BEHAVIOR OF A LANDSLIDE PRIOR TO INDUCING A VIADUCT FAILURE, CARACAS-LA GUAIRA HIGHWAY, VENEZUELA.

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ABSTRACT

A reactivation of a large ancient landslide identified in 1987 began to progressively damage the southern side of an important prestressed concrete span structure of a highway connecting Caracas, Capital of Venezuela, with its main port and airport. This paper deals with the behavior of the landslide over a period of 19 years, and the final failure of the structure in March 19, 2006. Results of geotechnical investigation, evaluation of inclinometers readings and surface control points displacements, as well as behavior of the structure subjected to the slide thrust, are discussed. Several causes that may explain the reactivation and behavior of the landslide, and some aspects about failure time prediction are also described.

INTRODUCTION

The Caracas-La Guaira highway connects Caracas, capital of Venezuela, with its main seaport and the Simon Bolivar International Airport. This highway was built during the years 1950-1953 with a 60 million dollar cost along a geologically complex zone. By that time it was considered as the costliest highway in the world. 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. Various major structures were built as part of the highway: 3 bridges called Viaduct No. 1, 2, and 3, and two tunnels. The area is characterized by a rainy season from May to October with an average rainfall of 1000 mm/year, with maximums in August and October of 110 mm and 125 mm.

This paper deals with the behavior of a landslide which was loading the Caracas side (southern hill) of Viaduct No. 1. The landslide was identified in 1987, and progressively caused serious damages to the structure inducing its final collapse. After some rehabilitation and structural works which extended the service life of the Viaduct, it finally was declared out of service on January 5, 2006, and its final collapsed occurred on March 19, 2006. Previous information about the landslide have been published by Salcedo (1989), Salcedo & Ortas (1991, 1994), and Salcedo (1994).

STRUCTURAL ASPECTS OF VIADUCT No. 1

Viaduct No. 1 was designed and constructed by Campenon Bernard Enterprise using Dr. Eugene Freyssinet methods and under his supervision. The structure was successfully completed in January 1953 and by the time it was built, the bridge was considered to have the largest prestressed concrete spans in the world and the largest concrete arches in the Americas. Structural details of the Viaduct have been published by Freyssinet, Muller and Shama (1953). The total length of the Viaduct is 308 m and the main part of the structure consists of three parallel arch ribs of a hollow box type with a hinge to hinge span of 152 m and a height of 70 m. Pilasters have been placed at each side of the arches and despite their thin hollow shell type (41.8 m high, 6.1 m x 6.1 m in plan) they have high bending and torsion strengths, which provide resistance against wind loads and play a major role in stabilizing the whole structure. The precast deck with eight longitudinal pre-stressed beams, was designed as the major member for carrying wind loads and it was necessary to exclude all joints throughout its entire length from abutment to abutment. The pilaster and the arch on the La Guaira side were founded on o raft foundation in the shape of a hollow prestressed concrete box. For the pilaster at the Caracas end of the arch, the nature of the ground required special foundation consisting of a system of seven vertical concrete filled shafts and three inclined concrete piles on top of which a light structure was built to receive and distribute the arch thrust and the weight of the pilaster. The seven vertical shafts, 1.93 m in diameter, were dug by hand to a depth of 18 m to penetrate 3 to 4.5 m in rock. At the bottom these shafts were enlarged 3.3 m diameter in the shape of an elephant's foot. The three -30º inclined piles, one behind each arch rib, were constructed by excavating galleries (2.3 m x 2.3 m) to a depth of 29 m, in order to penetrate 3 m into sound rock.

On the Caracas side the end abutment has a hollow-box shape with four buttresses founded to depths varying between 7 and 10 m, and a rocker on top of which a cross beam transfer the loads coming from the longitudinal prestressed beams and the deck. The other two piers, between the abutment and the pilaster of the arch, are founded directly on special foundations at depth of 4.8 m. Figure 1 shows a longitudinal section of Viaduct No. 1.

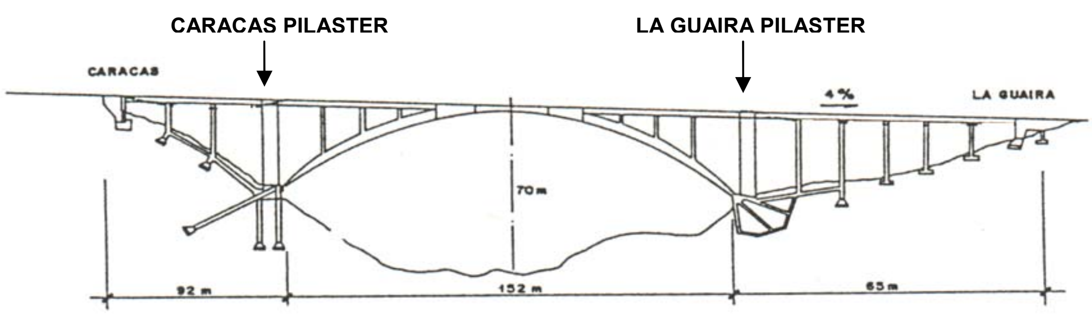


Figure 1. Longitudinal section of Viaduct No. 1. Note different type of foundations for the Caracas Pilaster and La Guaira Pilaster.

EVENTS OF 1987 AND LANDSLIDE IDENTIFICATION

At the end of March 1987 an asphalt bump formed along the southern abutment (Caracas side) expansion joint of the Viaduct No. 1. The bump was immediately cut and leveled and a detailed inspection of the structure was made. During this technical inspection, rotation of the Caracas abutment and other damages in the structure were observed. After evaluating possible causes of the problem, the Ministry of Transportation and Communications decided in 1987, to contract a geotechnical investigation oriented to investigate the stability of the southern hill. Geotechnical investigation included photogeology, surface geology, borings and installation of piezometers, inclinometers and extensometers A topographical control of the ground surface and the structure as well as the excavation of exploratory adits were later included as part of the investigation.

Photogeology

Several aerial photographs missions from 1936 to 1983 were studied in detail. From this evaluation and field inspections, several conclusions can be pointed out:

- The area where the southern side of the Viaduct was located has a morphology with clear evidences of an ancient landslide. Scarps, stepped topography, different color tone of the hill in comparison with the surrounded area, and a curious change of slope were stereoscopically observed in the 1936 mission. (Figure 2a). The ancient landslide has a length of approximately 500 m and a height of about 225 m. Smaller slides within the whole unstable area can easily be identified. Figure 2b shows approximate limits of the ancient landslide.
- An outstanding morphological feature of the area is an unjustifiable meander in the Tacagua creek at the bottom of the slide, which has been considered as a physiographic evidence of the hill movement, changing the original course of the Tacagua creek. It is probable that the ancient slide have almost completely filled the valley with slide debris and forced the Tacagua stream against the north bank.
- In the years of 1970 to 1971 the hill was invaded by poor people building small houses called locally “ranchos” without any kind of sewage disposal and drainage facilities.
- In 1974 the upper hill area was affected by a landslide and many “ranchos” were destroyed. By that time a heave was observed at the central island of the highway in a section located 50 m to the South from the Caracas abutment. An important earthwork was made on the upper hill.
- In 1981 an important landslide 130 m wide and 300 m from base to top, occurred in the upper hill, located 250 m south of the Caracas abutment of the Viaduct.

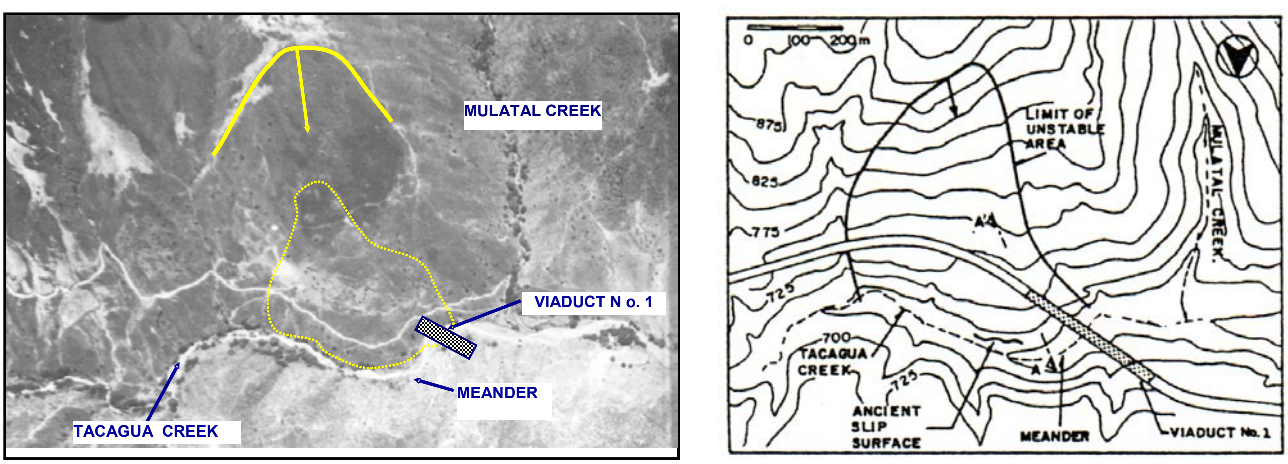


Figure 2a and 2b. Aerial photograph (year 1936), showing physiographic evidences of an ancient landslide where the Caracas side of Viaduct No. 1 was built. A meander apparently induced by the slide mass can be observed. Figure 2b: Plan of the slide and location of the bridge.

Surface geology and exploratory borings

Most of the hill, mainly the lower hill under the highway is covered by a colluvial material derived from ancient landslides. This material is characterized by angular boulders and fragments of metamorphic rocks of different lithology (schists, gneiss and marble) in a silty sand matrix. 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 which runs parallel to the alignment of the first 4 kilometers of the highway close to Caracas. This fault belongs to the Tacagua–Avila Fault system which has been considered geologically active and responsible for some earthquakes in central Venezuela. Tacagua fault has been classified as a dextral strike slip fault. Foliations planes are very well developed having a strike parallel to the strike of the hill and dipping predominantly 15º-40º towards the slope. Two main joint planes are easily determined in outcrops and rock cores recovered from boreholes. One set of joints has a strike parallel to the foliation planes and dips either vertical or 60º opposite to the foliation, and the other set has a strike perpendicular to the foliation surfaces, dipping vertically.

During the field geological site exploration, several important factors were observed:

- Movement of rock blocks along foliation planes, even at low dips (10º-20º).
- Tectonic slickensides on foliation and joint planes, local fault breccias and microfaults at hand sample scales, all of them evidences of the intense tectonic history of the area.
- An ancient landslide trace cropping out at elevation 670, approximately at 55 m under the Viaduct deck elevation. Material in the failure surface is formed by a typical clayey soil with brecciated fabric, slickensides and polished surfaces, and fragments of metamorphic rocks. This finding was considered as an important proof of the hypothesis of an ancient landslide interpreted from physiographic evidences observed in aerial photographs.

Emergency works (1987-1988)

Once the diagnostic was completed, emergency remedial measures consisted in the installation of 245 passive anchors 1 3/8” in diameter grouted bars, inclined 55º under horizontal, and lengths between 30-36 m in order to assure that anchors would pass through the identified failure surface. This work was done on an area of 25 m x 70 m covering a zone between the Caracas abutment and the second southern pier below the bridge and part of the hill. During grouting of the anchors it was observed that in many of them abnormal amounts of cement were necessary; 200-300 bags (42.5 kg) for most of them and some in the range of 1000 to 2000 bags. Additionally, 20 prestressed cable anchors 60 m long, were drilled, grouted and stressed between 45 and 60 tons. These active anchors were installed at the base of the Caracas end abutment and at the base of the two intermediate piers (Pier 9 and Pier 10) between the abutment and the Caracas pilaster. This remedial measure was only aimed to divert the landslide thrust from the bridge since it was known that the applied support force was only a fraction of the force needed to stabilize such a large landslide.

Figure 3 shows results of deck deformation measurements from June 87 to December 87 revealing that the deck was deforming asymmetrically at a rate of 5.5 mm/month. Calculations made by Jean Muller International with a computer model of Viaduct No. 1, revealed that the bending moment estimated at the quarter part of the arch on the Caracas side was negative and especially alarming.

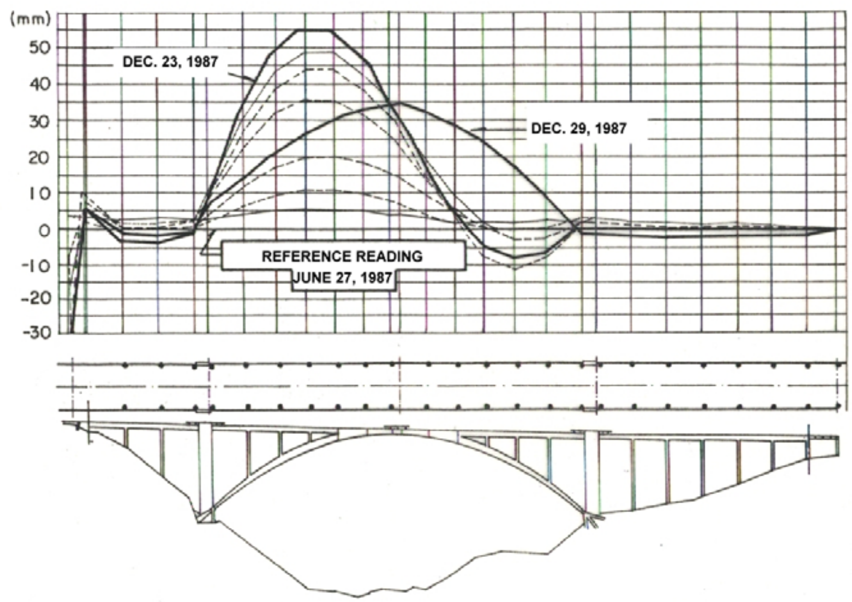


Figure 3. Deck deformation before and after joint cuttings in Caracas (left) and La Guaira (right) abutments.

On the La Guaira side, calculations gave positive bending moments which tended to induce tension at the bottom fiber and would be increased under live loads. This situation placed the arch in a “limit state” for serviceability requirements. Based on these results another emergency work that had to be accomplished was opening of the expansion joints of the structure between the abutment and the deck in both sides of the bridge. Before concrete cutting of the joints, 8 jacks providing each one 125 tons were used, applying a total force of 1000 tons. This operation decreased the bending moment in the critical section from -4500 t-m down to -1600 t-m. Opening of the joint was made in the Caracas side first, beginning in November 1987. The whole job was completed on December 24, 1987, when the jacks on the La Guaira side were released. Just in that moment everybody on the bridge could feel the movement of the deck changing its asymmetrical deformation to a symmetrical shape with its maximum vertical displacement at the crown of the arch. Figure 3 also shows results of spirit leveling measurements after the joint operations. Since 1987, opening of Caracas and La Guaira joints had to be accomplished several times because the moving mass closed the opening which generally had a width of 25 cm.

Exploratory borings and adits

Seventeen borings between 40 and 50 m deep were drilled in the area in 1987, installing open pipe piezometers and 15 inclinometers. In order to investigate factors that could trigger the ancient landslide, during the years 1992 and 1993, 410 m of exploratory adits, including main and secondary adits, were excavated, installing 11 extensometers. A very detailed geological description of the exploratory adits was made and results have been published by Salcedo, 1994. Two important results can be summarized as follows:

- The existence of a fault breccia with a length of 50 m measured in the direction of one of the main galleries, which was excavated parallel to the displacement vector of the landslide. It was possible to differentiate at least three periods of tectonism. The first period characterized by petrified slickensides, probably contemporary with the fault age, and the second and third period characterized by polished clays with slickensides of recent appearance which according to the writer can be assigned to recent tectonic processes (neotectonic). Figures 4 and 5 show recent slickensides (striated and polished surfaces) found during excavation. Two generations of slickensides were also observed in a scanning electron microscope.

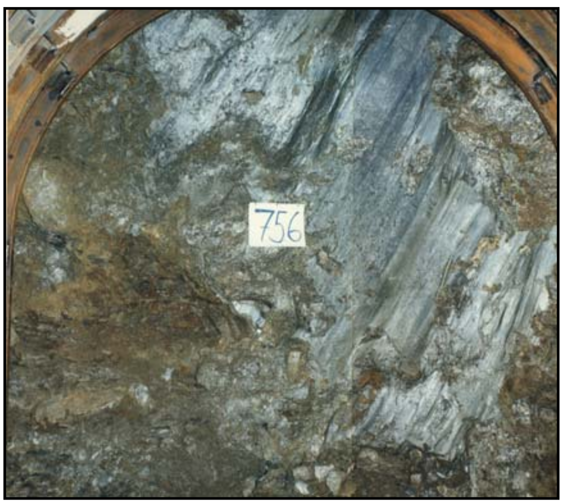


Figure 4. Polished striated surfaces (slickenside), inside the fault breccia found in the exploratory adits. Width of exploratory adit was 2.9 m.

Figure 5. Two generations of polished surfaces and slickensides in a sample taken inside the exploratory adits.

- No important water flows suggesting this factor as a triggering cause of the ancient landslide were found during the time the adits were excavated. Some water drops detected in the colluvial material revealed an abnormal salt content and very high pH value. This type of water has been identified by Rodríguez et al (1984) as fossil waters trapped in ancient landslides. Water chemical analyses did not reveal any bacterium or fecal particles that could suggest that they could come from the non formal houses built on the upper hill.

Instrumentation

Slope indicators installed in 1987 revealed an essentially planar main failure surface as can be seen in Figure 6. This geological profile has been elaborated taking into account results from borings, exploratory adits and instrumentation.

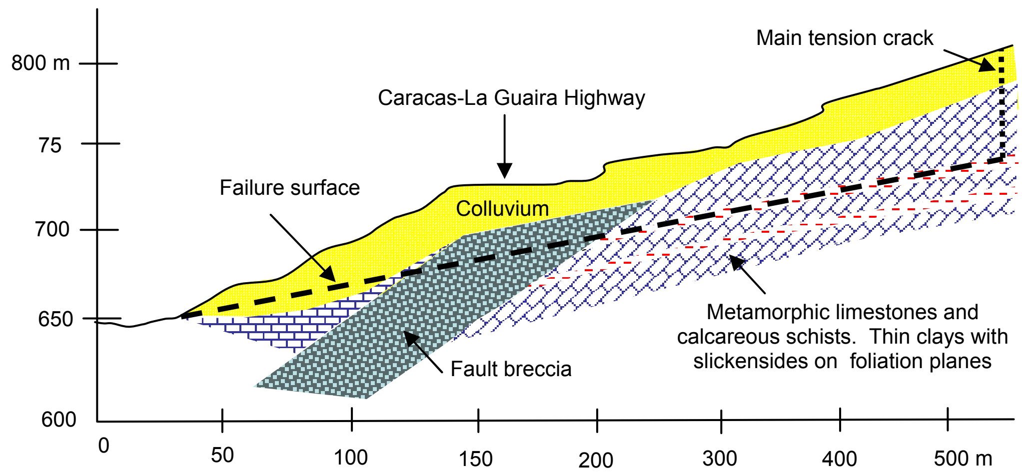


Figure 6. Generalized geologic section of the landslide.

Two inclinometers installed in June and September 1987, under the Caracas abutment and at the toe of the upper hill, close to the same abutment, revealed a well defined shear failure surface located 37 m deep and 25 m deep, respectively, below the deck elevation. Velocities of the landslide in 1987, estimated from slope indicator measurements reached 1.0 cm/month, and were decreasing progressively. From year 1990 to 1993 landslide velocities kept in a range from 1 to 2 cm/year. From this year on, measurements were suspended until year 2000 when 3 new inclinometers were installed, revealing velocities from 0.17 to 0.27 cm/month. During the first three weeks of August 2005, seven new inclinometers were installed in the unstable area. According to inclinometers readings and new field inspections of the lower and upper hill, it was possible to define the boundary of the active area which has been indicated in Figures 7 and 8. Approximate volume of the whole unstable area identified in aerial photographs is 6 million cubic meters; however, active sliding mass in 2005 has an approximate volume of 4.3 million cubic meters.

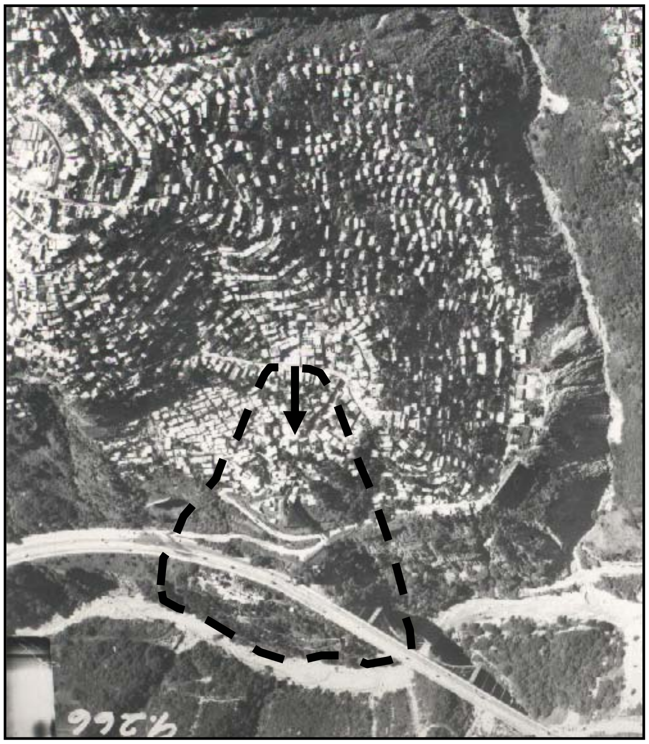


Figure 7. Aerial photograph (year 2000), showing boundary of the active landslide in year 2005.

Figure 8. Panoramic view of the southern hill of Viaduct No. 1, showing limits of the ancient landslide.

Slope indicators installed in year 2005 showed initially, velocities from 2 to 4 cm/month from August to October, however, beginning the second week of November an important increase of the rate of movement was measured, reaching value up to 19.75 cm/month. Six of the seven inclinometers installed in August 2005, which were located in the active portion of the landslide, were finally sheared off by the movement during the second week of November 2005. New technical inspections of the upper hill revealed an increase of the landslide movement and breakage of the main sewage pipe systems. Important cracks and deformations were observed in houses and access road to the urban development. The main tension crack on top of the hill revealed only a vertical displacement of about 1.5 m. It is important to mention that deformation measurements on the Viaduct's deck also revealed an increase in the vertical deformation of the arch. In view of these facts in November 2005, our consultant group recommended to the Ministry of Infrastructure to apply a contingency plan previously elaborated and to evacuate immediately approximately 400 houses that were built on the active landslide area.

Surface movement monitoring

Topographical survey made by Tranarg Consulting Company, consisted of the following activities: a) Installation of 3 bench marks on the opposite side of the hill considered to be stable. b) Installation of 15 control reference points on the surface of the ground. These points were distributed between the lower and upper hill and were subjected to vertical and horizontal positioning by triangulation, distance observations and spirit leveling. c)

Installation of 56 control points located on the deck and uniformly distributed on both lateral sides of the Viaduct. These points were periodically measured in order to know the deck deformation due to the slide thrust. d) Installation of control points on the pilasters and on both abutments of the structure.

Control points measurements were made from 1987 to 1993, and recently from year 2005 to 2006. Pilperca Construction Company was also in charge of monitoring deformation of the structure between years 1997 and 2006. During 1987, landslide horizontal velocities of surface control points varied from 1 to 3 cm/month, however from the beginning of 1988 a decrease of the rate of movement was observed down to 1-1.5 cm/year. This value remained almost constant until 1993, year where measurements were suspended. The average horizontal displacement vector revealed an approximately 40º-50° angle with respect to the Viaduct axis which is the same orientation of the slickensides measured on the ancient landslide trace cropping out at elevation 670, previously mentioned. At the beginning of year 2005, measurements of the deck deformation made by Pilperca, detected an important increase in the vertical displacement of the center of the arch reaching rate of movements between 3 to 4 cm/month. By the first week of November 2005 the velocity of the deck vertical displacement revealed, as it was also interpreted from the inclinometers measurements previously described, an acceleration of the landslide, increasing the velocity progressively from 4.7 cm/month to 30.7 cm/month in the period from November 2 to December 12, 2005. Since the inclinometers were soon sheared off and the high rate of movement value did not economically justify installation of new inclinometers, measurements were only made with control points on the surface of the hill and on the structure. New surveys made from December 13 to 29, 2005 revealed an average velocity of the upper hill of 1.2 cm/day and an average of 2.47 cm/day in the lower hill. These velocities increased in January 1, 2006 up to 1.85 cm/day and 38.4 cm/day, respectively, suggesting that the hill was in a process of rapid collapse.

MOVEMENT RATES AND FORECASTING OF FAILURE

In view of the landslide acceleration during the last months of 2005, an estimation of the collapse time was made based on Fukuzono (1985) and Voight (1989). Taking into account that the inclinometers were sheared off, Fukuzono's theory was applied to the daily measurements of the vertical displacement of the crown. These measurements had been previously correlated with the slope indicators and surface reference point's results, revealing that from November to December 2005, the ratio of soil movement to arch closing was approximately 1:1. Figure 9 shows a relation between the inverse of the velocity (month/cm) against time; the extension of a straight line fitting the data, would intersect the time axis at December 16, 2006, date of the forecast possible collapse.

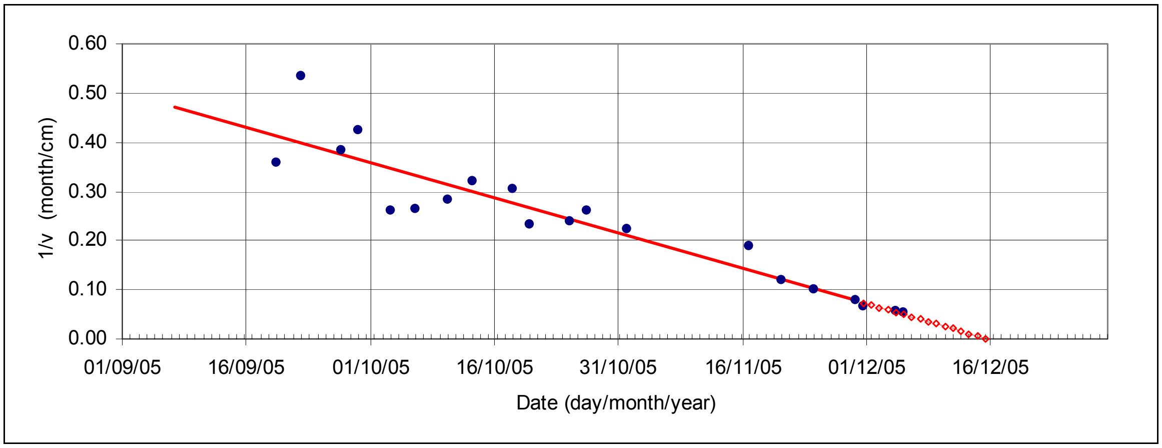


Figure 9. Inverse of velocity-vs-time (September 2005 to December 2005).

It has to be mention that all measurements that were progressively incorporated to Figure 9, coincided with the extension of the straight line towards December 16. However from December 11, the rate of movement remained approximately constant with a value of approximately 30 cm/month, as it can be seen in Figure 10. It can also be seen in this Figure that on January 5, 2006, rates of movement have an important increase up to a moving average of 150 cm/month, date in which an important movement affected the structure causing severe damages. In January 5, 2006 it was decided to declare Viaduct No. 1 out of service.

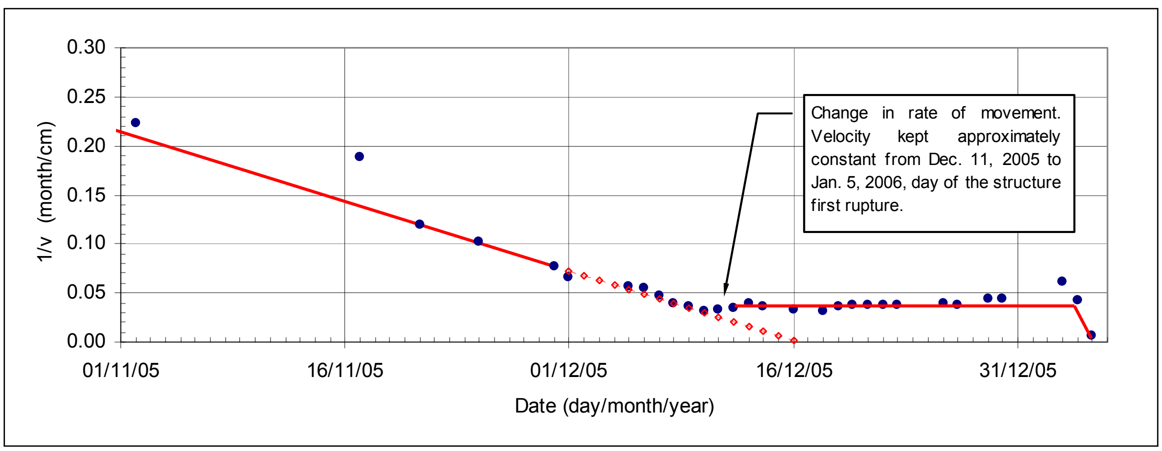


Figure 10. Inverse of velocity-vs-time (November 2005 to January 5, 2006).

Even though the aforementioned theories did not predict the exact day of the acceleration of the movement and the date of final collapse, there is no doubt that their use provided valuable information for decision making processes. In other case histories as in Vaiont, Italy (1963), if these theories would have been used, failure could have been predicted over a week in advance (Voight, 1989). It will be necessary to investigate if those theories are more effective in case of landslides with steep inclined failures surfaces that in the case of failure surface with low angle of inclination as it is the case under consideration. Voight (2005, personal communication), advises to apply his theory with caution.

EVALUATION OF DIFFERENT REMEDIAL MEASURES

Evaluation of several solutions to the problem have been discussed previously by Goodman, Salcedo & Sancio (1993). In summary they can be described as follows:

- Alternatives to stop the slide before it destroys the bridge. (Drainage, excavation, anchoring, filling the valley).
- Alternatives to stop only a portion of the slide (Anchoring, excavation or combination of both solutions).
- Alternatives aimed to isolate the Viaduct from the slide.
- Alternatives addressed to adapt the Viaduct to the slide displacements. These alternatives would be effective only if the behavior of the landslide could be predictable. Taking into consideration that the hillside may move with displacements greater that those tolerated by the structure, an exclusively structural approach would not solve the problem.
- Rerouting the highway.

In 1987 our consultant group recommended to build a new bridge located out of the sliding mass, founded on competent rockmass at the lower level of Tacagua creek. This new bridge was also aimed to avoid all the landslides affecting the first 4 kilometers of highway. It was recommended that as a first step a Viaduct of 800 m long could be built substituting Viaduct No. 1 as well as 500 m of highway also subjected to landslides problems. In October 1990 a high level technical panel recommended to fill the valley with a 2.5 million cubic meters embankment, building previously a 500 m long, 9 m in diameter concrete culvert. None of these recommendations were finally accomplished. From 1993 to 2005 only structural rehabilitation works were carried out on Viaduct No. 1, trying to adapt the structure to the slide displacements. These structural works certainly extended the structure service life, but they could not guarantee its final stability.

Due to an increase in the velocity of the slide mass, the Ministry of Infrastructure, (former Transportation and Communication Ministry) again consulted our professional group in May 2005. After reviewing data concerning bench marks, slope indicators and structure deformation, it was recommended to initiate as soon as possible the construction of a new Viaduct suggested in 1987. Considering that time needed to build the new structure was approximately 1.5 years, it was also recommended to build a detour of 2 kilometers which could be built in about 6 months. This detour was identified as a "contingency road", because it was designed to be used only in case of a collapse of Viaduct No. 1, before the new bridge was finished. The Ministry of Infrastructure decided to go ahead with these recommendations and earthwork for the contingency detour began in September 2005. In the mean time efforts continued to adapt the structure to the slide mass movements, trying to extend its service life. Unfortunately the Viaduct had to be closed about two months before the contingency road was finished, generating a difficult crisis; during this time it was necessary to use the old Caracas –La Guaira road which did not have the same traffic capacity of the highway. The contingency road was opened to traffic in February 25, 2006, and it is expected to be used until the new bridge is put into service on June 2007.

EFFECTS OF THE LANDSLIDE THRUST ON THE VIADUCT No. 1 STRUCTURE

Loading of the Caracas side of Viaduct No. 1 due to the moving mass progressively caused increasing distress in the structure. Loading on the longitudinal direction of the Caracas side reduced the clear span between arch hinges, thus raising the crown. Computer simulations and field measurements indicated an approximately 1:1 ratio between the arch closing rate and the raising of the crown rate. At the same time the deck, which is attached to the crown, rises together with the crown and moves 0.5 cm horizontally towards La Guaira side when the arch closes 1 cm. Data gathered from computer simulations indicated the initiation of collapse at a crown vertical deformation of 67 cm; two fractures were expected to develop at both side of the crown producing an unstable structure with four hinges. Recommendations were made in 1988 to stop traffic on the structure when that deformation reached 50 cm. It is important to mention that the arches were designed to resist only compression stresses sections and hence no reinforcement was provided to resist tension forces. Due to the fact that the average displacement vector of the active landslide has an angle of about 40º-50º with respect the Viaduct's longitudinal axis, and considering that the La Guaira side is stable, the structure had also been affected in its transverse directions causing a horizontal flexural deformation which reached 60 cm in November 2005. According to the structural engineers besides the possible failure of the arches, this flexural deformation had to be corrected as soon as possible because it might cause the final collapse of the bridge. Structural rehabilitation measures began in 1992 and continued to the end of 2005.

Geodetic measurements revealed that at the end of November 2005, with most of the rehabilitation measures completed, the vertical deformation of the crown was 90 cm, and at the end of December 2005, this deformation reached 115 cm. Figures 11(a) to 11(d) show vertical deformation of the crown due to arch closing at different dates.

Figures 11a, 11b and 11c. Comparative views of deck deformation at different dates (August, 2005, December 2005, and January 5, 2005. Figure 11d: Closed view of deck vertical deformation which had reached 130 cm at the crown and 167 cm at the ruptured section; picture taken from the Caracas pilaster in March 9, 2006.

In addition to the deck deformation, since 1987 many other damages were observed in Viaduct No. 1, such as rotation of the Caracas abutment, cracks in several sections of the structure and inclination of piers 9 and 10 located in the Caracas side. The following rehabilitation measures, all led by the structural engineer Rosendo Camargo-Mora, and the extensive communication between geologists, geotechnical engineers, structural engineers and the contractor, made it possible to extend the life of the Viaduct, beyond 67 cm of longitudinal displacement:

- Replacement of original concrete rockers located in the Caracas abutment and in the upper chamber of Caracas and La Guaira pilasters, by a set of steel struts.
- Replacement of Caracas concrete abutment by a steel truss supported by a group of micropiles and placed on top of rollers that could freely move in the Viaduct's transverse direction, and allowed recovering of progressive displacements.
- Construction of new foundations and piers to substitute Piers No. 9 and 10 between the Caracas abutment and Caracas Pilaster. The new foundations consisted of a group of micropiles. Each new pier included four concrete columns that supported a pair of steel trusses located on both sides of the existing piers and placed on rollers that can freely move in the Viaduct's transverse direction.
- Placement of telescopic beams in the upper chamber of La Guaira pilaster supported by Teflon plates that allows the free displacement of the deck over the pilaster in the longitudinal direction.
- Construction of a cable system placed in a U shape grabbing piers 9' and 10' located on the Caracas side. This system served two purposes: a) Provide the piers with a redundant mechanism to supply lateral support in case of a potential transverse shear failure at the base of the piers (caused by the transverse landslide thrust), and b) To help releasing the potential energy stored in the deck (cause by its bending) by intentionally eliminating the shear resistance at the base of the piers, thus transferring the transverse reaction of the deck to the cables. Then by slowing releasing the tension in the cables, the deck returned approximately 25 cm towards its original position and hence unloaded a considerable amount of potential energy stored in it.
- Postensioning of the three hollow box type parallel doubled-hinged arches and the design of a plastic hinge at the crown. The postensioning was designed to delay cracking of the arches. The postensioning and the reinforcement layout provided at the crown, promoted the formation of a plastic hinge at this location. The plastic hinge itself was self-activated by the progressive increase in longitudinal displacements, and anticipated to form at a vertical deformation of approximately 80 cm. The plastic hinge together with the two real hinges converts each arch into a statically determinate structure, partially immune to the detrimental effects of the longitudinal thrust. This premise was true as long as the deck was free to move in the longitudinal direction. Thus, on numerous occasions it was necessary to demolish the concrete deck when it reached the abutment in order to create a gap so the deck could freely move without restraining the crown from doing so. Unfortunately in January 5, 2006, a sudden and significant landslide displacement closed the existing gap between the deck and the La Guaira abutment. As a consequence the crown was restrained from movement and the arches were no longer immune to longitudinal displacements. The deck acted as a horizontal trust restraining the longitudinal advance of the crown and the arches

fractured at a distance of approximately 37 m measured from the crown to Caracas side, where the demand/capacity ratio resulted maximum. Anyhow, due to the structural remedial measures accomplished, the vertical deformation of the crown reached 120 cm, instead of 67 cm as it was estimated without reinforcement, providing the structure an important extended life.

THE JANUARY 5, 2006 EVENT

After three days of an intense rain accumulating 107 mm, on January 5, 2006, a sudden acceleration of the landslide occurred. A first evidence of this acceleration was noticed at 2:30 am after a new cutting of 25 cm was completed in the Caracas abutment joint. This event can be summarized as follows:

- At 2:30 am the open joint at the Caracas abutment began to progressively close and by 7 am, closing reached 25 cm.
- At 7:30 am a sudden movement of the deck was noticed changing its symmetrical shape to an asymmetrical shape, and as a result an important compression generated in the arches, breaking them at 37 m from the crown in the Caracas side. Figure 12 shows a measurement of the deck deformation recorded in the afternoon of January 5, 2006.
- As it was mentioned before, the gap at the expansion joint between the deck and La Guaira abutment closed as a consequence of the sudden landslide movement. Probably after the arch fractures occurred, the deck bounced back and an opening of 6 cm was noticed at this location.
- It is worth mentioning that due to the event of the morning of January 5, 2006, it was decided to prohibit vehicular and pedestrian transit on the Viaduct, and immediately proceed with a detailed inspection of the structure. It is also important to mention that according to the estimated displacement, during the morning of January 5, 2006, the threshold velocity (5 cm/day) selected by our professional group to prohibit traffic on the structure, was widely exceeded.

Results of inspection of the structure can be described as follows:

- Breakage of the three arches at a distance of 37 m measured from the crown towards the Caracas side. Crushing of concrete by compression at the fracture section was clearly observed inside and outside the arches.
- Cracking and concrete crushing of the longitudinal beams that support the deck under the fracture section.
- Plastic deformations in the steel trusses that supported the deck in Piers 9’ and 10’.
- Severe cracking of the grade beams located on the Caracas, between Piers 9 and 10.
- Rigid body rotation of the Caracas pilaster, reaching a maximum of approximately 2%.
- A significant change in the geometry of the arches (Figure 12). The maximum deformation originally at the crown changed to the section where rupture of the arches was observed. Also a transverse crack on the deck in the same section where the arches were broken, was also noticed
- Cracks in the concrete elements located in the lower chamber of Caracas pilaster.

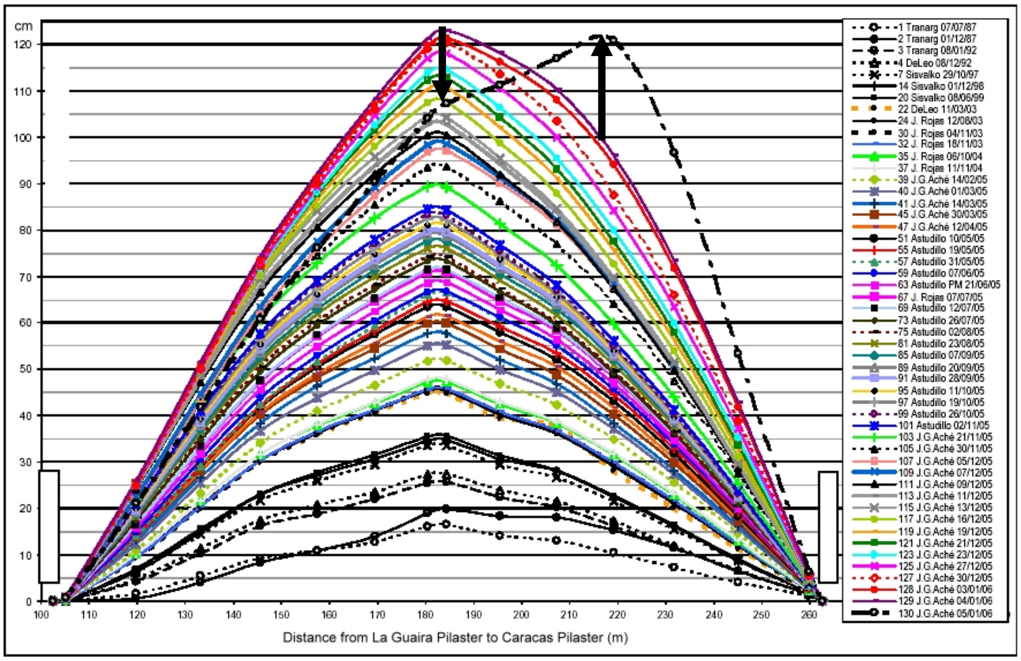


Figure 12. Vertical deformation of the deck before and after the January 5, 2006 event. Measurements between La Guaira Pilaster (left) and Caracas Pilaster (right) from June 7, 1987 to January 5. 2006. Note that the crown descended 17 cm and the broken section raised

In summary the event of January 5, 2006 initiated by a sudden acceleration of the moving landslide mass and subsequent stages of rotation of Caracas pilaster, a horizontal displacement of the deck towards La Guaira side, a deck rebound by the restriction at the La Guaira abutment joint, a change in the vertical deformation shape of the deck, and the rupture of the three arches. Figures 13 and 14 show some damages in the East arch and on the deck, as a consequence of the January 5, 2006 event.

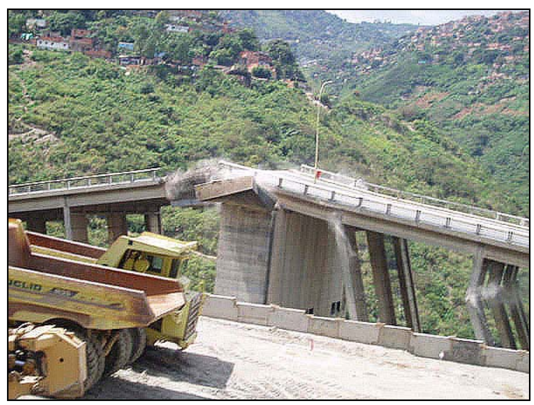
Figure 13. Panoramic view of the East arch rupture. (January 5, 2006).

Figure 14. Cracking of the deck due to the January 5, 2006 event in the same structural section of the arch rupture. (January 5, 2006).

Concerning the behavior of the landslide, from December 25, 2005, it was observed a clear progress of the tension cracks at the lower part of the hill close to the Caracas abutment. Intense rainfall beginning January 2, 2006, allowed water infiltration through tension cracks and important vertical and horizontal displacements were observed in the lower hill.

THE MARCH 19, 2006 VIADUCT FINAL COLLAPSE

Alter prohibiting cars and pedestrian on Viaduct No. 1 in January 5, 2006, monitoring of the deck deformation showed that the new ruptured section was rising progressively. Risk of collapse of the structure was evident and therefore it was not recommended to accomplish any repair measure that could represent risk for workers lives. The last measurement of deck deformations was made in March 17, 2006 and due to the clear progress of cracking it was decided to prohibit all activities at the bridge area. On Sunday March 19, 2006 a collapse of the structure occurred. Two persons who were coincidentally taking pictures of the Viaduct at this moment, could provide valuable information about the process of collapse (Figures 15 and 16). Three seismic stations located at 3, 14 y 19 kilometers, respectively, from the Viaduct, allowed the impact of the fallen structure to be estimated as an equivalent earthquake of magnitude Mw = 1.6.



Figures 15 and 16. Pictures taken during the Viaduct No. 1 collapse. March 19, 2006 (11 am, local time). (Pictures taken by A. Fonseca and F. Federici).

Figure 17 graphycally represents the inverse of velocity (day/cm) of the ruptured section that occurred in January 5, 2006, against time. It can be seen that fractures in the structure of the Viaduct No. 1 could have been approximately anticipated when readings were close to the x-axis.

LANDSLIDE MECHANISM

Concerning the landslide mechanism, there are sufficient evidences to conclude that the hill movement affecting the Viaduct No. 1 is due to the reactivation of an ancient landslide. However it has been difficult to establish with certainty which causes originated the initial landslide and its first reactivation on a gently dipping sliding surface (12°-16°). Several possible causes were considered, including the structural geologic setting influence and rainfall infiltration. These possible causes are discussed in the following paragraphs.

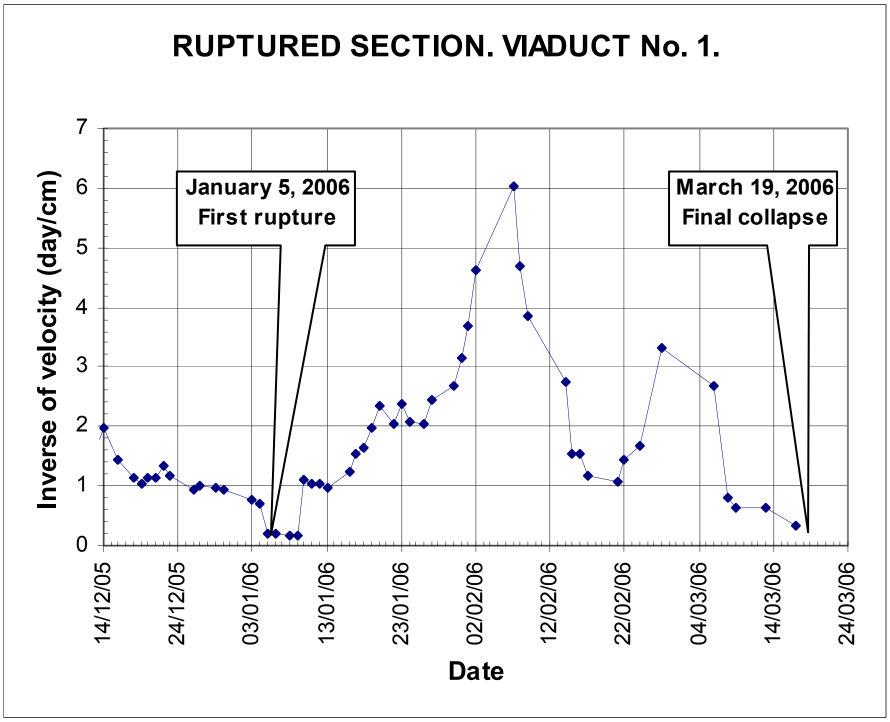


Figure 17. Inverse of velocity –vs– time. Measurements of two reference points installed on each side of the January 5, 2006 ruptured section.

Characteristics of the colluvial soil outcrops originated by the ancient landslide represented by large and small variously oriented rock blocks distributed in a chaotically way and encased in a soil matrix, suggest that the ancient landslide probably occurred very rapidly without time for the rock blocks to rest in a position according to its weights and dimensions. According to this fact, it is probable that the landslide could be associated with tectonic deformations, a hypothesis that is compatible with the evidences observed in the exploratory adits. There is no doubt that the existence of a wide tectonic breccia has a direct influence on the failure mechanism because due to the abundant slickenside and polished striated clay surfaces, this interval must be considered as a very low shear strength material. On the other hand the indirect influence of the fault breccia would be associated with its possible recent activity. Even a very small strike-slip movement of the fault could justify an instantaneous loss of shear strength in the direction of the gravitational landslide which of course could explain its reactivation (Goodman, Salcedo & Sancio, 1993, and Salcedo, 1994).

Figure 18 shows a map including the first 4 kilometers of the Caracas-La Guaira Highway, indicating locations where landslides of important dimensions have occurred or where they are recently active. It can be observed that most of the unstable areas are closed to geologic faults interpreted by the writer in aerial photographs, one of them observed inside the exploratory adits. Even realizing that it seems reasonable to expect landslides close to fault traces due the high degree of shearing of rocks, the fact that landslides of important dimensions occurred close to faults, also suggests its activity. Landslides of important dimensions with failure surfaces of low inclination have been also reported along active faults in other countries of frequent tectonic activity such as Japan (Hasegawa, 1992).

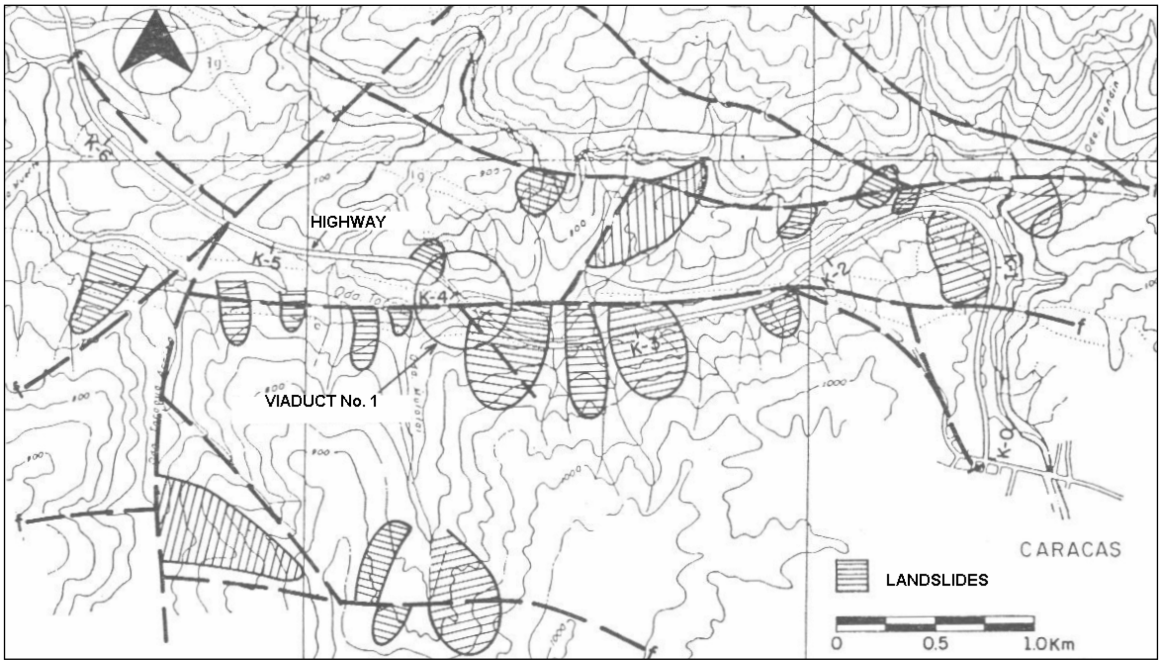


Figure 18. Map showing relationship between location of large landslides and geologic faults interpreted by the writer. Km 0 to km 4. Caracas-La Guaira Highway.

Another hypothesis that could explain the landslide reactivation could be the effect of dynamics forces coming from earthquakes that not necessarily must have their epicenter in the Tacagua fault system. Speculating, one could think that the landslide was reactivated during the Caracas July 29, 1967 earthquake (Mw = 6.5 Richter Scale). A mental exercise assuming that estimated displacement in the Caracas Pilaster before the beginning of measurements in 1987, initiated in July 1967, lead to the conclusion that the hill must have moved between 10 and 15 mm/year, values in the same order of magnitude as the ones measured in long periods of times (2 years). This circumstance made the Caracas July 29, 1967 earthquake, a possible cause of reactivation.

Figure 19 shows measurements of arch vertical deformation which in a way is an indirect measurement of the slide mass movement, extrapolating results to July 1967. From 1967 to 2006 all earthquakes recorded around the area have magnitudes ranging from Mw = 2.1 to 2.9, with the exception of two small earthquakes of Mw = 3.5 and 3.2, recorded in August 1983 and September 1986.

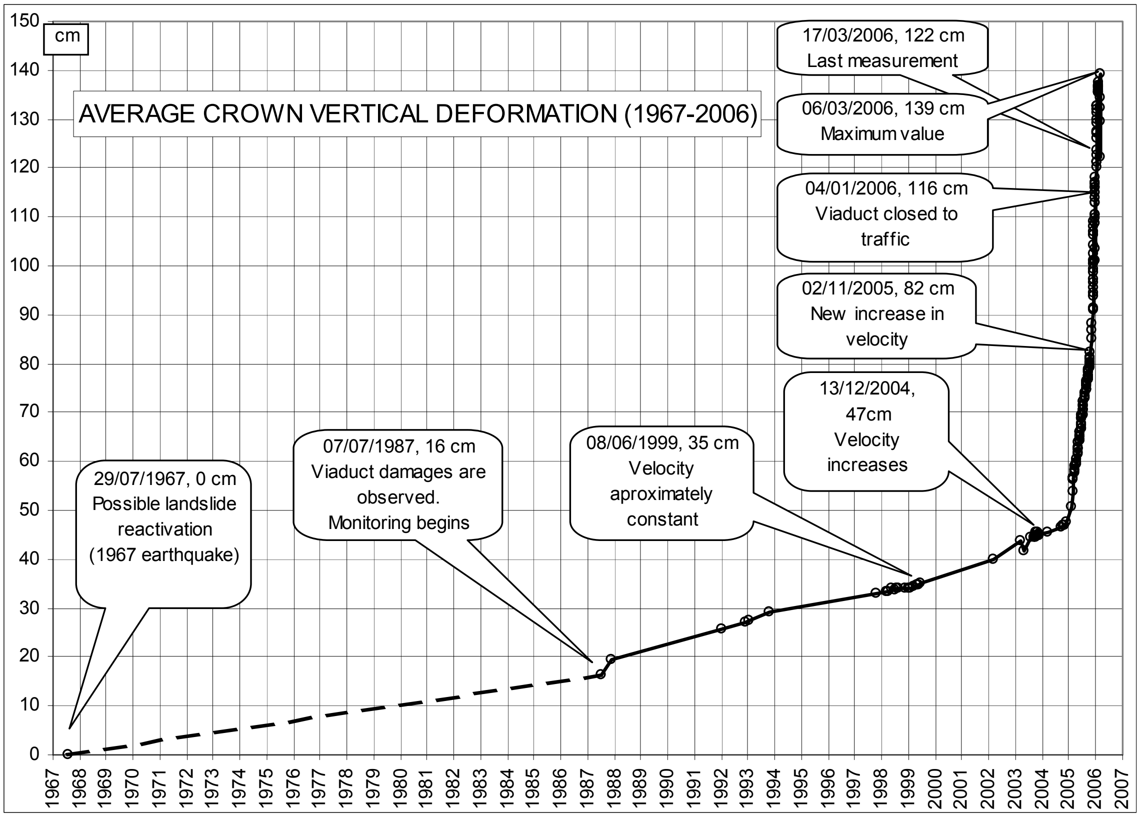


Figure 19. Average crown vertical deformation from 1967 to 2006. Monitoring began in 1987.

In order to evaluate if rainfall and as a consequence development of water pressures in the failure surface, was a significant factor in the development of the slide, several correlations were made between monthly rainfall and rate of movement of the sliding mass taking into account reference surface control points and inclinometers. Considering that accumulated rainfall over a period of time is more likely to be significant in affecting the movements of the sliding mass, multiple correlations were also made between a 10-day cumulative rainfall and rate of movement of inclinometers and surface reference points, as well as structure deformations. None of these correlations revealed results suggesting rainfall as main cause of landslide behavior. This lack of correlation does not discard the influence on the rate of movement, of possible water pressures induced by rainfall infiltration and from houses without adequate drainage service and limited sewerage systems. The writer believes that only the intense January 2-3, 2006 rainfall had an important influence on the landslide behavior because many tension cracks were already developed, allowing water infiltration. However, considering the aforementioned lack of correlation and the fact that very little water was found in the exploratory adits, this unique factor could not be accepted as responsible for the reactivation of the landslide and for its rate of movement variations.

In summary, even though it must be considered just a hypothesis, the writer believes that a factor such a neotectonic movements could explain reactivation and behavior of the landslide. Reactivation of the large landslide could have occurred during the July 1967

Caracas earthquake, and successive and very small neotectonic movement (creep) could explain its behavior with time. In this way research is needed to understand this factor, installing a monitoring system along geologically active faults and correlating small displacements (creep) that not necessarily generates earthquakes, with large landslides displacements.

FINAL REMARKS

Many lessons could be learned from the described case history. Some of them are:

- The well known importance of understanding the nature of geological factors before building a civil work, in this specific case a highway and an important bridge. This case confirms once again how aerial photographs are excellent tools at early stages of site selection for any civil work.
- Large landslides on a gently dipping failure surface, mainly in those areas where water pressures are not present, can not be explained by means of traditional slope stability methods, which neglect lateral in situ stresses. The influence of these stresses also cited as "tectonic stresses", "residual ground stresses" and "initial stresses" in the technical literature, must be taken into consideration whenever we are dealing with landslides in tectonized areas. The role of initial stresses in natural slope analysis has been discussed by Chowdhury (1976).
- Even though the possibility of neotectonic deformations is simply a hypothesis between the factors that could explain the origin and reactivation of the landslide, there is no doubt that the abundant striated polished clay surfaces and slickensides produced by shearing of apparent neotectonic origin, have a direct influence on the landslide mechanism. This factor has to be taken into account in similar geologic conditions.
- The case history also teaches that whenever we have to deal with problems associated to natural hazards, we can not completely trust the monitoring results, and postpone the decision-making process. Landslides may suddenly accelerate due to factors such as water pressure induced by rainfall infiltration, earthquakes or even creep deformations along active faults in the area.
- Finally it can be concluded as shown in other case histories, that failure forecast theories used with caution, are excellent tools during landslide investigations. There is no doubt that at least a rough estimate of the date of failure can be predicted from these theories which is of a great help during the decision making processes.

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