# The restoration of Rio Tinto pier: functional and foundation aspects

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ABSTRACT

The pier of the Rio Tinto railway, 120 years old, is being restored. The foundation consists of a timber platform, bolted to cast-iron screw piles resting on a very soft mud. As an underpinning was expensive and problematic, a thorough investigation, including a study of soil-structure-foundation interaction, has been carried out to check the state of the foundation.

The conclusion is that the materials are in good conditions and the stresses remain within allowable limits.

## INTRODUCTION

In 1873, the British Rio Tinto Company purchased from the Spanish government the cupriferous iron pyrites mine of Rio Tinto. During 1877 the output of ore was 750.000 t. The Huelva pier of the Rio Tinto railway was constructed by this company in the 1870's for the shipment of mineral from the mines, and also of goods from the railway Seville-Huelva. The pier is placed in the estuary of River Odiel, in the S.W. of Spain (fig.1).

The designer followed the general lines of other British piers of the 19th century. The construction was carried out under the direction of T. Gibson, which wrote a paper on the pier [2].

We shall review briefly some of the most important characteristics of the works.

## GEOLOGY

The pier rests on holocene deposits (mud and shells) followed by pleistocene terraces (v. Justo [3]).

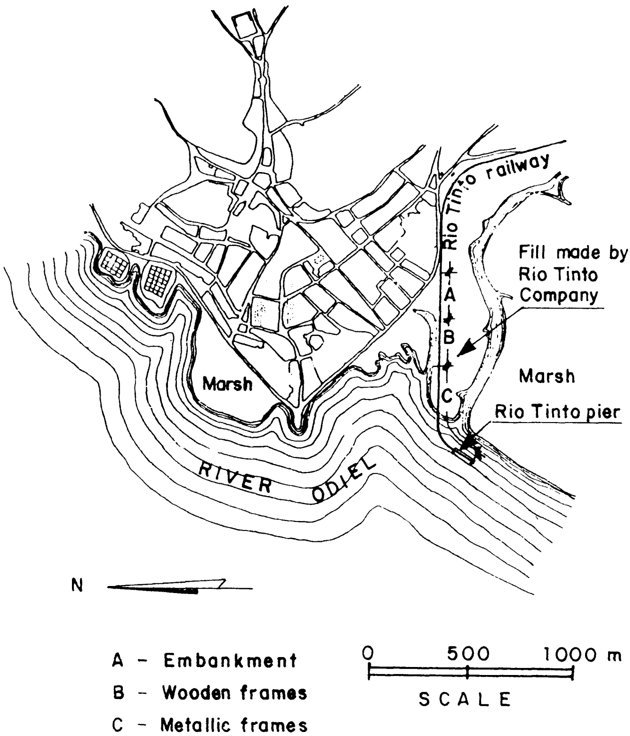


Figure 1. Plan of Huelva at the end of the 19th century, showing Rio Tinto railway and the marshes (Gonzâlez Vilches [1]).

## LAYOUT AND FOUNDATION

The description of the characteristics, construction and loading tests of the foundation occupies most of Gibson's paper. The pier is founded on soft blue day (fig. 2).

Figure 1 shows the layout of the pier. The part that originally was founded under high water was formed by 60 frames with wrought-iron girders, struts, stays and tie rods, and cast-iron columns, with a total length of 579 m between the axes of the first and last frame (fig. 1, 2 and 3). This distance is made up of twenty-nine spans of 15.24 m each, and thirty groups of cast-iron screw piles and columns, these being 4.57 m from centre to centre of the columns (fig. 2 and 3). Each group of eight piles is joined by a wooden platform, forming a piled slab (fig. 4). At deep water the transverse distance between piles was 3.66 m (fig. 4 and 5).

The cast-iron screw-piles are 41 cm in diameter, and 32 mm thick. The lower screw blade is 1.5 m in diameter, with a 15 cm pitch.

The depth of the piles ranged from 4.6 m (frames 1 to 16) to 9.8 m (frames 49 to 60).

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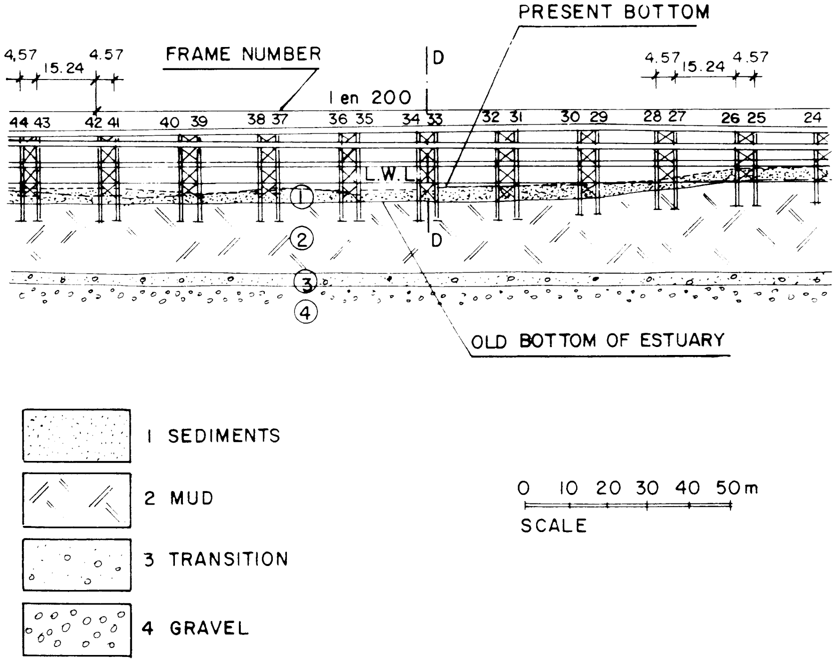


Figure 2. Longitudinal section of pier showing the foundation, old bottom and present bottom of estuary, and twenty metallic frames.

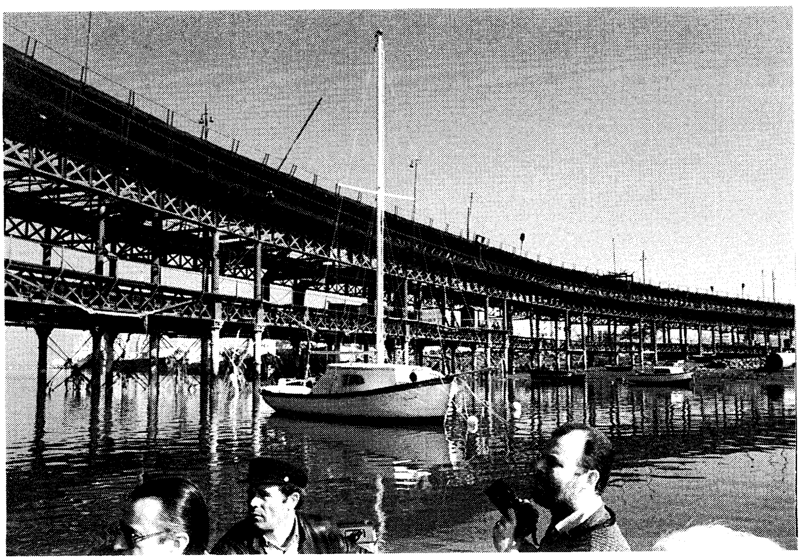

Figure 3. View of the pier during low tide.

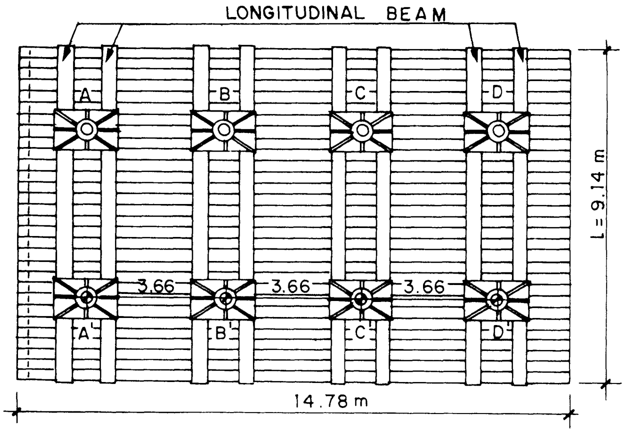


Figure 4. Plan of deep water platform.

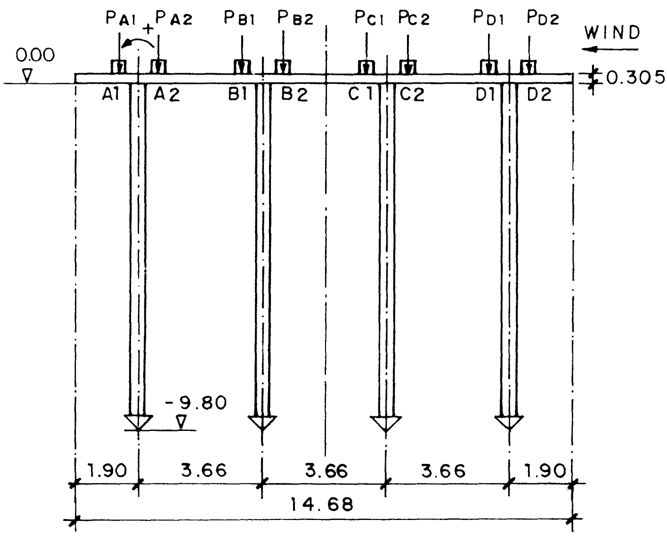


Figure 5. Cross-section of platform 29-30.

In the 29 former frames, the material at the bed of the river opposed relatively little resistance to pile driving.

At the shore end it was found that the piles were incapable of supporting more than 33 kPa.

As a reasonable bearing capacity could not be obtained with a reasonable number of piles, it was decided to screw together these to timber platforms, formed by beams 30x30 cm placed transversely to the central line of the pier (Fig. 4). The base plates of the cast-iron columns are supported by eight beams placed in the longitudinal direction of the pier, one at each side of the screw piles (fig. 4, 5 and 6).

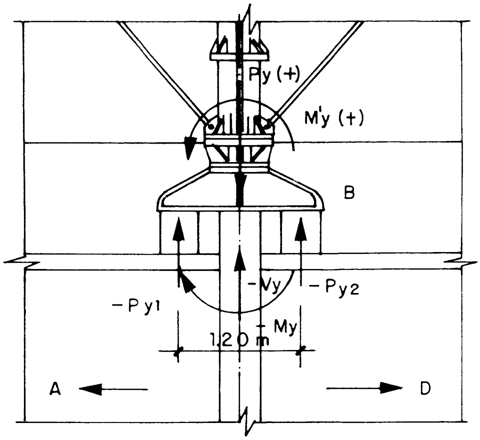


Figure 6. Interconnection node of structure-platform-pile.

## TIMBER PLATFORM

Compression and bending tests are described by Gibson [2]. These tests, carried out before the pier was constructed, have been evaluated by Justo [3], with the results indicated in table I.

Table I. Parametes of Baltic Red Wood obtained in the 1870's

| | Elasticity modulus (MPa) | Limit of proportionality | | Strength (MPa) |
|---|---|---|---|---|
| | | $\sigma_y$ (MPa) | $\epsilon_y$ (%) | |
| Compression | 12.000 | 25 | 0.17 | 32 |
| Tension | | | | 40 |
| Bending | | 31 | 0.28 | |

Samples of the platform have been carefully taken in 1990. The average compressive strength (41 MPa) is larger than the one of the 1870's. The aspect of the recently cut wood is excellent (fig. 7).

## LOADING TESTS AND SOIL PARAMETERS

Loading tests were carried out during contruction, on the 30 groups of cast-iron screw piles and/or platforms. An evaluation of these tests carried out by Justo [4] allows us to obtain the soil parameters.

The undrained shear strength increases linearly with depth according to the equation:

$$c_u\ (kPa) = 2.5\ z(m) \tag{1}$$

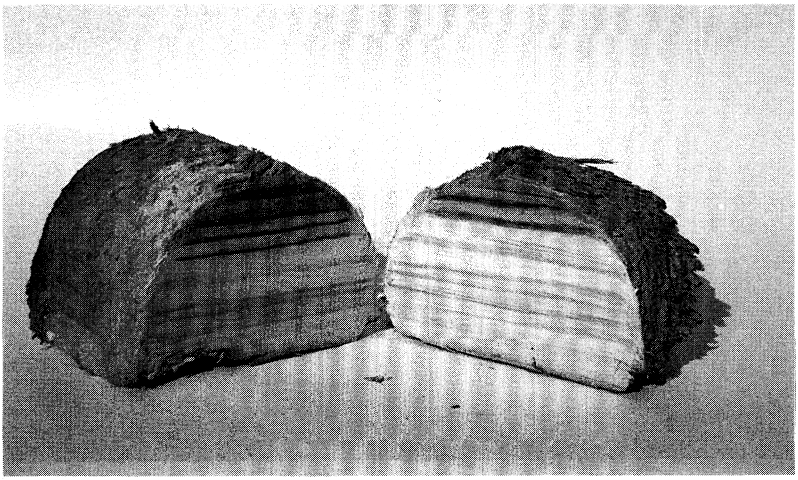

Figure 7. Aspect of recently cut wood from the platform.

## METHOD OF CALCULATION

### Generalities

The construction sequence and history of the pier has been closely followed so as to find the stress history of the different materials.

The method is described by Justo [4], and the detailed calculation included in [3]. The structure is described by Picón et al. [6].

A study of soil-structure-foundation interaction has been carried out, so as to check the state of the foundation. The method is essentially linear-elastic, although deviations are produced due to the following reasons:

1. Buckling of the tie-rods.
2. When the bearing capacity at the pile point is reached, the pile load stays at this value.
3. The pile load is also limited by the non-appearance of tensions above the pile blade. This happens in long term calculations due to the low values of effective stresses.

Owing to this and to differences in the moduli of deformation of the soil for loading and unloading-reloading, an incremental method must be followed.

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Loading tests

Three kinds of loading tests have been carried out.

In the first series the platform was charged without being fastened to the piles.

In a normally consolidated soil, as the mud of Huelva is, the undrained modulus of elasticity increases linearly with depth:

$$E_u = mz \tag{2}$$

Under these circunstances it may be shown that the soil behaves as a Winkler model (v. Jiménez Salas et al. [7]). The m coefficient above is related with the modulus of subgrade reaction by the equation:

$$m = \frac{3}{2} k_s \tag{3}$$

In the loading tests, an average modulus of subgrade reaction, for the deep water platforms, of 180 kN/m³ was measured. Equation 3 permits to find m.

In the second series of tests only the piles of each platform have been charged. It was assumed that the whole load was applied on the blade, as, when the pile is scre wed down, a complete remoulding of the sensitive soil along the shaft is produced. The settlement produced by the loaded pile under its axis has been calculated by integration of Mindlin equations in the blade area. The undrained modulus of deformation, $E_u$, of the soil at the point of the piles is found. Equation 2 allows us to find m, which approximately coincides with the value obtained from the first series of tests.

The third series of tests was carried out on the platform joined to the piles. For this case the interaction platform-pile-soil was fully taken into account as explained later.

Calculation of platform 29-30

A detailed calculation of platform 29-30 has been performed, complemented by simpler calculations of other platforms. The depth of mud at this platform is 18.7 m.

Following classical concepts of Soil Mechanics, two calculations have been made: short term (including the constructio) and long term, corresponding respectively to undrained and drained conditions of the soil.

The following steps have been considered.

1. Short term.
   a) Construction of superstructure on the piles only
   b) Wind action
   c) Loading test on platform separated from piling and structure
   d) The platform is joined to piling
   e) Unloading of platform

f) Train loading
g) Wind action
2. Long term.
a) Load of sediments
b) Unloading due to closing of pier

At any stage the following elements are considered separately (fig. 6):
1. The structure up to its connection with piling and platform.
2. The piling and adjoining ground.
3. The platform and support ground.

Apparently the ground participates jointly in the second and third elements, but it has been possible to separate the roles of the soil owing to the following reasons:

1. The platform is affected mainly by the vertical deformations of the surface soil. It has been assumed that the shear strength of the contact platform-soil is nil, as the corresponding stress only appears during wind action (short term), and, as indicated by equations 1 and 2, under these conditions the shear strength and modulus of deformation of the soil at surface is nil. So the wind action is supported by the piles.

2. On the other hand, the piles are supported mainly by the vertical action of the soil at the point and by the horizontal action of the soil surrounding the pile. The settlement produced by the load transmitted by the platform at the points of the piles has been introduced in the calculations.

The three "elements" indicated above are interconnected at 8 nodes (fig. 4) Y (A, B, C, D) and Y' (A', B', C' and D').

The following conditions must be fulfilled:
1. Compatibility of deformations at nodes
2. Equilibrium of forces at nodes

The first condition means that the deformations of the "elements" connected at the node <u>at the stage of calculation</u> must be equal.

Only loading conditions with transverse symmetry for each platform have been considered, so that the "forces" for Y and Y' are equal.

Equilibrium of vertical forces at node Y:

$$\Delta P_Y - \Delta V_Y - \frac{1}{2} (\Delta P_{Y1} + \Delta P_{Y2}) = 0 \qquad (4)$$

Equilibrium of moments:

$$(\Delta M'_Y - \Delta M_Y) + \frac{1.22}{2} 1 (-\Delta P_{Y1} + \Delta P_{Y2}) = 0 \qquad (5)$$

where:
$\Delta P_{Y1}$, $\Delta P_{Y2}$ = vertical action increment per linear meter of the node on the longitudinal beams of the platform.

For gravity loads there is symmetry, and the number of unknowns is so reduced fo one half.

For each stage, the force-displacement relationship for the structure will be:

$$\begin{Bmatrix} \Delta P_Y \\ \Delta M'_Y \\ \Delta H_Y \end{Bmatrix} = [K] \begin{Bmatrix} \Delta s_Y \\ \Delta \theta_Y \\ \Delta \rho_Y \end{Bmatrix} + \begin{Bmatrix} \Delta P_{Yo} \\ \Delta M'_{Yo} \\ \Delta H_{Yo} \end{Bmatrix} \qquad (6)$$

where:
Y = A, B, C, D
$\Delta s_Y$ = vector of support settlement increment
$\Delta \theta_Y$ = vector of support rotation increment
$\Delta \rho_Y$ = vector of lateral displacement of support

$\begin{Bmatrix} \Delta P_{Yo} \\ \Delta M'_{Yo} \\ \Delta H_{Yo} \end{Bmatrix}$ = increments of support actions for nil deformation of it.

There will be equations similar to (6), but based upon flexibility matrices, for platform and piling.

For the platform:

$$\begin{Bmatrix} \Delta s_{Y1} \\ \Delta s_{Y2} \end{Bmatrix} = [F] \begin{Bmatrix} \Delta P_{Y1} \\ \Delta P_{Y2} \end{Bmatrix} \qquad (7)$$

$\Delta s_{Y1}$, $\Delta s_{Y2}$ = vectors of longitudinal beams settlement increments
$\Delta P_{Y1}$, $\Delta P_{Y2}$ = vectors of vertical action increments on the longitudinal beams
[F] = flexibility matrix of longitudinal beam on elastic foundation

$$\Delta s_Y = \frac{\Delta s_{Y1} + \Delta s_{Y2}}{2} \qquad (8)$$

$$\Delta \theta_Y = \frac{\Delta s_{Y1} - \Delta s_{Y2}}{1.2} \qquad (9)$$

Equations 4 to 9 allow the solution of the problem.

For long term calculation, given the non-elastic characteristics of the mud, it was necessary to calculate the settlement of the soil using the edometric method and to find equivalent values of $k_s$ and E' corresponding to these settlements.

## CONCLUSIONS

The main stresses suffered by the platform correspond to short term, train loading - wind action, and correspond to:

$$M_{max} = 32 \ m{\cdot}kN/m$$
$$Q_{max} = -46 \ kN/m$$

The corresponding bending moment and shear resistance at the limit of proportionality are:

$$M_l = 470 \ m{\cdot}kN/m$$
$$Q_l = 4700 \ kN/m$$

The present state of platform and cast-iron piles is also excellent.

So, although pending from a loading test, it will not be necessary to underpine the foundation.

## ACKNOWLEDGEMENTS

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