Missouri University of Science and Technology
Scholars' Mine

International Conference on Case Histories in Geotechnical Engineering

(2008) - Sixth International Conference on Case Histories in Geotechnical Engineering

14 Aug 2008, 7:00 pm - 8:30 pm

Ancient Landslide Reactivation at the Viaduct No. 1 Located on the Caraca-La-Guaira Highway in Venezuela

Luis B. Fargier-Gabaldón
Universidad de Los Andes, Mérida, Venezuela

Daniel A. Salcedo
Universidad Central de Venezuela, Caracas, Venezuela

Rosendo Camargo-Mora
Universidad de Los Andes, Mérida, Venezuela

Follow this and additional works at: https://scholarsmine.mst.edu/icchge

Part of the Geotechnical Engineering Commons

Recommended Citation

Fargier-Gabaldón, Luis B.; Salcedo, Daniel A.; and Camargo-Mora, Rosendo, "Ancient Landslide Reactivation at the Viaduct No. 1 Located on the Caraca-La-Guaira Highway in Venezuela" (2008). *International Conference on Case Histories in Geotechnical Engineering*. 2.
https://scholarsmine.mst.edu/icchge/6icchge/session08c/2

This Article - Conference proceedings is brought to you for free and open access by Scholars' Mine. It has been accepted for inclusion in International Conference on Case Histories in Geotechnical Engineering by an authorized administrator of Scholars' Mine. This work is protected by U. S. Copyright Law. Unauthorized use including reproduction for redistribution requires the permission of the copyright holder. For more information, please contact scholarsmine@mst.edu.

ANCIENT LANDSLIDE REACTIVATION AT THE VIADUCT NO. 1 LOCATED ON THE CARACA-LA-GUAIRA HIGHWAY IN VENEZUELA

Luis B. Fargier-Gabaldón
Universidad de Los Andes
Mérida-Venezuela

Daniel A. Salcedo
Universidad Central de Venezuela
Caracas-Venezuela

Rosendo Camargo-Mora
Universidad de Los Andes
Mérida-Venezuela

ABSTRACT

Reactivation of an ancient landslide detected in 1987 affected the southern side of the Viaduct No.1 located in the Caracas-La Guaira highway, which connects Caracas, capital of Venezuela, with its main seaport and the Simon Bolivar International Airport. The Viaduct was built in 1953 and covered a gorge of approximately 300 m. It consisted of three parallel double-hinged arch ribs made of plain concrete spanning over approximately 152 m and two smaller access Viaducts on either side of the arch rib span. This paper summarizes the results from geotechnical investigation, the evaluation of inclinometers readings and surface control points and the main rehabilitation measures conducted on the structure.

INTRODUCTION

The Caracas-La Guaira highway connects Caracas, capital of Venezuela, with its main seaport and the Simon Bolivar International Airport. It was built by Campenon Bernard, between 1950-1953 with a 60 million dollar cost along a geologically complex zone. It climbs 915 m. in 17.2 km. of length, from the town of La Guaira at sea level to the western side of Caracas, capital of Venezuela. Various major structures were built as part of the highway, including 3 bridges called Viaduct No. 1, 2, and 3, and two large tunnels.

The Viaduct No. 1 consisted of three paralleled double hinged arch ribs designed to resist compression stresses (and thus practically without longitudinal reinforcement) and covered central span of 152 meters measured from hinge to hinge as shown in Figure 1. Two smaller access Viaducts, covering spans of 49.7 m and 102 m, connected the arch span to the Caracas and La-Guaira side, respectively.

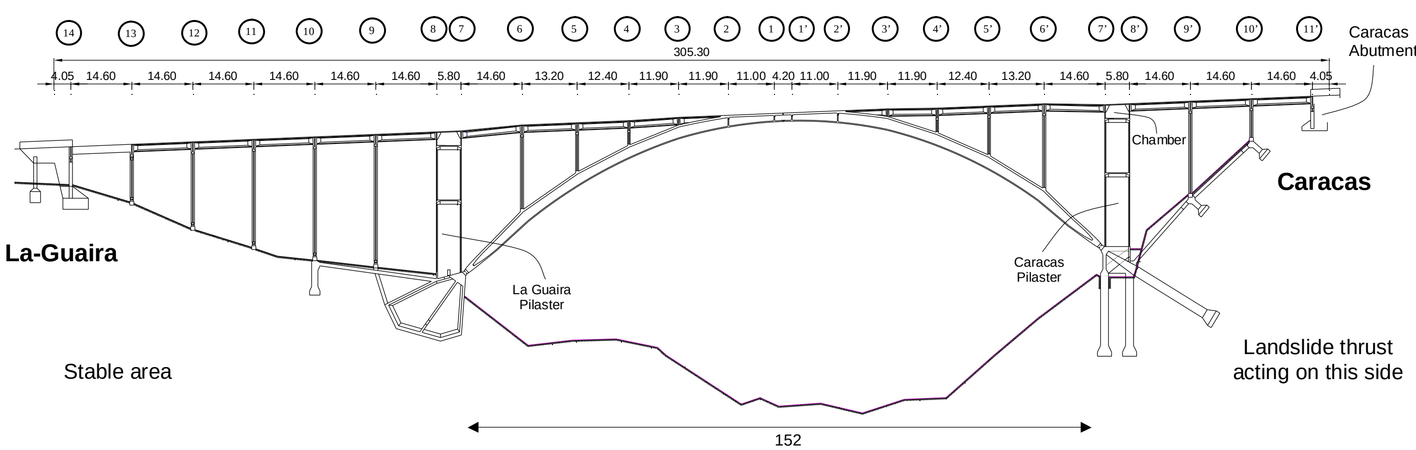

***Fig. 1** Viaduct No. 1*

VIADUCT DESCRIPTION

An excellent description of the structure can be found at Muller (1953). The three parallel double-hinge arch ribs consist of hollow-box type placed 8.40 m. apart measured on centers, as shown in figures 2, and 3. The deck was chosen as the primarily member to resist wind loads, and thus requiring the exclusion of all joints in the deck from Abutment to Abutment (Muller 1953). Each arch rib was 3.2 m. wide and varies in depth from 2.9 m. to 3.0 m. at the supporting points of the deck. Longitudinal arch rib reinforcement can be considered insignificant and consisted of 1.0 cm diameter bars spaced at 50 cm. Deck gravity loads were transfer to the arch ribs at concentrated points by a series piers consisting of three columns supported on the three arch ribs, as observed in Figure 4.

As described by Muller (1953), the longest piers shown in Figure 1 (5, 5', 6 and 6') had sufficient flexibility in the longitudinal direction to allow displacement of the deck without developing important bending moments. The shortest piers (4 and 4'), were hinged at their ends to provide sufficient flexibility in the longitudinal direction, as shown in Figure. 4. Two Pilasters were used to connect the two access Viaducts to the central arch span and play a major role in stabilizing the whole structure. The Pilasters had a rectangular hollow cross section of 6.0 m. x 24.0 m., were 41.8 m. tall, and had a wall thickness of 0.15 m. barely reinforced with plain bars. An upper chamber was built on the upper part of both pilasters. Inside this chamber, a concrete rocker hinged at it both ends received the precast girders.

The Pilaster and arch ribs on the La Guaira side were founded on a raft foundation as shown in Figure 1, which consisted of a 80 cm. deep postensioned concrete slab to transfer loads from the arch ribs to surrounding the soil. The postensioned slab was connected to the base of the arch ribs through a series of orthogonal and parallel postensioned concrete walls. Five walls were placed parallel to the longitudinal axes of the Viaduct and the three walls that radiate from the base of the arch ribs were placed perpendicular to the longitudinal axes of the Viaduct, as shown in Figure 1. Special foundations were required for the Caracas pilaster and the Caracas end of the arch ribs. Foundations for the Pilaster consisted of seven vertical concrete filled shafts, and three inclined concrete piles that received and distributed the arch thrust. The seven vertical shafts, 1.93 m in diameter, were dug to a depth of 18 m to penetrate from 3 to 4.5 m in rock. The bottom portion of these shafts was enlarged to 3.3 m in diameter. The three inclined piles were constructed to a depth of 29 m, in order to penetrate 3.0 m into sound rock, and carried most of the arch thrust. The Abutment located on the Caracas side, consisted of a concrete rocker that provided support for the precast girders.

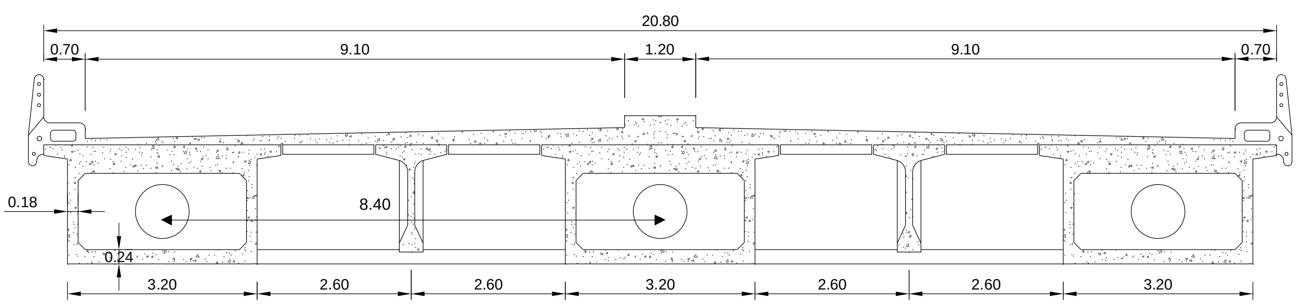

Fig. 2 *Cross section of the Viaduct at the crown*

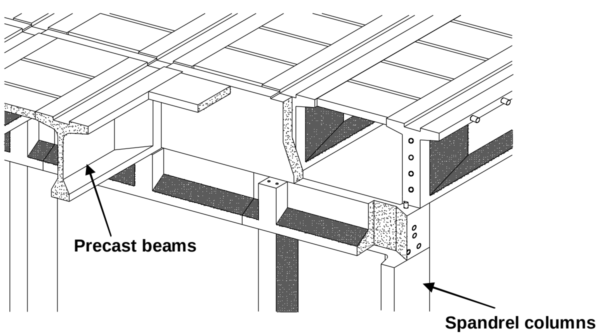

Fig. 3 *Cross section of the deck (Muller 1953)*

THE PROBLEM

An asphalt bump was noticed along the expansion joint located on the Caracas Abutment in March 1987. The bump was immediately eliminated and the pavement levelled, followed by a detailed inspection of the structure. Several cracks were observed on the grade beams located on the Caracas side. Shortly after, the Ministry of Transportation and Communications initiated a geotechnical study on the Caracas side of the Viaduct, which included photogeology, surface geology, borings, installation of piezometers, inclinometers and extensometers, ground surface, topographical monitoring and the excavation of exploratory adits.

Fig. 4 *Spandrel columns*

Photogeology

Morphological studies from aerial photographs missions conducted in 1936 (Figure 5) show a stepped topography, and different color tones of the hill adjacent to the Viaduct when compared to the surrounding area, which suggests the existence of an ancient landslide at the Caracas side of the Viaduct. The unstable area was roughly estimated in 500 m. by 225 m., as shown in Figure 6. In 1970, sixteen years after the Viaduct was built, the hill and unstable area were invaded by informal houses, referred to as “ranchos”, as shown in Figure 7, which lacked of sewage disposal and drainage facilities, and thus drain the sewage system into the ground.

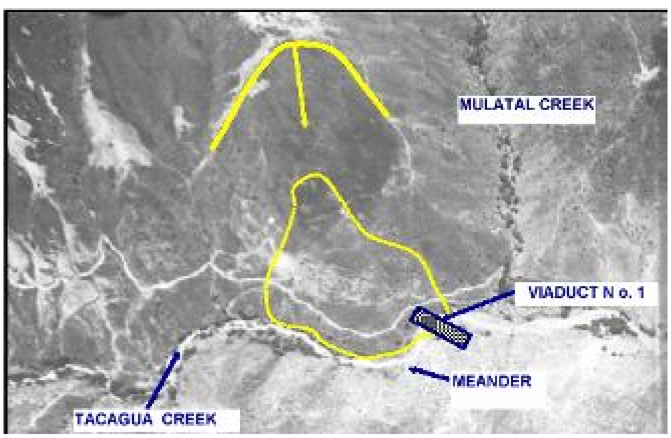

Fig. 5 *Aerial photograph of 1936. (Salcedo 2006)*

Surface Geology and exploratory borings

The lower hill under the highway is mainly covered by a colluvial material derived from ancient landslides. The material is characterized by angular boulders and fragments of metamorphic rocks of different lithology (schists, gneiss and marble). The rock mass below is a sequence of Jurassic-Cretaceous metamorphic limestones, interfoliated with calcareous mica schists, quartzites and some amphibolites. The main geological feature of the area is a fault running East-West parallel to the alignment of the first 4 kilometers of the highway close to Caracas. The fault belongs to the Tacagua-Avila fault system which has been considered geologically active and responsible of several recent small earthquakes in central Venezuela. The Tacagua fault has been classified as a dextral strike slip fault.

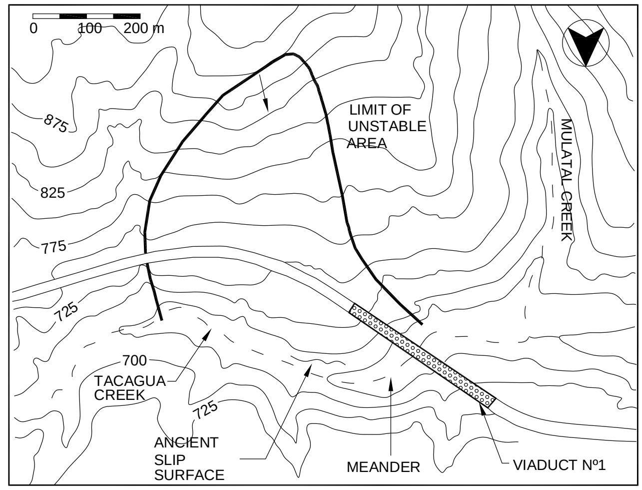

Fig. 6 *Limits of the unstable area*

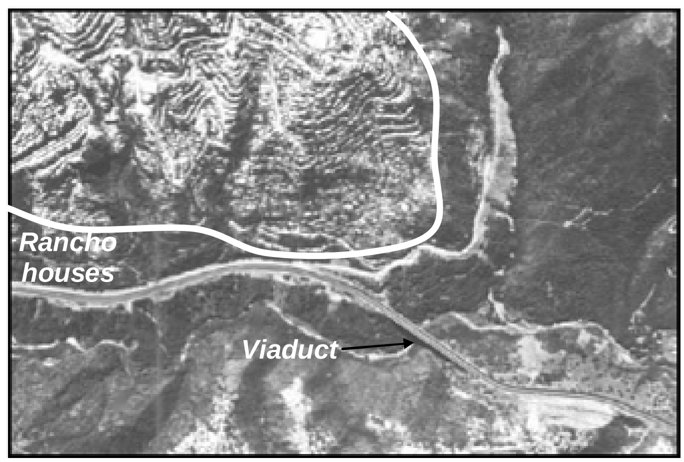

Fig. 7 *Aerial photograph of upper hill invaded by “ranchos” (Salcedo 2006)*

An important finding was an ancient landslide trace cropping out located at approximately 55 m below the Viaduct deck elevation which helped confirming the premise of an ancient landslide reactivation as was initially suspected based on the interpretation of the physiographic evidences observed in aerial photographs.

Exploratory borings and adits

Seventeen borings, with depth ranging from 40 to 50 m were drilled in 1987 to install several open pipe piezometers and 15 inclinometers. In addition, 410 meters of exploratory adits were excavated during 1992 and 1993 to investigate the causes that could have triggered the ancient landslide activation. Eleven extensometers were installed in the adits. The main field observation inside the exploratory adits was the existence of a 50 cm fault breccia running parallel to the landslide displacement vector. Results from inclinometer readings indicated that the average landslide displacement vector intersects the structure at a 40 degree angle, thus inducing transverse and longitudinal displacements on its foundations. Based on the results obtained from borings, exploratory adits and field instrumentation, it was possible to elaborate the geological profile shown in Figure 8.

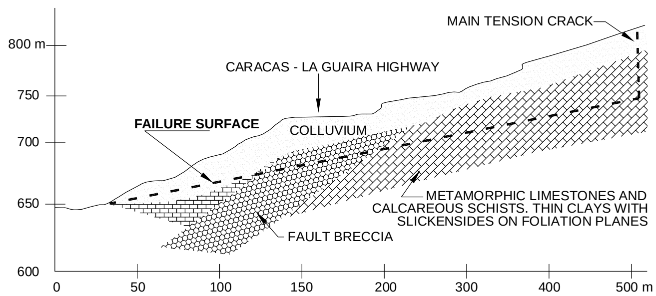

***Fig. 8** Interpretation of geological profile. (Salcedo, 2006)*

<u>Surface movement monitoring</u>

A surface displacement monitoring system conducted by Tranarg Consulting Company was based on topographical survey of 3 Bench Marks (BM) placed on the opposite side of the hill which was considered stable, in combination with 15 control reference points on the unstable area. The information gathered from surface monitoring indicated landslides velocities ranging from 1 to 3 cm/month in 1987. A decrease in the rate of displacement was detected at the beginning of 1998 with velocities ranging from 1.0 to 1.5 cm/year that remained almost constant up to 1993, when monitoring was suspended. In early 2005, monitoring of deck deformation made by Pilperca (Construction Company responsible of the Viaduct rehabilitation), indicated a significant increase of the landslide velocities reaching values of approximately 3 - 4 cm/month. Six of seven inclinometers installed in August 2005 were sheared off by the displacement. Technical inspections of the upper hill confirmed an accelerated landslide progress and the breakage of the main sewage pipe systems that served the “ranchos” houses.

An estimation of the landslide collapse time was made in November of 2005 based on Fukuzono (1985) and Voight (1989). The proposed methodology is based on the hypothesis that estimated date of collapse corresponds to the date when the inverse of the velocity versus time intersects the time axis. Such plot is shown in Figure 9. The trend indicated a possible landslide collapse on December 16th, 2005. However, constant velocities of approximately 36 cm/month were monitored between December 11th and January 5th. On January 5th, 2006, a significant landslide displacement with velocities up to 150 cm/month affected the structure, and it was decided to close Viaduct No. 1.

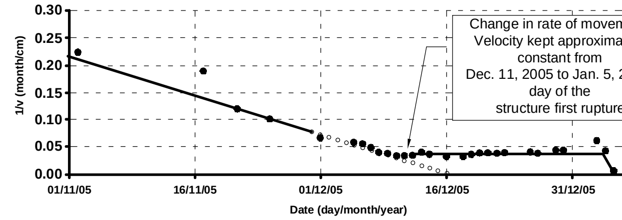

***Fig. 9** Estimated landslide collapse. (Salcedo, 2006)*

LANDSLIDE MECHANISM

Evidence has been presented to conclude that the hill displacement affecting the Viaduct No. 1 was caused by the reactivation of an ancient landslide. It is probable that the landslide activation mechanism could be associated to tectonic deformations, as evidenced by the field observations gathered inside the exploratory adits.

As mentioned by Salcedo (2006), an alternative that could explain the landslide reactivation would be the effect of earthquakes. Speculating one could think that the landslide was reactivated during the Caracas July 29, 1967 earthquake (Mw = 6.5 Richter scale). A mental exercise based on the measured landslide displacement based on the position of the Caracas Pilaster in 1987, leads to the conclusion that the hill must had moved at a rate of approximately 1.0 and 1.5 cm/year in the longitudinal direction of the Viaduct, velocity that was also measured in the following years, as shown in Figure 10. As will be described later, a 1:1 ratio was monitored between the average vertical deformation of the arch ribs, measured at the crown and the landslide displacement in the Viaduct’s longitudinal direction.

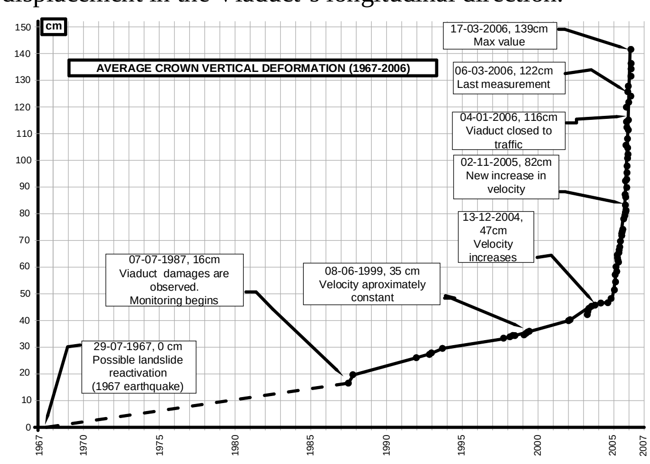

***Fig. 10** Landslide displacement vs. time (1967-2006) (Salcedo, 2006)*

EMERGENCY WORKS (1987-1988)

Passive and active anchors were placed on the Caracas side of the Viaduct with the aim of deviating the landslide thrust away from the Viaduct. A total of 245 passive anchors, consisting of 1 3/8” diameter grouted bars, inclined at 55° with lengths varying from 30 to 36 m were installed on an area of approximately 25 m x 70 m located between the Caracas Abutment and Pier 9’ in order to ensure that anchors would pass through the identified failure surface. It is important to emphasize that this measure was aimed at deviating the landslide thrust from the bridge because the applied support force by the anchors was only a small fraction of that required to stabilize such a large landslide mass. More information related to the geotechnical aspects can be found in Salcedo, D. (2006), Salcedo, D. (1989), Salcedo, D. (1994), Salcedo, D. and Ortas, J. (1994), Salcedo, D. and Ortas, J. (1991).

EFFECTS OF THE LANSLIDE THRUST ON THE STRUCTURE

Results from computer simulation and engineering judgement indicated that the most detrimental effects of the landslide thrust on the structure were its transverse and longitudinal components. The transverse thrust component pushed transversely the structural components founded on the Caracas side. Note that the piers and spandrel columns that connect the foundations to the deck were significantly stiff on the transverse direction, as can be observed in Figure 3. It is important to note that the deck itself is also a very stiff component. It can be visualized as a cantilever beam, with a fixed end at the La-Guaira side pushed on its tip sideways by the stiff piers founded on the Caracas side, and thus subjected to bending on its own plane. Thus, in the transverse direction the structure had a significant stiffness which makes it develop large forces when subjected to small transverse displacements. On the other hand, the longitudinal thrust component induces tensile stresses on the top fibers of the arch ribs. However, the structure is considerably more flexible longitudinally than transversely, because of the flexibility of the large and slender spandrel columns in the longitudinal direction, and the addition of hinges at both ends of the short spandrel columns, as mentioned before. Because the arch ribs were designed to resist compression stresses and practically no longitudinal reinforcement was provided, there was a major concern of cracking of the arch ribs which would lead to collapse of the structure, at approximately 60 of longitudinal landslide thrust, as described in the report elaborated by Jean Muller Interntational (1987).

It is considered of main importance to visualize the kinematics of the Viaduct when subjected to the longitudinal landslide thrust. Results from field measurements indicated that a unit displacement measured at the base of the arch ribs in the longitudinal direction, induces a unit vertical and half horizontal displacement at crown of the arch ribs. Because at the crown the arch ribs and deck become a solid member, the deck also advances half a centimetre towards La-Guaira. This information was very useful in validating the different computer models used to understand the structural behavior during the rehabilitation of the Viaduct.

In 1987 the expansion joints between the deck and both abutments were found to be blocked restraining longitudinal deck displacements as consequence of the landslide reactivation. It was considered of major importance to unblock the expansion joints so the deck and arch ribs can better accommodate the displacement regime imposed by the soil thrust. The deflected shape of the deck as obtained from surface monitoring while both expansion joints were blocked in 1987, is shown in Figure 11. The first rehabilitation measure conducted on the Viaduct was on December of 1987 and consisted of a concrete demolition of the deck to create a gap at the expansion joint so the structure can freely move longitudinally.

Designers of the Viaduct expected compression stresses in the rage of 6 kg/cm^2 - 85 kg/cm^2 and 7 kg/cm^2 - 71 kg/cm^2 for the bottom and top fibers of the arch ribs, respectively, and thus compatible to construct an arch rib without reinforcement, as it is described in a report elaborated by Jean Muller International in 1987 and presented to the Venezuelan Ministry of Transportation and Communications. In order to evaluate the arch rib stresses when subjected to the longitudinal landslide thrust, a computer model was created using the software Microsafe and SAP 2000. The model included the effects of permanent deformations arising from creep and shrinkage. Based on results obtained from the computer simulations it was estimated that cracking of the arch ribs would occur at a longitudinal landslide thrust of approximately 60 cm.

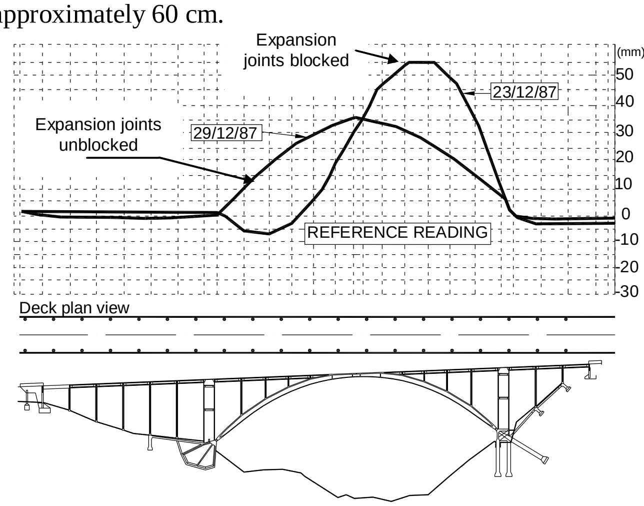

Fig. 11 *Deformation profile of the deck in 1987*

REHABILITATION MEASURES

In 1993 a small gap was observed at the base of piers 9’ and 10’ specifically at the connection between the precast hinges and their foundation blocks, as shown in Figure 12. The gap appeared at the east side of each column indicating the piers were “pulling” the deck towards the east and the deck was opposing trying to pull it back to the west. Because no reinforcement crossed the connection between the precast hinge and its foundation block (Figure 12), as was indicated on the Viaduct’s blue prints, the shear induced by the landslide transverse component was transferred to the piers through friction between both elements and a concern existed of a potential shear failure at the connection interface. In attention to this problem, an external cable mechanism was designed as a redundant lateral force resisting system to prevent shear failures at the base of piers 9' and 10'. The mechanism is shown in Figure 13 and consisted of a pair of overlapping set of tendons placed in a U shape wrapping around the external edges of each Pier (note that redundant piers were also built adjacent to the existing piers (Figure 15a), as will be described later). Both cables ends were anchored into two heavily reinforced concrete squat walls located on either side of the Pier. A total of 22 and 21 A-270 low-relaxation strands with a 0.6" diameter wrapped the east and west external edges of the external spandrel columns, of piers 9' and 10', respectively. Strands were stressed to approximately 30% of their ultimate strength. In addition, a very stiff horizontal strut element was placed between the external spandrel columns to transfer the axial force introduced by the tendons, as shown in Figure 13. The strut consisted of a 4 cm thick, 60 cm x 60 cm steel hollow section filled with a concrete mixture and additives to minimized long term deformations such as shrinkage and creep. The mechanism described could serve for two applications (Camargo, 2006): **a)** As a passive system to prevent a transverse shear failure at the base of piers 9' and 10' by providing a redundant load-path capable of resisting the shear force produced by bending of the deck, in case the friction strength between the base of the pracast hinge and the foundation block would be insufficient. **b)** As an active system as it was activated on December of 2005, when lateral deformations of the deck measured at the Caracas Abutment passed 100 cm. By controlling the tension on the cables, it was possible to recover approximately 23 cm of transverse displacement (Figure 18) allowing the deck to release a part of the potential energy stored in it, as will be described later.

A redundant gravity load resisting system was constructed for the entire access Viaduct located on the Caracas side. New gravity load resisting elements were placed at the Caracas Abutment, piers 9' and 10' and inside the chamber located at the upper part of the Caracas Pilaster. A redundant steel truss was placed on the Caracas Abutment, as shown in Figure 14, that could eventually replace a concrete rocker that receive the beams at this location. The steel truss was supported on Hillman rollers placed transversely to the longitudinal axes of the Viaduct. A set of redundant reinforced concrete piers were constructed at either side of piers 9' and 10' to support paired trusses which received the precast girders, as shown in figures 15a and 15b. The trusses were also supported on rollers free to displace in the transverse direction. A redundant gravity load resisting system was placed inside the upper chamber located inside the Caracas Pilaster. It consisted of metallic struts, which received the precast girders. Similarly as it was done at the Abutment and piers 9' and 10', the steel struts were supported on Hillman rollers placed transversely to the Viaduct's longitudinal axes, as shown in Figure 16.

In December of 2006 it was decided to activate the redundant gravity load resisting system and to release the tension on the cable system installed in 1993. For this purpose, the base of piers 9' and 10', the concrete rockers that received the girders inside the Pilaster shown in Figure 16, as well as a concrete rocker located at the Caracas Abutment, were

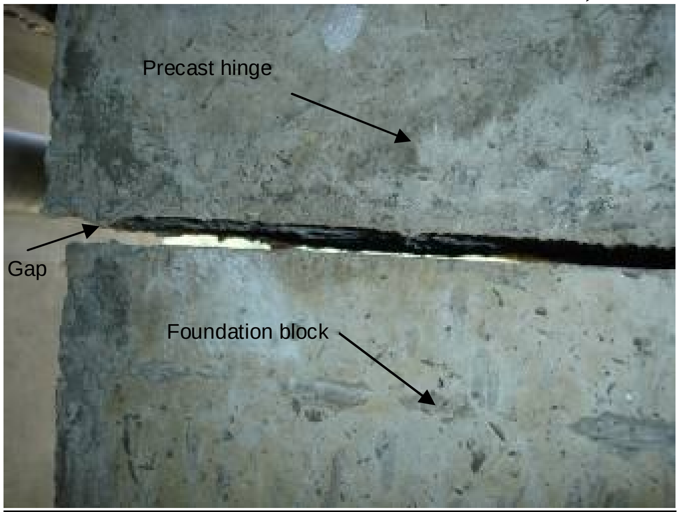

Fig. 12 *Gap at the connection between precast hinge and its foundation block*

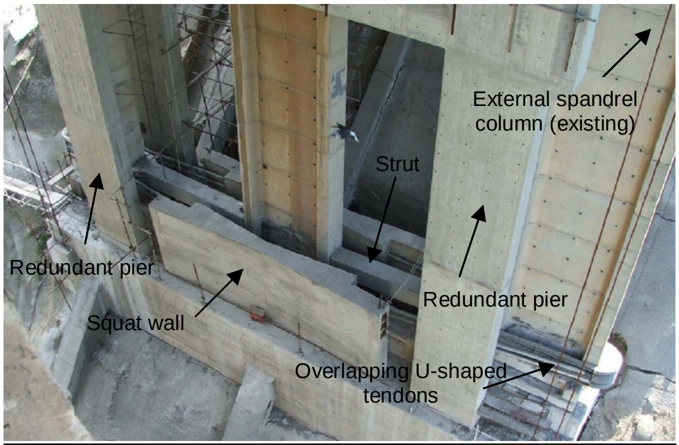

Fig. 13 *External cable mechanism at Pier 9'*

Fig. 14 *Redundant truss at the Caracas Abutment*

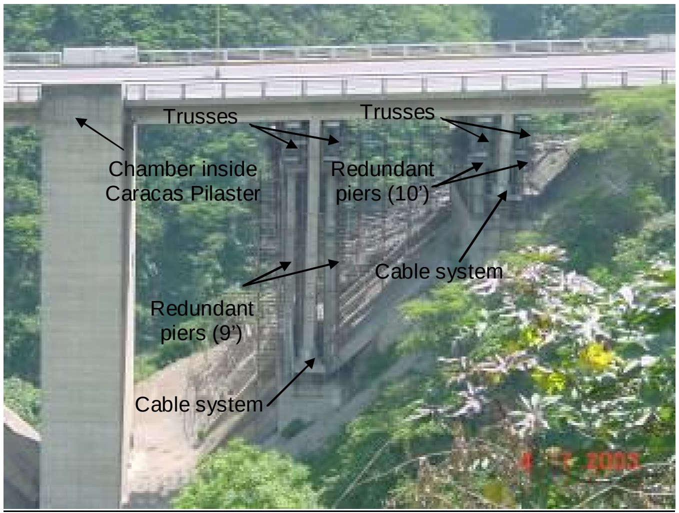

Fig. 15a *Redundant piers 9' and 10'*

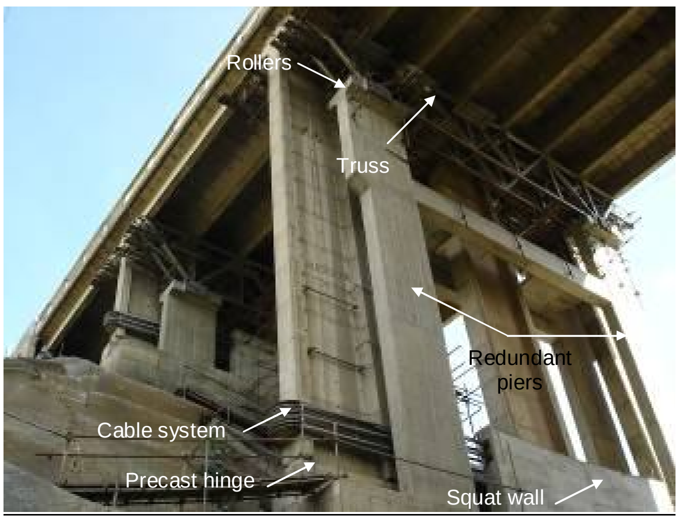

Fig. 15b *Redundant piers 9' and 10'*

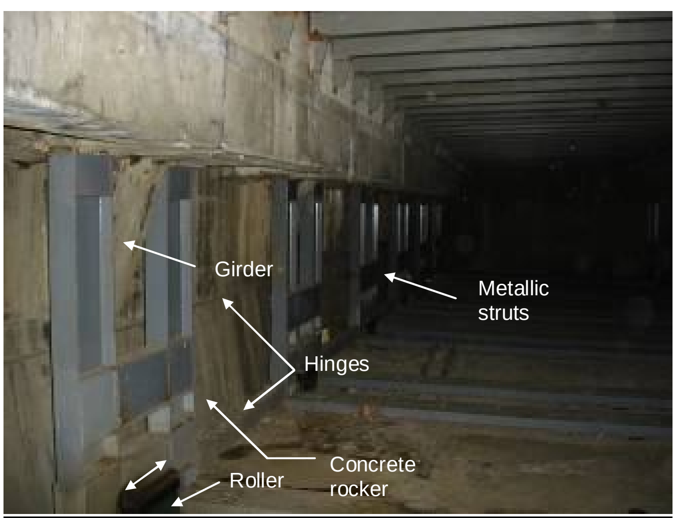

Fig. 16 *Struts inside the chamber located at the upper part of the Caracas pilaster*

intentionally cut at their bottom bases to allow the structure rest on the redundant gravity load resisting system described before. Subsequently, the tension on the cable system at piers 9' and 10' was slowly release. Figure 17 shows sketches of the concrete surfaces that were cut using a diamond saw, at the base of the piers (Figure 17a) and at the base of the concrete rockers located inside the upper chamber at the Caracas Pilaster (Figure 17b). As a result, it was possible to recover approximately 23 cm of the Viaduct's transverse deformation at Pier 9', as shown in Figure 18.

In October of 2003, at an average longitudinal deformation of approximately 45 cm measured at the base of the arch ribs located on the Caracas side, maximum probable tensile stresses in the order of 25 kg/cm^2 were obtained for the arch ribs section adjacent to the crown. It is worth mentioning that the average tensile strength as obtained from a split cylinder test of the arch ribs was of in the order of 32 kg/cm^2. In order to prevent the possibility of a sudden collapse of the arch ribs, it was decided to postension them to reduce the existing stresses to those suited for an unreinforced arch rib. For this purpose, two set of tendons were placed near the top fibers of the each arch rib, as shown in Figure 19. The first set of tendons was placed between the Pier 4 and the precast hinge located on the Caracas side and consisted of a total of thirty tendons, 0.6 in. diameter, made of Grade A-270 steel, placed inside two ducts near the top fibers of the arch ribs, as shown in Figure 21. Similarly, the second set of tendons consisted of twenty two strands, 0.6 in. in diameter, placed in between Pier 4' and the precast hinge located on La-Guaira side. The active ends were located on piers 4 and 4' and the passive ends at the precast hinges. Tendons were stressed to 70% of their ultimate capacity. That is 588 tons and 410 tons for cables anchored at piers 4 and 4', respectively. The trajectory followed by the tendons is described by a straight line between piers and an angular change at the intersecting points between arch ribs and spandrel columns. To effectively transfer the forces from the cables to the structure where the angular change occurs, the thickness of the lateral walls for the arch ribs was enhanced to 25 cm with the use of shear connectors. In between piers the trajectory remains straight, and a small beam encased the ducts. Ducts were grouted after the tendons were stressed. Results from a computer analysis indicated a reduction in the probable stress in the top arch fibers after post-tension of the arch ribs from 25 kg/cm^2 to 10 kg/cm^2.

Advantage was taken from the available ductility of the prestressing steel and a plastic hinge was designed and constructed at the crown of either arch rib. It was expected that the formation of a plastic hinge at the crown of the arch ribs will change a statically undermined structure to the first degree (an arch with two hinges), to a statically determine one, (three hinged arch) practically immune to the longitudinal thrust (Camargo 2006). The plastic hinge was designed to self activate by the moment demand increase produced by the continued longitudinal thrust.

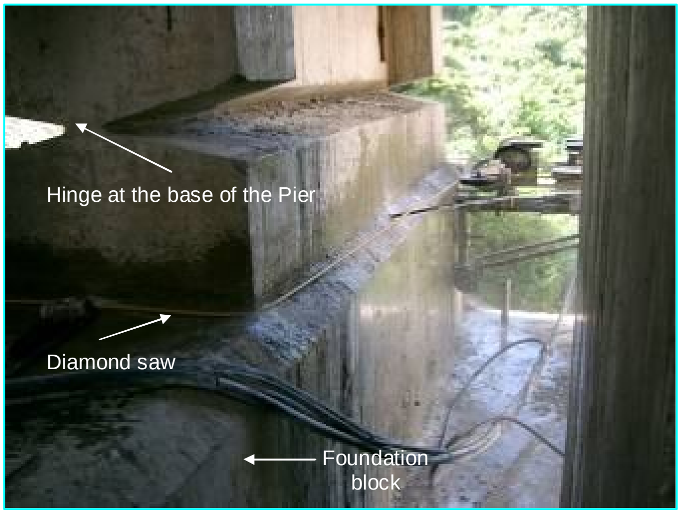

Fig. 17 a *Diamond saw performing a cut at the base of Pier 9'*

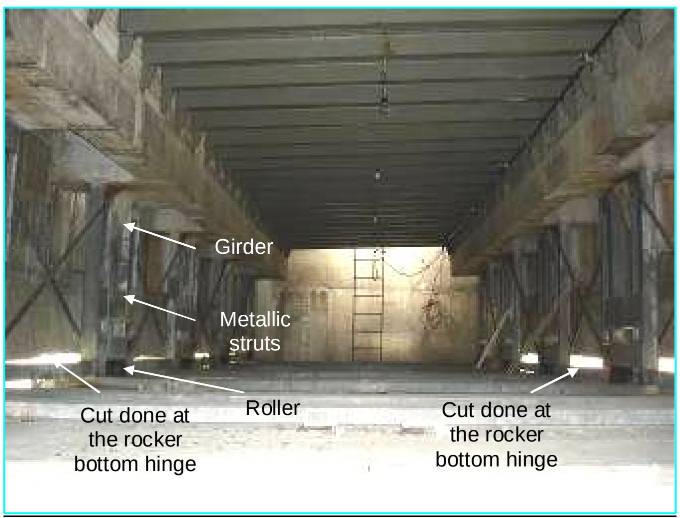

Fig. 17 b *Cuts done at the bottom hinge of the concrete rockers located inside the chamber at the Caracas Pilaster*

Fig. 18 *Transverse displacement recovered at pier 9'*

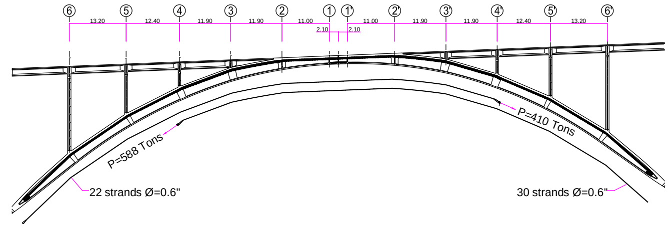

Fig. 19 *Postension tendons to strengthen the arch ribs*

The crown or its vicinities was believed the ideal location for the plastic hinge formation. Thus, this spot was designed to posses the minimum capacity-demand ratio (C/D). Reinforcing bars surrounded the ducts to increase the flexural capacity of the ribs, as shown in Figure 20. The plastic hinge at the crown was expected to form at a longitudinal deformation of approximately 80 cm. Arch rib reinforcement adjacent to the plastic hinge region is shown in figures 21 and 22.

This proposal established a feasible solution to extend the life of the structure for a lapse of time long enough to construct a new Viaduct. At a longitudinal deformation of approximately 80 cm, several cracks were observed outside and inside the arch ribs, given and indication of the efficiency provided by rehabilitation measures conducted on the structure. It is important to mention that by date in which the project was completed, the deformation monitored on the ribs was of approximately 45 cm and the velocity the landslide thrust was in the order of 2.5 cm/year. Despite the project was completed and presented to the authorities in October of 2003, it was late implemented in March 2005, at a longitudinal deformation of approximately 55 cm.

As reported by Salcedo (2006), the intense rainfall beginning January 2, 2006 and the existence of wide open tension cracks on the ground, allowed a significant amount of water infiltration into the soil and major vertical and horizontal displacements were observed few meters away from the Caracas Abutment. On January 5 of 2006, at a longitudinal thrust of approximately 120 cm, and after three days of an intense rain accumulating 107 mm, a sudden acceleration of the landslide occurred. A first evidence of this acceleration was noticed at 2:30 am after completing a new concrete demolition to open a gap of approximately 25 cm at the expansion joint located on the Caracas Abutment. Next day, at 7:00 am., when works where initiated on the Viaduct, the 25 cm gap was completely closed and so the gap between the deck and La-Guaira abutment. Thus the deck and crown were restrained to move longitudinally, similarly as occurred in 1987 when the problem was first detected. As a result, the arch ribs became significantly stiff, and the structure no longer behaved as a statically determinate one. The three arch ribs fractured at a section located approximately 35 m away from the crown, as shown in figures 23 and 24. The fractures were characterized by a significant amount of crushed concrete. The arch ribs experienced significant change in their geometry and so did the trajectory of the pressure line and thus the internal distribution of stresses within the arch ribs. Immediately all traffic and circulation over the Viaduct was strictly prohibited.

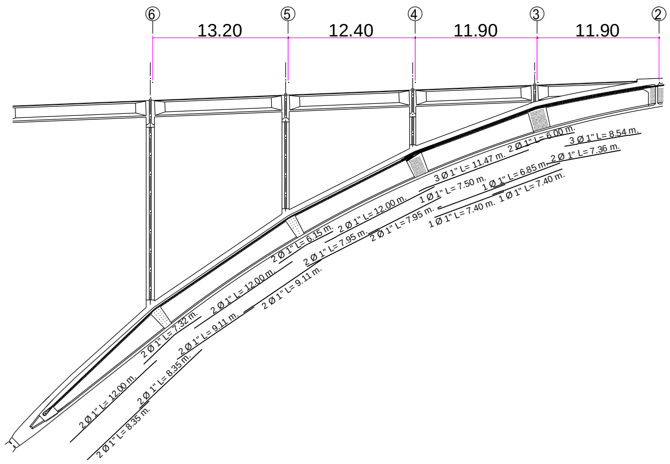

Fig. 20 *Reinforcing bars to strengthen the arch ribs*

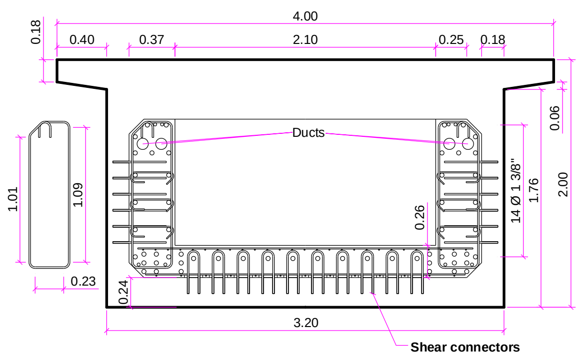

Fig.21 *Cross section of the ribs adjacent to the plastic hinge*

The thrust continued at a rate of approximately 0.8 cm/day, and the Viaduct remained closed. In March 19th 2006 the Viaduct collapsed, at a vertical deformation of the arch ribs of approximately 1.90 m. Immediately after the collapse, three seismic stations located at 3, 14 y 19 kilometres from the Viaduct allowed to estimate the impact of the fallen structure as an equivalent earthquake of magnitude Mw = 1.6, as reported by Salcedo (2006). No deaths nor injures were associated with the structural collapse.

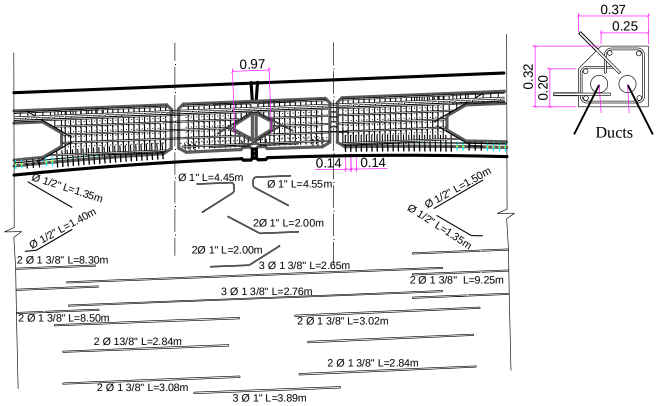

Fig. 22 *Longitudinal reinforcement at the plastic hinge region*

Fig. 23 *Location of the fracture at the arch ribs*

Fig. 24 *Fracture of the arch ribs*

Several lessons could be learned from the described case history.

1. The well known importance of conducting extensive geological studies before building civil infrastructures.
2. Based on the information presented in this paper, it is concluded that the reactivation of the landslide could have been triggered by the 1967 Caracas earthquake.
3. Based on the experience gathered from this case history, it is recommended that whenever dealing with landslide hazards one can not completely trust monitoring to make long term estimations of the landslide velocity. Landslides may suddenly accelerate due to factors such as water pressure induced by rainfall infiltration or earthquakes.
4. The main rehabilitation measures conducted on the structure consisted of a cable system to prevent transverse shear failures at the base of piers 9' and 10', the postensioning of the arch ribs and the design of a plastic hinge at the crown of the structure. In terms vertical deformation, the structure was capable of accommodating deformations of approximately 190 cm, compared to 60 cm as expected if no rehabilitation measures would had been conducted.
5. Failure forecast theories used with caution are valuable tools to facilitate the decision making processes in complex scenarios.

ACKNOWLEDGEMENTS

Many professionals collaborated to extend the life of the Viaduct in different areas of engineering. Andres Pesti, Celso Fortoul, Claudio Ramirez, Claudio Ramirez Jr., Eli Abadí, Isaac Salcedo, José Capobianco, Josefina Ortas, Mathías Brewer, Nelson Rodriguez and Salvador Pulido, deserve lots of credits for their valuable support. The workers from Pilperca Construction Company made possible to carry out the rehabilitation measures conducted on the structure. Their dedication and professionalism is greatly acknowledged. The first author of this paper would like to acknowledge the help of Tadeo Vera Mora in preparing this manuscript.

REFERENCES

Camargo, R. [2006]
"Reforzamiento del Viaducto 1 de la autopista Caracas-La Guaira". Ingeniería forense y estudios de sitio. Guía para la prevención y gestión de riesgos. José Grases Editor, p.p. 257-272.

Fukuzono, T. [1985]
"A new method for predicting the failure time of a slope." Proceedings IV International Conference and Field Workshop on Landslides, Tokyo, pp. 145-150.

Jean Muller International [1987]
Technical report requested by the Venezuelan Ministry of Transportation and Communications about the Viaduct No. 1.

Muller, J. [1953]
"Largest concrete spans of the Americas. How the three bridges were designed." Civil Engineering, March, pp. 45-48.

Salcedo, D. [1994]
"Observaciones geológicas en galerías exploratorias excavadas en la zona del Viaducto No. 1. Autopista Caracas-La Guaira." The 1994 International Symposium. Integral approach to applied Rock Mechanics. IV Congreso Suramericano de Mecánica de Rocas. Santiago de Chile, Vol. II, pp. 701-713.

Salcedo, D. [1989]
"*¿Es predecible el comportamiento del macrodeslizamiento que afecta al Viaducto No. 1 de la Autopista Caracas-La Guaira?*." Boletín Sociedad Venezolana de Mecánica del Suelo e Ingeniería de Fundaciones, N° 58, Caracas, pp. 3- 33.

Salcedo, D. [2006]
"El deslizamiento de la ladera sur de la autopista Caracas-La Guaira. Ingeniería forense y estudios de sitio". Guía para la prevención y gestión de riesgos. Edited by José Grases p.p. 227-256.

Salcedo, D. and Ortas, J. [1991] *"Investigation of the slide at the Southern Abutment hill of Viaduct No. 1. Caracas-La Guaira Highway. Venezuela."* Proceedings of the 6th. International Symposium on Landslides. Christchurch, Vol.1,. pp. 189-198.

Salcedo, D. and Ortas, J. [1994]
"Comportamiento del deslizamiento de la ladera Sur del Viaducto No. 1. Autopista Caracas-La Guaira. Venezuela." Memorias I Symposium Panamericano de Deslizamientos, Ecuador,Guayaquil, pp. 60-90.

Voight, B. [1989]
"Materials science law applies to time forecasts of slope failure." Landslide News, No. 3. pp. 8-11.