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MEXICANA DE PRESFUER 20 SA DE CV

FIRST REPORT OF THE SPECIALIZED TECHNICAL ANALYSIS ON THE
POSSIBLE CAUSES THAT CAUSED THE BRIDGE DEPLOYMENT
CHIRAJARÁ

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INTRODUCTION

• Background.-
On January 15 of the current year, minutes of the meditem. the sudden fall of the Tower 1 (axis B) of the Chirojaró Vioduto structure that formed part of the sector 4 A called "Chirojara". that leads from Bogotá or Villovicencio. at department of Cundinomorca and that currently is under the administration of the Concessionaire Vial de los Andes SA (COVIANDES).

• goals
The Interconcestones Interventor Consortium. entrusted Mexicana de Presfuerzo, SA with preparation of this report, with which the possible causes of the They caused the failure of the structure.

In addition to presenting a concept approach to the actions of revision and mitigation of risks of the collapsed and non-collapsed structure.

INFORMATION AVAILABLE

For the preparation of this report, the Audit Consortium has provided Next information;

• Studies and Detailed Redesigns fose 111. for the second road of sector 4a "Chirajara" de lo Vio Bogotá - Villovicencio Contract 444-039-12 STRUCTURAL DESIGN FOR SECTOR 4 or VOLUME VIII. JUNE Review No. 1 of the EC Villovicencio Consortium

• Calculation memories of the cable-stayed Chirajaró bridge, dated August 2016. prepared by the company Ingenieros Consultores Area

• Project plans (mixed board) elaborated by Ingenieros Consultores Area with the following numbering:

039-I2-S4A-EST-CHI-01-55
039-12-S4A-EST-CH1-2-55
039-12-S4A-EST-CHI-3-55
039-12-S4A-EST-CHI-03A-55
039-12-S4A-EST-CH1-04 -55
039-12-S4A-EST-CHI-04 A-55

or 039-12-S4A-EST-CHI-04 B-55

or 039-12-S4A-EST-CHI-5-55
or 039-12-S4 A-EST-C HI-05-55
or 039-12-S4 A-EST-C Hl-05 A-55
or 039-12-S4A-EST-C H1-05 B-55
or 039-12-S4A-EST-CHI-06-55
or 039-12-S4A-EST-CHI-07-55
or 039-12-S4A-EST-CHI-08-55
or 039-12-S4A-EST-CHI-09-55
or 039-12-S4A-EST-CHM 0-55
or 039-12-S4 A-EST-CHI-11-55
or 039-12-S4A-EST-CHI-12-55
or 039-12-S4A-EST-CH1-13-55
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or 039-12-S4A-EST-CHI-15-55
or 039-12-S4A-EST-CHI-16-55
or 039-12-S4A-EST-CHI-17-55
or 039- 12-S4 A-EST-CHI-18-55
or 039-12-S4 A- EST-C HI-19-55
or 039-12-S4A-EST-CHI-20-55
or 039-12-S4 A-ESTCHI-21 -55
or 039-12-S4A-EST-CHI-22-55
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or 039-12-S4A-EST-CH1-24-55
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or 039-12-S4A-EST-CHI-26-55
or 039-12-S4A-ESÍ-CHI-27-55
or 039-12-54A-EST-CHI-28-55
or 039-12-S4A-EST-CHI-29-55
or 039-12-S4 A-EST-CHI-30-55
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or 039-12-S4A-EST-CH1-32-55
or 039-12-S4A-EST-CHI-33-55
or 039-12-S4A-EST-CHI-34-55
or 039-12-S4A-E5T-CHI-35-55
or 039-12-S4 A-EST-CHl-36-55
or 039-12-S4A-EST-CHI-37-55
or 039-12-S4A-EST-CHI-38-55
or 039-12-S4A-EST-CHI-39-55
or 039-12-S4 A-EST-CHl-40-55

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or 039-12-S4 A-EST-CHI-43-55
or 039-12-S4A-EST-CHI-44-55
or 039-12-S4A-EST-C HI-45-55
or 039-12-S4 A-EST-CHI-46-55
or 039-12-S4 A-EST-CHI-47-55
or 039-12-S4 A-EST-CHI-4 & -55
or 039-12-S4A-EST-CH1-49-55
or 039-12-S4 A-EST-CHI-50-55
or 039- 12-S4A-EST-CHI-51-55
or 039-12-S4 A-EST-CHI-52-55
or 039-12-S4 A-EST-CHI-53-55
or 039-12-S4A-EST-CHI-54-55
or 039-12-54A-EST-CHI-55-55

• Project plans (concrete board) prepared by CONSORCIO EDL Ltdo
- CEISA with the following numbering:

613-06-S4 A-CHI-01-15
613-06-S4A-CHI-01 A-15
613-06-S4A-CHI-01 B-15
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613-06-S4A-CHI-02-15
613-06-S4 A-CH1-03 * 15
613-06-S4 A-CHI-04-15
613-06-S4 A-CHI-05-15
613-06-54A-CHI-06-15
613-06-S4A-CHI-07-15
613-06-S4A-CHI-0 & -15
613-06-S4 A-CHI-09-15
6I3-06-S4A-CHM0-15
613-06-S4 A-CHI-16-15
613-06-S4A-CHI-12-15
613-06-S4A-CHI-13-I5
613-06-S4 A-CHI-14-15
613-06-S4 A-CHI-15-15

Folders with Geotechnical Information! draft:

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- Photographic Record and Videos tornado with drone overflight on January 18th.

- FIRST WORKING HYPOTHESES

Of general mañero in the bridges cable-stayed with towers type "diamanle" can present several types of size

I. FAHa due to lack of tension capacity in the "crossing" area and diaphragm wall due to the change of direction of the load that goes down the columns of the tower due to its "diamond" type geometry.

»This is the main hypothesis in the CHIRAJARA case because it is observed very little presfueno in the slab "crossbar" and low density of reinforcement to take tensions in the diaphragm wall in the direction parallel to the crossbar slab.

In the videos that have been observed of the size it is appreciated that the lower columns are open from the connection with the slab crossbar and consequently the connection is broken with the diaphragm wall provoking the collapse of the tower and its cantilevered board.

2. Failure of the soil per cargo capacity sheet or slope of the slope causing possible settlement of the tower

»There is no evidence that this has happened in the CHIRAJARA case because notes that the foundation remained in its position even after colopso

3. Folio per capacity sheet in tironles or in their anchors due to overload of the same

»There is no evidence of this situation in case CHIRAJARA was not observed Detachment of the braces prior to the failure of the tower. In fact, it was observed that Start the fault in the lining the braces lose load and fall Along with the board.

• WORK METHODOLOGY.

On January 18 in the company of! Consorcio Interventor, a visit to the §ite of the Chirojoró Bridge. in which it was possible to carry out a visual inspection of the area of

collapse, in this visit, with the help of a drone a series of tomes were made photographs and videos, in which the various elements that could be identified they formed the cantilever of Tower I (axis B). Lower columns. Top Columns Tone of Atirontomiento. Board. Zapóla Cross Traverse Slab. Diafrogma wall and in which they could observe the condition of some eBas: stop with this veriícor and Try to check the first hypothesis of what fails.

A compilation was made of the information that was available, plans, calculation memories field photographs, field record, to re-roll a modeling of structure and refining an estimate of charges to carry out a verification of Is states of effort of the various elements at the time of collapse

• EVALUATION OF THE LOADS

o Estimation of Charges acting at the time of Collapse

Since in the calculation memory of the project under construction with board mixed, we did not count on the mechanical elements of the analysis of the Bridge, a estimation of the discharges or tower 1 (axis B) at the moment of the fa & a. It was considered soto the dead load and the estimated tensile load of the 4 pairs of rear suspenders that are attached to the counterweight stirrup.

Download the board to the tower

From the information of the manufacturing drawings of the metal board, a total weight of the oprox structure. 912 Ion pore a length of 418 m. whereupon obtains a linear board weight of 2,182 ton / m. The concrete slab on the board is of a width of I3m and a thickness of 20cm. with which a linear weight of 0.2m is obtained x 2.4 ton / m3 x I3m = 6.24 ton / m.

The permanent load due to the mix board is estimated at 2182 lon / m ♦ 6.24 lon / m = 8,422 ton / m.

I download it from the board or tower 1 is obtained considering its tax weight Towards the suspenders:

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the length of the board that unloaded or the tone 1 to the section corresponding to brace 13 is 52.67 ♦ 122.3 = 174.97m so the total charge is of 174.97 mx 8.422 torv / m = 1473.6 ton

Unloading to the tower due to the load of tensioning of the straps to the counterweight

The tension load of the 4 pairs of rear tyrolles that go to the rear is unknown. counterweight, so you make a guess of your stress load and you get the vertical component of these towards the tone.

According to the project specification, the 4 pairs of rear braces are conformed by 28 strands of diameter 0.62 "with breaking stress of 1860 MPa. Approximate angle of the braces with respect to or vertical is 49.4 °. A tension charge of 35% of your copacidod is assumed for the rupture.

With these dolos you get the following charge in the tower:
Tu = 28 x 1.5 cm2 x 18960 kg / cm2 = 796.32 ton = load of rupture of 1 tie
= 0.35 x 796.32 ton = 278.7 ton = estimate of tension load
Tysst = T skx eos 49.4 * = 181.38 ton = vertical component of ta load of 1 tie

The total vertical charge of the 4 tie pairs or tower is Ty = 1451.03 ton

Additionally, the weight of the suspenders with a weight of 1.3 kq / m is considered by lorón = 71.7 ton

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Discharge to the lone due to live construction load on the board

One living construction load of 50 kg / m2 was coasidered on the board. Considering the same tributary length of the board to the tower one gets a charge of 50kg / m2x 174.97 mx 13m = 113.73 Ion

The own weight of the tower will be obtained from the model to be made.

Summary of charge on the tone at the time of collapse '

- Own weight of the tower
- Mixed Board »1473.6 ton
- Vertical load of counterbalanced straps * 1451.03 ton
- Weight of suspenders * 71.7 ton

Sumo of permanent loads = 2996.33 ton (or the own weight of the tower)

- Construction live load = * 11373 Ion

Additionally and locally on the diaphragm wall, it is taxed by board and live load of construction was considered for a length of 12.33m.

- Mixed toblero * 12.33 mx 8.422 lon / m = 103.87 ton

* Corga live construction ■ 50 kg / m2 x 12.33 mx 13m = 8.02 Ion

• REVISION OF STATES OF EFFORTS AT THE MOMENT OF COLLAPSE

Since the main hypothesis of the situation corresponds to the failure of the connection Between cokrmnas and loso crossbar and diaphragm wall in ta zone of change of direction of the tower type '• diamond ", a finite element model of the tower has been re-solved with The estimated loads at the time of the fucks.

This model has been made with solid elements with the MIDAS FEA software (Finite Element Analysis). All the elements of the tower are modeled, including the Reinforcement on the crossbar slab.

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Reinforcement in struts (cross slab): 12 strands of 0.6 *

Loads in the model

1 %

The own weight of the extraction tower was considered and the following charges on the tower of otirontamiento:

• Permanent charge ■ 2997 ton
• Living construction charge * 114 tons

(} In the position of support of the board on the other side, they were applied and \ / distributed the following charges:

• Permanent load = 114 ton
\\ * Construction live load = 8 ton

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Results

For a better understanding of the efforts obtained in the area of interest, the Efforts in the direction YY corresponding ol ol axis cross bridge or ol axis longitudinal of the trovesaño slab

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Distribution of efforts in diaphragm wall and brace (cross slab)

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By analyzing the mathematical model of the tower, stress efforts are obtained in the brace and the diaphragm wall.

In the figure we show a detail of the concentration of efforts in the union of riostra'y wall diaphragm with the pore column the combination of permanent loads and live construction loads at the moment of failure.

Not necessarily the maximum efforts are in that zono. but there is evidence that the reinforcing steel of the brace (the cross piece) is not in the column. Therefore, the tension that develops in the union was determined and compared with the stress that resists the steel of pressure. In addition, the tension force developed was calculated in the diaphragm wall and compared with the strength of the horizontal reinforcement steel.

Acting and resistant forces in brace (crossing it)

The transversal area of the brace where the SYY efforts are observed is 600 cm x 60 cm = 36,000 cm2. The value of the tensile stress shown in the figure is 10.43 kg / cm2. therefore the tension load that this area had to resist at the time of the failure is of 36000 cm2 xlO.43 kg / cm2 = 375,480 kg = 375.48 ton.

The existing prestress in the brace is 12 strands of 0.6 "whose capacity to break is 12 x 1.39cm2 x 19000 kg / cm2 = 316920 kg = 316.92 ton which is less than tension load of 375. 48 ton and so tonlo it is verified that there would be fucks Presence in this zone.

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Acting and resistant forces in diaphragm wall

According to reinforcement plans (plan 039-12-S4A-EST-CH1-28-55) the steel of horizontal reinforcement in diaphragm wall corresponds to stirrups C54 that are of 2 branches with varóla de. «4 (Av = 1.27cm2 per bouquet) and placed every 20cm.

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The thickness of the wall is 50 cm. so Ionian the tronsversol area pore the width tributary that corresponds or elbow eslribo is 50 cm x 20 cm = * 1000 cm2. The value of SYY tension stress shown in the figure is 10.43 kg / cmZ so Ionium loads it tension that had to resist this zone ol moment of the failure is of 1000 cm2 xl0.43 kg / cm2 «10,430 kg = 10.43 Ion.

The existing reinforcement in e! wall is I Stirrup # 4 with 2 blunt whose capacity to creep is 2 x 1.27cm2 x 0.4x4200 kg / cm2 * 6400.8 kg = 6.4 Ion which is less or solicitation of 10.43 Ion which implied that cracks would appear in the diaphragm wall that would give the indication of a next folio.

If one considers the capacity to break of said horizontal reinforcement, one would have resistance of 2 x 1.27cm2 x 4200 kg / cm2 - 10668 kg = 10.67 ton which is openas 2% above the required charge of 10.43 Ion. that however is not being considered no charge tactor so I check that I would have the reinforcement *hole* horizontal of the diaphragm wall.

It remained chlorine that once the phallus was presented in these resistant elements, presents a redistribution of corgos that overload other elements of the tower and I unleash the instability of the entire structure.

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Detail of the prestress in the cross slab

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The plan of the tosa travsoAo is shown with the presfuerzo indicated in plan and confirmed as existing exertion, liorna the attention that in the short toso or longitudinal direction of the bridge indicated a prestrain of 14 cables of 8 strands of 0.6 "that equals 112 strands of 0.6".

Besides that I do not remain chlorine which demondane solicitation is contidod e in this direction, being such a short element in that direction | 6.0 m) would present very large losses, so, it is possible that this is a drawing error and possibly this effort should go in the long direction of the tile or direction transverse to the axis of the bridge.

The same model previously described is being worked on, opaqueting the effort under this assumption. It is to be expected that the stress in the crossing and Reduce and redistribute the tension in the diaphragm wall. This was re-written only as exercise to check if said effort would have been sufficient and will be shown results in a next report.

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It is recommended to extend the structural revision of this project to carry out a complete model with all its constructive stages as well as to decide the actions to be made in the 2 tone (C axis) that may be in one Emile failure condition.

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• CONCLUSIONS

Although this is a report product of the first overiguooons in little mós of one semono. and there is a lot of material to review, we consider that they can be State some conclusions:

1. After analyzing the results with a simplified model of the Tower, the insufficiency of the structure was confirmed in the presence of static charges construction progress.
2. There is no evidence that it was produced due to Geotechnical issues. from according to the photographic record, in which the foundation head is appreciated intoxicated after the accident (see Geotechnical summary and observations. 2018)
3. The presence of an exogenous event such as an earthquake or gust of wind is ruled out that will cause the collapse.

4. The video of the security camera that recorded the collapse, shows that the fucked in the same tower, not detecting any break in the board and frontes ol start the collapse.
5. Today deficiency in the design of creep system between tower columns in the change of address section.

to. Not today steel reinforced through between the inner side of the columns and the ends of the trovesoño slab.

b. The above was confirmed not only in terms of design but also in the visit to the site and in photographs during the construction of the crossing

c. The trovesoño loos are only crossed by 12 single cores of strand of 0.6 ", which are insufficient in the case of static stresses produced by the components of upper columns.

d. In the longitudinal direction (parallel to the axis of the bridge), the same slab trovesoño contains much greater amount of effort when in this meaning is unnecessary.

and. The crossing slab must contain in its section, the change section of direction of the columns, that is: your Ironsversal axis should have coincided the axis of the section change, when in this case, it is uslo under it.

r. The diaphragm wall 50 cm thick that I link the lower columns, It has designed the main reinforcing steel vertically and the secondary in the horizontal sense, when its function is inverse, with tensions (tractions) in fronsversal form.

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• RECOMMENDATIONS

1. Assuming that this section has been designed and constructed under the same parameters of the collapsed tower. and with the knowledge that it is find a previous construction phase or the collapsing pile, surely the Tower that still stands today. possibly it is in an initial state of failure.
2. Although there are reinforcement techniques that could rescue the structure that I remain standing. we recommend not to take more risks, and make it immediately possible a rational and safe demolition, which avoids the risk for both people and for the surrounding structures-

3. The dismantling of the debris of the collapsed structure. It has to be done from way careful, previous previous assurance to ground of their components since they show an incipient instability. We recommend use a system of a Cable via or Blondín system of capacity between 15 and twentyTon from capacity with enough area from acluation cross large enough to remove the sectioned remains of the structure.

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Cd. De Mexico January 25, 2018

Baborated

Ing. Christian Balcózar Benitez

Ing. Miguel Arifael fought Chang

Revised

/ V /

Ing. Pedro Guardado Pérez

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Approved

z Aguilar

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