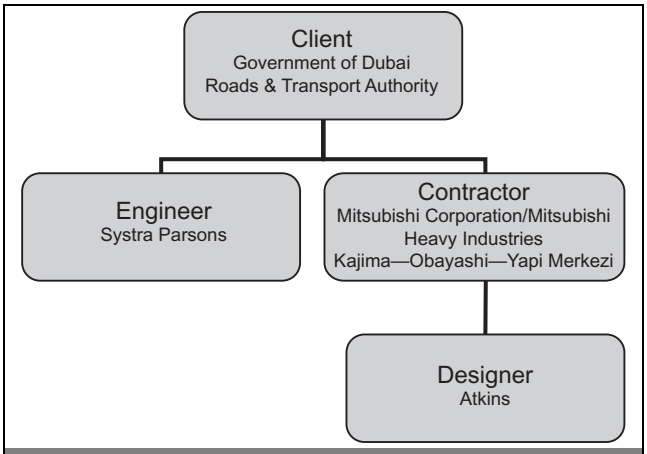

Figure 1. Project organisation

depots and bifurcations of the main lines at the largest stations

The viaduct substructures generally comprise reinforced concrete piers, with flared pier heads to support the deck, and reinforced concrete abutments. Pier heads for the single span, twin spans and station spans were constructed using thin precast reinforced concrete shells which were infilled with in situ concrete and prestressed in stages once erected on site. The pier heads for the single track and three-span continuous internal piers are of in situ reinforced concrete construction. All piers and abutments were founded on large-diameter bored piles.

3. PILED FOUNDATION DETAILS

Dubai lies directly within the Arabian Desert and much of the geology comprises fine sand overlying sandstone and mudstone. The fine, upper sand layers consist mostly of crushed shell and coral and are a combination of mobile dune sands and *sabkha* deposits. These overlie calcarenitic, Aeolian deposits (sands, weakly cemented sands and weak sandstone) and calcisilitite. Beneath these, between around 20 and 40 m below existing ground level, are Jurassic conglomerates, mudstones and siltstones.

The geotechnical design parameters were derived by Atkins' dedicated geotechnical team in Dubai and agreed with the engineer's representatives locally. Results of the extensive ground investigations and their interpretation were collated into several ground reports for typical sections along the full metro route. The geotechnical design parameters in the soil and rock layers were derived from Standard Penetration test (SPT-N) values and unconfined compressive strength (UCS), respectively.

The vast majority of the viaduct spans are supported on single circular reinforced concrete columns with flared pier heads to support the decks, although a few portal structures are used in specific locations. Most single columns are supported on bored monopiles of diameter 2·2 and 2·4 m for speed of construction and to minimise the footprint required for excavations in the congested urban environment. Figure 7 shows a typical monopile detail.

The use of large-diameter piles suited the interface with the circular piers. Typically diameters of 1·75 or 2·0 m were used for the piers supported on 2·2 m diameter piles, and 2·2 m for piers supported on 2·4 m diameter piles. The connection detail between the pile and pier was constructed like that for a pile cap; the pile was broken down and pier starter bars introduced, making allowance for piling tolerances. Figure 8 shows a typical monopile foundation under construction with the column starter bars in place and Figure 9 illustrates the alternative cranked reinforcement connection details that were adopted, depending on the relative sizes of pier and pile and the percentage reinforcement content to minimise potential reinforcement clashes due to piling tolerances.

The piles needed to be large enough to resist the significant moments that are generated from lateral seismic loading (see Sections 6 and 7 below) and from out-of-balance forces from the deck due to horizontal alignment curvature, wind loading, eccentric train loads and other effects. These moments increase down the length of the pile towards a peak at the effective point of fixity and reinforcement was provided and curtailed to suit

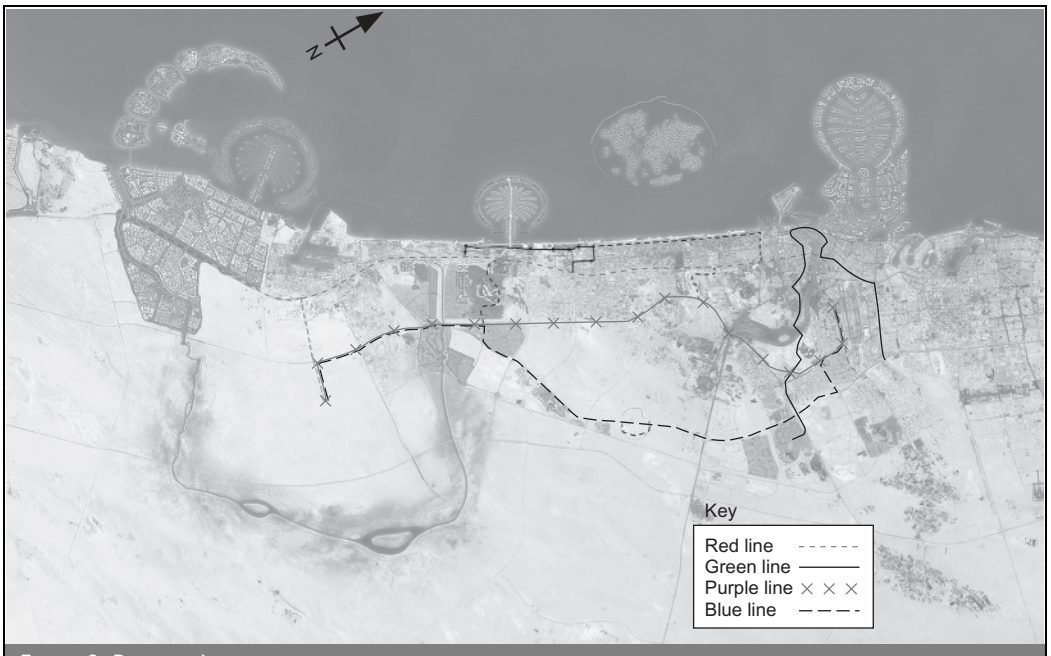

Figure 2. Proposed route map

Figure 3. Red line route map

Figure 4. Conceptual viaduct form

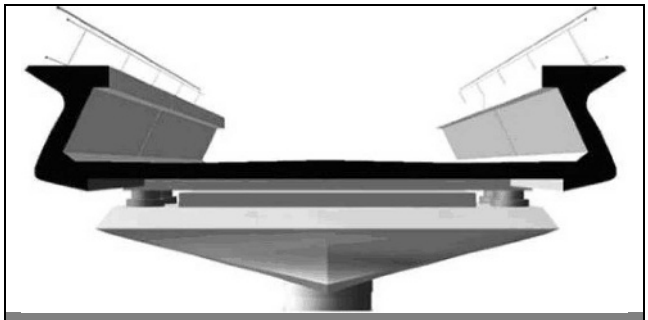

Figure 5. Typical section of U-trough deck and pier head

the specific force and moment envelopes generated from a range of load cases for each foundation.

The soil–structure behaviour for the pile design was determined using readily available geotechnical software. Repute[2] was used as the main tool for each individual foundation. It allowed the input of multiple strata parameters using a boundary element method restricted by pile diameter and was able to model dynamic behaviour. Its relative simplicity enabled a large number of models to be analysed quickly and was therefore highly suited to the tight design-and-build programme. Exact locations of peak moment and force were not always easy to

Figure 6. Prototype deck segment illustrating typical section and finish

obtain, however, due to the inability to introduce a finer modelling mesh. Calibration exercises were therefore undertaken using the more rigorous finite-element capabilities of Plaxis[3] to enable conservative distribution factors to be applied to each Repute model accounting for soil type, strata depths, foundation location, pile length, soil liquefaction potential under seismic loading and load case type.

Critical to the use of monopile foundations was the calculation of pile length. For foundations with only a single pile, these needed to be suitably conservative as there is clearly no potential for load distribution between adjacent piles as is possible within a pile group. The original design basis proposed an allowable pile working load ($\boldsymbol{Q}_{\text{allow}}$) as the sum of the shaft resistance ($\boldsymbol{Q}_{\text{s}}$) divided by a factor of safety of 2 and the end bearing resistance ($\boldsymbol{Q}_{\text{b}}$) divided by a factor of safety of 3. The final resistance calculation method, as agreed with the engineer, used $\boldsymbol{Q}_{\text{s}}/3$ and ignored end bearing due to the potential problems that could arise if a monopile were to bear into a local void in the weak sandstone or mudstone layers. Initial pile testing indicated that twice the calculated ultimate shaft resistance and 50% of the calculated ultimate end bearing resistance would give good agreement to the test results and thus potentially a more refined $\boldsymbol{Q}_{\text{allow}} = \boldsymbol{Q}_{\text{s}} + \boldsymbol{Q}_{\text{b}}/2{\cdot}5$ might have been adopted, but the design was completed using the more conservative calculation method previously approved ignoring end bearing.

Pile shaft resistances were calculated using the following relationships depending on soil type and pile construction method

(*a*) $1{\cdot}6 \times \mathit{SPT}\text{-}N + 6$ kPa (after Decourt[4]) for soils
(*b*) $k \times (\mathit{UCS\ design\ value})^{0{\cdot}5}$ (after Zhang and Einstein[5]) for rock, with $k = 0{\cdot}35$ for polymer-modified, water-supported

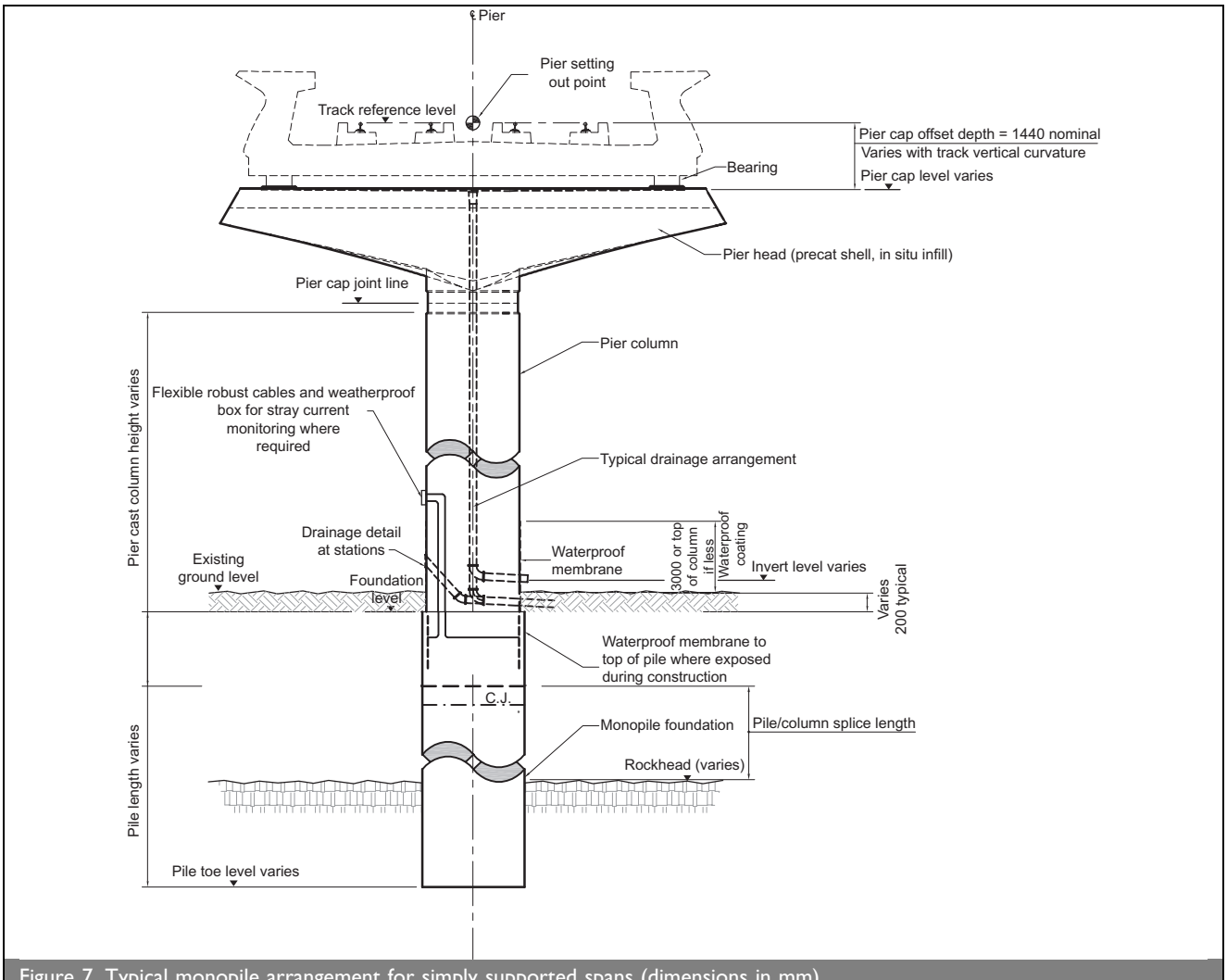

Figure 7. Typical monopile arrangement for simply supported spans (dimensions in mm)

Figure 8. Construction of typical pier on a monopile foundation

pile shafts and $k = 0{\cdot}25$ for bentonite-supported pile shafts as minimum values.

Several foundations for the simply supported spans required the use of twin pile groups with a conventional pile cap for spanning over existing large services where a monopile location would clash. Additionally, four-pile pile groups with conventional pile caps (illustrated in Figure 10) were adopted for the internal piers of continuous two-span and three-span structures and for simply supported spans where a monopile foundation was not feasible due to very tall piers being required to support spans with tight deck radii.

4. DESIGN METHODOLOGY FOR ULTIMATE LIMIT STATE AND SERVICEABILITY LIMIT STATE

The design of the viaducts was based on BS 5400: Part 4[6] and associated British Standards, with additional International Standards used to supplement the scope in such areas as seismic loading and detailing, and rail dynamic factors. The American Concrete Institute technical design standard ACI 358.1R–92[7] was used to determine the dynamic factors to be applied to the vertical train loading for deck longitudinal design for the continuous spans. For the simply supported spans, the dynamic factors were derived from bespoke dynamic analyses for the respective span lengths. For transverse design, the recommended dynamic factors of BS 5400: Part 2[8] for reduced rail loading (RL) of 1·2 to 1·4 were verified again using a finite-element dynamic analysis. In accordance with the ACI code, the dynamic impact factors were not applied to the design of viaduct foundations, but were included in the pier head and bearing design. The maximum operating speed of the trains is intended to be 90 kph; the maximum design speed was taken as 100 kph.

The usual BS 5400 load combinations from 1 to 5 were assessed to determine critical design load effects. In addition, a sixth load combination was added to cover seismic loading (see Section 6 below). Other specific load cases considered included temporary loading from gantries (Section 5 below) and vehicular collision.

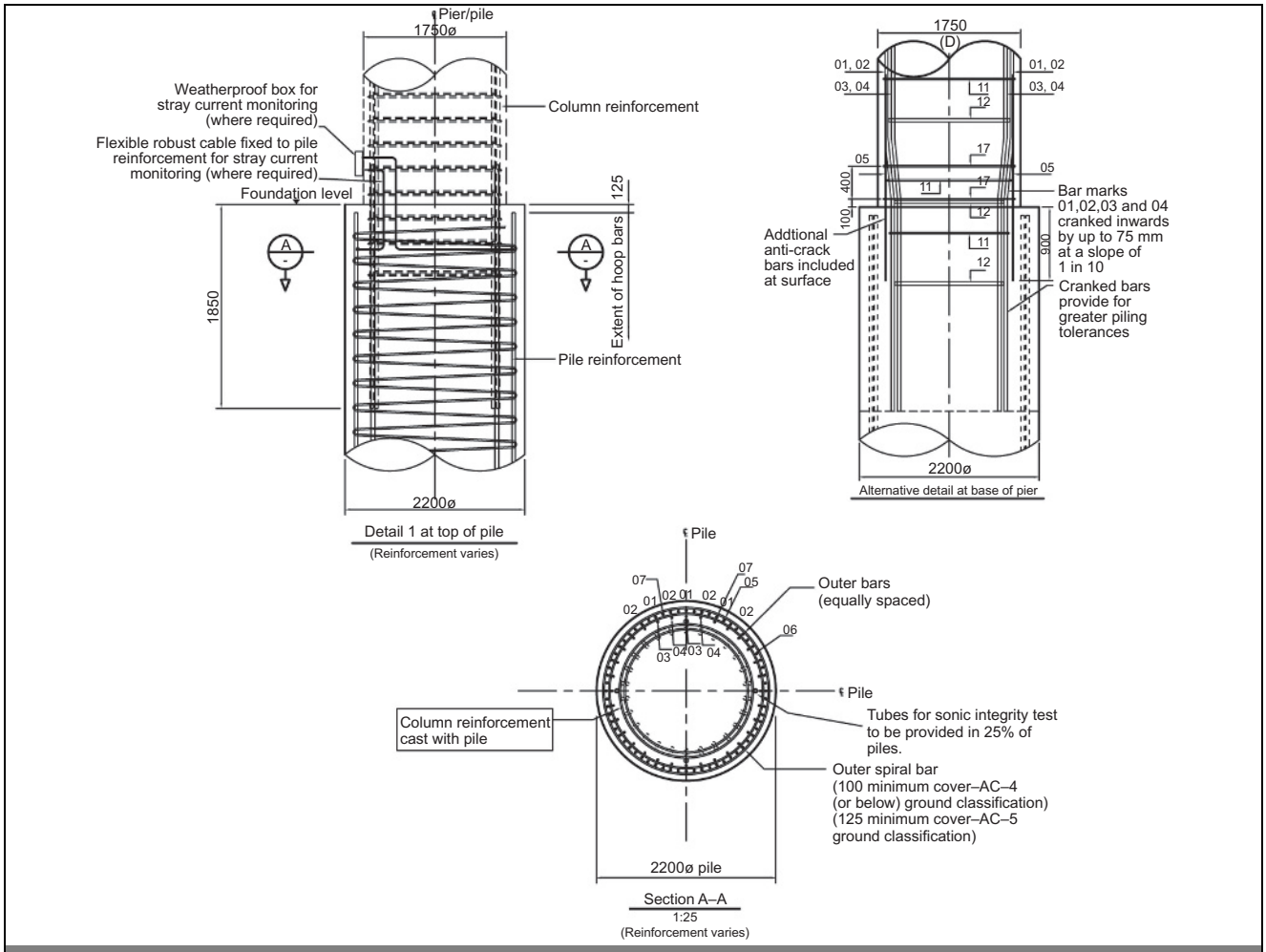

Figure 9. Typical pier–pile connection details (dimensions in mm)

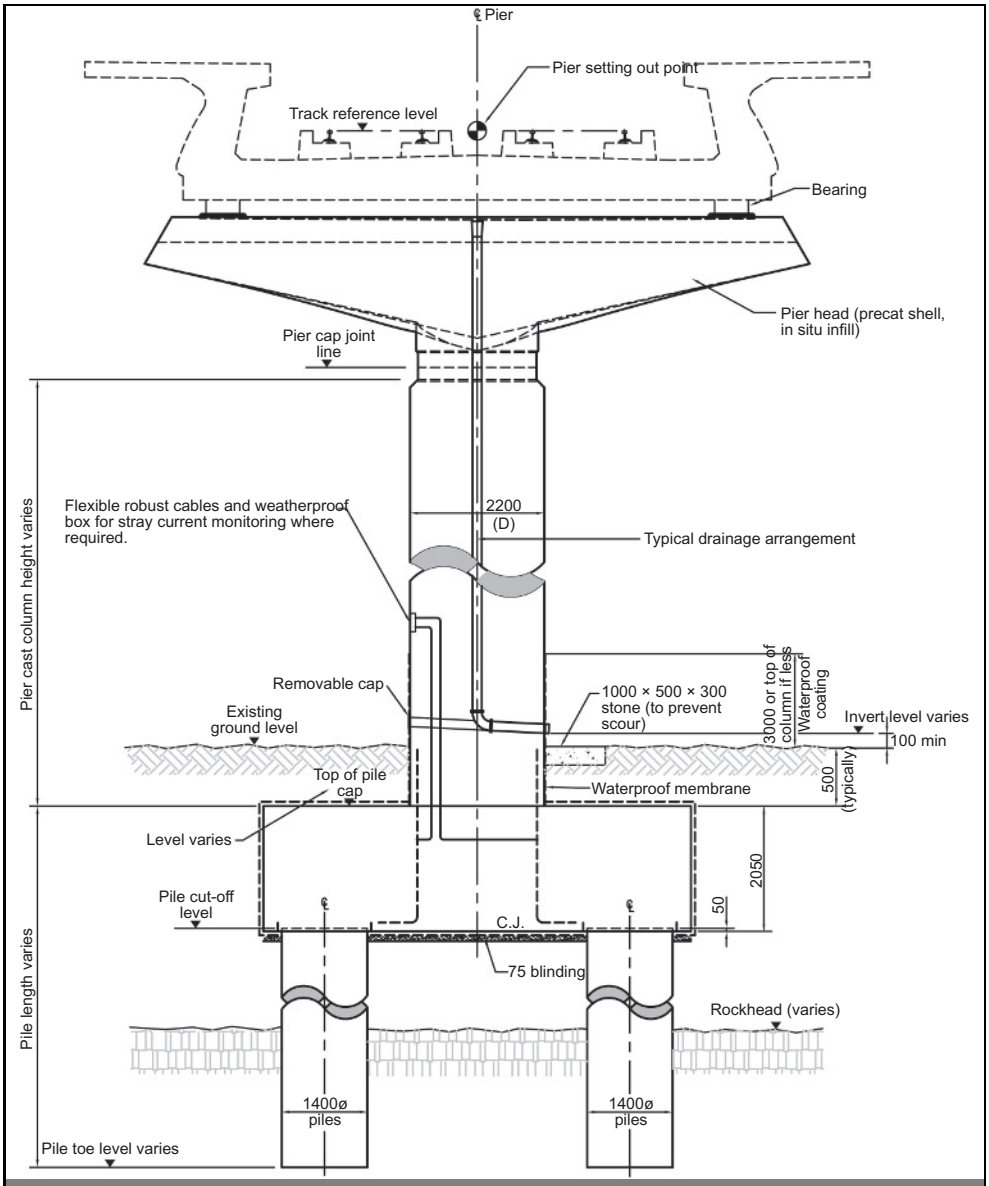

Figure 10. Typical four-pile pile group arrangement for internal piers (dimensions in mm)

Typically, class C32/40 was used for the pier and pile cap concrete. Class C32/40 concrete was also used for the piles, but to account for possible weakening during the placement (under bentonite or polymer-modified suspension fluid) the cube strength design value used was reduced by 10 MPa. All reinforcement used in the substructure design was high yield, type 2 deformed bars with a yield stress of 460 MPa. The aggressive ground conditions meant that durability considerations were paramount. As a result, additional waterproofing was applied to the top 5 m of all piles to improve resistance to chloride attack and pile cover to reinforcement of 120 mm was used to improve resistance to sulfate attack. Generally crack widths (under combination 1 loads) were limited to 0·2 mm. Fatigue of reinforcement was also a critical design consideration in some locations (see Section 8 below).

The construction programme called for the initial design of over 1200 unique foundations in the first 9 months of design to take the viaduct construction off the critical path. This was achieved through automation of the bulk of the design process and the use of conservative simplifying assumptions in the early stages of design. As the team got ahead of the programme, the conservatism was removed from the process and more refined calculation methods introduced into the automated procedures to optimise the designs for the foundations yet to be constructed.

The design programme capitalised on the locations of the UK-based structures team and the Dubai-based alignment team, which handled setting out and local issues such as utility diversions. Advantage was made of the staggered weekends between Dubai and the UK; the alignment of a given section was distributed at the start of the UK week and the appropriate design data added and sent back to Dubai at the end of the UK week. Coupled with the automation process developed, this allowed a peak output of 100 bespoke designs per week to be achieved.

5. DESIGN DURING CONSTRUCTION: GANTRY LOADING

The majority of the simply supported decks and two-span continuous decks were constructed by overhead gantries (illustrated in Figures 11 and 12). The temporary loading from the various gantries used on the scheme was defined by the temporary works subcontractor, VFR (a consortium comprising VSL, Freyssinet and Rizzani De Eccher), appointed to undertake the

Figure 11. Use of overhead gantries to construct typical viaduct sections

Figure 12. Use of overhead gantries to construct typical viaduct sections

deck construction. These loading regimes were continually developed throughout the design programme as various configurations of gantry were developed to cater for the many permutations of span configurations and access restrictions on site. The gantry loads included the effects of the most severe loading configuration carrying deck precast elements and also the unloaded case when the gantry was potentially subject to stronger winds. In some locations, the gantries were also

required to travel over previously constructed (by the balanced cantilever method) three-span continuous decks and the temporary effects of these conditions needed to be designed for. Precast deck segments were mostly delivered to their required location at ground level, but in some locations where access was more difficult, some segments were delivered over the previously constructed deck using special transporters. The additional load effects from these cases on permanent works also needed to be considered in the detailed design.

For the substructure, the temporary construction load cases were generally not governing for the pier and pile designs as they were typically less onerous than the seismic design effects. The design of the pier heads however, was extremely sensitive to the gantry loads (and in particular to the torsion induced) as discussed in Section 9 below.

6. SEISMIC DESIGN METHODOLOGY AND DETAILING

Seismically, Dubai is in a very stable zone. The nearest seismic fault line, the Zargos Fault, is 120 km from the UAE and is unlikely to have any seismic impact on Dubai. Nevertheless, the employer's requirements specified that the viaducts should be designed for earthquakes in accordance with AASHTO LRFD,[9] classing the viaducts as 'essential', the site as Zone 2 and using an acceleration coefficient of 0·12. The site coefficients were determined in accordance with AASHTO LRFD on the basis of the relevant geological profile and geotechnical data for the foundations. Soil profile types I and II were found to be appropriate for the entire route, giving site coefficients of 1·0 and 1·2 respectively (modified to 1·0 and 1·5, respectively, where the flexibility of the elastomeric bearings was included in the seismic analysis).

AASHTO LRFD was only used to determine the load effects from a seismic event. Once obtained, these effects were combined with the other appropriate coexistent load effects and the foundations designed in accordance with the resistance rules of BS 5400:Part 4 as for other non-seismic load cases.

In general, the single-mode elastic method was used and the fundamental period of vibration was determined by modelling individual piers using the computer software lusas[10] as described in Section 7 below. Multimodal analyses were also used for continuous structures and irregular single-span arrangements (see Section 7). The horizontal elastic seismic response coefficient for a given natural period of vibration, C_{sm}, was obtained from AASHTO LFRD

$$C_{sm} = \frac{1{\cdot}2AS}{T_m^{2/3}} \leq 2{\cdot}5A$$

where T_m is the period of vibration of the mth mode (in s), A is the acceleration coefficient and S the site coefficient.

There were no contract requirements for seismic serviceability performance so columns were designed to form plastic hinges at their bases under a seismic event at the ultimate limit state. Potential plastic hinge zones were confined to the base of the piers and the top of the piles so that the substructure could be readily inspected for damage after an earthquake. Plastic hinges were promoted in these regions by dividing the elastic seismic design forces by appropriate response modification factors, R, for the respective elements (based on the provisions given in AASHTO LRFD) with

(*a*) $R = 1{\cdot}5$ for elastomeric bearings where account of the bearing flexibility had been included in the natural frequency analysis
(*b*) $R = 2{\cdot}0$ for continuous structures with mechanical bearings.

The onset of plasticity controls the magnitude of forces that can be transmitted to the rest of the structure, which is particularly beneficial in the event of an extreme earthquake whose magnitude exceeds the design value. However, over-strength of the hinge zone in bending could lead to greater forces being attracted than expected. For such cases, it is important that the piers have adequate shear strength to still enable a ductile response to develop, rather than permit a brittle shear failure to occur. To guard against such failure, the piers were designed for a shear force that corresponds to the achievement of the over-strength moment of resistance in the plastic hinge zone at the base of the piers. AASHTO recommends using an over-strength factor of 1·3 to account for the difference between mean material properties and 5% lower fractile values. In addition, further over-strength is provided because of the use of partial material factors. These need to be removed to determine the potential difference in strength between design values and realistic in situ values. The over-strength factor to account for the removal of the material partial factors will be between the values of 1·15 and 1·5 (the BS 5400: Part 4 values for steel and concrete, respectively). For the typical column reinforcement percentages used for Dubai Metro, this factor was approximately 1·22.

To ensure that the plastic hinge zone is confined to the base of the pier, the flexural design of the pier above the plastic hinge was based on bending moments which are consistent with the over-strength moment of resistance in the plastic hinge zone and the corresponding over-strength shear force.

Superstructure–pier connection forces were based on the lesser of the elastic seismic design forces divided by a response modification factor, R, of 1·0 or the shear force that corresponds with the over-strength moment of resistance in the plastic hinge zone at the base of the pier. This was applied to the bearing design too.

Reinforcement detailing for the viaducts was generally in accordance with BS 5400: Part 4. However, special additional requirements for seismic detailing were taken from AASHTO LRFD. In particular there is a need to ensure that the piers have adequate ductility as discussed above. This was achieved by providing adequate transverse reinforcement in the potential plastic hinge zones, to prevent buckling of the longitudinal reinforcement and to provide confinement to the concrete core. This confinement reinforcement was continued into the body of the pile caps, where used, in accordance with the code. Transverse reinforcement could be formed by either spiral or hoop reinforcement, provided that conventional laps were not used in the plastic hinge zone and hoop reinforcement was anchored into the body of the column with adequate hooks. The latter would have given increased congestion and reduced space for concrete vibrators, thus spiral reinforcement was used for the majority of foundations. Lapping of longitudinal reinforcement

was also kept outside the potential plastic hinge zones so as not to compromise ductility and over-strength shear and moment design away from the plastic hinge zone.

7. SEISMIC ANALYSIS

The seismic analyses of the straight sections of simply supported spans were carried out in accordance with AASHTO LRFD for multi-span bridges with 'regular' spans provided they met the span ratio and pier stiffness criteria even though the number of spans exceeded six. The single-mode elastic analysis uniform load method was used. Other structures were designed using a multimodal response spectrum analysis in which the presence of the rails was ignored.

All of the seismic analyses were performed using several finite-element beam and shell models analysed in lusas. The models included the pile supports either with equivalent cantilevers or complete piles with soil springs. Equivalent cantilevers were determined from the results of the soil–structure interaction analyses. This was an iterative process to ensure that the stiffness derived was appropriate to the mean load level. Accelerations were determined using the seismic response spectrum defined in AASHTO LRFD. The site coefficients used in the response spectrum definition were dependent on the bearing type used as noted in Section 6 above.

Initially, parametric studies of the single-span models for the simply-supported structures were developed for a range of variables to enable rapid automated design of individual, unique foundations. Variables included pier diameters, pier heights, soil properties, span arrangements, bearing types (from rigid mechanical bearings to a range of elastomeric bearing stiffness values) and acceleration direction were considered. Results for a typical set are illustrated in Figure 13, where 'type 1' and 'type 2' refer to two different elastomeric bearing designs. The beneficial effect of introducing flexible bearings on reducing natural frequency is apparent.

Macros were developed to automatically write the model file data for an individual pier, enabling further quick refinement of the design to be undertaken which was essential to the rapid redesign of sections of the alignment subject to change.

For the multimodal analyses, the seismic load effects from each mode were summed using the complete quadratic combination. These results were obtained directly from the lusas output and verified against simpler hand calculations based on the most dominant modes. Directional combination of effects was achieved using a simple combination rule whereby 100% of the response in any one direction was considered with 30% of the response in the other two orthogonal directions.

8. FATIGUE OF CRANKED REINFORCEMENT

Due to limitations on maximum pile sizes, reinforcement content was particularly high for the most heavily loaded monopiles. In several cases, two layers of longitudinal reinforcement were required. This called for an additional detail to cater for piling tolerances and led to the cranking of longitudinal bars (as indicated in Figure 9) in some instances. The high centrifugal loads on the tallest, slender piers resulted in particularly high fatigue stress ranges in the cranked bars.

It is commonly appreciated that bent bars have lower fatigue resistance than straight bars but design codes give limited guidance on their design. BS 5400: Part 4 covers fatigue of reinforcement inadequately by limiting stress ranges to 325 MPa and does not distinguish between bent and straight bars. The Highways Agency document BD24,[11] as amended by Interim Advice Note (IAN) 5, also fails to distinguish between straight and bent bars and its limiting stress ranges are inappropriate for rail loading; they are based on calibration studies using static highway loading. As a result of these deficiencies, Eurocode EN 1992-1-1[12] was used for the fatigue assessment because it contains rules for bent bars with different bend diameters. For the typical bar arrangements used for the Dubai Metro, the fatigue stress range limits are 116·5 MPa for straight bars and around 65 MPa for the cranked bars, which constitutes a large reduction in effectiveness. This predicted magnitude of reduction in fatigue performance for bent bars has actually been shown to occur in tests.[13] These limits were used for all the substructure reinforcement throughout the project.

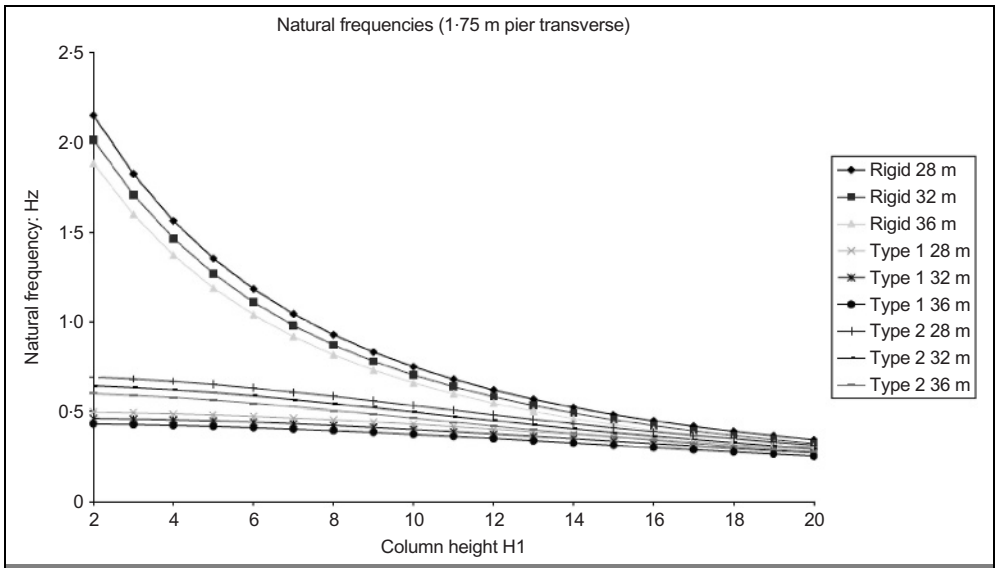

Figure 13. Typical natural frequency analysis results for a range of pier heights, bearing types and span arrangements

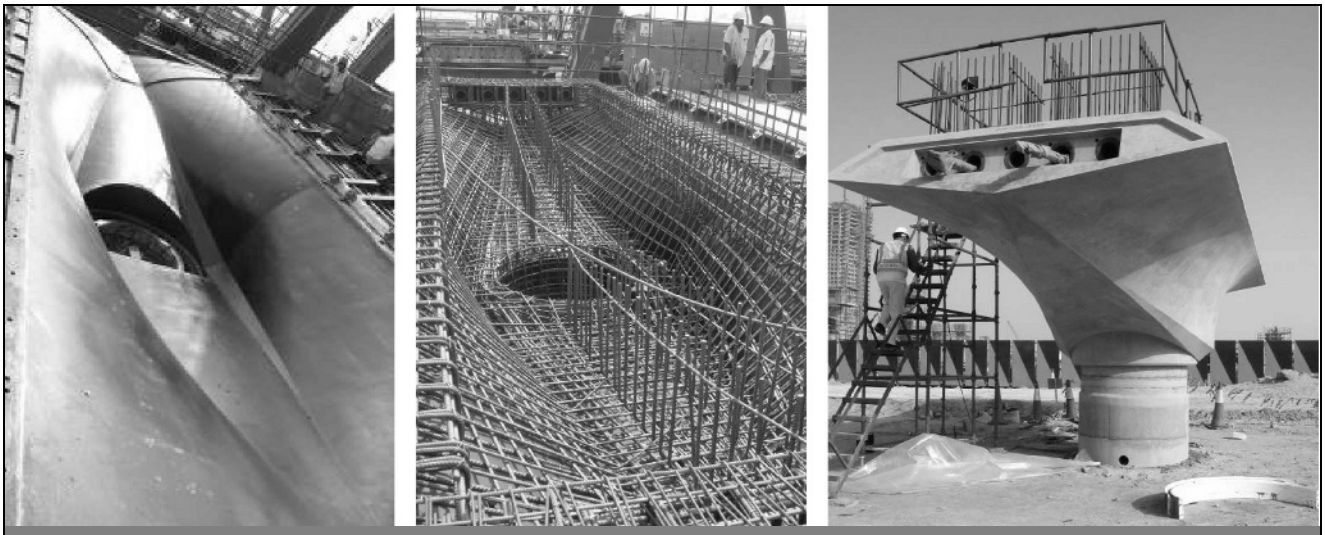

Figure 14. Typical pier head arrangement

In order to comply with these relatively low permissible limits, the fatigue stress ranges were calculated from the likely mix of actual train loads rather than from the full characteristic loading used in the static design. For the predicted 20 million load cycles within the design life, the latter would be too onerous. Realistic operating train speeds were also used in the calculation of centrifugal forces, rather than the enhanced design speed used for ultimate limit state calculations.

The above approach was enough to justify the use of cranked bar details apart from one or two worst cases where separate bespoke details were developed without bending the bars.

9. PIER HEAD DESIGN

The design and construction of the pier heads was subcontracted by JTMjv to ETiC, but Atkins retained the design responsibility and was instrumental in developing the final design solutions to satisfy the particularly onerous temporary load conditions. The design interfaces were complicated further with close coordination required between the designers, the bearing suppliers and VFR, who were responsible for the supply of gantry loads and erection sequences. Iteration with the bearing design was subsequently simplified by landing the decks on temporary supports before jacking onto permanent bearings after the gantries had moved on to erect other spans (see Section 10 below).

The appearance of the pier heads was architecturally led (see Figure 14) and required precast outer shells to achieve the desired finish consistently. The outer shells were cast using steel moulds and stored with the deck segments in the dedicated casting yard at Jebel Ali. The outer shells comprised thin-walled, reinforced concrete sections which could be transported to site when required, before being in filled with in situ concrete and prestressed in controlled stages on site as the permanent load was added.

Designed initially for the permanent condition only, the pier heads had several shear and torsion deficiencies identified under construction load cases due to the high eccentricity of the temporary gantry loading. In this temporary condition, all the self-weight of both spans was applied on one side of the pier head through temporary supports (Figure 15). In the final condition, the load from each span is more balanced.

For the temporary condition, the maximum shear and torsion could not be resisted by either the shell or the in situ infill concrete alone and thus the pier head had to be designed to mobilise a contribution from both the shell and the infill, despite minimal interface reinforcement being provided between the two.

To achieve an adequate design without modifying the pier head moulds, which had already been fabricated, several departures from BS 5400: Part 4 were required. In particular, the inclined components of prestressing force and compression chord force were included in the shear resistance, the tensile serviceability limit state stresses during construction were limited to class 2

Figure 15. High eccentricity of temporary loading from overhead gantries

limits, rather than the.1·0 MPa limit stipulated, and stresses during bearing replacement were limited to class 3 limits following agreement to this relaxation by the client.

10. BEARING DESIGN

The majority of the single-span decks are supported on elastomeric bearings. Pot bearings have been used for all the continuous structures for both internal supports and end supports and therefore also the ends of the single spans sharing pier heads with continuous spans.

The philosophy for the design of the elastomeric bearings was complicated by two opposing design objectives. The bearings needed to be stiff enough to comply with the continuous rail deflection limits (of International Union of Railways technical standard UIC 776-3[14]) and strong enough to withstand the vertical and horizontal loads. However, they needed to be flexible enough to minimise the seismic forces attracted to the substructure since lower stiffness leads to reduced natural frequencies and reduced lateral accelerations. The range of adequate elastomeric bearings that could satisfy these requirements was particularly narrow and also very sensitive to span arrangements.

Typically, bearings of around 600 mm by 650 mm to 750 mm by 750 mm in plan and 150 mm to 210 mm tall were adopted. The elastomeric bearings were generally designed in accordance with BS 5400: Part 9,[15] but this only gives guidance on design at the serviceability limit state and does not cover ultimate limit state design which was required for the usually critical seismic load cases. The similar design rules given in BS EN 1337-3[16] were therefore used to check the adequacy of the bearings at ultimate limit state.

For speed of construction and simplicity, the contractor was keen to avoid any mechanical fixing of the elastomeric bearings to the deck. Consequently the bearings were designed to be placed dry between the deck and the pier head plinths and to rely on friction as the sole means of lateral restraint. Where transverse forces on the elastomeric bearings exceeded 10% of the vertical load (as occurred for all locations), the bearings were fitted with an interfacing chequered plate to provide a minimum coefficient of friction of 0·5 between mating surfaces. This attachment was capable of carrying the entire transverse load.

Since reliance was placed on friction as the sole means of restraint against lateral seismic loads, a coexistent vertical acceleration was considered in the design of the elastomeric bearings, despite not being specified in the employer's requirements. This acceleration was derived from a response spectrum similar to that used for the horizontal accelerations, but with values equal to two-thirds of the horizontal ones.

As the design of the elastomeric bearings was tailored to the permanent condition, there was little capacity left to accommodate deflections, rotations and translations that would be locked into the bearings due to the use of launching gantries and the construction sequences adopted. Consequently, the majority of the decks had to be landed on temporary supports during the initial erection by gantry before they could be subsequently jacked up onto their permanent bearings when the gantry had moved on. Although not ideal for construction, this solution had benefit in reducing the temporary load effects in the pier heads and was thus adopted as a compromise to overcome several problems.

11. RAIL–STRUCTURE INTERACTION

As discussed above, the viaducts comprise a combination of simply supported spans on elastomeric bearings and continuous spans with fixed and free guided sliding pot bearings. The rails are continuously welded across all the decks and deck joints and are connected to the deck by regular track fixings, thus the rails interact with the deck causing relative movements to occur which induce stresses in the rails and forces in the deck.

The temperature range of the continuous welded rail (CWR) was considered relative to its neutral setting temperature of 40 $\pm$ 37°C and the maximum and minimum rail temperatures which were assumed to be +75 and +3°C, respectively. This gave CWR extreme ranges of +38°C and −40°C. The rail–structure interaction (RSI) temperature range is governed by the change of structure temperature relative to deck temperature at the time of installation of the rail. Based on the specified design temperature ranges, the maximum and minimum deck temperatures were assumed to be +55 to +5°C. It was further assumed that the rails were fixed to the deck at deck temperatures between +20 and +40°C, which gave maximum and minimum temperature ranges of +35°C and −35°C. This corresponds with the International Union of Railways technical standard UIC 774-3R[17] requirements of maximum and minimum bridge temperature ranges of $\pm$35°C. The build-up of CWR forces in the rail was considered at points where rail breather joints are located and for the case of a rail break. For viaducts with horizontal curvature the effects of the radial forces arising from the full CWR forces were considered.

The rails were checked against the recommendations of UIC 774-3. These requirements limit the stress ranges due to this RSI behaviour to prescribed values. The checks are only necessary for in-service conditions, thus no analysis of rail forces was undertaken for seismic events. Checks were, however, also made on the additional stresses induced in the rails due to the end rotations of the decks and the differential vertical and transverse movements permitted due to the flexibility of the bearings. The values of these additional forced rail displacements and rotations were considered by the trackwork designers in the final rail design.

The rail–structure interaction behaviour is non-linear because the track fixings allow slip to occur with an elasto-plastic relationship illustrated in Figure 16. The decks also interact with each other through the continuous rails and the rail connections and so the overall global behaviour is complex. The effects can

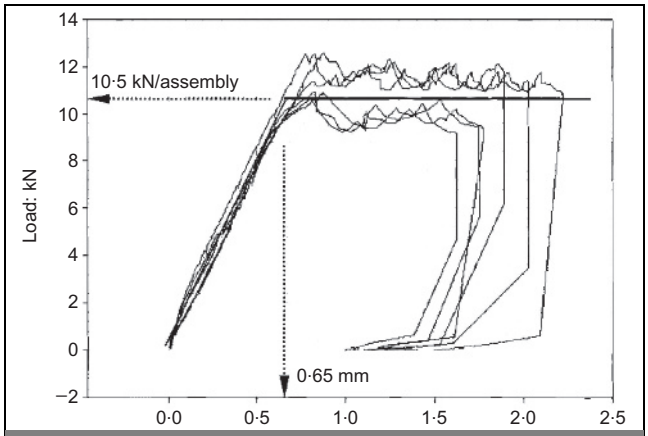

Figure 16. Track fixings slip characteristics

be rationalised, however, such that simplified approximate methods can be used initially to identify the most critical viaduct lengths which required further detailed analysis.

The global behaviour was analysed using two-dimensional models of both the structure and rails to examine the longitudinal load distributions. The rails were modelled as beam elements with non-linear springs connecting them to the deck. The out-of-balance effects of different adjacent span lengths were analysed using a simple elastic two-dimensional model of at least five spans either side of the design pier in order to quantify the forces on the bearings and piers. This approach was also used for vertical load effects, seismic loading and traction and braking.

A series of simplified spreadsheet approaches to estimating the rail–structure interaction forces and for checking compliance against the UIC code requirements was developed. Where the simplified checks showed borderline cases, the more sophisticated non-linear analysis was used to demonstrate adequacy. In some instances, this required the flexibility of the fixed piers and their foundations to be included in the non-linear models.

Changes to the overall viaduct setting-out continued throughout the construction process where previously undisclosed existing utility locations were identified requiring span arrangements to be adjusted. The development of the simplified approaches to RSI analysis enabled a rapid decision to be obtained on the impact such setting-out variations had on the overall RSI conclusions without the need to undertake more complex analysis every time the span arrangements altered.

In addition to the RSI analysis, absolute and relative displacement checks were carried out against the UIC 774-3 requirements for braking, acceleration and deck end rotation due to vertical loading. The relative vertical deflection across adjacent decks at rail locations was limited to 3 mm.

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REFERENCES

1. Smith D. A., Hewson N. R. and Hendy C. R. Design of the Dubai Metro Light Rail Viaducts—superstructure. *Proceedings of the Institution of Civil Engineers—Bridge Engineering*, 2009, **162**, No. 2, 55–62.
2. REPUTE v1.54, Geocentrix Ltd, Banstead, Surrey, 2007.
3. PLAXIS 2D v8.6, Plaxis BV, the Netherlands, 2007.
4. Decourt L. Prediction of load–settlement relationships for foundations on the basis of the SPT-T. *Proceedings of the Leonardo Zeevaert Conference*, National Autonomous University of Mexico, Mexico, 1995, pp. 85–104.
5. Zhang L. and Einstein H. H. End bearing capacity of drilled shafts in rock. *Journal of Geotechnical and Geoenvironmental Engineering*, 1998, **124**, No. 7, 574–584.
6. British Standards Institution. *Steel, Concrete and Composite Bridges, Part 4—Code of Practice for Design of Concrete Bridges*. BSI, London, 1990, BS 5400.
7. American Concrete Institute. *Analysis and Design of Reinforced and Prestressed Concrete Guideway Structures* (Chapter 3—Loads, p. 358. 1R-15). ACI Committee 311, Michigan, 1992, ACI 358.1R-92.
8. British Standards Institution. *Steel, Concrete and Composite Bridges, Part 2—Specification for Loads*. BSI, London, 1978, BS 5400.
9. American Association of State Highway and Transportation Officials. *LRFD Bridge Design Specifications*, 3rd edn. AASHTO, Washington, DC, 2004.
10. LUSAS Finite Element Analysis Software, Finite Element Analysis Ltd, Kingston-upon-Thames, 2007.
11. Highways Agency. *Design Manual for Roads and Bridges, BD24—The Design of Concrete Highway Bridges and Structures—Use of BS 5400: Part 4: 1990*. Highways Agency, London, 1992.
12. British Standards Institution. *Eurocode 2: Design of Concrete Structures—Part 2-1-1: General Rules and Rules for Buildings*. BSI, London, 2004, BS EN 1992-1-1.
13. Bennett E. W. Fatigue tests of spliced reinforcement in concrete beams. In *Fatigue of Concrete Structures,* ACI Publication SP-75 (Shah S. P. (ed.)). American Concrete Institute, Detroit, 1982, ACI SP-75.
14. International Union of Railways Technical Standard. *Deformation of Bridges*. UIC, Paris, 1989, UIC 776–3.
15. British Standards Institution. *Steel, Concrete and Composite Bridges, Part 9—Bridge Bearings*. BSI, London, 1983, BS 5400.
16. British Standards Institution. *Structural Bearings, Part 3—Elastomeric Bearings*. BSI, London, 2005, BS EN 1337–3.
17. International Union of Railways Technical Standard. *Track/bridge Interaction, Recommendations for Calculations,* 2nd edn. UIC, Paris, October 2001, UIC 774–3.
18. Smith D. A. and Hendy C. R. Design of the Dubai Metro Light Rail viaducts—substructure, 2008. *Proceedings of Concrete Bridge Conference HPC: Safe, Affordable and Efficient*, 4–6 May 2008, St. Louis, MO, National Concrete Bridge Council, c/o Portland Cement Association, Skokie, IL.
19. Smith D. A. and Hendy C. R. Design of the Dubai Metro Light Rail bridge viaducts—substructures. In *Concrete Durability: Achievement and Enhancement* (Dhir R. K., Harrison T. A., Zheng L. and Kandasami S. (eds)). IHS BRE Press, Watford, UK, 2008, pp. 781–800.

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