BOTHASFONTEIN INTERCHANGE BRIDGE, SOUTH AFRICA – AN ABC COMPOSITE STEEL/CONCRETE 4-SPAN BRIDGE ERECTED IN TWO CONSECUTIVE WEEKENDS OVER 29 YEARS AGO

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ABSTRACT

The Bothasfontein Interchange Bridge was constructed almost 30 years ago over the busiest freeway in Africa, which is the Ben Schoeman freeway, connecting Pretoria to Johannesburg. There was a very natural concern for traffic interference on a road with highly congested traffic and severe restraint regarding the depth of the bridge. Innovative solutions had to be found, which passed by the consideration of a composite steel concrete structure and precast concrete deck panels for the full width of the bridge with transverse unbonded prestressing. In the end a very elegant structure was achieved, with aesthetically pleasing post-tensioned V columns, which was put in place over the course of two consecutive weekends, one for the steel bathtubs, and the other for the precast concrete deck panels, for which profit was taken of the two weeks that many of the Pretoria and Johannesburg residents would take off for the coast during the Summer/Christmas vacation. They have left without any bridge, and when they returned a whole new 122 m (400') long bridge was, all of a sudden, there.

At the time, ABC wasn't commonly used, or wasn't even recognized as a procedure, it was the author's intention to draw the attention of the South African engineering community to the positive aspects of this type of construction, and to promote its use in applicable situations, thereby providing the Client with a better engineering, faster execution time, less disruption of existing traffic, and a cost effective solution.

INTRODUCTION

In order to cope with expected traffic volumes, it was necessary to double road P66-1 through the interchange over the Ben Schoeman freeway, which links Pretoria and Johannesburg and is one of the busiest roads in the whole of Africa, if not the busiest, since in 1989 it already carried a number of cars in excess of 50,000/day (about 3,600 v/h).

This necessitated a whole new bridge inside the interchange, while under full traffic conditions.

Therefore, in order to limit road closure and traffic interference in general, the bridge was designed to facilitate a rapid construction, according to what we, nowadays, call ABC (Accelerated Bridge Construction).

Tenders for this bridge were invited in August 1989 and construction started in November 1989, with the contract completion and opening to traffic in December 1990, i.e. about 2 years after start of design.

Additionally, there were strong geometric constraints limiting the depth of the bridge, in spite of spans of 40 m (131 ft), due to the needed concordance with the other direction of traffic of the interchange, on top of another close by parallel bridge, keeping the needed clearance over the Ben Schoeman freeway, and considering the presence of relatively low power lines over the bridge.

Figure 1 - Perspective View of Bridge

For the purpose above, and to satisfy the site geometric constraints, after the consideration of different types of structures, it was concluded that the type that would satisfy all the requirements would be a steel-concrete composite deck, with the steel part materialized in two relatively shallow steel troughs, and the deck in precast pretensioned concrete planks, 2 meters (6.5 ft) wide, and the full width of the bridge long.

In fact, apart from minimal traffic interference for the construction of the central pier, making use of a quite wide median, some 40 ft wide, and the side piers and abutments, sufficiently away from the traffic lanes, it

only took the closure of the Ben Schoeman freeway, with traffic diverted through the on and off ramps, for two consecutive weekends to put the bridge in place.

In one weekend the steel troughs were erected in place and duly connected, while in the next weekend the precast concrete planks were also placed on top of the steel troughs and solidarized between themselves.

For minimizing traffic interference, the two weekends for closure of the road were strategically chosen as coinciding with the dates in December when most South Africans take their Summer/Christmas vacation, thus allowing minimal traffic during the closures.

Special consideration was also given to the bridge aesthetics, owing to its high visibility, giving the great volume of traffic on the Ben Schoeman freeway.

OVERALL FEATURES OF THE BRIDGE

The composite steel deck structure comprised four skew spans of 20.5 m (67.3 ft), 39.5 m (130 ft), 39.5 m and 20.5 m of uniform thickness, in a double spine open steel box girder, topped by precast concrete planks.

The overall deck width is 13.95 m (45.8 ft), carrying the northbound direction of traffic of road P66-1.

In the Bothasfontein bridge, the designer believes that his two main goals were achieved, i.e. a structure that is aesthetically pleasing and relatively straightforward in design and construction.

The superstructure is a composite steel and concrete design, with the steel girders as a double spine steel trough in weathering COR-TEN steel and 1.0 m deep, while the deck slab consists of precast, prestressed concrete planks in 50 MPa (7.3 ksi) concrete, only 8” deep.

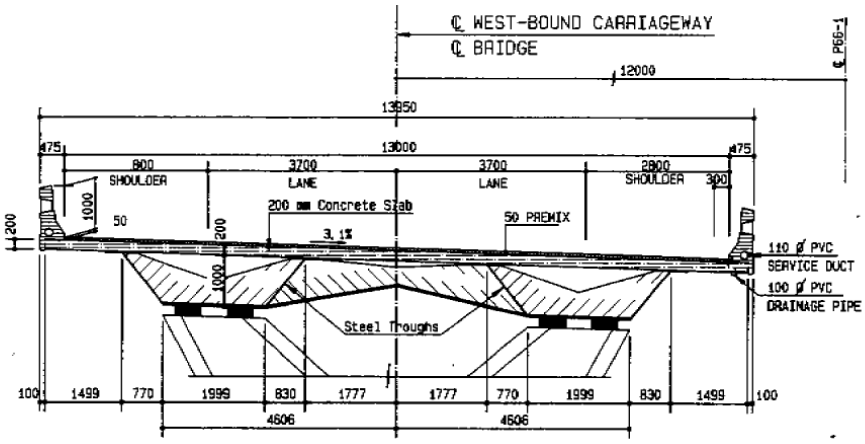

Figure 2 - Typical Cross-Section over the Piers

SUPERSTRUCTURE

The steel girders in the Bothasfontein bridge are open-top trapezoidal boxes, or “bath tubs” as more commonly used in the U.S., with dimensions shown in Fig. 3.

In order to retain the existing adjacent bridge and ramps in the interchange complex, as well as providing sufficient clearance on the Ben Schoeman freeway underneath, the new bridge, depth had to be kept as low as possible. This dictated a deck with an overall depth of circa 1,200 mm (3’-11 ¼”), resulting in a L/d (span/depth) ratio of 32.9 (or 3% d/L) for the long inner span of the continuous beam (Fig. 1).

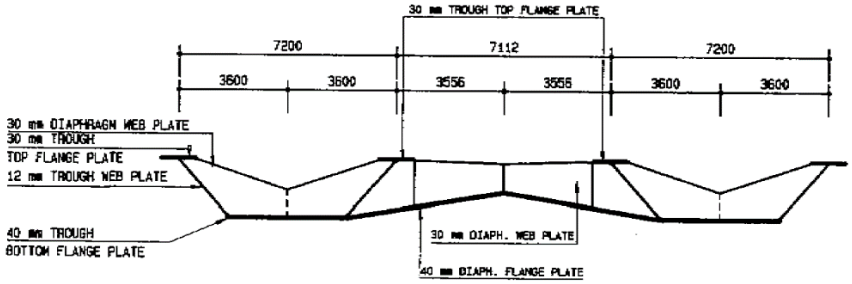

Figure 3 - Cross-Section Through Span Diaphragms

The steel troughs are only 1.0 m (3.3 ft) deep, see Fig. 2, with 600 mm x 30 mm (23 5/8” x 1 3/16”) top flanges and 2,000 mm x 40 mm (6’-6 3/4” x 1 9/16”) bottom flange, reduced to 30 mm (1 3/16”) thick in the jack spans.

The webs are made out of 12 mm (15/32”) plates, which slope inward from a width of 3,600 mm (11’-9 3/4”) at the top to a width of 2,000 mm (6’-6 3/4”) at the bottom, thus in an angle with the vertical of 38º 40’.

The deck comprises two bath tubs in order to optimize the spacing of the transverse supports for the concrete deck slab. This facilitated the use of a uniform thickness, reduced to the possible minimum – 200 mm (8”) for a slab post-tensioned in both directions.

In the case of a straight structure, it might have been cheaper, even though less aesthetical, to use a multiple-beam system of single-web girders, and, in fact, this hypothesis was analyzed at the design stage. However, due to the existing 52º skew, added to a radius of curvature of 450 m (1,480 ft), this solution would necessitate piers with large crossheads which would extend over the freeway, when the planned 6 lane widening would be implemented, thus infringing with the vertical clearance.

To avoid this problem, it was decided the use of twin box girders, as this structural system provides ample torsional rigidity, capable of withstanding the large torsional moments caused by the offset of the bearings.

With the open-top boxes, there were in the past distortion and twisting problems with the boxes, before they were connected and made integral with the concrete slab. To avoid this problem in the interim handling and construction phases, a trussed bracing system was used for bracing the bath tubs at the top flanges level. The use of the twin tubs has the added advantage of providing narrow bottom flanges, which do not require stiffeners.

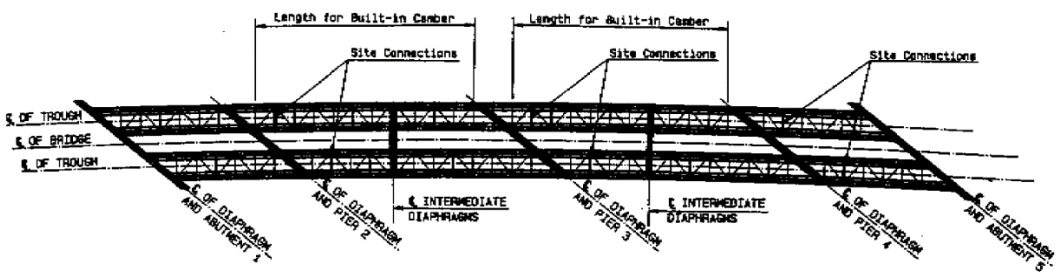

Figure 4 - Plan of the Steel Structure with the Horizontal Bracing

HORIZONTAL BRACING

The used horizontal bracing consists of a trussed system of diagonals and cross-elements comprised of back-to-back unequal double angles.

The bracing was made stronger than required by calculation (wind analysis), in order to have a reserve for the self-weight torsional moments during transportation and erection, partly resulting from the severe skew and curvature, in order to avoid any warping or geometric change, quite critical in the present case where the troughs have to accommodate and support a prefabricated concrete deck.

The connection points of the bracing were chosen to coincide with the vertical stiffeners of the webs, in order to avoid the introduction of additional stresses on the top flanges.

Initially the bracing was specified as to be removed once the concrete slab would be monolithical with the steel troughs. This was intended to avoid major loss of longitudinal prestress in the concrete slab, due to the restraint that the bracing would introduce against creep and shrinkage.

At the construction stage, both the contractor and the designer have realized the difficulty of this task, due to the little space available inside the boxes, to manipulate the bracing elements of over 3.5 m (11.5 ft) in length and weighing about 80 kg (176 lb) each.

Further analysis on the creep and shrinkage of the concrete panels was made, taking the bracing restraint into account, in an attempt to find a better solution for the removal or not of the horizontal bracing.

Finally, the decision was to leave the A40 steel bracing inside the troughs, provided that they would be painted to avoid electrochemical reaction with the COR-TEN steel of the tubs. Also, in order to reduce the bracing restraint, the connection bolts would be alleviated by a quarter of a turn when construction was

complete and, to avoid their local unscrewing and possible noise due to vibrations, they were provided with Nyloc rings.

Creep and shrinkage losses were also minimized by stressing the transverse prestress of the precast planks as late as possible, i.e., when they were, at least, 3 to 4 weeks old, which was achieved within the contractor's work schedule. The interaction due to composite behavior with the steel structure was also possible to postpone until the planks were 2 months old at the earliest, therefore significantly later than initially assumed for the design.

DIAPHRAGMS

The diaphragms were reduced to a minimum and were provided only over the supports (3 piers) and in the center of the two main spans.

These diaphragms are connected to the steel troughs by full moment connections, in order to increase their effectiveness, and were designed with triangular cut-outs of 0.5 m (1'-7 ½") to allow access inside the troughs for inspection.

Initially all the diaphragms were idealized as single flanged and triple webbed tee sections, which were thought to be a simplification from a construction point of view. In fact, they would provide the required stiffening, while locally rigidifying the bottom flange over the bearings, allowing ample access for welding as the webs were spaced at 400 mm (15 ¾").

However, the wholly welded conception of the site splices created a time problem for the site operation, as the whole erection and fastening of the bridge components was to be attempted in a single weekend.

To accommodate the scheme proposed by the contractor, the diaphragms over the piers and in the mid-spans were changed to I sections and the site connections to High Strength Friction Grip (friction critical in the U.S.) bolted connections.

The diaphragms over the abutments were kept with their initial shape, as the space between the webs was to be filled with concrete of high-density aggregates (magnetite ore, in the present case), with an increase in the unit weight by almost 2.5 times (circa 0.350 kip/ft^3).

This requirement was intended to counteract the uplift tendency shown by the bearings under the spines at the acute angle side, caused by the severe skew of the bridge, in accumulation with small length of the jack spans in relation to the adjoining main spans (approx. 50%, instead of the desirable 70% to 75%).

CONCRETE DECK

The concrete deck comprises two very distinct parts: one cast in-place corresponding to the trapezoidal regions at the ends, created by the skew; the other composed of precast rectangular concrete planks.

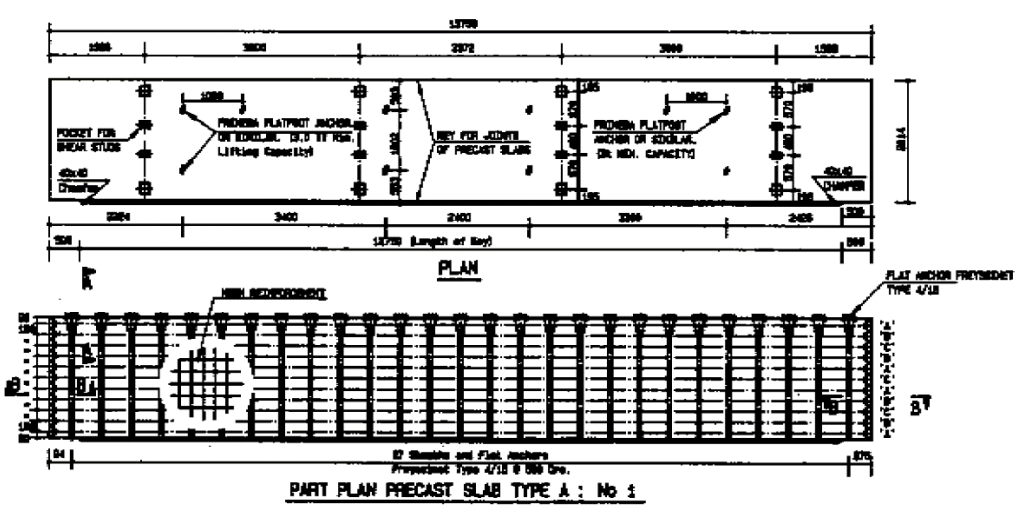

Figure 5 - Plan View of the Slab Planks (pockets for stud connectors & PT cables)

The part cast in-place was executed on left-in-place steel sheathing, supported on the top flanges of the troughs and on the steel bracing, which was still not a common procedure in the RSA as considered costly at that time (lost wood formwork being used), and had only started being experimented on in Europe.

The precast concrete planks are 2.0 m (6'-6") wide over the entire width of the deck (13.95 m) and only 200 mm (8") deep. This reduced thickness caused some difficulty in accommodating the prestressing tendons in both directions, in accumulation with the rebars, also in two directions on top and bottom.

The concrete planks were provided with pockets for posterior inclusion of the stud steel connectors that made them composite with the steel troughs. Between the panels, joints were created with a recessed configuration to facilitate the threading of the longitudinal PT tendons, and which were grouted once all the planks would be in place and prior to the longitudinal post-tensioning.

Both the in-place portion and the precast portions of the deck were post-tensioned in the transverse direction of the deck, using plastic coated unbonded monostrands. Each panel was post-tensioned with 11 x 15.9 mm (0.6") strands, of which 4 are straight and run near the top face, other 4 are also straight and run near the bottom face, and the remainder 3 are draped to enhance the flexural and shear capacity of the slab at the critical stress points.

Owing to the high susceptibility to corrosion and ensuing failure of unbonded prestress systems, particular attention was paid to the anchor details, providing them with grease filled screwed-on caps.

The sequence of erection, solidarizing together and attachment to the steel troughs of the concrete planks, was defined in as great detail as possible. At construction stage, it was further discussed with the contractor and adapted to suit his available means and to accommodate suggestions for possible simplification.

LONGITUDINAL POST-TENSIONING

1) By using prestressing tendons, placed inside the concrete slab;

2) By pre-cambering, where the steelwork is placed at a position above the final level of supports prior to erection of the deck, and, once the structure has become composite, lowering it on to the final support level.

The reason for such apparent sophistication was to satisfy the requirement of a fully prestressed structure at the concrete level, i.e. no tensile stresses, during the service life of the structure, and also to meet the ultimate limit state requirements, as the bonded longitudinal PT cables are the only continuous "reinforcement" provided, owing to the utilization of precast concrete panels. Also, the lack of space to provide for more longitudinal cables, if only prestressing tendons were used, was another deciding factor.

If one takes into consideration both the tensile stresses from hogging moments over the supports, as well as the tensile stresses caused by the restraint opposed to free shortening from creep and shrinkage of the concrete slabs, it may be appreciated the necessity for the high degree of prestress as was introduced in the structure.

It must be noted that the province of the Transvaal (nowadays Gauteng) is subjected to very low levels of humidity, at circa 40% average, partly due to its altitude (about 6,000 ft), which makes the effects of creep and shrinkage particularly significant.

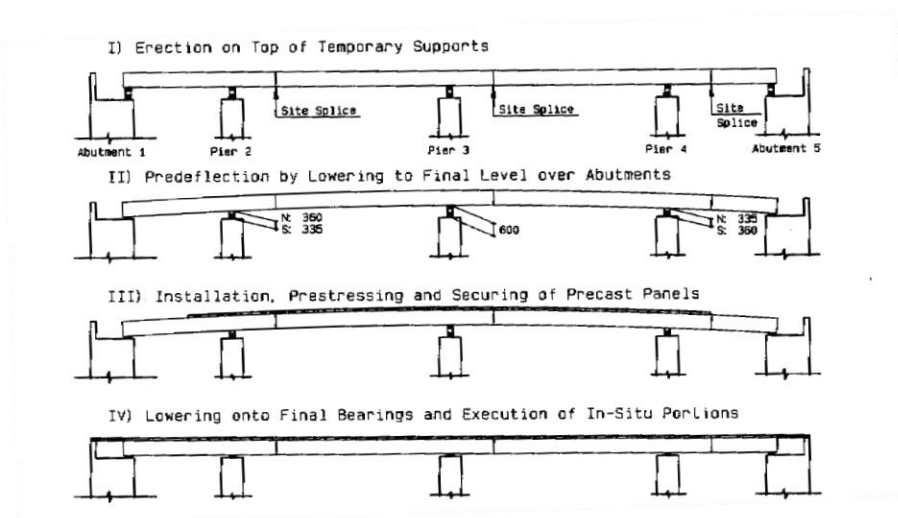

Figure 6 - Precamber Sequence

Thus, the structure was post-tensioned with 38 cables of 4 x 12.9 mm (0.5”) high strength (270 ksi), low relaxation strands and by precambering the structure from a height of 600 mm over the central support, varying to zero over the abutments (see Fig. 6).

The post-tensioning introduced by the cables is applied before the concrete slabs are integral with the steel troughs. This procedure, which brought about some additional complication of execution, was followed in order to avoid the redistribution of prestress into the steel structure.

SUBSTRUCTURE

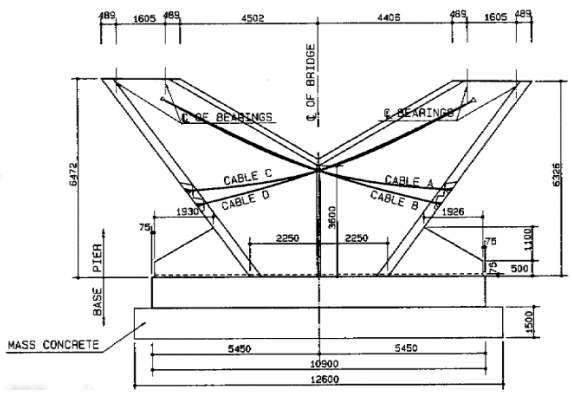

Figure 7 - Elevation of Piers

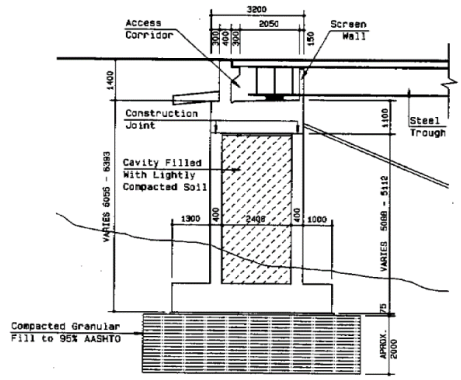

Figure 8 - Section through Northern Abutment

Concrete Piers

The piers were detailed in a very wide “V” shape, created not only as an aesthetic feature, but also to minimize foundation work, and, especially, to take them as far as possible from the freeway traffic lanes.

Because of the strong inclination of the pier’s legs, intended to match the inclination of the webs of the steel troughs, they had to be prestressed to avoid congestion of reinforcement along their upper faces, as well as long term deformations due to creep of the concrete, which also had an advantage for durability.

The piers are founded on spread footings of relatively small dimensions (virtually no seismic demand in the RSA), particularly in the transverse direction, taking advantage of the reduced self-weight of the bridge in relation to a concrete solution.

Because the bearing stratum is some 3 to 5 meters (10 to 17 ft) below ground, and in order to increase the stability of the foundations, the difference in level was made up of mass concrete, lightly reinforced at the bottom and connected to the footings by dowels around the perimeter.

Abutments

The abutments are of a completely closed box configuration, which was partly due to the need to create an access corridor to the ends of the steel troughs for access to their inside for inspection, which required a fairly wide cap beam. By taking advantage of this extra width, it was possible to have fairly thin walls both at the front and at the back of the abutment, thereby providing support at both ends for the cap beam.

The boxes so created were filled up with fill material to increase their stability and to create a surface against which to cast the cap beams.

This type of abutment has the advantage to be easily detailed and constructed, and obviates the need for counterforts, especially at the higher side of approximately 8.0 m (26.2 ft); furthermore, it allowed to eliminate the need for wingwalls. In a way, the abutments reflect the main philosophy behind the whole conception of this structure, which was to try and simplify design and construction of the same, by using somewhat thicker or larger elements in favor of reducing labor to a minimum, but at the same time not compromising on aesthetics.

CONSTRUCTION

The modular steel design meant that sections of the bridge could be constructed elsewhere and then brought to the site by road, assembled, and bolted in position as preparation for the final on-site welding.

The sections of the steel girders were transported from Dorbyl’s factory, located some 60 km from the site of construction, on low-beds semi-trailer trucks, the largest being 40 m (131.2 ft) long and 60 tonnes heavy.

Erection of the steelwork was carried out on the 21.st July 1990, in one full day (Fig. 9 and 10), using a 225-ton crawler crane and two 40-ton mobile cranes for the erection of the steel sections.

Figure 9 - Construction - Early Morning (7:00 am) on 21.st July 1990

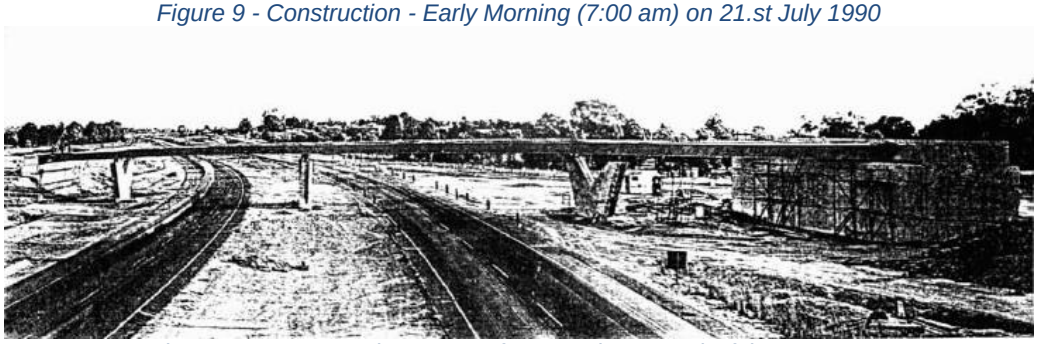

Figure 10 - Construction - Late Afternoon (5:00 p.m.) of the same Day

As each section was installed and joined with temporary bolting, welding was done on the outside of the troughs, above the closed roadway. Inner welding was completed the following week.

The rectangular precast deck slabs were constructed on site while the steel troughs were being fabricated. A formwork platform that accurately matched the final deck alignment and profile and transverse elevations was used to cast the slab panels against, and these were partially post-tensioned in the transverse direction with unbonded individually protected monostrand tendons.

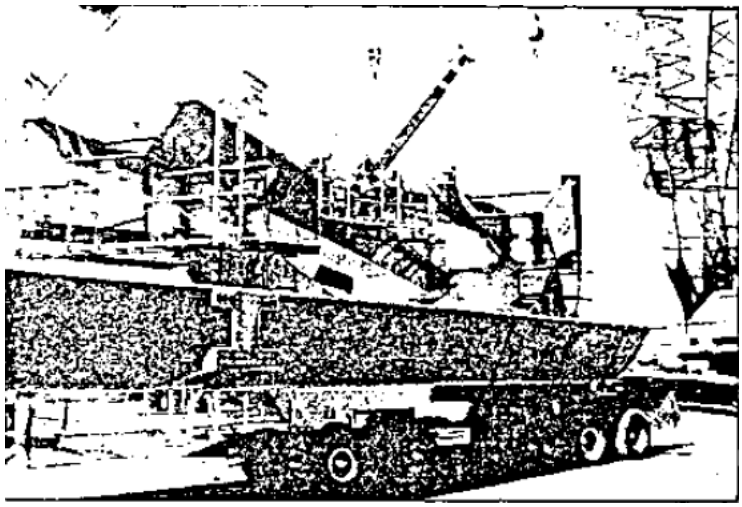

Figure 11 - Placement of the Open Trough Girders

After the deck slabs were lifted in position, the longitudinal tendons were threaded through and tensioned to provide a slab compression of 8.0 MPa (1.16 ksi). The stud connectors were installed, and openings at the joints were closure grouted.

The transverse tendons were then fully tensioned to provide a compressive stress of 3.6 MPa (0.522 ksi) in this direction and uniformly distributed along the entire bridge.

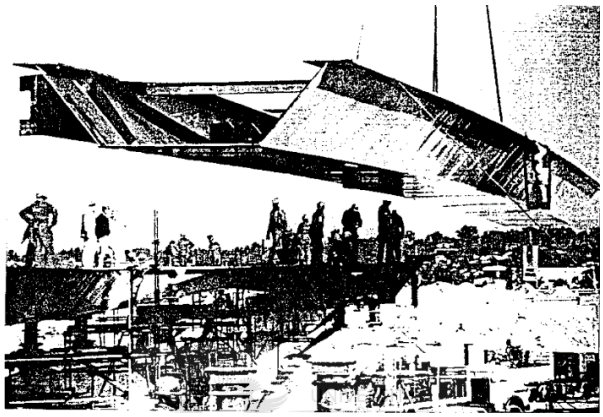

Figure 12 - Large Girder Element being handled by Single 225t Crawler Crane

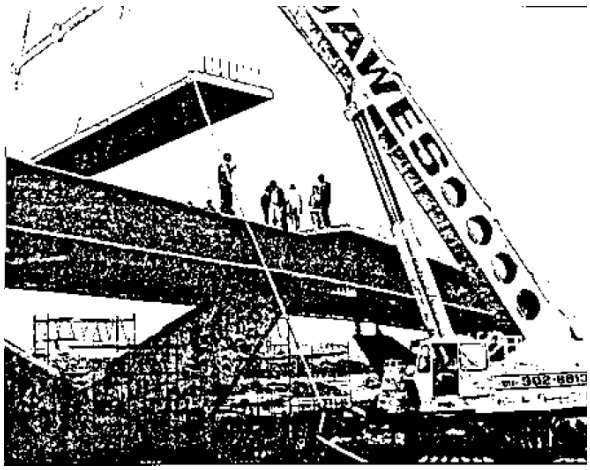

Figure 13 - Precast Concrete Deck Planks were placed with Minimal Disruption to Traffic on the Freeway

DYNAMICS

From a dynamic point of view the deck exhibits enhanced dynamic behavior, which needed to be properly taken into account, owing to a very slender deck associated with a relatively lightweight structure.

Design Phase

Due to the high slenderness of the bridge, particular attention was already paid to its dynamic behavior.

Therefore, it was aimed in design to keep the natural frequencies of the bridge out of the range where wind excitation could be a problem, and also so that the human perception or discomfort to vibrations would be minimal, or, at least, acceptable. Unfortunately, at the time the dynamic recommendations relative to bridges, especially the ones incorporating pedestrians' traffic, were very simplistic and were more directed to pedestrian bridges than to mixed usage of pedestrians and vehicular traffic.

Thus, the 1.st natural frequency of the bridge, which happened to occur in the vertical direction, was tried to be kept above 3.0 Hz. The horizontal vibrations did not prove to be of concern, given the much higher stiffness of the bridge in the horizontal direction, for which the concrete deck acts as a very rigid diaphragm.

In a dynamic analysis of a preliminary design the natural vertical frequency of the bridge was found to be too low, around 1.8 Hz, and it was decided to use double bearings under the steel troughs transverse diaphragms, as well as to strengthen the diaphragms to take better advantage of their stiffening influence on the structure, in particular owing to the strong skew according to which they are oriented, and it was further decided to make the concrete barriers/parapets integral with the deck, additionally, to avoid cracking in the long continuous balustrade, to pre-tension them axially with adherent cables of 4 x 12.5 mm strands.

Through the accumulation of all the modifications made to the preliminary design, it was possible to achieve a final natural frequency for the structure of about 5.5 Hz, thus fulfilling the proposed vibrational requisites.

Experimental Campaigns for Measuring Vibrations

Right after the completion of construction and before opening the structure to traffic, in October 1990, the Witwatersrand University in Johannesburg (Wits) was commissioned by the Transvaal Provincial Administration (TPA) to conduct a series of tests to monitor the long term static behavior of the bridge deck, including the effects of concrete creep and shrinkage, temperature and deck deformations during construction. These measurements were extended by Dr. G. Krige to include preliminary dynamic measurements.

About three years later after construction, the Rand Afrikaans University, later incorporated in the University of Johannesburg, took the initiative of undertaking a dynamic evaluation of the bridge, in order to establish some possible implications of the present type of design.

Results:

- Measured fundamental frequencies: in the range of 2.5 Hz to 3.2 Hz, with average of 2.78 Hz;
- Accelerations: 1.3 m/s^2 (4.27 ft/s^2 = 0.13 g) to 3.7 m/s^2 (12.14 ft/s^2 = 0.38 g) for accelerometers in the mid-span;
- Damping: was determined to be in the region of 2.7% to 4.4% of critical damping.

As seen, the measured natural frequencies were conforming with present AASHTO recommendations of a minimum of 3.0 Hz. Although the minimum vibration frequencies come out a bit low, the measured accelerations, which are the really important parameter to measure pedestrian comfort, according to the SETRA criteria, still fall within tolerable levels of comfort for the passage of trucks on the bridge.

The measured accelerations, were measured somewhat higher than the desirable 0.10 g to about 0.4 g in the critical spots (mid-points of the central spans). While for 0.10 g (1.3 m/s^2) we would still be in the region of Min. comfort (see Fig. 14), for 0.4 g (3.7 m/s^2) we are already somewhat into the uncomfortable zone.

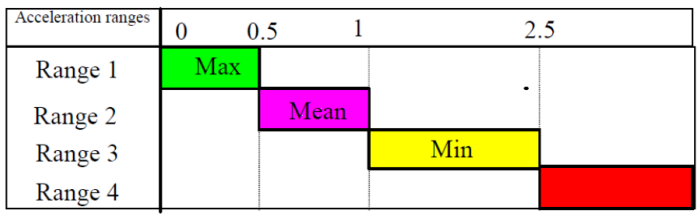

Figure 14 - Acceleration Ranges (in m/s2) for Pedestrian Comfort to Vertical Vibrations according to SETRA

Note: SETRA is the French Authority for Technical Studies of Roads and Highways.

The bridge is located between traffic lights and upon the green signal, traffic is released to pass over the bridge at high density and low speed. The frequencies thus imposed are of the same order as the natural frequencies of the structure and this explains the pronounced effect of heavy slow-moving trucks which can be felt, although without feeling concern, by standing at mid span.

Imposed Frequencies by Passing Vehicles (Hz)					
Speed		**Spacing between vehicles**			
(km/h)	(mph)	2.0 s	1.5 s	1.0 s	0.5 s
120	75.0	0.60	0.73	---	---
60	37.5	1.05	1.67	1.86	---
30	18.8	1.84	2.29	3.01	4.36
15	9.4	3.00	3.56	4.36	5.65

Figure 15 - Excitation Frequencies exerted by Traffic (Hz)

Above in Fig. 15 a table is presented, where a correlation was made between the velocities and spacing of 5.0 m (16.4 ft) long vehicles (between axles) with the resulting imposed frequencies, while driving over a bridge with 40 m long spans, where the critical frequencies were highlighted in orange:

As can be seen, for this particular situation, even if the natural frequencies were above 4.0 Hz, or even 5.0 Hz, the bridge would still be excited by the slow-moving traffic leaving the traffic light after being stopped.

With nowadays availability of sophisticated analysis software and powerful computation resources, a dynamic time step analysis for moving vehicles on a 3-D model could have been done and this situation possibly identified. However, given all the existing constraints those same would still not be easy to overcome, unless by making the steel structure much stiffer by increasing the thicknesses of the flanges of the steel troughs, at the cost of significant extra weight.

At present there is much more guidance regarding the perception by humans of vibrations and the resulting accelerations of structures on which they might be standing and the author recommends, in addition to the SETRA recommendations, the provisions in ISO 2631 and 10137, associated with a dynamic vehicle analysis on a 3-D model of the structure, covering the possible different conditions of traffic movement.

As for the calculated damping, the results came in line with what could be expected, which was assumed to be of the order of 2% to 3% of critical, to which might have added the beneficial effect of the asphalt overlay.

In conclusion, in spite of being very light and slender, the Bothasfontein bridge would still marginally satisfy pedestrian comfort requirements in today's terms for normal speeds and densities of traffic, and marginally fall in the uncomfortable zone for dense low speed conditions of traffic.

COMPARISON WITH INTERNACIONAL PRACTICE

The type of structure selected – concrete deck on steel girders – although at the time of design commonly used in Europe and the United States, had very seldomly been used in the R.S.A.

In fact, only 4 such structures were found in the search carried out by the author, of which only one of them carrying vehicular traffic, in the form of a double-decker urban viaduct in the city of Johannesburg.

Therefore, giving the novelty in South Africa at that time, it was just natural and prudent to look for international examples to compare with the present design and check whether the design parameters were in the previously used ranges of values.

Coincidentally, the author had come across a survey conducted in Europe, the United States and Japan, which covered 82 structures of the box girder composite steel and concrete type, in which ranges or trends for the main parameters were identified.

The parameters for the comparison basis were identified as (numbering in accordance with table below):

1. Area of boxes in relation to the controlling span;
2. Depth-to-span ratio;
3. Ratio of box depth to bottom flange width;
4. Ratio of bottom of flange width to its thickness;
5. Concrete slab thickness;
6. Bridge weight per unit area of deck.
7. Area of tension bottom flanges;
8. Inverse of spacing of diaphragms.

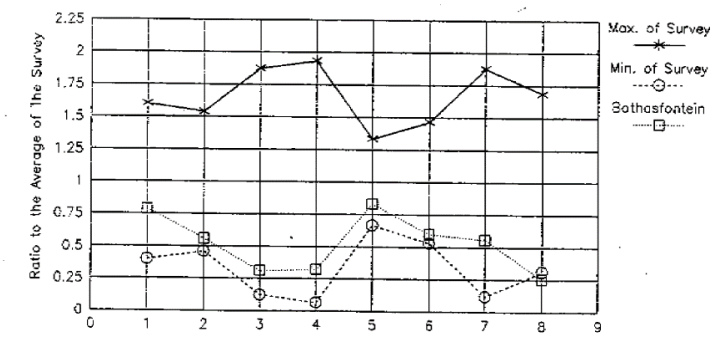

Figure 16 - Composite Bridges of Steel and Concrete - Curved and Skew Box Girders Results from Survey

The comparison made can be summarized in the graph above (Fig. 16) where it can be seen how the Bothasfontein bridge fits within the ranges of variations that were reported, lying significantly below the average values, which is a fact that inspires a certain confidence in the design, not only from a safety point of view, but also considering the economic factor.

It must also be noted that the structures surveyed were mainly constructed between 1965 and 1976 and, therefore, some reduction on the structural dimensions were to be expected on more recent structures owing to improvements in construction technologies, materials and computation means.

CONCLUSION

The bridge at the Bothasfontein interchange, designed and constructed in the Republic of South Africa in 1990 was a precursor in today's approach to design and construction that we conventionally call ABC, or Accelerated Bridge Construction, and, indeed, this design of almost 3 decades ago, would quite fit as a good example in today's practice.

For me, my senior supervisor and senior partner of VKE – Mr. Marthinus Rautenbach – and the whole design team, the design of this structure and its construction with our dedicated technical support to the site, was an exciting endeavor that has left a mark for the rest of our professional lives.

In our opinion, in spite of a possible better dynamic behavior, owing to the limited computational resources back then, it was well deserved the two professional awards given to this structure: SAICE's (South African Institute of Civil Engineers) **Award for the Most Outstanding Civil Engineering Achievement for 1991**, and the SAISC (South African Institute of Steel Construction) **Winner 1990 Steel Construction Awards**.