LOS CARAS ISOLATED BRIDGE IN THE 2016 MUISNE ECUADOR EARTHQUAKE: BEHAVIOR OF PIERS AND DEEP PILE FOUNDATIONS

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ABSTRACT

The Los Caras Isolated Bridge is located in the Ecuadorian province of Manabí between the cities of Bahía de Caráquez and San Vicente. It is the longest bridge of Ecuador with a 2-km long main and 48 piers. This paper focuses on the bridge foundations that consist of friction steel pipe driven piles. The piles penetrate the soils for 40 to 74 m and did not reach bedrock that is at an estimated depth of 100 m. A Soil-Structure Interaction (SSI) model was developed to study the year-2016 observations especially at Pier 12 that suffered the highest lateral displacements, and Piers 10 and 44 which deformed according to the original design that relied on P-Y spring modeling. The bridge was resilient and remained in service immediately after the earthquake, with minimal repair damage required. Hence, the Los Caras Bridge is a resilient example of a major project, highlighting the use of advanced seismic protective technologies in anticipation of major earthquakes.

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Los Caras Isolated Bridge in the 2016 Muisne Ecuador Earthquake: Behavior of piers and deep pile foundations

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ABSTRACT

Los Caras Bridge is located in the Ecuadorian province of Manabí between the cities of Bahía de Caráquez and San Vicente. It is the longest bridge of Ecuador with a 2-km long main and 48 piers. The design was undertaken by the Ecuadorian Army Corps of Engineers and construction was completed in 2010. Each span consists of six beams, two of which are supported on triple pendulum seismic isolation systems, totaling 152 bearings in the central section. Following the 2016 Mw7.8 Muisne earthquake, the bridge remained functional after experiencing high accelerations, possible liquefaction, and the largest ever recorded triple pendulum bearing displacement of 65 cm at Pier 12, with an average of about 35 cm.

This paper focuses on the bridge foundations that consist of friction steel driven piles. The piles penetrate the soils from 40 to 74 m and did not reach bedrock that is at an estimated depth of 100 m. The upper 10 m of subsurface conditions are poor, typical of river estuaries, including a top 4-m thick soft river mud layer, followed by potentially liquefiable sand deposits. These layers do not practically contribute to the axial capacity and additionally provide minimal lateral resistance.

A Soil-Structure Interaction (SSI) model was developed to study the 2016 observations especially at Pier 12 that suffered the highest lateral displacements, and Piers 10 and 44 which deformed according to the original design that relied on P-Y spring modeling. The results agree with the observed behavior, showing that the bridge foundations did not exhibit significant vertical settlements, but the foundation moved horizontally due to inadequate upper soil lateral support. This affected the bridge performance, especially at Pier 12 where the accelerations and bearing deformations exceeded design levels. Despite this distress, the bridge was resilient and remained in service immediately after the earthquake, with minimal repair damage required. Hence, the Los Caras Bridge is a resilient example of a major project, highlighting the use of advanced seismic protective technologies in anticipation of major earthquakes.

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Introduction

The Los Caras Bridge is located in the Ecuadorian province of Manabí between the cities of Bahía de Caráquez and San Vicente. It is the longest bridge of Ecuador with a 2-km long main and 48 piers, and is shown in Figure 1. The design was undertaken by the Ecuadorian Army Corps of Engineers and construction was completed in 2010. Each span consists of six beams, two of which are supported on triple pendulum seismic isolation systems, totaling 152 bearings in the central section. This bridge is located near the epicenter of the 1998 \( M_w \) 7.1 Bahía de Caráquez earthquake, and 120 km away from the epicenter of the 2016 \( M_w \) 7.8 Muisne earthquake, remaining functional after this earthquake.

![Figure 1. Plan of View of Los Caras Bridge, Courtesy of Ecuador’s Army Corps of Engineering (GPS coordinates: 0°36’33.58”S, 80°24’58.68”W)](image)

Geotechnical Conditions of the Bridge

The site is characterized as an estuarine environment formed by alluvial deposits with depths between 25 and 85 m overlying weathered rocks. Unaltered rock is found at depths generally greater than 100 m. Twenty (20) deep borings with Standard Penetration Tests (SPT) and collecting soil samples with Shelby tubes were used to determine the generalized stratigraphic soil profile showed in Figure 2. The foundations of the bridge are open-end steel friction driven pipe piles with diameter of 1.22 m, wall thickness of 20-25 mm and length varying from 32 to 65 m, also shown in Figure 2. Figure 3a and 3b is based on the data from pile drive monitoring on piers 10, 12 and 44, during the construction of the bridge. Three high dynamic tests in each pile were carried out because of the high variability of the soils conditions in each pile.

![Figure 2. Soil Stratigraphic Profile and Pier Piling Depth for the Los Caras Bridge](image)
The conditions of the implantation site required the use of deep foundations. The heterogeneous soil of the site and the length of the bridge forced to perform an individual geotechnical design for each pile. Two different methods were used to calculate the skin friction (pile’s capacity) and tip capacity. The design of the pile was carried out in accordance with LRFD Design (AASHTO, 2007) [2]. Skin Frictional piles were employed, to stand 38 bridge piers (Pier 7 to 44) in the central segment. Only one pier foundation (Pier 6) could use end bearing piles. Steel piles employed in the bridge foundation were tube-shaped; piers were built on reinforced concrete; the superstructure supporting system was based on triple frictional pendulum (TFP) isolators; the superstructure was built on posttensioning beams and reinforced concrete decks; pier span was 45 m.

To control the strength detrimental effect of the steel degradation, under ocean waves (in the splash zone), the upper part of the steel piles had a reinforced concrete pile inside from 8 m to 14 m long. Each pier foundation required between 8 and 9 piles (Figure 4) that were installed in the perimeter of the pier, separated from each other 4.60 m. Figure 4 shows a plan view of the pile distribution.

The theories of Matlock and O’Neil & Reese (1999) [3] were used to calculate the lateral load design. The stress - strain curves (P - y) at variable intervals up to the bottom of the planned pile are shown in Figure 5a, 5b and 5c. Lateral capacity of piles was reduced during design, due to the shadow effect.
Frictional and top capacity of piles were verified and adjusted during construction, by pile dynamic tests, with Pile Dynamic Analyzers (PDA), and CAPWAP technique. The central segment of Los Caras Bridge (1710 m long), between Bahia and San Vicente, was a typical Soil-Structure Interaction (SSI) problem, which became more complex with the introduction of a seismic isolation system between sub-structure and super-structure. The following sections will cover these aspects.

**Structural Design**

**Basic Seismic Criteria for Structural Design**

The seismic hazard study for the bridge area revealed that the peak ground acceleration (10% probability exceedance, in 50 years) at the rock was around 0.40 g, and as such, it was considered as the referential design acceleration for sub-structure and super-structure. The analysis was performed for the Design Basic Earthquake (DBE) and Maximum Considered Earthquake (MCE), hazard levels for the design of the bridge piers and seismic isolators. The analysis of the bearings was developed on the basis of assumed mechanical properties of the Friction Pendulum
Coefficients, Figure 6 shows the idealized behavior of Triple Pendulum Bearing of the Los Caras Bridge at both hazard levels.

![Figure 6. Idealized Behavior of Triple Pendulum Bearing of the Los Caras Bridge](image)

All bearings were verified by prototype testing. Results from analysis indicated that the expected displacement at DBE was of 35 cm (14”), while at MCE was of 52 cm (20.5”). The minimum factor of safety for vertical resistance, under seismic plus gravitational loading, was 1.50, with no earthquake reduction factor. The structure was analyzed by spectral modal analysis, in accordance with the 2008 Ecuadorean Construction Code and verifications were carried out using seismic wave propagation models, to detect possible problems of asynchrony within the structure.

**Structural design of the seismically isolated bridge**

The soil influence on the structural behavior of the bridge was established through the use of soil-structure interaction models (Figure 7a, 7b), where soil discrete element system was represented as springs, having inelastic behavior, according to P-Y curves.

![Figure 7a. Computer Model of Bahia Access and Central Segment of the Bridge](image)  
**Figure 7a. Computer Model of Bahia Access and Central Segment of the Bridge**

![Figure 7b. Computer Model of San Vicente Access and Central Segment of the Bridge](image)  
**Figure 7b. Computer Model of San Vicente Access and Central Segment of the Bridge**

The starting point for the elimination of soil strata that would not provide lateral capacity, was the average between the expected mean and maximum scours (Figure 8).

![Figure 8. Riverbed, Mean Scour and Maximum Scour Level](image)  
**Figure 8. Riverbed, Mean Scour and Maximum Scour Level**

The analyses included the lateral contribution of soil (springs in the models) at different depths.
Pier #10 (P10) included a depth of 12 m, P12 of 14 m and P44 of 8 m. In the remaining liquefiable strata (refer back to Fig. 2), the lateral bearing capacity was reduced to 20%. The specific design of the seismic isolators indicates a fundamental vibration period of 2.6 seconds. The combined fundamental vibration period of the bridge section near Bahía (between P6 and P24 piers) with isolators was estimated to be 2.65 seconds in the longitudinal direction, and 2.59 seconds in the transverse direction. The lateral relative displacement of the isolators was estimated to be 29 cm, which was consistent with results obtained from an independent analysis by others. The displacement results of the pile heads and of the isolators, for the design conditions, are described in the following Table 1:

<table>
<thead>
<tr>
<th>Pier ID</th>
<th>Longitudinal displacements</th>
<th>Transversal displacements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pier head disp. (cm)</td>
<td>Top isolator displ. (cm)</td>
</tr>
<tr>
<td>P10</td>
<td>17.72</td>
<td>44.60</td>
</tr>
<tr>
<td>P12</td>
<td>19.67</td>
<td>44.57</td>
</tr>
<tr>
<td>P44</td>
<td>12.57</td>
<td>48.67</td>
</tr>
</tbody>
</table>

The relative displacements of the isolators in the bridge structure reached up to 32 cm, in the analytical modeling (Figure 10a, 10b), quite close to the quasi-static displacements of the isolator of 35.6 cm. Relative displacements in isolators reached 28.7 cm in digital modeling of P10, 27.5 cm in P12 (Figure 9a), and 36.1 cm in P44 (Figure 9b); close enough to 35.6 cm quasi-static displacement estimated in the isolator alone.

The design of the isolators was made without reducing the seismic force, while the design of the substructure and the superstructure managed a factor of seismic force reduction of 1.10, somewhat more severe than the recommended by the Ecuadorian Norm NEC15 [4], which allow up to 1.50, because the analytical models had smaller displacements than those of the independent isolator.

**Measured Displacements on Isolators**

The internal review of the 152 isolators of the Los Caras bridge (Figure 10), allowed to establish the real relative maximum displacements in each of the isolators, through the traces left on the sliding surfaces.
There were minor differences in measured displacements, among the 4 isolators in every pier. Measured maximum relative displacements in transversal direction (direction of major acceleration), under the April 16, 2016 Muisne Earthquake, for relevant piers were:

Table 2. Measured Maximum Displacements in Isolators

<table>
<thead>
<tr>
<th>Pier</th>
<th>Pier 10</th>
<th>Pier 12</th>
<th>Pier 44</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measured Displacement (cm)</td>
<td>31.5</td>
<td>52.0</td>
<td>31.5</td>
</tr>
</tbody>
</table>

The only important difference between maximum relative displacements calculated during design, and maximum relative displacements measured after the Muisne earthquake, occurred in P12 (27.5 cm calculated, and 52 cm measured), which represent 2.5% of all piers.

**Diagnosis of Pier #12 Problem**

After the April 16, 2016 earthquake, the bridge remained operational, as it revealed no visible damage. The source of the problems in P12 could be structural, geotechnical, or a combination of the two. The structure of the central section of the Bridge was revised in 3 instances, with 3 different objectives: 1) detect damage that can affect continuous operation of the bridge; 2) each seismic isolator was revised, and 3) each structural element was checked, in search of traces of plastic hinges, or cracking.

All four isolators of P12 reached their maximum design capacity to lateral displacement. The entire detailed revision process allowed discarding any structural constructive defect that could have caused the greatest displacement, including the possibility that an inelastic behavior had occurred at the head of the piles (Figure 11), so the exhaustive examination included piers, isolators and superstructure.
The challenge in P12 had a geotechnical nature. The pier structure was more flexible than expected during design, due to a lack of lateral support of the soil layers. Sand liquefaction and soft clays in upper layers did not bring enough lateral support to piles.

**Refined Digital Models**

A few months after the earthquake, 16 compaction piles were driven around P12 (Figure 12) in order to densify the soil around this pier.

![Figure 12. Piles Driven for Soil Densification, around Pier #12](image)

Two new models were built to increase the certainty of the global structural behavior and the observed displacements: 1) a SAP2000 model (which was used in the original design), but with global adjustment of the liquefaction criteria and the lateral contribution of the liquefied soils; and 2) an OpenSees model. Compaction pile monitoring allowed having an additional tool to identify soft and dynamically liquefiable layers, and the extension of such layers (Figure 13a, 13b, 13c).

![Figure 13a. Compaction Pile monitoring, on pier 12 (PD4)](image)
![Figure 13b. Compaction Pile monitoring, on pier 12 (PD12)](image)
![Figure 13c. Compaction Pile monitoring, on pier 12 (PD15)](image)

Densification pile driving revealed that, at required depths for pile lateral support (up to 30 m under water level or 20 m under riverbed), there was no clear stratigraphic limits. There was only local heterogeneity, even at distances lower than 5 m between piles. This fact had no impact on the vertical loading capacity of piles, since the main support is given by layers more than 30 m
deep, but affected their lateral support. A total of 9 out of 16 compaction piles detected liquefiable or soft soil layers at depths of 26 to 30 m under water level (16 to 20 m under riverbed).

**A Model with New Global Criteria to Estimate Liquefaction and Lateral Support of Soft Clays**

The first improved SSI-SAP model increased the thickness of soft or liquefiable layers, and eliminated its lateral support, up to 24 m under water level (the average depth of weak strata during pile compaction driving shown in Table 3). Results of such modeling in P12 are as follows (Figure 14):

![Figure 14. Pier #13 Displacements with Adjustments in Liquefaction Parameters](image)

The new model predicted a relative displacement in the isolator, of 37 cm, in the transverse direction, much better than the 27.9 cm of the original model, but still halfway, compared to the 52 cm measured on the bridge, after the earthquake. Certainly, permanent eccentric load was produced in the section P12 to P16, by the presence of a balcony, which was executed during the construction stage, and that was not contemplated in the design, but the main component of the change of behavior was looser and softer soil that did not provide lateral support in the upper layers of soil, and at the same time amplified locally soil accelerations. An additional component needs to be included when analyzing piles on P12, to improve displacement prediction. The soil category for digital modeling need to be reclassified as a softer soil, and the base acceleration needs to be recalculated to take into account local amplification. Transversal performance is much more affected than longitudinal performance when soil is softer or liquefiable, since in the longitudinal direction foundations from other piers limit superstructure displacements.

**A Model using OpenSees**

An additional analytical model was developed using the OpenSees software [5] and corresponds to P12. It is important to highlight the use of fiber templates for steel and concrete sections. Specifically, the “Steel02” model was used for structural steel, the “Reinforcing Steel” model for rebar steel, and the “Concrete01” model for unconfined and confined concrete. They were optimized according to reference [6]. In the top of the pile, TFP isolators were placed. The friction coefficients were assumed for minimum and maximum values. To represent the soil and the likelihood of liquefaction, the “PyLiq1” and “TzLiq1” models were used. The loads were gravitational and seismic. In the latter case, the acceleration time histories were scaled to reach lateral TFP’s displacements of around 50 cms. The results were similar to those obtained with the previously indicated models. This would confirm that the structural behavior was essentially elastic, but with displacement values affected by the liquefaction of the soil.
Conclusions

During the seismic event of Muisne, Los Caras bridge had peak ground accelerations, very close to the design accelerations, except for P12.

The characteristics of the foundation soil, forced the use of deep foundations supported by friction piles.

Among all 38 piles of the central section, only P12 presents lateral transversal displacements superior to those foreseen in the digital interaction models, used for the analysis and design.

SPT tests and the soil characterization tests conducted at the time of the design, were not enough to describe the soil behavior of the foundation on P12, under lateral seismic loads. In P12, the depth of the liquefiable strata and soft strata was greater than that detected during the geotechnical studies prior to the design of the bridge, and the traditional verification tests during construction.

The density compaction piles provided a better description of the presence of wide horizontal layers, which allows to considerably improve the modeling of the foundation structure.

The depth increase of inadequate soil layers, had no impact on the vertical bearing capacity of the piles, since the ability to support vertical loads depends on the resistance of deeper layers, but it had an impact on the lateral flexibility of the foundation structure due to horizontal seismic loads.

The structure of the foundation did not suffer significant impact because it was designed with a seismic force reduction factor of only 1.1, which left capacity reserve in both the elastic and the inelastic range.

The criteria and recommendations used to determine if certain granular strata levels are liquefiable or not, are probabilistic correlations, in which it is possible that a small percentage of such cases, are not met. In the case of Los Caras bridge the probabilistic correlation failure was 2.5% of all cases studied.

Local acceleration amplification may be very important when dealing with soft or liquefiable soils, in river deposits.

To improve bridge performance, under large seismic loads, there is a feedback need of test results made during construction, to make small adjustments to foundation design.

References