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Experimental Determination of Damping Ratio of a Transparent Pier with Steel Piles and Reinforced Concrete Board

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SUMMARY

This paper presents the results of Pull-Back tests realized during December, 2002 in Port Ventanas, one of the mains solid and liquid bulk ports located at Quínteros harbor, V region of Chile. The tests objective was to determine the damping ratio of a transparent pier of steel piles and reinforced concrete slab. The access bridge of the pier was constructed in 1964 and it has 375 mts. length and 8.3 mts. width. It consists of five independent decks with trestle support structure, in which are supported pipelines for fuels, asphalts, chemicals, sea water intake, and different types of conveyors belts. Every deck is formed by a slab of 30 cms. of thickness, steel girders inserted in concrete and steel piles. To realize the tests a steel pull back system was made, installed in longitudinal direction in between two decks. It consisted mainly of a hydraulic jack, four steel flat bars, fuses and two steel base plates with bolts anchored set to the floor slab. Its operation was realized through the hydraulic jack, generating the forces through the system, and the corresponding decks displacements. Reaching the system to a specific force, the fuse broken was obtained. The pull back tests generate decks oscillations and records of acceleration, speed and displacements was obtained. Seven Pull-Back's tests were done applying loads from 17 tons of fuse break up to 43 tons of pull back load.

Besides the pull back tests, microvibrations measurements were done. The records analyses of the Pull-Back test result in a damping ratio for the pier deck is between 2,6 % and 3.1 % and a main period of vibration of 0.61 seg. From the microvibrations measurements damping ratio of 3 % +/-0.5 % was obtained, of similar magnitude than the pull back test results. Besides the previous damping ratio, the tests displacements results were lower than the expected calculation and the pier desks oscillation took place in more than two decks simultaneously, because of a non independent behavior of the pier desks for the longitudinal seismic displacements.

INTRODUCTION

This study correspond to an investigation of the Civil Engineering Departament of the Chile University, guided by Mr. Hugo Baesler, specialist in ports and professor of the same University, and the participation of professor Rubén Borocheck and the engineer Mr. Carlos Vega.

The interest of this memory was to realize a contribution to the structures design of transparent wharfs, solution wich has been buildt in several ports along the Chilean coast in the last years.

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When these structures are designed, the supposed damping ratio varies between 2 % and 5 %, due to the effect of concrete floor slab, transversal and longitudinal frames of beams and steel piles. After an specific analysis the designer decides by a value, but he does not have certainty if this value is the real one for the transparent wharf. This fluctuation affects the value of the displacements of the structure, that can be bigger if the real damping ratio is smaller. With this the facilities and equipments which are supported along the wharf as for example the polipipes of fuel or conveyors can suffer impacts between each other for seismic events, since they are separated in each section by expansion joints and to produce damages and possible liquids spills that go through them.

This study tried to solve that doubt, analyzing the specific behavior of a transparent wharf located in Ventanas Port. It is located in Ventanas, bay of Quintero, V region of Chile. It has about 35 years of operation and its main function is the loading and unloading of solid and liquid bulks in the central zone of the country. The reason by which this structure was chosen is that all the wharf technical information was available and its located in a high seismic zone, with the the seismic records of Valparaíso 1985, with 7.8 of magnitude.

Objectives
To obtain the damping ratio of the transparent wharf structure.

Description of the Ventanas Port's Wharf
The wharf is formed by several independent structures between each other, with differents dates of construction. The first construction period of the wharf corresponds to year 1965, with access bridge, sites Nº 1 and Nº 2, wharf and mooring dolphins. In a second construction phase more sites were available. The access bridge has 375 mts. length and 8,3 mts. width. It consists of 5 independent boards, 4 of 76 mts. length and 1 of 71 mts., with similar structures that acquire more depth as the bridge goes into in the sea.

Fig. 1 Location of Ventanas Port's Wharf

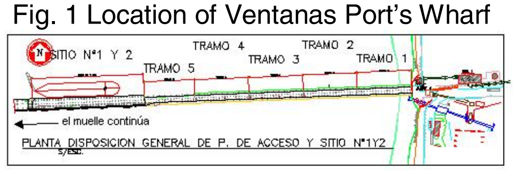
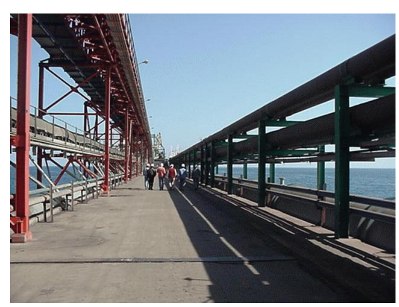


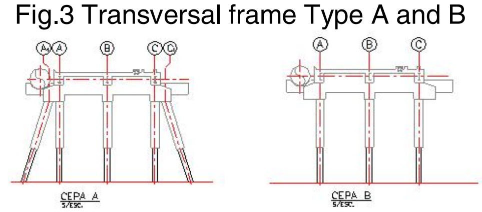
Fig. 2 View of Access Bridge.



The trestel bent are of two types:
The trestel bent Type A have two inclined piles of 12" diameter and three vertical piles of 18" for the boards Nº 4 and 5, and of 16" diameter for boards Nº 1, 2 and 3. These piles, are connected at board level with a steel transversal beam (W36*135) embeded in concrete with a total cross section of 90 * 186 cm.

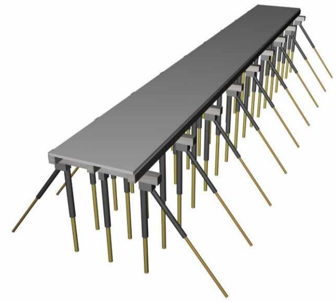
The trestel bent Type B do not have piles and they form a frame of three vertical piles of 18" for the boards Nº 4 and 5 and of 16" diameter for the boards Nº 1, 2 and 3. They connect these three piles at board level, with a steel transversal beam (W24*68) embeded in concrete, with a total section of 90*96 cm. Both types of trestel bent have an alternative distribution along each span of the access bridge of the wharf.

Fig.3 Transversal frame Type A and B



Over the board floor slab exist steel structures which support three conveyor belts that transport solid bulks like, coal, clean grains, copper concentrated, clinker, other. Also, there are pipelines and ducts at each sides of the board along de access bridge. There are three types of pipelines, fuel and asphalt and their derivatives, sulfuric acid, sea water for the intake of the cooling water system of Ventanas thermoelectric power plant. The CWS pipes for the thermoelectric plants, are located at each side of the board access bridge, along the first up to the fourth span.

Fig. 4 Span Nº4



The steel structures are marks that are to the sides of the section. Alongside North they support to the asphalt pipes and their derivatives. Alongside South they support to the conveyor belts and the line of sulfuric acid.

Obtaining of rigidities and periods through SAP 2000

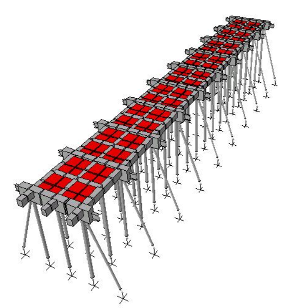
There were modelated five decks in independent form in SAP 2000, obtaining the following rigidities, with their respective periods.

Table Nº 1 predominant Frequencies and observed dampings.

Rigidity and periods of the different decks from the Bridge Access					
	Deck 1	Deck 2	Deck 3	Deck 4	Deck 5
Rigidit (ton /cm)	114	54	30	23	20
Total weight (ton)	2062	2062	2062	2062	1642
Period (seg.)	0,79	1,15	1,55	1,67	1,55

For the test the deck Nº 4 was chosen, like representative structure, because the first and second they are near to the earth, being its structure very rigid and non representative for this investigation, as it can be seen in the table Nº 1. Also the fourth deck has piles longer than the previous sections. The fifth deck has an smaller mass because it does not support the ducts of refrigeration, which also can be observed from the table Nº 1 through the total weight existing in each deck. In addition as it can be seen in the following chapters, the rigidity value of the fourth deck of 23 ton/cm, helps the design of the shot system that will move this deck.

Fig. 5 Span model N°4



In addition calculated in manual form the rigidity of the deck 3 Nº and Nº 4 to reaffirm the obtained value of program SAP 2000. For the deck Nº 3 gives a value of 29 ton/cm and for the deck Nº 4 gives 22 ton/cm.

Test in the wharf

There was made a shot system that applied a force of 50 ton., to the deck. The reasons by which it was decided by this value are the following ones:

Analyzing the floor slab of the deck Nº 4, it can be applied an exact maximum moment of 41 ton/mt. The force for that mentioned moment is 150 ton. Supposing an impact requirement, it would mean an static force of 75 ton, in order to the impact were double of that value, as security factor.   Therefore for the test was taken as real value 50 ton, like maximum force to apply.

Design of the Shot System

In order to be able to make the test it was designed a shot system that was located at the edge of two chosen decks. This consisted in two main plates fixed to the floor slab, one over and two under the floor slab, in each deck; connection elements in the superior plate and shot elements wich connect the two main plates ; a bar wich connect the shot element with the hidraulic jack and several fuses that were designed so at they were broken to specific forces of shot. The idea comes from a previous study in where dynamic tests were realized, applying to a system similar.[9].

The system was designed for the cylinder force of 50 ton., as static force and for a impact force due to the fuse cut of 100 ton.

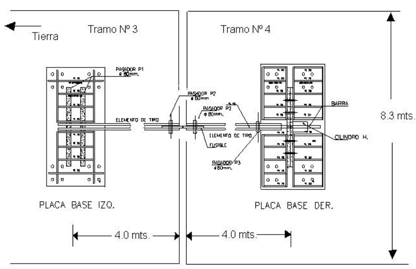


Fig. 6 Location in Plant of shot system in the Bridge access

Fig. 6 Location in Plant of shot system in the Bridge access

Design of the Fuses

The Fuses were designed to avoid fault by traction in the edge holes , where it has a diameter of 83 mm and to have the smaller deformation by flow in the center of the fuse. In that zone should produce a fuse breakage to an specific traction force and that is the reason why it was designed an special fits.

Table N° 2 Technical specifications of the Fuse material

Fuse	
Brand	Thyssen
Material	Allloy steel
Name	XAR Prime
Abreviation	20MnCr6-5
Flow Tension	7.0 Ton /cm2
Deformation	13 %
Breakage tension	10.0 Ton/cm2

There were designed several fuses, changing their thickness , wide and forms of fits.

It was investigated the form of fuse fits in order to the breakage took place to the exact force of design. Since the enough information about the material were not had, the behavior beyond the linear rank was not known. Reason why the fusible test was made with different fits in order to arrive to the necessary one. The forms and the fuse result was the following:

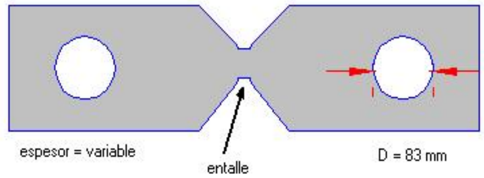


Fig. 8 Design of the final Fuse

Installation of the shot system in the deck
The installation was made by parts. It was programmed for one week of duration but by postponement of the test it was counted on extra time.
A first layout was made in the floor slab, in the edge of the decks 3 Nº and Nº 4, of the access bridge, marking the main axle and the location of the two main plates. Concluding this, it was placed all the shot system on the floor slab according to the layout, it means the two main plates the shot elements and the fuse in the center, at the edge of the two sections. For this, it was asked for a cargo crane for transporting the main plates of around 200 kgs., weight each one. Located all the system, it was come to mark the points of perforation on the floor slab. 32 perforations would be made. It was observed that everything were well located according to the plan and proceed to remove the shot system for perforating the floor slab. Made the previous thing the, it was requested to the port for positioning two scaffolds in the two edges of the two decks. These ones were placed under the board floor slab and their main function were help in the perforation and later in the positioning of the inferior plates, nuts and the passage of boltswich would be tightened over the floor slab.

When the main plate was designed in office, the perforations could not be done anywhere since they could hit the floor slab's ironwork of the wharf. Reason why the floor slab was inspected, to locate the ironwork. It was roughened in order to find the superior mesh and it was founded, but it was supposed that the inferior mesh was also placed to the same distance. In land when one was in the installation, it was discovered that it was not true. It was hit with ironwork, in several perforations. Reason why it had to cut the iron to follow with the same location of all the system. This problem delayed all the perforation. Another additional thing that was solved in land was that perforations would not be straight in the vertical way, reason why the holes of the inferior plates were enlarged and put bolts so that the head of the bolt fixed well. This solution helped enough to not waste time installation.

Finished the previous thing, the shot system was located again and bolts were tighten definitively. The final tighten was made in the day of the test, previous general inspection. The required time in all this process was smaller than the real one, since the test had to be postponed several times, reason why there was time to solve each problem. The scaffolds were dissambled in order to not interfere during the test.
For the day of the test the hydraulic cylinder was put in the shot system , wich was under technician support. Also, he realized tests in laboratory in Santiago, where he assured about the well functioning of pressure hoses and obtained the maximum force that the cylinder could give during the test, this value was 52 ton. The pressure hoses had 10 meters length approximately. This was due to protection rules were taken. Since it could happen that during the test the shot elements, of around 4 mts. of length, they could stand up or parts of the fuse were cut suddenly and then explote. A remote possibility but not impossible to happen. Reason why the technician, the nearest person to the shot system had to protect himself. This was made with hoses extension and protection with sand bags, in the place wich he would be located.

Fig 9 Final tighten of nuts to the floor slab

Fig. 10 Location of the shot system

Installation of the measurement equipment

All the installation was made during the test day, beginning during the morning with the instruments for the microvibrations measurement and continue at noon with the instrumentation of the Pull-back tests. Something that should be considered in that day, should that it required favorable weather conditions (without rain or excessive humidity, without excessive winds or dust) and also realize the test all the day.
The measurements were in charge of Professor Rubén Boroschek who offered his services for the test. Next to him Pedro Soto, Engineer in charge of all the computer system, and a group of students who would be for help in the complete installation of the equipment in the sections 3 Nº and Nº 4 of the Bridge Access.

Instruments installation for microvibrations measuring

For microvibrations measuring 4 speed sensors were settled (seismometers), Kinemetrics brand, Model Ranger SS-1 as a whole with a system of data acquisition IOTECH brand, Model Daqbook 200. This system registered the history of speed of each sensor in parallel form (common time) and stored them, in digital form, for its later processing. The signals processing was made using a temporary programm and spectral analysis. The speed sensors anchored to the superior part of the board and they were connected at the same common time, placed throughout the deck Nº 4.

Installation of Pull - Back measurement instruments

For Pull-back measurement there were settled inertial and relative instrumentation. The inertial instrumentation consisted on accelerometers of two types: FBA-11 Kinemetrics and Kistler. Accelerometers FBA-11 were connected to a registry unit SSR -1 and located in deck Nº 4. The located Kistler accelerometers between decks 3 Nº and Nº 4 were connected to a registry unit, National Instruments, the accelerometers located in deck Nº 4 near deck Nº 5 were connected to a data system acquisition brand IOTECH, Model Daqbook 200.
For the relative measurements potentiometers were placed at the edges of deck 3 Nº and Nº 4 and decks 4 Nº and Nº 5 were used, wich measured relative displacements.

In all the measurements were only checked the horizontal directions (parallel to the floor slab). In the following graph are the positions of the sensors for the Pull - Back tests:

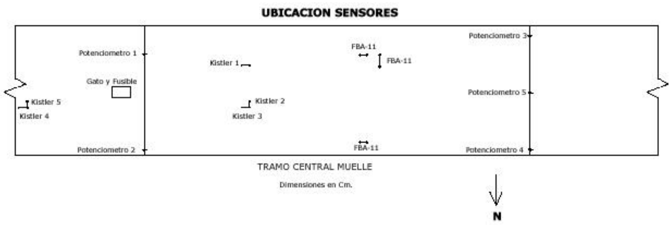


Fig. 11 Location of sensors in deck N°4

Fig. 12 Location of seismometers

Analysis of all connections type between deck Nº 3 and Nº 4.
There were identified couples, joints, and separations stockage at the edge of these decks. These were the following:

- Couples of refrigeration tubes
- Couples of asphalt pipes
- Couples of expansion of the conveyor belts support
- Couples of floor slabs expansion

Test Realization
The test was made the day Friday, December 26, 2002. Arriving to the wharf at 10:30 a.m. and the measurement equipment arrived at the 11:00 a.m.. It was a sunny day and there was mist. Also, there were no operation works in the port. Began immediately with the instruments installation and finished preparing the shot system. At 13:00 p.m. began with the microvibrations and during one hour the passage of workers by the section was cancelled. The acceleration registries recorded the waves of the sea that the wharf felt periodically, being confirmed in the computers monitors. Finished the measurements in the section, the installation for the measurements of the Pull-back was completed.
Before continue, measurements of vibrations with the abrupt braking of a current vehicle were made, this was outside programm. In a distance of 40 mts., a vehicle would arrive at 45 Km/hr. of speed and it would suddenly brake . Several tests with the vehicle became, but when registering itself the waves, these hardly were a little superior to the waves of sea.
At 16:00 p.m., began with the Pull-back. First, were proved the two fuses common steel made. These ones were not be tried before. Their breakage was produced in 50 % more than the designed one and also had deformation of one centimeter, approximately. Results can be observed in the table Nº 3. With this shot system was verified that it was working well.

Table N°3 Results of the fuses test A37-24 (Common Steel)

Results of the test to traction of the current steel fuses and its deformation			
Fuse	Load of teorical breakage	Load of study breakage	Deformation
	Ton.	Ton.	cm
P1	10	15	1.0
P2	10	17	0.8

Deformations wich were expected of these two first tests, were less. 3 mm of deformation than deck Nº 4 were expected, but it only get 1 mm.,eformation, obtained this value from instant measurements by a located digital instrument at the edge of the deck. This result began to show that deck was working in a different way than designed one.

Testing with high resistance steel fuses. The positioning of the fuses was very simple. The pins were removed to test fuses and replace for a new one. Later the pins were located and they were tightened with rubber ring, to avoid the displacement of these ones. When trying first, this one had an excellent value of breakage, only one ton. Also, its deformation was of 0.4 cm. The breakage value of the tested fuse and the other ones was very good since they were near the values of design. In the following table Nº 4 the results are shown:

Table Nº 4 Results of the Thyssen fuses test

Results of test to the high resístance steel fuses tractión			
Fuse	Charge of teorical breakage	Charge of test breakage	Deformation
	Ton.	Ton.	cm.
P3	21	22	0.4
P4	33	29	0.6
P5*	32	29	0.2
P6	42	39	0.5
P7*	42	39	

*Note: Modified Fuses

Every each test was verified that definitively the model of the section was different, to the others realized. The deformations did not arrive beyond one millimeter, for the fuse with bigger breakage charge. Began thinking that the rigidity of the section was bigger. But checking each test was discovered that the fifth deck, that assumed unremovable, due to the independence of the sections, also oscillated with the others. This produced the doubt of the correct measurement of the displacement measures, that were supported in the fifth section. Also, it was not possible to place instruments to measure the fifth deck, since the discovery was almost at the end of the tests. Finished all the tests, it remained with the restlessness that it seemed they had moved more than two deck of the Bridge Access.

Analysis of the results

Analysis of the displacement registers

The previous graph corresponds to the displacement register for the fuse of 39 ton., breakage with a total displacement registered of 1,75 mm.

The analysis of the displacement registers indicates that exists an entailment between decks 3 Nº and Nº 4 of the Bridge Access and possibly exists with other decks. The Table Nº 4 shows the displacements registered between decks Nº4 and Nº 5. The maximum value corresponds to the displacement before the fuse breaking, the minimum to the first maximum observed in vibration, D is the sum of previous absolute values and D/2 represents the average value of both.

It is convenient to use the medium value as reference since the wave action causes displacements that are important, in terms of the observed value of the study. Deck Nº 5 is assumed as fixed point, but actually apparently it showed displacements, which can produce errors in the value of real displacement. Nevertheless the damping is determined of the accelerometers of reliable way.

Fig. 13 Registers of the fuse displacement P 7

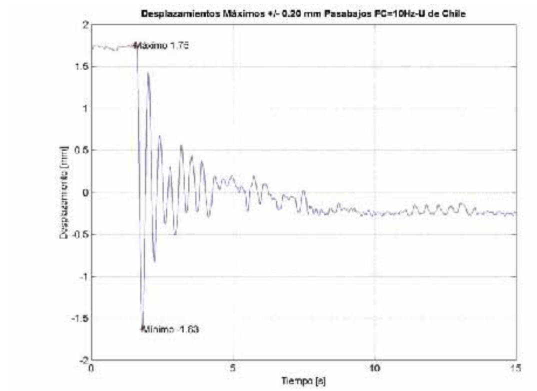


Fig. 14 Registers of the fuse displacement P1

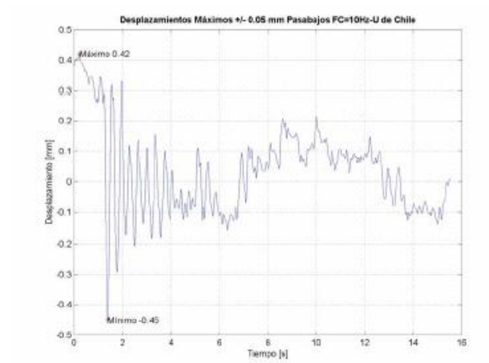


Table Nº 5 Result of relative displacements between section 4 Nº and Nº 5

Relative displacements in the connection between deck Nº 4 and Nº 5				
Real Load	Maximum	Mínimum	D	D/2
Ton.	mm.	mm.	mm.	mm.
15	0.42	-0.45	0.87	0.44
17	0.60	-0.53	1.13	0.57
22	0.77	-0.74	1.51	0.76
29	1.30	-1.01	2.31	1.16
29	1.04	-1.42	2.46	1.23
39	2.06	-1.11	3.17	1.58
39	1.75	-1.63	3.38	1.69

From the table Nº 5 it is observed that the maximum displacement registered is near 1,7 mm, smaller than the hoped one, indicating clearly bigger rigidity to the predicted one. The entailment between decks implies that the wharf, between the decks 3 Nº and Nº 4 moves at least longitudinally altogether. This can happen by the presence of elements, so far not identified, but, of enough rigidity and resistance for controlling the movement between the wharf decks.

Analyses of the observed frequencies and dampings

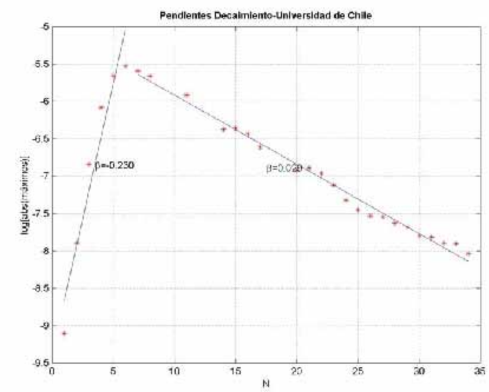


Fig.15 Obtaining of damping in base to the registries

Each acceleration registry was processed to determine the critical damping and predominant frequencies of the structure. The previous graph is the result of that proces. From the analysis can be also observed two predominant frequencies, these are the following:

Table N° 6 Predominant frequencies and observed dampings.

Predominat frequencies and noticed dampings				
Real Load	Frecuency 1	Damping	Frecuency 2	Damping
Ton.	Hz.	%	Hz.	%
Environmental	2.77 – 2.81	*	1.66	3 % +/-0.5 %
15	2.857	**	1.633	**
17	2.796	4,3	1.664	2,8
22	2.765	4,3	1.613	2,3
29	2.722	4,3	1.633	2,6
29	2.754	4,5	1.635	3,1
39	2.686	4,3	1.602	2,6
39	2.634	4,4	1.598	2,9

* Frecuency of low answer in excitation, environmental does not show damping
** Not identied in trust way

The associated longitudinal period to the joint movement of wharf segments is 0,61 seconds (1,6 Hertz). This movement can be assumed because the involved sections oscillate in the same direction, (the phase among them is zero degrees). The number of segments that participate in the movement with this period has not being determined.

The damping for this vibration for is located in a band from 2.3% to 3.1% with a 2,7% average. The variation band is small and it does not show a tendency with respect to the magnitude of the load and therefore is convenient to stablish only one value for the system.

Decline observed in the tests indicates that dispels energy for this way can be associated to a viscoelastic system. The associated longitudinal period to the relative movement between wharf segments varies between 0,36 and 0,38 seconds (2.6? 2,8 Hertz). This movement can be assumed because the involved sections oscillate in opposite direction but it corresponds to the same way, (the phase among them is 180 degrees). The number of segments that participate in the movement with this period has not been determined.

The damping for this vibration form is located in a band from 4.3% to 4.5% with a mean of 4,4% . The variation band is very small and it does not show a tendency with respect to the magnitude of the load and therefore is convenient to stablish only value for the system. Under this same frequency exist a change of state in dissipation which corresponds to lower amplitudes with a reason of critical damping equivalent from 2.0% to 2,5%.

Analysis of the acceleration registries

Can be observed in the following spectograms, the conduct that decks had before and after the tests. The red color marks the predominant frequency. It is observed two for after some seconds to stay with only one frequency, the natural frequency to vibrate of the decks.

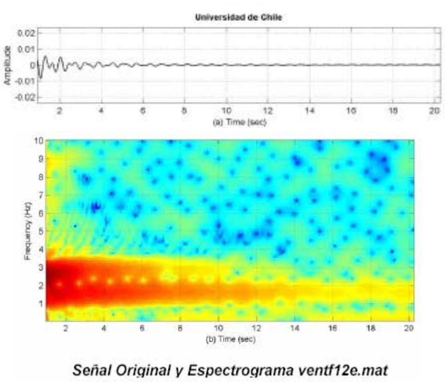


Fig. 16 Registry of the Fuse acceleration P 6

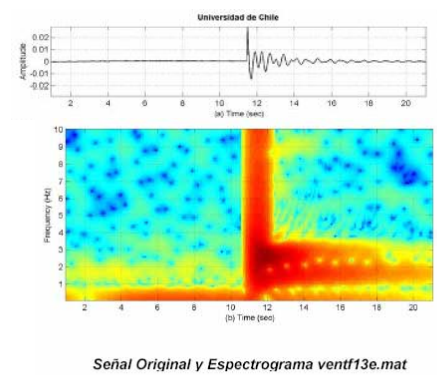


Fig. 17 Registry of the Fuse acceleration P 7

Analysis of the acceleration registries

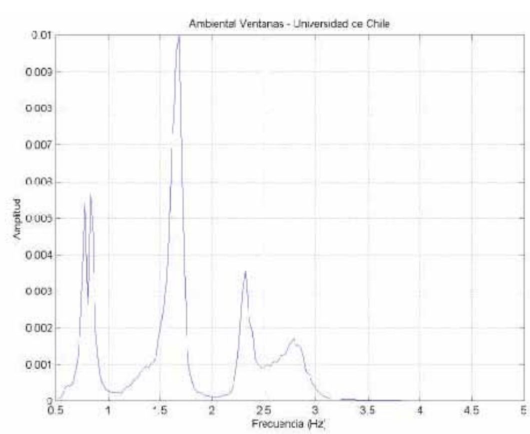


Fig 18 Power graph of the microvibrations registries

Effect of the swell in the registries

Is important to not that the studies of microvibrations and Pull- Back with low loads were not able to differentiate in important form of the movement of the board due to the swell. In order to reduce the effect of the swell the signal filtered under a band of 1Hz. This can be observed in the shift register of figure 13 and 14. If figure 13 is observed, in this registry decline is not distortioned , but continuous, then suffer this. While figure 14 is observed, in the shift register there is an immediate distortion, due to the swell.

Final analysis of the structure

Analyzing the real behavior which the sections had during the test, it is possible to be inferred that its modeling is much more complex. The sections 3 Nº and Nº 4 of the Bridge Access had a behavior together and it is possible to be supposed that also the sections participated Nº 1, Nº2 and N º 5. Therefore the sections are not independent between each other, but they act all together joined by elements that transfer forces to the other sections. In order to model the joined sections, it would be necessary to add the expansion joints of the refrigeration tubes, that they would have to be modeled by springs and a damping value, due to this is rubber made. The asphalt joints also would have to be modeled by some type of spring that reflect the behavior of these joints. In addition one is due to identify the elements that transfer the force to the continuous sections.

Final analysis of the observed period

The predominant period of 0.61 seg. corresponds to the symmetrical movement of the section Nº3 and Nº4, it means both sections move longitudinally all together, in the same direction simultaneously. This period differs from the model in Sap 2000 since it were considered as each independent section. This movement is the one that is possibly going away to have in an earthquake, reason why its period in this case would be of 0,61 seg.

The second predominant period of 0.35-0.38 seg. It is an internal case of the Bridge Access, is like applying an internal force. Reason why it does not correspond to one of the ways to vibrate that it is going to have the structure all together and will not be taken into account for the analysis.

Final analysis of the obtained damping

The obtained damping from the longitudinal movement of the sections correspons has an average of 2,7%. Now it must consider that the structure behaved together with all the sections. Since the sections are identical structurally, have the same facilities, have equal antiquity of construction, it can be said that each independent section has a damping of 2.7%.

Conclusions

The obtained damping is of an average of 2,7%, that varied between 2, 6% and 3.1%. Reason why it is recommended to use for the seismic design structures of similar transparent wharves type with steel piles, a damping not bigger than 3%. The damping value obtained from the microvibrations registries correspondent to 3 %, is similar to the obtained one with the Pull-Back test, reason why the measurement of microvibrations has been validated as a good method of test to obtain dampings in other wharves transparent type.

The sections of the access bridge are not independent, but there exists elements wich link them and make working in shared in common way.

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