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# SWING BRIDGE OVER SUEZ CANAL AT EL FERDAN
# SOIL - STRUCTURE INTERACTION AND DEFORMATIONS

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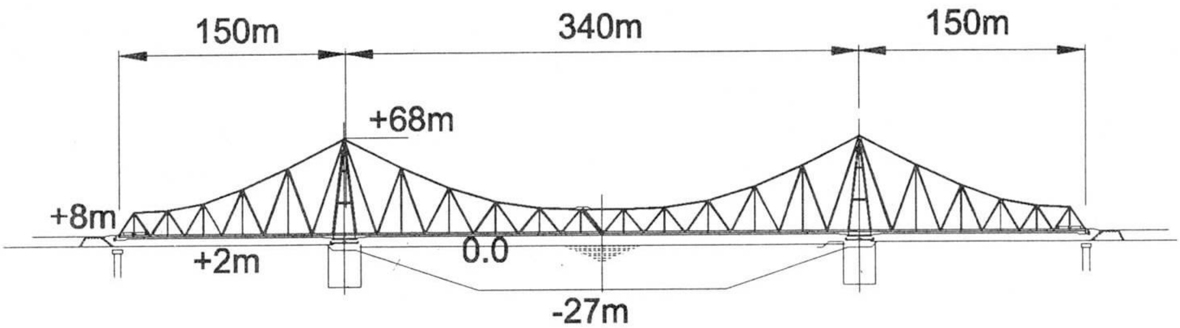
Luc Maertens got his civil engineering degree at the Catholic University of Louvain (KUL), Belgium.
Since 25 years he has been active in design of quay walls, bridges, and large concrete structures in Besix Design Department. Since 1991 he was also lecturing project management at the Catholic University of Louvain.

## SUMMARY




For the third time a swing bridge over the Suez Canal at El Ferdan (Egypt) is under construction. After completion it will be the largest moveable bridge in the whole world, and this bridge will reinstate the former road and rail link across the Suez Canal.
Taking into account the fact that foundations are in the slope of the Suez Canal (27,5 m deep) and that heavy wind (250 km/h) and a considerable earthquake with an acceleration =150 cm/sec$^2$ has to be considered, a solution with a steel superstructure and piled piers was proposed by the Consortium KRUPP – BESIX – ORASCOM for the design and construct tender organised by the Egyptian National Railways. Ship collision on the piled piers was also considered.
In the parked position parallel to the Canal bank each of the superstructure halves can be seen as a 150-m single-span girder having a cantilever of 170 m. In the closed position the main span length over the Suez Canal is 340 m both end spans are 150 m.
The foundations consist of a pile-raft foundation composed of 36 bored piles diameter 1,50 m and a rigid pilecap with a thickness of 4,5m. Soil structure interaction is considered for static as well as for dynamic loads.
Design and construction are governed by a quality plan according to the ISO 9000 standard.

The following subjects are developed in this paper:
1 Ship collision
2 Transfer of horizontal loads from superstructure to pile cap
3 Earthquake behaviour of the pile foundation
4 Soil – structure interaction
5 Comparison of calculation models.

# 1 SHIP COLLISION

According to the specifications, a protection jetty has to be constructed on both sides of the canal in order to protect the bridge in parked position against impacts of vessels. Vessels with a water displacement up to 300,000 ton and a sailing speed of 8 km/h have to be considered.
For the evaluation of the impact of a vessel against the pier, we considered the same several conditions.

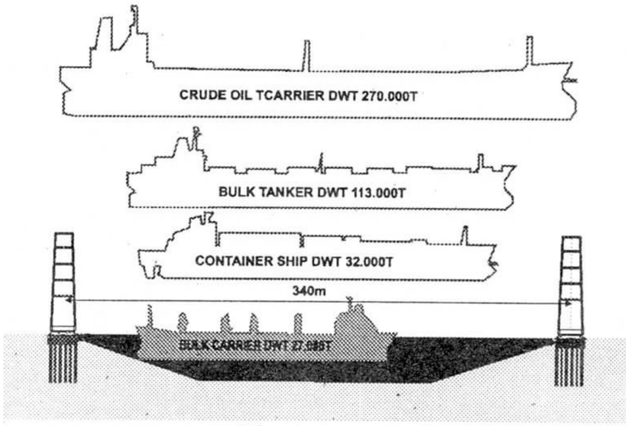



Fig.1.1

Vessels normally sail in the direction of the axis of the canal. If the vessel is out of control, it can deviate from this direction. In that case, the vessel will bump against the slope of the canal.
As long as the collision angle is less than 45° for tankers (cylinder bow) or 26° for container ships (sharp bow), the vessel will slide off the slope. At greater collision angles, the vessel will run up and if the slope is steep dig into the canal slope.
On figure 1.1 one can see that such large vessels cannot sail to the pier in a direction perpendicular to the canal axis. However, this extreme case was analysed to evaluate the impact forces on the pier. In figure 1.2 one can see the calculation method which was applied.

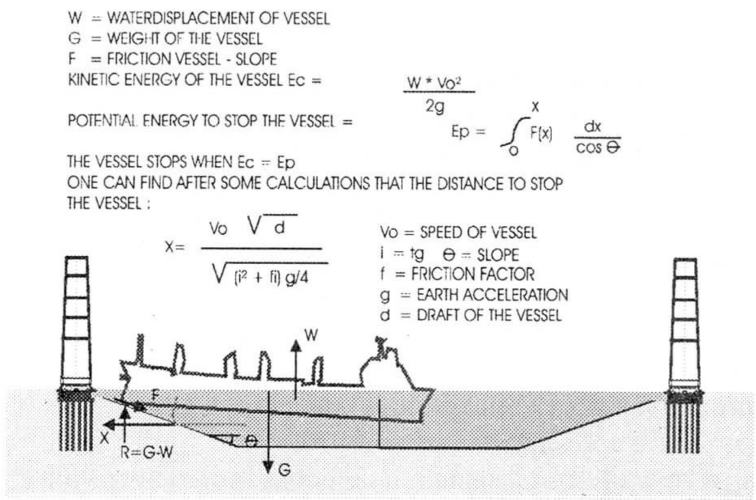



Fig 1.2

The "friction factor" for a vessel which forefoot slides on the slope is 0,4 (ref. 1). This "sliding" assumption is on the safe side, since "digging" generate passive earthpressure and thus also a larger friction factor.
Applying the general formula to the Suez Canal (f = 0,4; i = 1/3), and taking into account that the slope of the canal bank reaches the pier at water level we easily find:

$$\boxed{a = 3d - 1{,}29\, Vo\sqrt{d}}$$

With: a = distance to the pier the vessel stops (m)
Vo = speed of the vessel at the moment that the vessel touches the slope (m/sec)
d = draught of the vessel (m)

In figure 1.3 one can see that only vessels with a draught smaller than 3m can reach the pier.

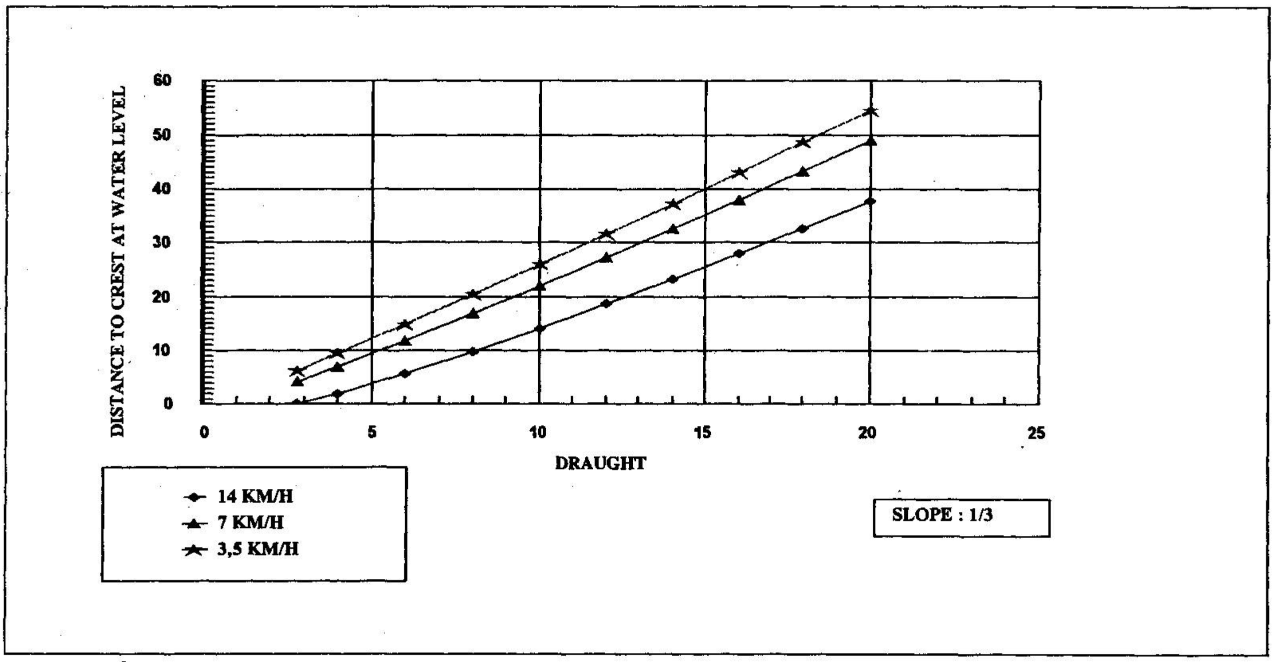



Fig. 1.3

On figure 1.4 and 1.5 (ref. 2) one can see that the impact load is 15 MN (for a speed of 14 km/h = 3,9 m/sec).

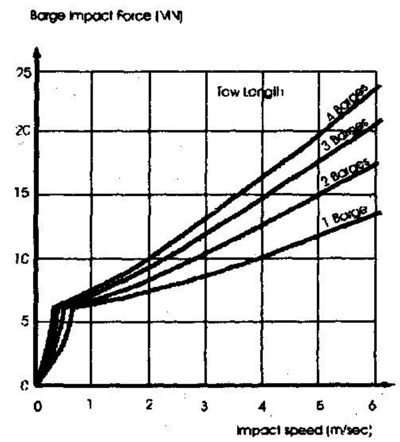



Fig. 1.4

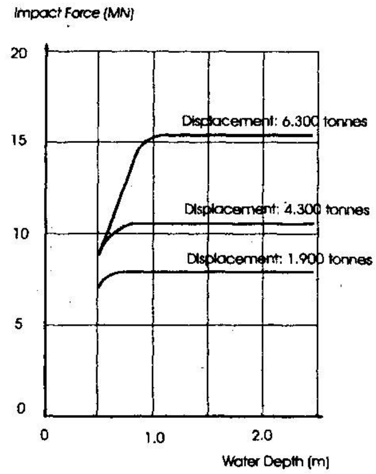



Fig. 1.5

## 2 TRANSFER OF HORIZONTAL LOADS FROM SUPERSTRUCTURE TO PILE CAP

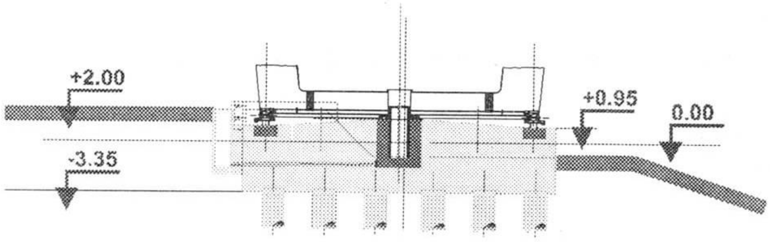



Fig. 2.1

Vertical loads are transferred from the superstructure to the pier cap by mean of a swing gear which consist of a circular structure supported by two layers of conical rolls. In the centre of the swing gear there is a pin to transfer the horizontal loads to the pile cap (Fig. 2.1). The pin itself is a steel cylinder with a diameter of 1,3 m and a wall thickness of 10 cm. The length is 3,5 m and is embedded in the concrete over 2,53 m. The horizontal load to be transferred is 9140 KN under wind load and 16.955 KN under seismic load.

Designers of the superstructure performed the predesign based on a former publication (ref. 3). For the final design more detailed calculations were performed by the designers of the substructure: a 3-D elastic-plastic model was considered (Fig. 2.2).

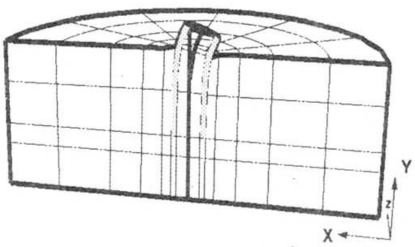



Fig.2.2

The steel tube is modelled as an elastic material and for the concrete we selected a Mohr-Coulomb model with $\varphi = 30°$ and $c = 15.000$ KN/m², which corresponds to an unconfined strength of 30 MPa

Two calculations were performed:
For an horizontal load of 9140 KN, a displacement of 0,66 mm is found assuming that concrete can resist to tensile stresses and 2,9 mm assuming that concrete cannot resist to tensile stresses.

To define the reinforcement around the pin, horizontal planes were calculated: in figure 2.3 one can see the deformations and the tensile stresses in the concrete assuming tensile can occur between pin and concrete, in figure 2.4 no tensile is allowed between pin and concrete, one can see a gap between concrete and pin. The reinforcements are calculated using the isolines of tensile stresses given in figure 2.4.

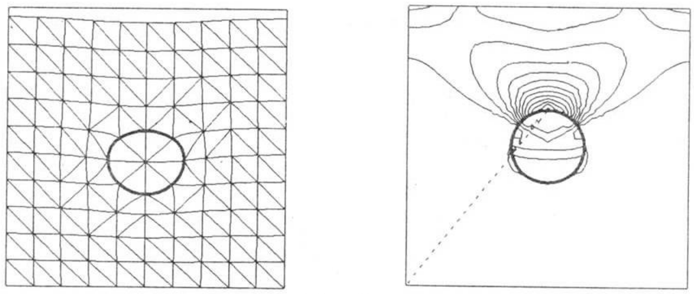


Fig. 2.3

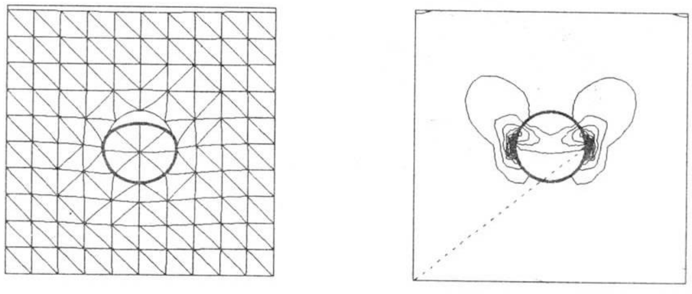


Fig. 2.4

## 3 EARTHQUAKE BEHAVIOUR OF THE PILE FOUNDATION

Earthquake calculations were performed according to different methods:

5.1 By the Consortium

5.1.1 For the superstructure using Response Model Analyse in SAPN based on the Acceleration Response Spectrum calculated by Dr. O, Ramadan.

5.1.2 For the substructure using DYNA IV (Novak) which is a 3-D Dynamic elasto-plastic model.

5.2 By the Consultant

The consultant performed own calculation.

It is interesting to evaluate the dynamic calculations by comparing the "earthquake coefficient" this is the factor you have to apply to the vertical loads to find the horizontal loads for pseudo dynamic calculations (for a ground acceleration of 150 cm/sec²).

| Location | Superstructure | | Pile Foundation | |
|---|---|---|---|---|
| | Consortium | Consultant | Consortium | Consultant |
| Horizontal load (KN) | 16.128 | 11.868 | 16.995 | 21.600 |
| Vertical load (KN) | 75.890 | 72.667 | 129.147 | 129.147 |
| H/V | 0,20 | 0,16 | 0,13 | 0,17 |

## 4 SOIL-STRUCTURE INTERACTION

For a major bridge more particularly when deformations are very important, one cannot assume that the superstructure, the pile cap and the piles behave independently i.e. that the stiffness of this structural elements together with the stiffness of the soil do not influence each other.

More advanced computer programs allow modelling the pile cap and piles as elastic elements together with the soil as elastic-plastic elements linked together by interface elements.

For the El Ferdan Bridge, various calculation methods were applied to evaluate the deformations of the foundations and the bending moments in the piles.

Intensive soil investigations including borings, SPT, triaxial tests, oedometer tests and PMT were carried out. The selection of the soil parameters for the calculations is very important and the input of experts is required for major structures.

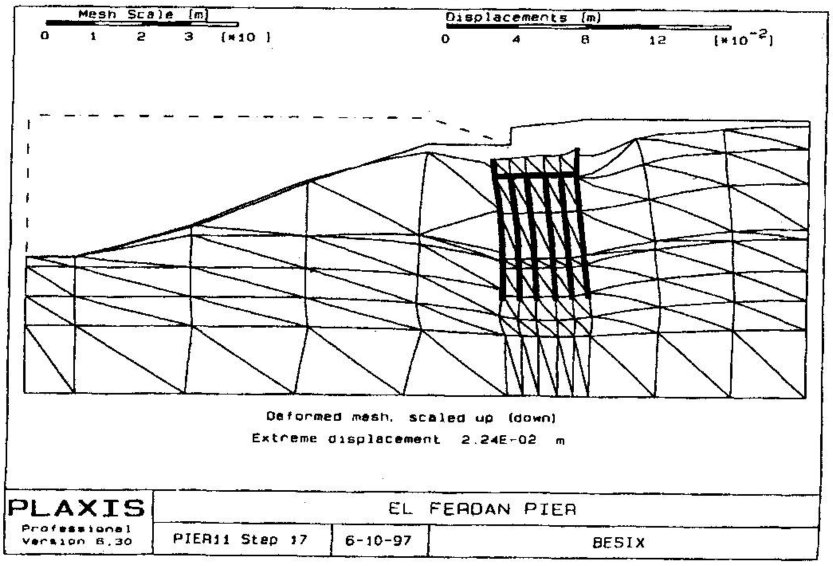



Fig. 4.1 Open Bridge Hmax – Deformations

For the dense sand layers ($D_r > 80\%$), we assume $\alpha = 32°$ and G = 20.000 + 800 z (KN/m²), with z = depth in meter. For the clay layers, ($I_p = 50$, $C_u = 250$ KPa) we assume $\varphi = 25°$ C = 25 KN/m² and G = 12.000 KN/m². The G-values are $G_{50}$ values for long term behaviour. The interaction factor (= tan $\alpha_{pile}$ / tan $\varphi_{soil}$) is 0,7 for sand layers and 0,6 for clay layers.
The history of the stresses in the soil is repeated in the different steps of the Plaxis FEM calculations. Starting from the initial situation (before dredging of the canal), initial stresses are generated. Then following steps are analysed: Dredging of canal, installation of piles, pouring of pilecap, erection of bridge, and other loadcases (see Fig. 4.1). The results concerning the deformations are given hereafter:

| Case | Horizontal displacements (mm) | Vertical displacements (mm) | Rotation (rad x 1000) |
|---|---|---|---|
| 1. Dead load bridges | 1,45 | 15,61 | -0,53 |
| 2. V max | 5,94 | 21,77 | -0,24 |
| 3. M max | 7,64 | 20,95 | 0,39 |
| 4. H max (closed bridge) | 4,75 | 15,53 | -0,34 |
| 5. H max + V max (closed bridge) | 6,95 | 21,01 | 0,07 |
| 6. H max (open bridge) | 8,33 | 16,67 | 0,15 |
| 7. Earthquake | 17,27 | 15,80 | 0,68 |

## 5 COMPARISON OF CALCULATION MODELS

Design is not only compute, it is also evaluate and decide. As an example, the values of Maximal Bending Moments in the piles are given according to different calculation models:

| **MAXIMAL BENDING MOMENT IN PILES (Serviceability state)** | | | |
|---|---|---|---|
| Horizontal load | Type of calculation | Maximal moment (KNm) | |
| | | H (V = 0) | H + V |
| Wind | Handcalculations according Franke | 607 | - |
| Wind | Simplified model | 2.114 | - |
| Wind | Plaxis calculations | - | 3.318 |
| Wind | M-Pile calculations | - | 4.363 |
| Earthquake | Handcalculations according French standard | 705 | - |
| Earthquake | Plaxis calculations (pseudo-dynamic) | 3.354 | 5.874 |
| Earthquake | DYNA IV (full 3 D – dynamic) | 3.516 | - |
| Earthquake | M-Pile (pseudo-dynamic) | - | 7.140 |

Finally a bending moment of 8707 KNm was considered for ultimate design state.

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