Monitoring System for Bridges

Final Report

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Krabbenhøft + Ingolfsson ApS
Efla

Vegagerdin

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1 Samantekt

Rannsóknaverkefni sem bar heitið Ástandsvöktun brúa var unnið 2012-2013 og styrkt af rannsóknasjóði Vegagerðarinnar. Í því voru 2 brýr skoðaðar, hengibrú á Ölfusá og bogabrú á Mjóafjörð í Ísafjarðardjúpi. Gerð var greining á hegðun Ölfusárbrú og sveiflufræðilegir eiginleikar hennar ákvarðaðir með mælingum sem notaðir voru til að kvarða reiknilíkön af brúnni. Einnig var skilgreind vöktunaráætlun fyrir brúna. sem gæti orðið fyrirmynd fyrir aðrar mikilvægar brýr á þjóðvegakerfinu. Fyrir bogabrú í Mjóafirði voru sveiflufræðilegir eiginleikar brúarinnar ákvarðaðir og bornir saman við reiknilíkön á hönnunarstigi.

Í lokaskýrslu til Rannsóknasjóðs Vegagerðarinnar er gerð grein fyrir niðurstöðum verkefnisins, í sérstakri greinargerð sem ber heitið: Brú á Ölfusá við Selfoss – Áætlun um vöktun, er skilgreindar kröfur til vöktunarkerfis og hvaða leiðir eru til vöktunar. Þar er gerð grein fyrir öllum þeim þáttum sem hægt er að vakta og var ætlað að vera fyrirmynd af því hvað hægt er að gera. Ekki er þó lagt til að fara í ýtrustu vöktun sem þar er fjallað um heldur einblína á mikilvægustu þættina sem gefa upplýsingar um hrörnun brúarinnar og þannig geta brugðist við í tæka tíð ef eitthvað kemur upp á.

Vöktunaráætlun brúar á Ölfusá

Skoðun á ástandi burðarkapla brúar á Ölfusá á Selfossi sumarið 2011 leiddi í ljós að vísbendingar eru um að kaplarnir hafi skerta burðargetu vegna tæringar auk þess sem eiginþunga- og umferðarálag hefur aukist verulega frá því brúin var tekin í notkun. Var brotöryggi kaplanna metið sem óásættanlegt. Gerðar voru mælingar og álagspróf á brúnni sumarið 2012. Á grundvelli mælinganna voru skilgreindar kröfur til sjálfvirks vöktunarkerfis fyrir Ölfusárbrú og mælt með því slíkt vöktunarkerfi yrði sett upp. Í greinargerð um vöktunarkerfi fyrir Ölfusárbrú voru fyrri rannsóknir á hegðun og ástandi burðarvirkis brúarinnar nýttar til þess að meta þörf á- og skilgreina raunhæfar leiðir til vöktunar. Niðurstöður fyrri athugana bentu til líklegrar tæringar í burðarköplum sem eykur óvissu við mat á brotöryggi og þótti gefa tilefni til vöktunar brúarinnar. Í greinargerðinni eru settir fram tveir valkostir við vöktun Ölfusárbrúar og þeir bornir saman með tilliti til bæði öryggis- og hagkvæmnisjónarmiða. Vöktun í formi sjálfvirks vöktunarkerfis sem gefur upplýsingar um hegðun brúarinnar í rauntíma annars vegar og hins vegar vöktun í formi reglulegra mælinga sem framkvæmdar eru með ákveðnu millibili. Var það metið svo að æskilegt sé að ráðast í uppsetningu vöktunarkerfis og er lausn sem þykir hentug sem grunnkerfi skilgreind í skýrslunni.

Sjálfvirkt vöktunarkerfi fyrir Ölfusárbrú

Aðalmarkmið vöktunarkerfisins eru að greina breytingar í burðarvirki brúarinnar vegna hrörnunar með því að:

- að greina sig brúarinnar vegna hrörnunar kapla í aðalhafi brúarinnar
- að greina breytingar í kapalkröftum brúarinnar með titringsmælingum
- að greina eiginsveifluform brúarinnar og hvernig eða hvort þau breytast með tíma

Aukamarkmið er að geta gefið upplýsingar um öryggi brúarinnar eftir stóran jarðskjálfta. Vöktunarkerfið er hugsað sem viðbót við reglulegar ástandsskoðanir en ekki til að koma í stað þeirra.

Fyrri áfangi verkefnisins eins og því var lýst í verkefnislýsingu er kominn til framkvæmda, þ.e. greina breytingar í kapalkröftum brúarinnar með titringsmælingum. Síðari áfangi er að koma upp mælibúnaði sem mælir hæðarferil brúarinnar sjálfvirkt. Hröðunarnemar voru settir upp á 2 burðarköplum brúarinnar í ágúst 2014 og hefur nú safnað gögnum frá þeim tíma með sítengdu mælakerfi sem aðgengilegt er af vef. Mælibúnaður kemur frá Krabbenhøft+Ingólfsson í Kaupmannahöfn og AMEG í Kaupmannahöfn.

Í greinargerðinni er fjallað um helstu forsendur vöktunarkerfisins og lýsing á mælibúnaði sem notaður er. Mælibúnaðurinn mælir titring kaplanna og eru eigintíðnir kaplanna greindar út frá mælingunum. Með reiknilíkani af burðarköplunum er samband eigintíðni burðarkaplanna og kapalkrafts fundið. Með því að greina breytingar í eigintíðni er hægt að greina breytingar í kapalkröftum brúarinnar til lengri tíma og á þann hátt hægt að meta hvort breytingar eigi sér stað í burðarvirki brúarinnar.

Gerð er grein fyrir nákvæmni mælinganna og hvernig nýta megi þær við vöktun brúarinnar. Þegar meiri reynsla verður komin á langtímamælingar þarf að huga að eftirtöldum atriðum:

1) Mældar eigintíðnir bakstaga byggja á 10-mínútna meðaltalsgildi og getur því umferð haft áhrif á mælda eigintíðni, frávik í mælingum geta því í einhverjum tilvikum stafað af breytingum í umferðarálagi.
2) Reiknilíkan af bakstögum er kvarðað miðað við mælda lóðrétta eigintíðni en ekki láréttrar eigintíðni bakstaganna. Munur er á mældri láréttu eigintíðninni.
3) Munur er á mældri eigintíðni hvors kapalplans og þar með kapalkraftinum. Munurinn er nokkuð stöðugur að undanskildum nokkra vikna mælitímabili í desember. Frekari gögn þarf til að greina þennan mun sem gæti stafað af ósamhverfri svörun brúarinnar vegna hitabreytinga.
4) Áhrif hitabreytinga á kapalkraftinn í reiknilíkaninu eru í nokkuð góðu samræmi við mælingar.
5) Á timabili vantaði gögn vegna straumleysis og rofs á gagnaflutningum, mikilvægt er að setja upp sjálfvirka vöktun á tengingum.

2 Introduction

This report outlines the main results from the research project entitled: *Monitoring System for Bridges* (in Icelandic: *Vöktunarkerfi fyrir brýr*). The project was carried out with funding from the Icelandic Public Road Administration (IRCA) research fund in collaboration between Krabbenhøft + Ingolfsson Engineering Consultancy (K+I) and AMEG in Copenhagen, Denmark and Efla Consulting Engineers in Reykjavik, Iceland. The project participants are listed below:

Einar Thór Ingólfsson, Krabbenhøft + Ingolfsson (Project Manager)

Guðmundur Valur Guðmundsson, Icelandic Road and Coastal Administration

Kristján Uni Óskarsson, EFLA

Baldvin Einarsson, EFLA

Bogi David Magnusson, AMEG

Asger Espersen, AMEG

Aron Bjarnason, Icelandic Road and Coastal Administration

2.1 Background

This research project is an independent continuation of the following previous projects which were carried through in the period from 2012 to 2013:

Ölfusá Bridge – Assessment of Cables (in Icelandic: *Brú á Ölfusá – Mat á ástandi kapla*),

Ölfusá *Bridge – Results from measurements* (in Icelandic: *Brú á Ölfusá – Niðurstöður Mælinga*)

Health Monitoring of Bridges (in Icelandic *Ástandsvöktun brúa*).

These projects were carried out with funding from IRCA Research fund and in collaboration between Efla Consulting Engineers, Technical University of Denmark (DTU Byg) and the University of Iceland (UI) and resulted in numerous publications on the subject, including three MSc theses [1]-[3] and technical reports [4]-[6]. In brief, the projects were centered on general methods of health monitoring, finite element modelling and updating as well as experimental modal identification of two road bridges (Ölfusa Bridge and Mjoifjördur Bridge) owned and operated by IRCA. Ölfusá Bridge is a suspension bridge built in 1945 and it desirable to install a monitoring system onto the bridge which is capable of determining potential long-term changes in the structural behavior of the bridge. In particular the distribution of forces in the cables is interesting as this is an influential parameter in the long-term reliability of the bridge. A potential change (e.g. increase) in the cable force may have an impact on the life-time of the structure.

2.2 Scope

The purpose of the project is to develop a robust and simple monitoring system for health monitoring of the Ölfusá road bridge at Selfoss, Iceland, with main emphasis on monitoring the long-term variation in the cable force in the main suspension cables of the bridge. A simple method based on the relationship between the natural frequency of the back stay of the cables and the cable force was developed. The method relies on an accurate Finite Element model of the bridge and a robust and automatic identification of the natural frequencies of the cables. For the purpose, a bi-axial accelerometer was developed and installed on the bridge for a continuous evaluation of the natural frequency of the bridge's back-stay and through back-calculation of the measured natural frequency, the force can be estimated.

3 The monitoring equipment

The monitoring system consists of two separate wireless accelerometers which are attached to the back-stay cables through fabricated clamps. The accelerometers transfer data to a common server through 2G GPRS mobile connection. The post-treatment of the data is carried out to extract the natural frequencies and other relevant quantities. The results are collected and visualized on a tailor-made web-interface.

The sensors are Dual-axis accelerometers (range ± 2 g) with sensitivity 0.6 mg ± 5% and noise 50 µg / √Hz @ 1 Hz. The sampling rate is fs = 50 Hz and the linear range is between 0 – 10 Hz.

The sensors are enclosed in IP67 rated enclosure and are supplied with 85 – 264 VAC electricity at 47 – 63 Hz. Both sensors are equipped with a SIM card for data transfer:

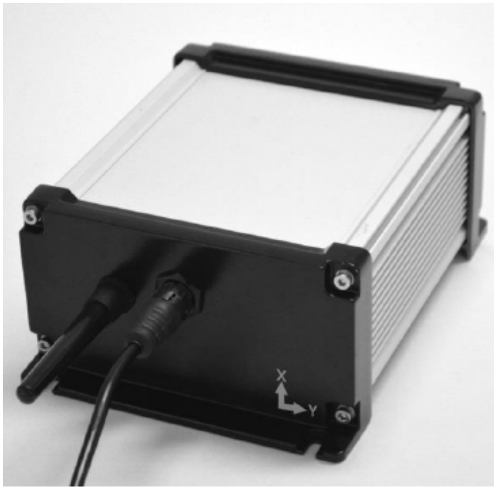


Figure 1: AMEG dual-axis accelerometer

Sensor #20

Sim Card No.: 89354030130601124017

Pin: 3172

PUK 05948003

Sensor #21

PIN: 2538

PUK: 05317312

Sim Card no.: 89354010130601124025

The data is accessed at the project server: vegagerdin.ameg.dk

3.1 Installation

The accelerometers were mounted onto steel brackets (plates) and installed onto the bridge on 22 July 2014. The steel brackets were mounted on the back stay of the suspension cable. Both sensors were located on the southern side of the bridge (Selfoss side) and one on each side of the road, denoted South-East (upstream) and South-West (downstream). They are positioned slightly below the bridge deck, to ensure that they are not visible for the users of the bridge.

The electricity is supplied from a technical room situated inside the bridge abutment.

It is worth noting that the suspension cable (and hence also the back stay) consists of the six individual locked-coil strands, but the accelerometers is attached to all six strands and thereby only captures vibrations of the combined mode shape for all six strands and not those of the individual strands. As such, only the total force in the back-stay can be measured using this particular setup (reference Figure 2).

Figure 2: Overview of sensor locations on the bridge.

4 Raw data processing

The cable vibrations are recorded continuously and the results are stored as 10 minute acceleration time series measured in three directions. However, only results for the two perpendicular cable directions are stored (X- and Y-directions respectively). The data is stored on a server and displayed on-line (vegagerdin.ameg.dk).

4.1 Single data file analysis

A single data file contains 10 minutes of data for the two directions (Horizontal and Vertical). With reference to the sensor axes (Figure 1) and the bridge geometry (Figure 2), the horizontal direction is defined as the horizontal transverse vibration of the cables (sensor Y-axis) and the vertical direction is defined as the vertical (inclined) vibration of the cables (sensor X-axis).

An example of an acceleration time history and its corresponding single sided Auto Spectral Density (ASD) is shown in Figure 4 and Figure 5 for the two directions respectively. The ASD shows how the energy of the vibration signal is distributed across the frequency range. This graph is used to identify the natural frequencies of the cable. The lowest natural frequency is in the range 2.5 -5.0 Hz and therefore Figure 3 shows the ASD particularly in that frequency range. It is noted that the horizontal black line represents the sensor noise floor and therefore data below and around this line is not reliable.

For the vertical vibrations, only one dominant mode appears around 3.7 Hz, whereas the horizontal vibrations feature a number of distinct peaks in the frequency range. The lowest frequencies are in the range 3.2 – 3.3 Hz.

These natural frequencies will be used as key indicators related to the condition of the cables. The natural frequencies are strongly related to tension force in the back stays and thereby the temporal variations of the frequencies indicate a proportional variation in the cable force. In the following sections, the relationship between the natural frequencies and the cable forces will be established.

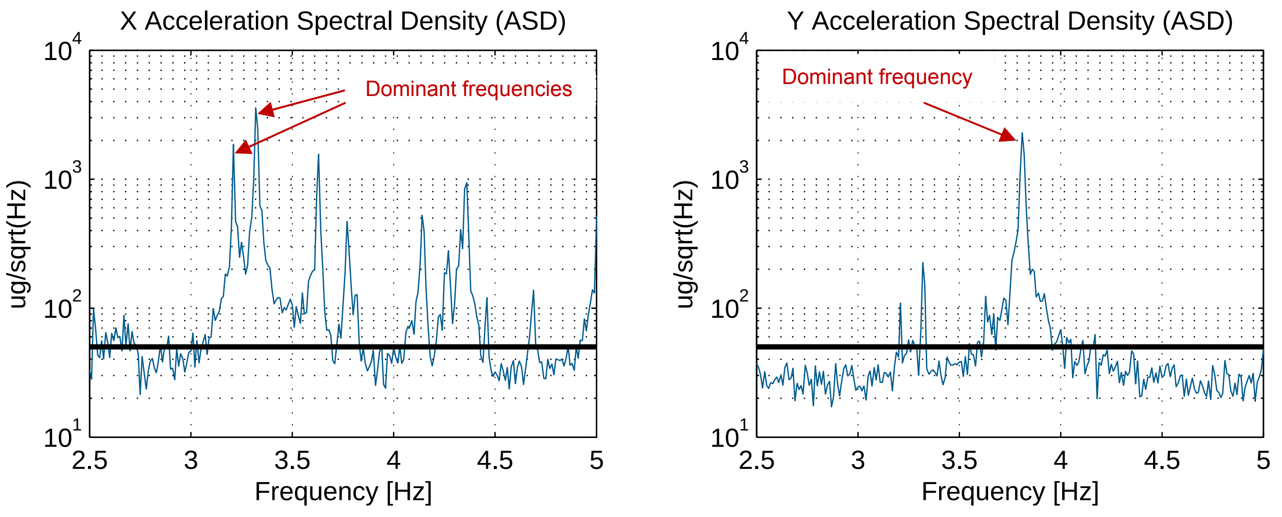


Figure 3: Auto Spectral Density from a 10 minute time series shown in the range 2.5 – 5.0 Hz.

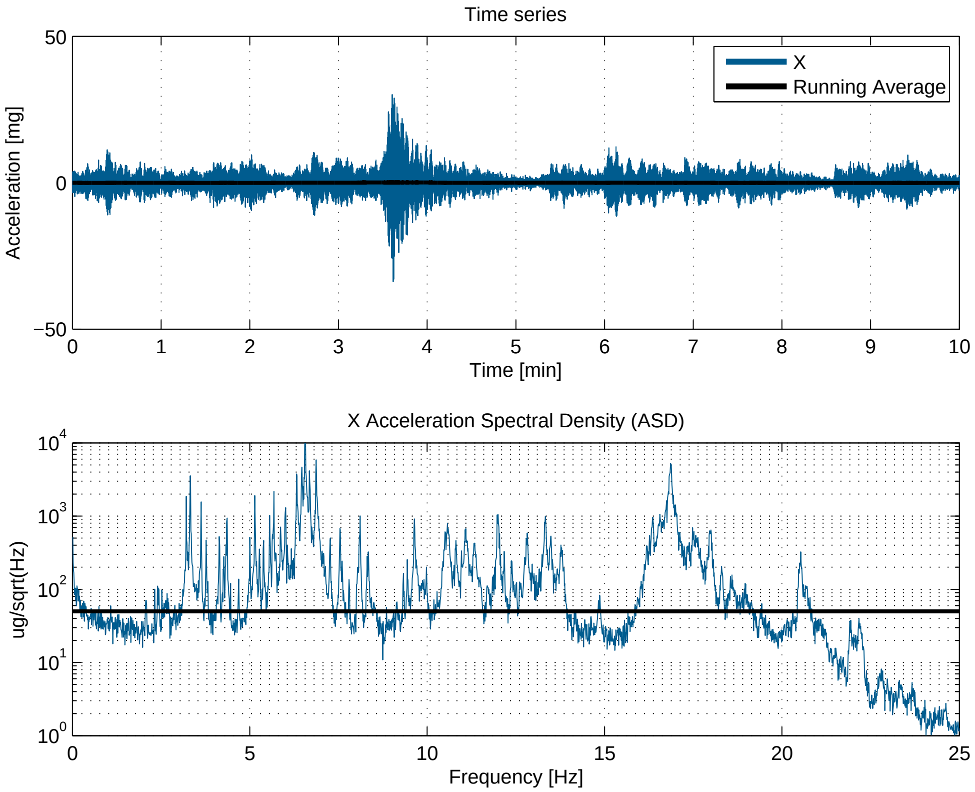


Figure 4: Example of 10 minute time series and ASD from accelerometer no. 20 – X-direction.

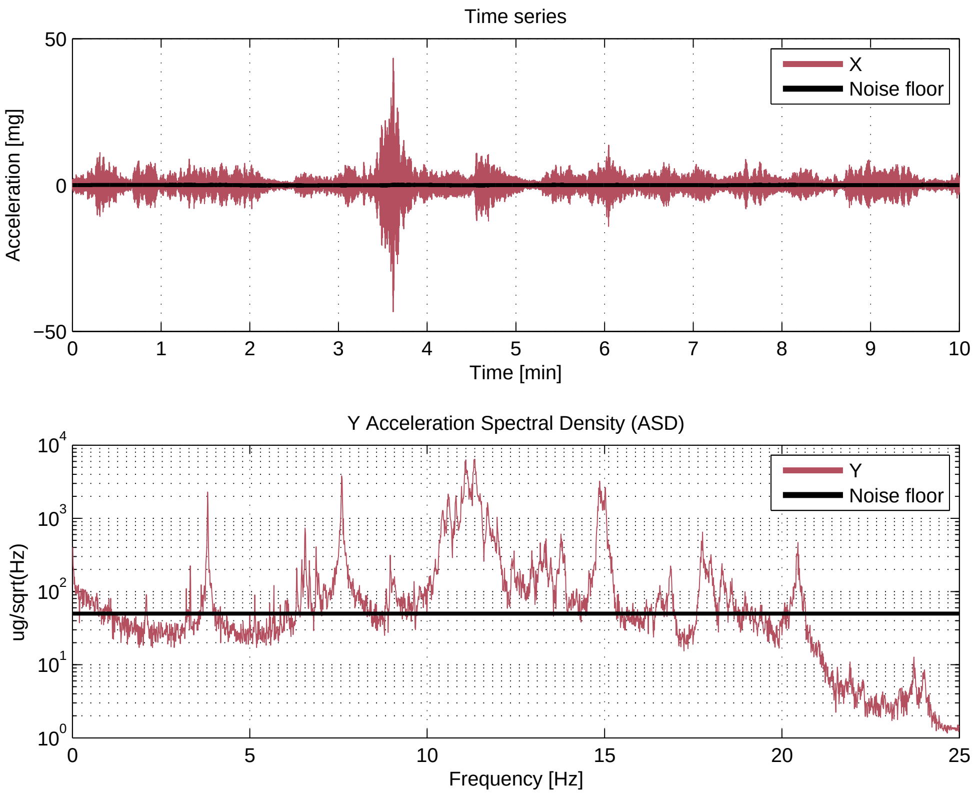


Figure 5: Example of 10 minute time series and ASD from accelerometer no. 20 – Y-direction.

4.2 Automatic detection of natural frequencies

All data from the two accelerometer boxes are saved on the AMEG server and subject to post-processing. Each data package contains a 10-minute acceleration time history as described above. For each data package, the measured acceleration signal is subject to Fourier transformation and the spectral density is used to identify the fundamental natural frequency of vibration in the two directions (X and Y).

It is assumed that the spectral peak in the bandwidth 2 – 5 Hz represents the fundamental vibration frequencies for horizontal and vertical vibrations of the cable, respectively. As such, the frequencies are identified at these distinct peaks. For each time series, the natural frequencies are identified and thereby the development of the natural frequency over time can be monitored.

In Figure 6, the extracted natural frequencies for the horizontal mode and the vertical mode are shown for a single time 10 minute event (or data package). The ASD as well as the extracted natural frequency is stored for each time history which is analysed.

In Figure 6 and Figure 8, the development of the natural frequencies over the entire monitoring period (i.e. from 22 July, 2014 to 18 October, 2015) is shown. There are few larger gaps in the data, which primarily caused by lack of connection to the sensor and or loss of power. An alert system for detecting lost connection has not been implemented into the system and thereby the lost connection was not identified immediately. Currently both sensors are in operation and a detection and alert system for ensuring a continuous data transfer is currently being developed.

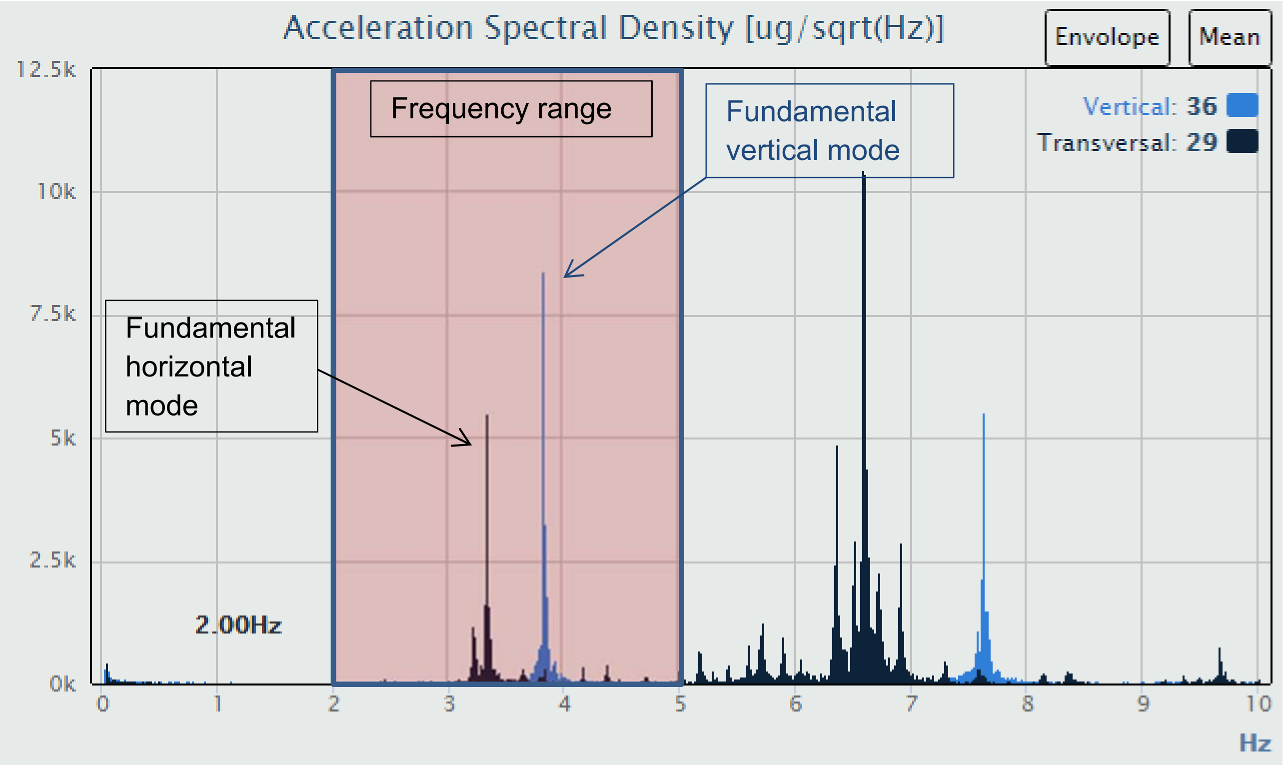


Figure 6: Example of an acceleration spectral density for a 10-minute time series (6. April, 2014 between 00:07 and 00:17, accelerometer #20).

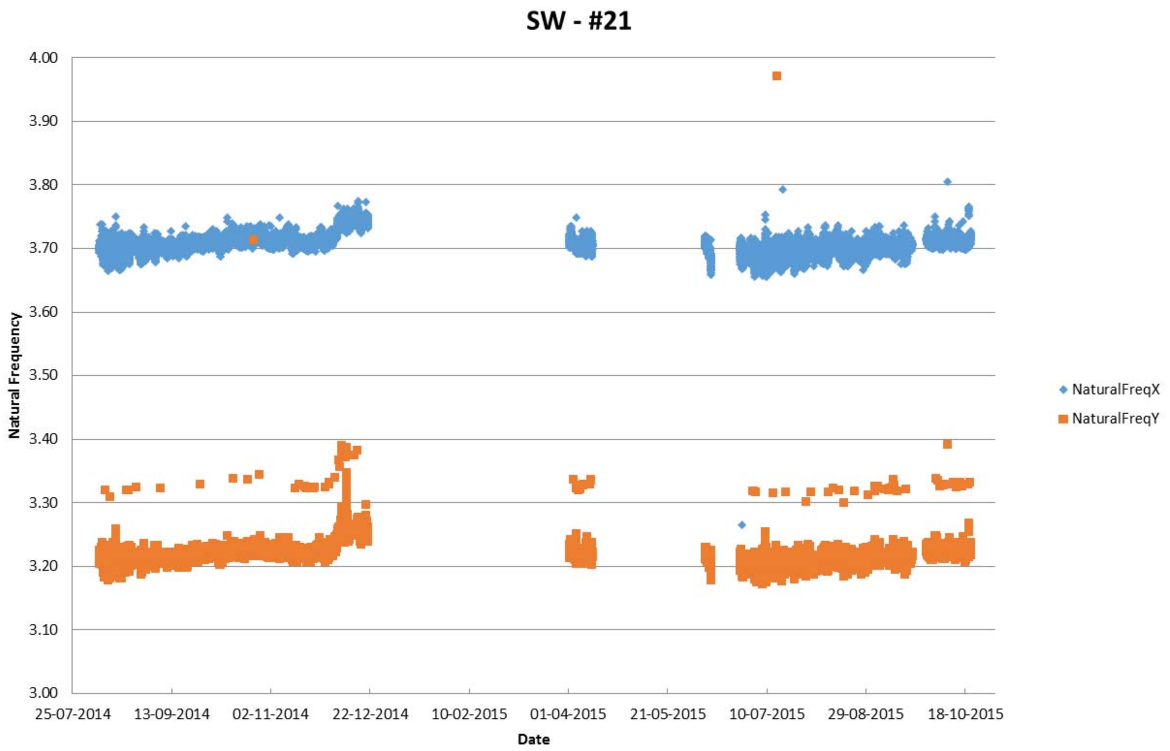


Figure 7: Development of the measured (extracted) natural frequencies over time for accelerometer #21 (South-West) for both X- and Y-direction.

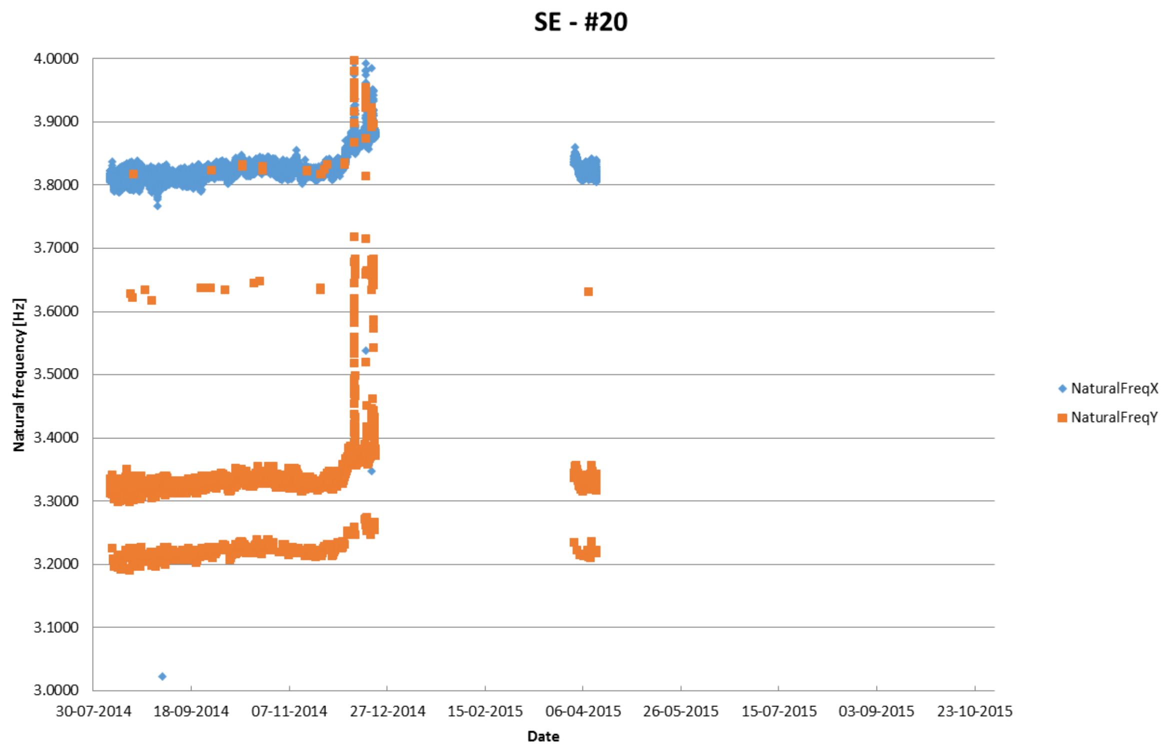


Figure 8: Development of the measured (extracted) natural frequencies over time for accelerometer #20 (South-East) for both X- and Y-direction.

5 Relationship between natural frequencies and cable force

5.1 Local finite element model of backstays

The relationship between the natural frequency of a string under tension and the tension force is well-know from commonly used chord theory [7] and is well applicable to cables and strings with well-defined end conditions. Since the results are strongly dependent on the particular end conditions and cable geometry, it was recognized that the string model would not be applicable to the back-stays of the Ölfusá Bridge. As such, previously reported methods of estimating forces in the backstays of the Ölfusá Suspension bridge [5] have included direct transformation of measured frequencies, extraction of element forces from an updated global finite element model and analysis using a local finite element model of the backstays. The global finite element model of the bridge (as described in ref. [5]) has been updated after modal identification of the bridge deck and is considered to provide an adequate estimate of forces due to vehicle loading while only indirectly describing forces due to self-weight of the structure itself.

A local model, considering only the backstays, but with an accurate representation of their geometry has been created [5] – see Figure 9. The local model of the backstays is considered to reduce the sensitivity of calculated forces to the element length while maintaining the direct coupling to the measured frequencies. The model itself comprises the six strings of the south end backstays, restrained at one end simulating the position at the tower saddle while at the other end distributed into the anchor block. A relatively stiff frame element is employed to simulate the splice connection, separating the strings from the main cable bundle out towards the anchor block. The elements representing the strings are frame elements of equivalent cross section to the ø60 locked coil cables. Estimated weight and elastic modulus of the steel cable section is taken as 82 kN/m^3 and 135.000 N/mm^2 respectively.

To determine the relationship between the natural frequencies and the cable force, various degrees of preloading has been applied to the model and the changes in the dominant horizontal and vertical natural frequencies has been recorded. Reference Figure 11, the modal frequencies of interest are the first horizontal frequency (mode 1) and the first vertical frequency, (mode 3) of the local model.

In Figure 11, the estimated relationship between the natural frequencies and the cable force are shown based on the local FE-model. Using linear regression, an expression for the cable force as function of the measured natural frequency can now be established:

Horizontal mode: $F_{Cable} = 402{,}4 \text{ kN / Hz} \cdot f_{mode\text{-}1} - 672{,}3 \text{ kN}$ (1)

Vertical mode: $F_{Cable} = 376{,}9 \text{ kN / Hz} \cdot f_{mode\text{-}3} - 758{,}2 \text{ kN}$ (2)

If above equations are correct, the derived cable force should be the same whether the measured frequency in mode 1 or mode 3 is used in the equations. This is examined in further details in the next section.

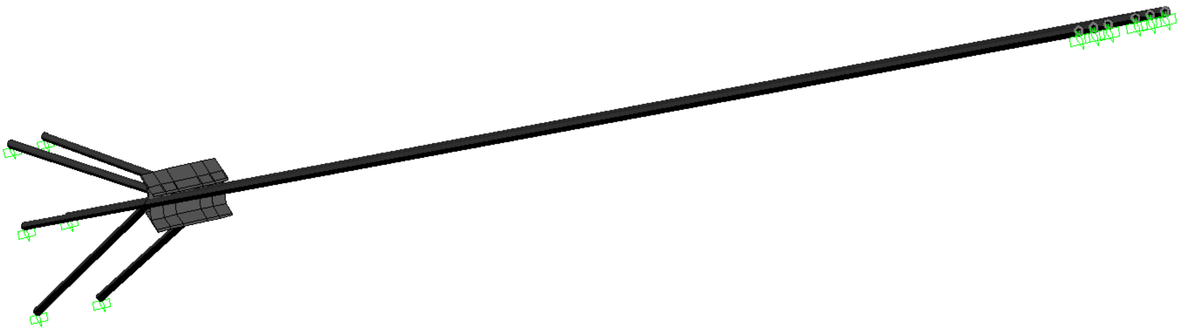
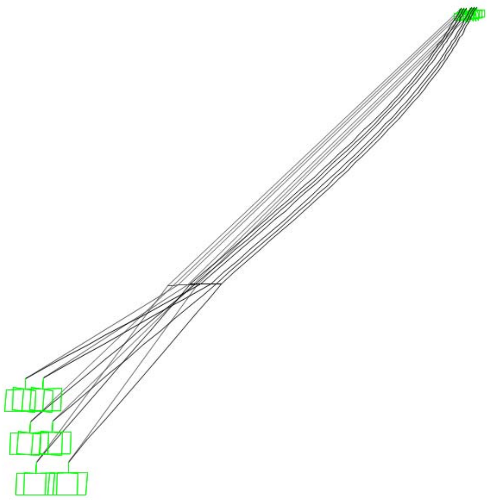
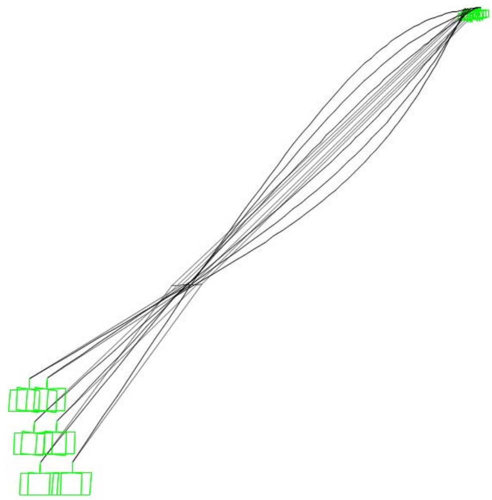


Figure 9: 3D view of the local finite element model of the Southern backstay.

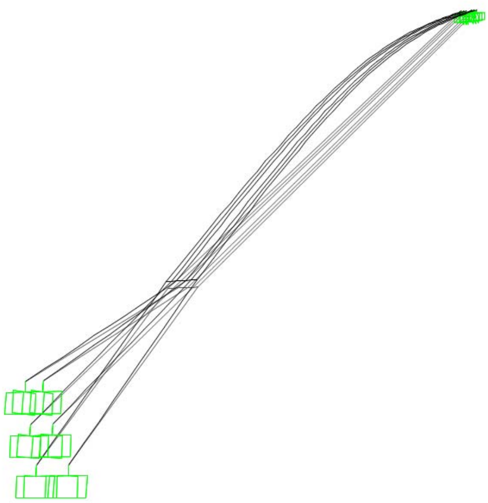
Mode 1: Dominant horizontal



Mode 2: Dominant torsion



Mode 3: Dominant vertical



Mode 4: Asymmetric horizontal

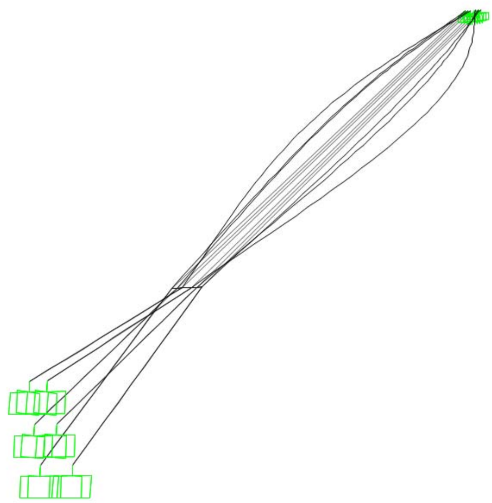


Figure 10: First 4 fundamental vibration modes for the local backstay model

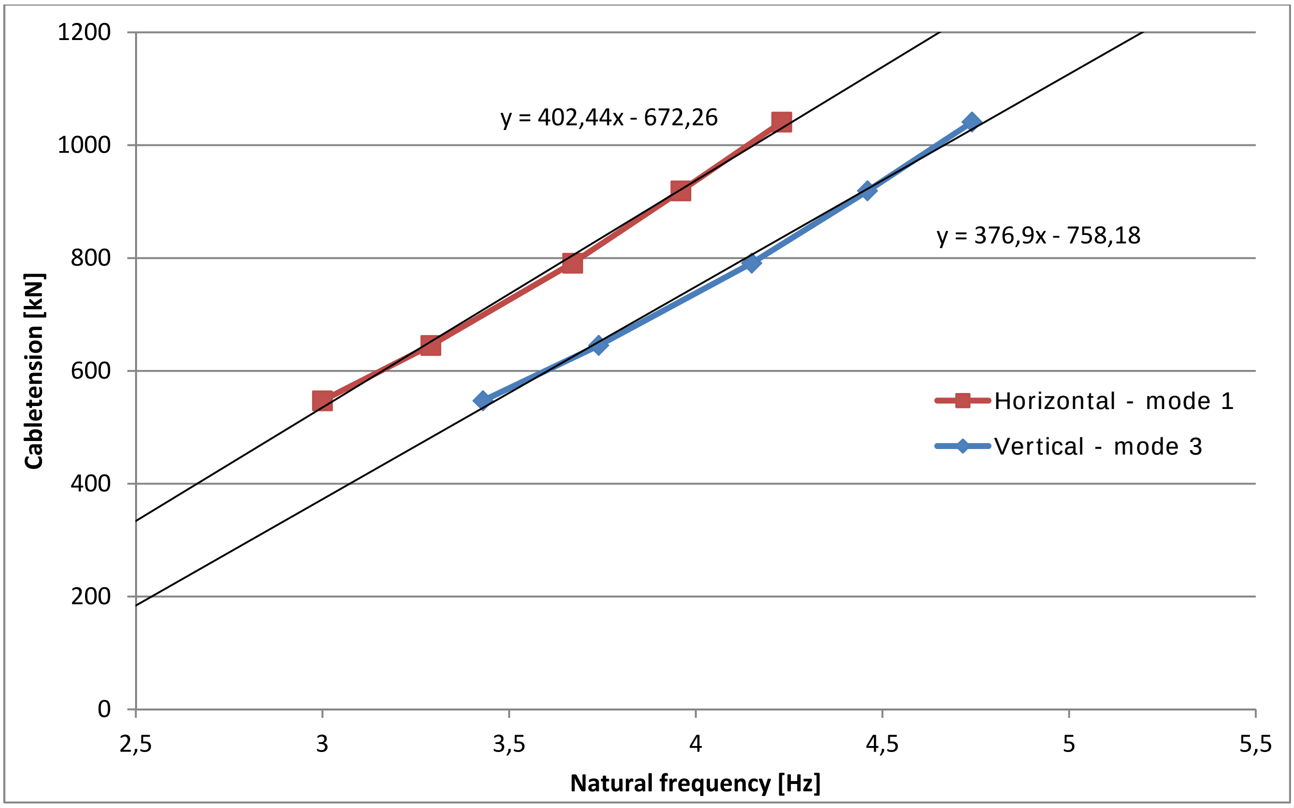
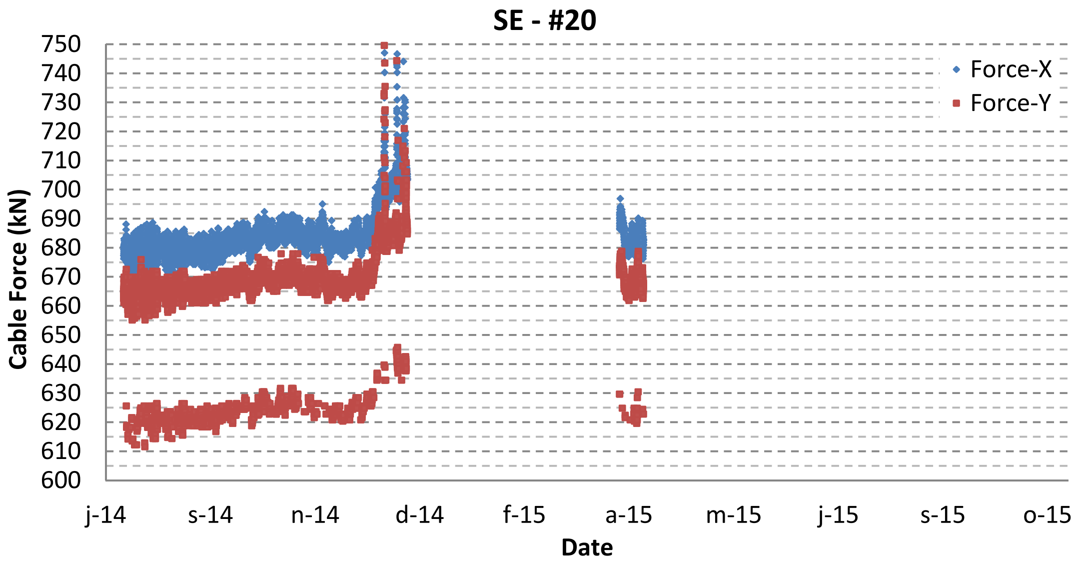


Figure 11: Relationship of cable forces to calculated frequencies in the local backstay model

5.2 Back-calculated cable force

The measured natural frequencies are transformed to cable force using the regression equations (1) and (2). There is some scatter in the data, but there is a consistent trend over time showing an increasing cable tension with time until the end of December 2014. The cable tension decreases again when the measurements are resumed in April 2015. Interestingly, the cable force in the downstream cable plan (SW) is consistently lower than in the upstream cable plan. This is consistent with the FE-model. It is noted that there is a difference between the force as estimated from the X-direction natural frequency or that of the Y-direction. As noted in Figure 3, the Y-direction ASD features two distinct and closely spaced frequency peaks, whereas the X-direction has only one peak within the frequency range of interest. As such, it only the X-direction measurement (i.e. vertical natural frequency) will be used from now on.



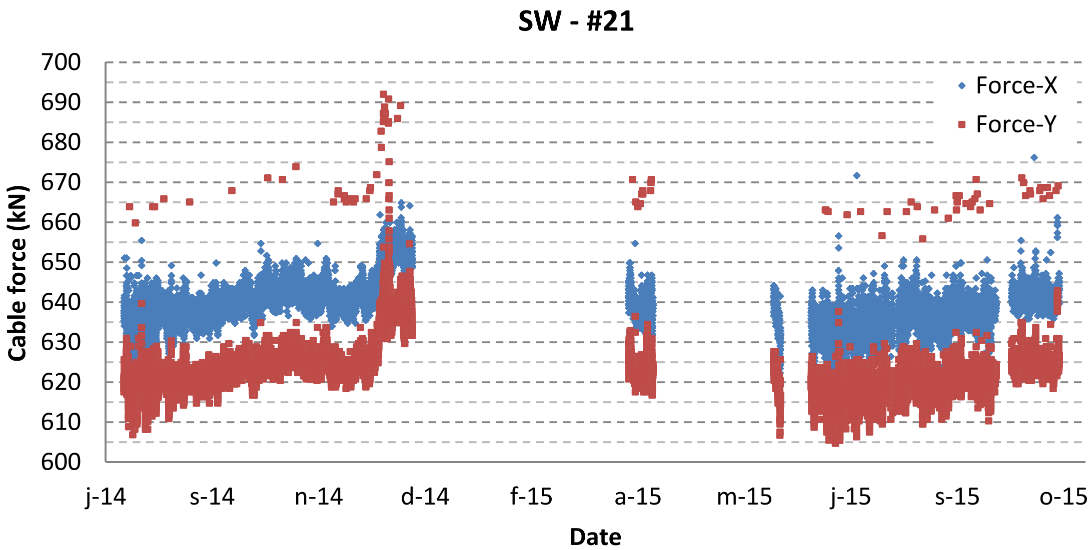
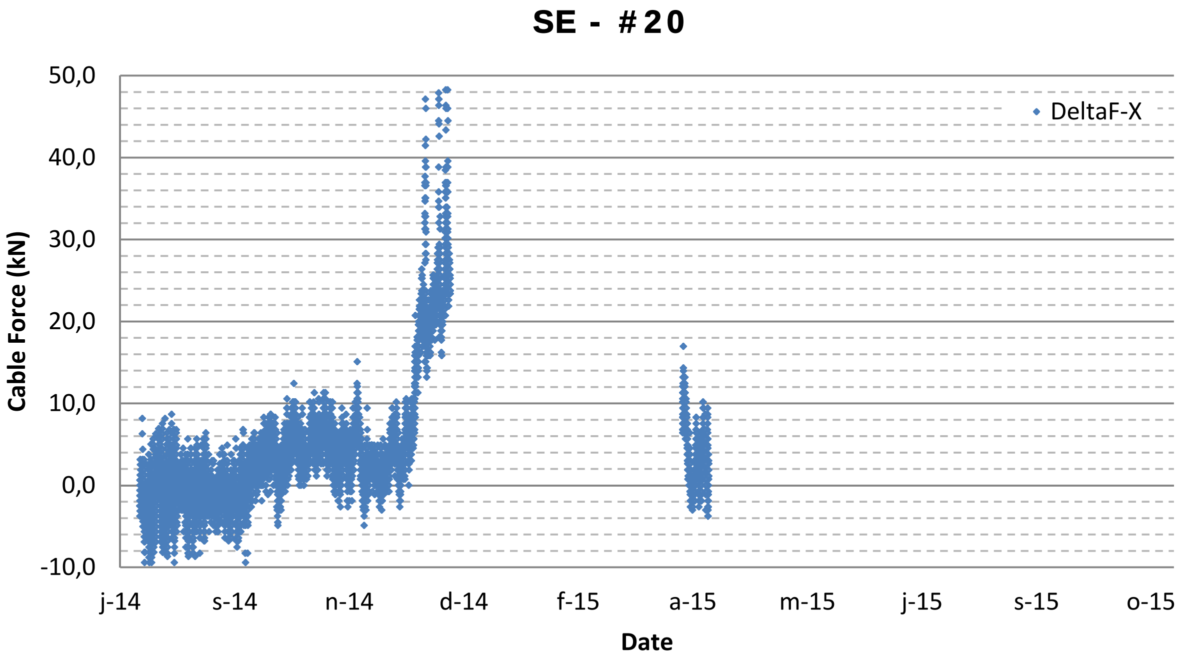


Figure 12: Estimated cable force as function of time from measured natural frequencies.

5.3 Variation in cable force

The variation in cable force is defined as the change in force (as measured from the mode 3 natural frequency) since the first measurement on 07-08-2014 (14:04). Such variation in the cable force may be caused by seasonal temperature changes which will be investigated further in the following.



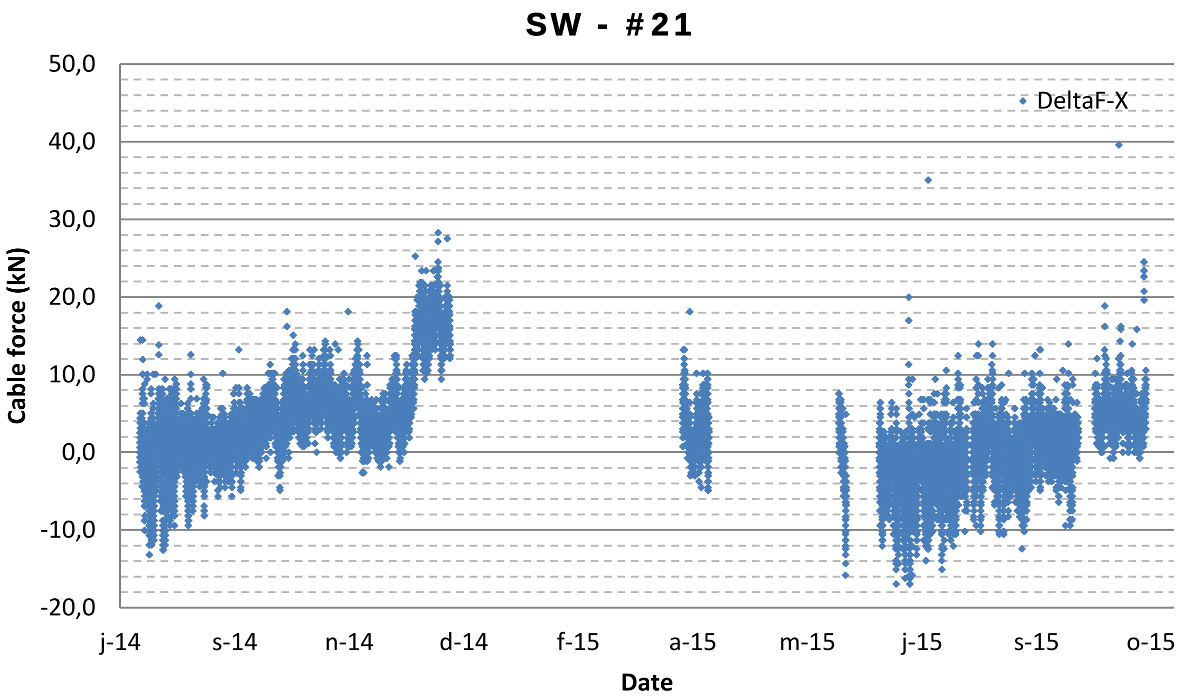


Figure 13: Estimated change in cable force as function of time from measured natural frequencies.

5.4 Variation between cable planes

It was noted that the cable force in the SE cable plane is consistently larger than that of the SW cable plan. Bearing in mind that the bridge is excentrically loaded, a difference in the cable force is expected. The interesting aspect is wheather the relationship between the forces in the two cable planes remains the same of changes over time.

In Figure 14, the relationship between the SW and SE cable plans is shown. In general the cable force relationship is constant, around December 2014, the cable force in the SW cable plan decreases gradually from around 94% to 92.5 % of the corresponding cable force in the SW cable plan.

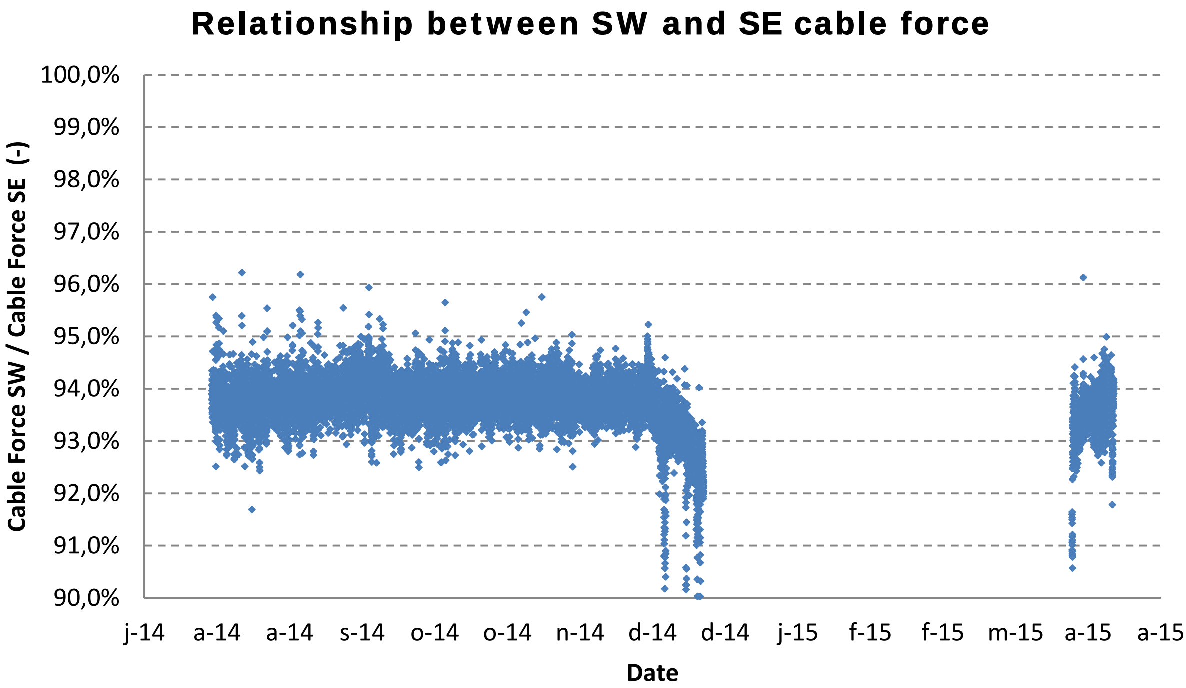


Figure 14: Relationship between the estimated cable forces in the two cable planes as function of time from measured natural frequencies.

5.5 Effect of temperature on cable forces

The effect of ambient temperature change is simulated in the global finite element model of the complete bridge structure, by applying temperature only onto the bridge cables in a 10 deg. C. interval. Analysis of temperature loads on the cable elements is considered sufficient as further assignment of loads on the remaining structure does not influence the results to a large degree.

In Figure 15, the expected linear relationship between the applied temperature and the resulting cable force (as percentage of original tension force) is shown and in Figure 16, a 3D view of the global FE model is shown.

Actual weather data has been collected from the nearby weather station at Ingolfsfjall (approx. 3,5 km from the bridge site), showing the seasonal variation in the ambient air temperature (Figure 17). Using this data together with the expression shown in Figure 15, the predicted change in the cable pretension can be determined – the results are shown in Figure 18Figure 17 together with the measured change in cable force. The actual measured change in cable force (SE and SW cable plans) is overlayed in Figure 18, showing a farily consistent trend, ie. the variation in the cable force as predicted by the FE-model is also obsevered in the measured cable force. Although the measured varations appear smaller than the predicted ones, the data gives no indication that the varation in the measured cable force is caused by anything other than the variations in the ambient temperature.

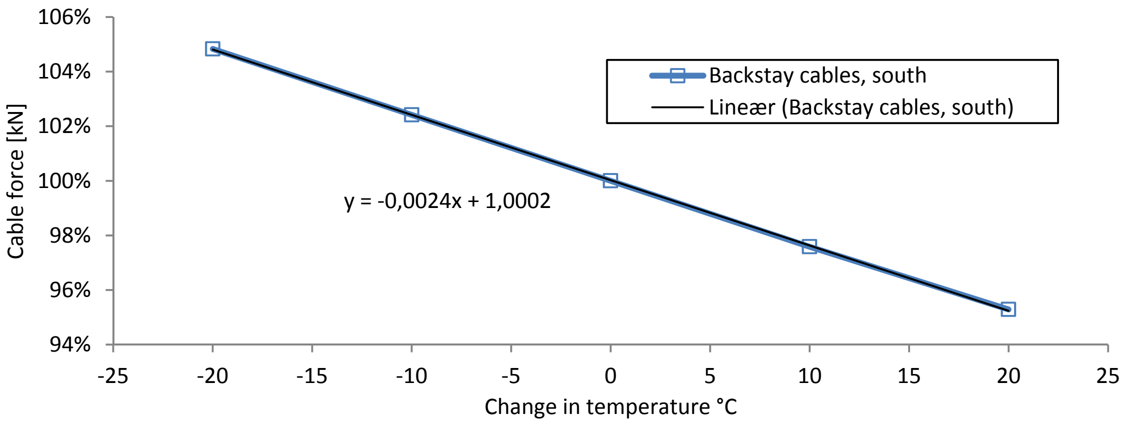


Figure 15: Proportional change in cable force due to temperature change.

Figure 16: Global finite element model of the Ölfusá Suspension bridge.

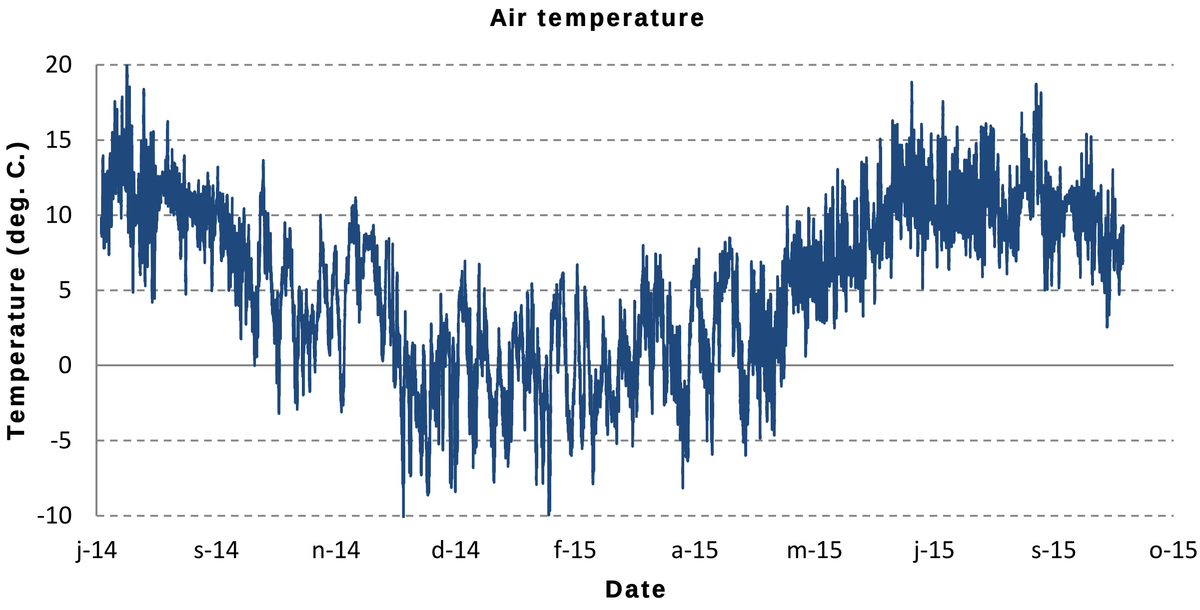


Figure 17: Measured ambient temperature at the Ingolfsfjall whether station nearby Olfusa Bridge.

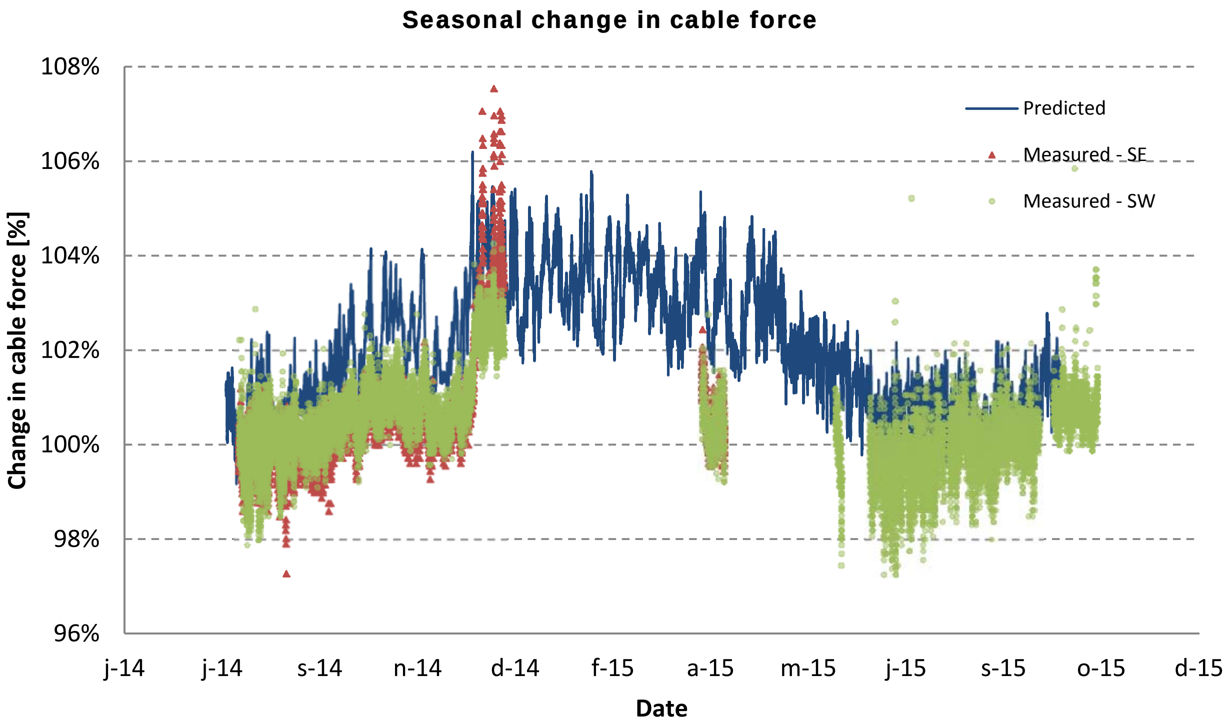


Figure 18: Comparison between the predicted (FE-model) and the measured change in cable force.

6 Discussion and conclusions

In general, the proposed methodology presented herewith appears robust and capable of detecting the variation of the cable force in the back-stays of the bridge. A number of uncertainties and limitations have been identified in connection with the extraction of natural frequencies and the back-calculation from natural frequency to cable force. In particular it is worth noting that:

1) Natural frequencies are estimated from 10-minute data files and thereby represent an average frequency of the cable rather than an instantaneous one. Since the average crossing time of a vehicle is only few seconds and that crossing of vehicles actually generate considerable vibrations in the bridge and the cables, the results will depend on the actual traffic situation. The observed scatter in the measured frequency may, to some extent, be caused by variations in the traffic. However, there is no way to distinguish between based on the current data.

2) The estimated cable force (Figure 12) is based on the local FE-model of the back stay and there is a difference between the predicted cable force using the horizontal natural frequency (y-direction) and the vertical one (x-direction), although they should yield the same results if the FE-model was perfectly correct. As shown in Figure 12, the difference between the two curves is not large and they both follow a similar trend suggesting that the prediction is qualitatively correct. In all further analyses, the vertical frequency has been selected as the indicator of cable force in the two cable plans.

3) As shown in Figure 14, there is a difference between the measured forces in the two cable planes (which is consistent with the global FE-model). This difference is fairly constant, but appears to drop from 94% - 92.5 % over a period of few weeks in December. There is a lack of data to analyse this further, but it is a trend that should be reviewed closely when more data becomes available. A possible explantion is an asymmetric response of the bridge to variation in temperature.

4) In Figure 18, the predicted change in cable force (based on measured temperature at a nearby whether station), obtained from the global FE-model of the bridge, is qualitately consistent with the measured change in the cable forces. Howver, the magnitude of the measured force is generally slightly smaller than the predicted one.

5) During the monitoring period, there are large gaps in the data, mainly caused by lack of power supply to the equipment and/or lack of wireless connection. This suggests, that an automated monitoring of data transfer is needed, to ensure that a warning is issued when data transfer fails.

7 Future work

Based on the work presented herewith, the following further improvements of the system and its predictive capabilities may include:

1) Creation and maintenance of automated monitoring of the data transfer from equipment to server. Set-up of warnings if data transfer is interrupted to ensure continuous data collection.
2) Creation and defintion of automated warning system when large changes in the measurements occur. Definition of suitable threshold values for warnings.
3) Set-uo data transfer directly from AMEG server to IRCA monitoring system.
4) Review of key measurement indicators. Currently 10-minute average is used for extraction of natural frequencies. Other indicators may be more suitable, e.g. one-hour average or 24-hour average.

8 References

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