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## PREFACE

This master's thesis is the final work of my master's degree in structural and material science at the University of Stavanger and marks the ending of 5 consecutive years studying engineering. This thesis aims to investigate the Lysefjord bridge's dynamic response by implementing real-life recorded traffic-induced acceleration of the bridge girder to a finite element model of the bridge. The traffic-induced acceleration data has been provided by the supervisors, and drawings of the bridge used for modeling are sourced from the Norwegian Public Road Administration, with whom this thesis has been developed in collaboration.

The primary focus of this thesis revolves around developing a finite element model in Abaqus suitable for reviewing the dynamic response of the bridge. Additionally, considerable effort has been dedicated to processing the recorded data in a manner that best aligns with the Abaqus model. A study of the eigenfrequencies and eigenmodes has been conducted, followed by an in-depth exploration of damping properties associated with various structural components. The dynamic response of the bridge is examined through discussions of time-based plots, encompassing displacements, torsional response and cable forces.

This research seeks to contribute a deep understanding of the dynamic response of the bridge under traffic-induced accelerations. Furthermore, this thesis lays the foundation for further investigation, utilizing the developed finite element model for additional extensions or comparative studies.

I want to express my sincere appreciation to my supervisors, Jasna Bogunovic Jakobsen and Jónas Þór Snæbjörnsson, for their valuable guidance and support throughout the thesis. I would also like to thank Ove Kjetil Mikkelsen for his guidance involving Abaqus's arrangement of the finite element method.

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# 1. Background

The Lysefjord Bridge is a suspension bridge connecting the two municipalities, Sandnes and Strand, at Highway 13 and was opened on December 18<sup>th</sup>, 1997. The bridge axis is 42° from the north in the counterclockwise direction, meaning one of the towers is placed in the northwest and the second one in the southeast. In this thesis, these two towers and their orientations will be referred to as the north and south towers to simplify the formulation.

The bridge has a total length of 637 meters, whereas the main span covers 446 meters, and the north and south viaducts are 34.5 meters and 156.5 meters, respectively. The overall bridge layout is somewhat unsymmetric (ref. figures 26 and 30), with the girder support at the south tower approximately 7m lower than at the north tower. In addition, the south viaduct is more than four times longer than the north viaduct; however, in the structural sense, the viaducts are separated from the main span. The bridge provides a sailing height of 50 m. Shortly after the bridge opening, the bridge design received the “beautiful roads” award from The Norwegian Public Roads Administration (NPRA) due to its esthetics and non-disruptive impact on the surrounding nature.



*Figure 1 View of the Lysefjord bridge from the southwest, from [1]*

After the bridge opening, a higher number of wire fractures than usual was observed in the main cables, ascribed to the cable defects during the production [2]–[4]. This eventually led to the need and desire for a more detailed survey in the form of structural health monitoring. Acoustic sensors to detect wire breakage were installed in 2009 to support visual inspections of the cables.

A separate monitoring project focusing on improved modeling of wind loads due to turbulence and wind-induced vibrations of long-span bridges was developed by the University of Stavanger in collaboration with the NPRA. Since 2013, several wind sensors along the bridge span have been in operation, and accelerometers inside the stiffening bridge girder, for simultaneous monitoring of the bridge deck vibrations. In collaboration with the Norwegian Centre for Offshore Wind Energy (NORCOWE) and the Technical University of Denmark, optical remote sensing of wind using so-called lidars (Light Detection And Ranging) has also been introduced on the bridge, which for bridges was a quite unique area of application [5]. The wind lidar measurements make it possible to study the wind flow above the sea surface in a broader scope than what is possible by anemometers fixed to the bridge structure, thereby observing the effects of the fjord topography.

Several analyses have already been carried out for the case of the Lysefjord Bridge; however, the effect of heavy traffic loading has not been studied using a detailed numerical model of the bridge. This will be the primary load case investigated in the thesis after creating a suitable numerical model of the bridge in the finite element software Abaqus. In general, the thesis aims to provide a better understanding of the load carrying characteristics of a suspension bridge, in particular in the sense of dynamic nonlinear effects. Eigenfrequencies and eigenmodes obtained with the new finite element model will be studied and compared to previous model analysis results and the measurement data. To increase the insight into the dynamic loading, the recorded time-series of the bridge deck accelerations due to traffic at full-scale have been applied as imposed constraints in the finite element model and the associated loads in the structure studies. Two previous bridge models created by Ragnhild Steigen, from [6], and Jan Tveiten, from [7], will be used as guidance when creating the model in Abaqus.

## **1.1 Scope of the work**

In relation to the existing plans for several major long-span bridges across the Norwegian fjords in complex surrounding topography, detailed field studies of ambient loading and the dynamic response of a long-span bridge are valuable and may be essential in developing more practical solutions [8]. This work will mainly focus on the dynamic responses of the bridge girder and the main cables associated with the traffic frequently crossing the bridge. The cable forces and the girder displacements will be simulated using the recorded deck acceleration data.

Although this thesis is directed upon a specific case study involving the Lysefjord Bridge, the essence of the discussion and results may also be applicable to general cases of cable-supported bridges.

## **1.2 Methodology**

A quasi-experimental approach has been adopted to investigate the dynamic behavior of the Lysefjord bridge under authentic loading conditions. This approach involves implementing recorded traffic-induced accelerations of the bridge girder into a finite element model, aiming to enhance the understanding of the bridge's dynamic response.

In the methodology, Abaqus software was utilized to create the finite element model of the bridge. Additionally, FEM-Design was employed to supplement with detailed information of the girder cross-section for input purposes in Abaqus. The process throughout these model creations is presented in later sections of the thesis. The integration of the developed finite element models, the recorded vibrations, and theoretical frameworks form the basis for this methodology, and it aims to provide a comprehensive analysis of the bridge's dynamic behavior. In addition, the work serves as a foundation for further studies in the field.



- Chapter 2: Suspension bridges are presented alongside their main structural components and are discussed in general and with specifications of the Lysefjord bridge.
- Chapter 3: This chapter presents the various structural health monitoring complemented on the bridge. A quick overview of the detection of fractures and defects on the main cables is presented, followed by a description of the structural health monitoring involving acoustic sensor measurements and anemometer- and accelerometer assemblies.
- Chapter 4: Time-varying loads acting on the bridge are introduced. The loads are presented briefly, in a general way, to identify how they have impacted the bridge throughout various periods of time. Weather stations are used to collect historical data for wind and temperature variations for comparisons and analyses.
- Chapter 5: This chapter presents and explains the finite element models in detail. The FEM-Design model is presented stepwise with three modifications to cover various properties of the bridge girder cross-section. Then, the Abaqus bridge model is presented with all assumptions and approaches made. A girder cross-section modeling approach is presented with calculations followed by a detailed discussion and summary of input loading and boundary conditions. Other software programs such as MATLAB and Mathcad have been used for various calculations and are briefly introduced as they are mentioned.
- Chapter 6: Here, the results of the Abaqus analysis are presented and discussed. Eigenfrequencies and eigenmodes are discussed, analyzed, and compared to previous calculations and measurements. Furthermore, the eigenfrequencies and eigenmodes form the basis for calculating Rayleigh's damping coefficients. Nodal and global displacements of the bridge girder are presented and analyzed, as well as the torsional response of the girder. Lastly, the simulated cable forces are presented, discussed, and compared to simplified calculations and results from design calculations.

# 2. Suspension bridges

Suspension bridges are bridges that use ropes, chains, or cables from the vertical suspender to hold and transfer the weight of the bridge deck, traffic, and other loads. Cable-supported bridges are typically distinguished by their ability to overcome large spans. By adding cable-supported side spans, the towers can maintain stability with equal cable forces on each side, preventing large tension forces on the longitudinal backstay side of the tower. Suspension bridges are remarkable structures that offer advantages such as flexibility in location and reduced material requirements. Today, there are more than 200 suspension bridges in Norway [6], with Hardangerbrua being the largest one reaching 1310 m across the main span. The longest and tallest suspension bridge in the world today is the 1915 Çanakkale Bridge, officially opened in March 2022 in Turkey, being a total of 4608 m long with a main span of 2023 m.

The main components of a suspension bridge are shown in Figure 2. However, the Lysefjord bridge does not include vertical suspenders on the backstay cables. The load-transferring system of a suspension bridge relies on the main cables, which efficiently transfer the loads from the girder via the hangers in pure tensional forces. Further, the forces are transferred to the tower leading it to the ground or directly to the anchorage supports. Overall, global stability heavily relies on the combined strength and rigidity of the towers and the stiffening girder.

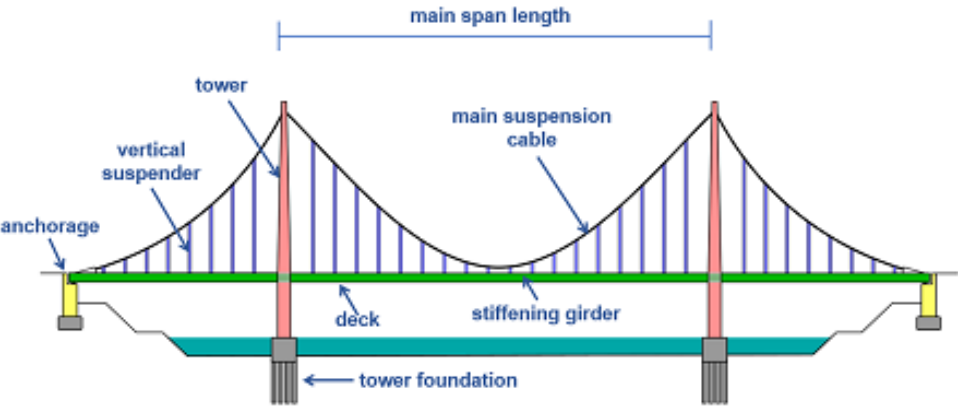


Figure 2 Main structural components on a suspension bridge, [9]

## 2.1 Towers

In a suspension bridge, the towers, or pylons, support the main cables and transfer the forces down to the foundation through compressive forces. It is often distinguished as pylons rather than towers because a free-standing tower's design will be dominated by the moment induced by the wind's horizontal loading (drag). However, as for pylons, the most decisive load will be the axial force originating from the vertical components of the forces in the main cables [10]. The cables are supported on a saddle-terminal at the tower at a specific height to provide the necessary inclination of the main cables. The main forces a bridge tower needs to withstand are compressive forces from the cable support, buckling, and torsional forces, which makes reinforced concrete a natural choice of material composition.

The Lysefjord Bridge consists of two reinforced concrete towers, which is the most common application for suspension bridges. Each tower consists of two tapered columns connected by two horizontal cross beams, as seen in Figure 3. The two towers are not identical in height due to the lower ground foundation on the south side of the bridge. Both towers reach 102.25 m above sea level, with the north tower being just above 95 m in height and the south tower being six meters taller at just above 101 m.

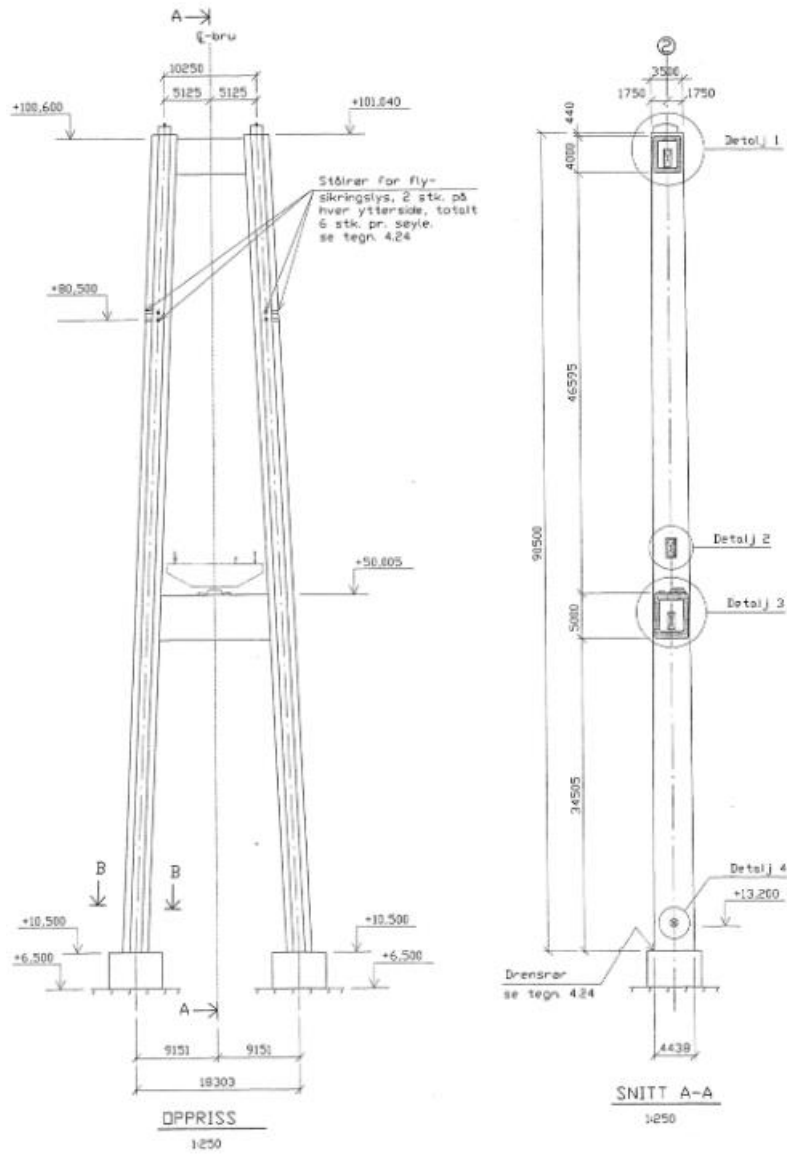


Figure 3 Elevation drawing of the north tower belonging to the Lysefjord bridge

## 2.2 Stiffening girder

The stiffening girder is the component that transfers the direct traffic load and other applied loads from the deck to the main cable via the hanger cables. The girder is subjected to the majority of the external load due to the total traffic load, and in most cases, both the dead load and the wind area are larger for the deck than for the cable system [10]. The girder is also crucial for the global stability of the deck and the bridge by providing stiffening along the global longitudinal axis. For the traditional cable-supported bridges with vertical cable planes, it has

been shown that the support from the cable system is at its most efficient for the dead load, somewhat lesser efficient for the traffic load, and least, if not non-existent, for horizontal loads [10]. Since the girder is suspended from the cables, it is essential to obtain low self-weight by utilizing the girder design geometrically. Typically, this results in closed sections with design characterized by aerodynamic needs, which will reduce torsional effect and the formation of vortex, which causes additional vibrational loads on the structure.

The stiffening girder on the Lysefjord Bridge is, as explained above, an aerodynamically characterized closed section with trapezoidal stiffeners, as shown in Figure 4. In addition to the trapezoidal stiffeners, there are two stiffening plates on each side of the cross-section. Both the trapezoidal- and the plate-stiffeners runs throughout the whole main span as continuous plates. The girder is 10.7 m wide in the upper horizontal plane, with a height of 2.7 m at its highest point. The steel box appears hollow between each hanger point, reaching 12 m in span. A diaphragm at each hanger point replaces most of the hollow area, resulting in an 8 mm thick steel plate across the cross-section. This diaphragm supports the 12 m span hollow steel box and can be thought of as a web supporting the flanges at each side. In addition, the diaphragm provides stiffness on a global scale for the girder and the cables as a result of preventing a chain reaction from girder instabilities.

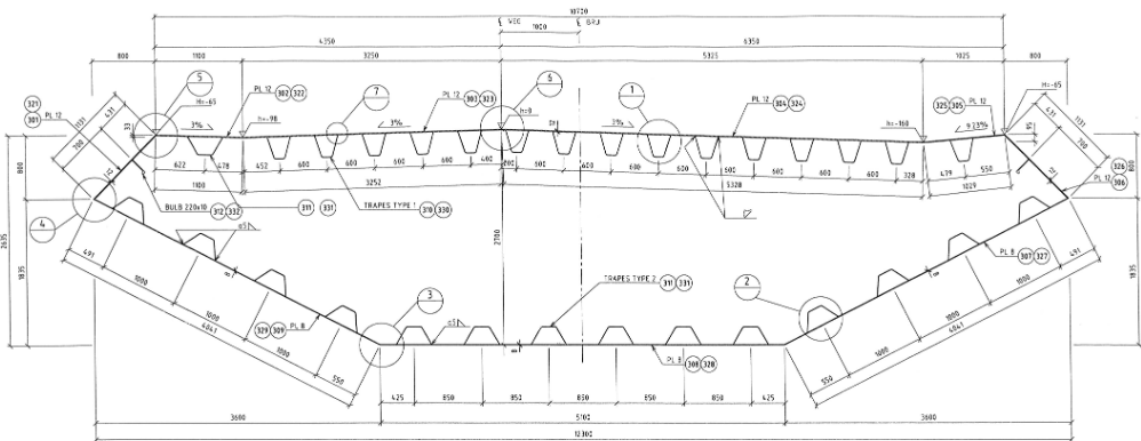


Figure 4 Cross-section of the bridge girder for the Lysefjord bridge

## 2.3 Main cables

The primary function of the main suspension cable is to support the deck via the hanger cables and transfer the loads to the towers or the anchored cable support through tension forces in the cables. The main cables act as the main structural element for carrying the suspension bridge's vertical loads. Other than the direct vertical loads from vehicles, pedestrians, and dead loads, the cables must also be able to withstand vibrations and other deflections from e.g., dynamic wind loads or vehicle-induced vibration.

Typically, the cables consist of several wires, often in different shapes, which are shop-assembled to form one large, solid strand. The combined wire usually forms a parallel wire strand, locked coil strands, or spiral bridge strands, as shown in Figure 5.

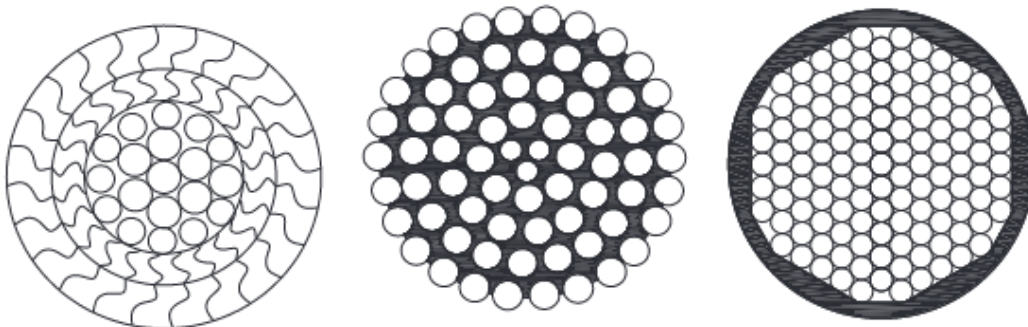


Figure 5 Different assemblies of bridge strands: locked coil, spiral, and parallel wire, respectively, from [11]

The support of the main cables in the Lysefjord Bridge consists of four points; the two pylon tops and the two anchor blocks. This is the typical application of supports for traditional earth-anchored suspension bridges [10]. The main cables are measured to be 713 m long before dead load is applied and consist of six fully locked-coil strands at each side of the bridge. Locked-coil strands are composed of regular round wires arranged as in a normal helical strand in the core and with a special Z-shape wire in the outer layers. This arrangement makes the locked-coil strand more compact than any other strand type. The Z-strands are self-locking to minimize the effect of strand breaks. The Z-shaped wires also make the strand less sensitive to

side pressures at the supports due to a genuine surface contact rather than a point contact, as is the case of the other round wires [10].

The arrangement of the cables in the main span differs from the arrangement in the backstay-side. In the main span, the cables have plated connections at each hanger point in a parallel three-by-three arrangement. In the backstay side, all six cables reach from the saddle to the anchorage undisturbed in a straight-line arrangement.

### 2.4 Hanger cables

The hanger cables are the load-transferring element between the bridge girder and the main cables. They act as pure tensional connecting elements with spacing calculated regarding the specific bridge girder. In the Lysefjord Bridge, the hangers are connected through a bolted plate connection on the main cables and anchored on the bridge girder with a spacing of 12 m, starting 19 m from the saddle point.

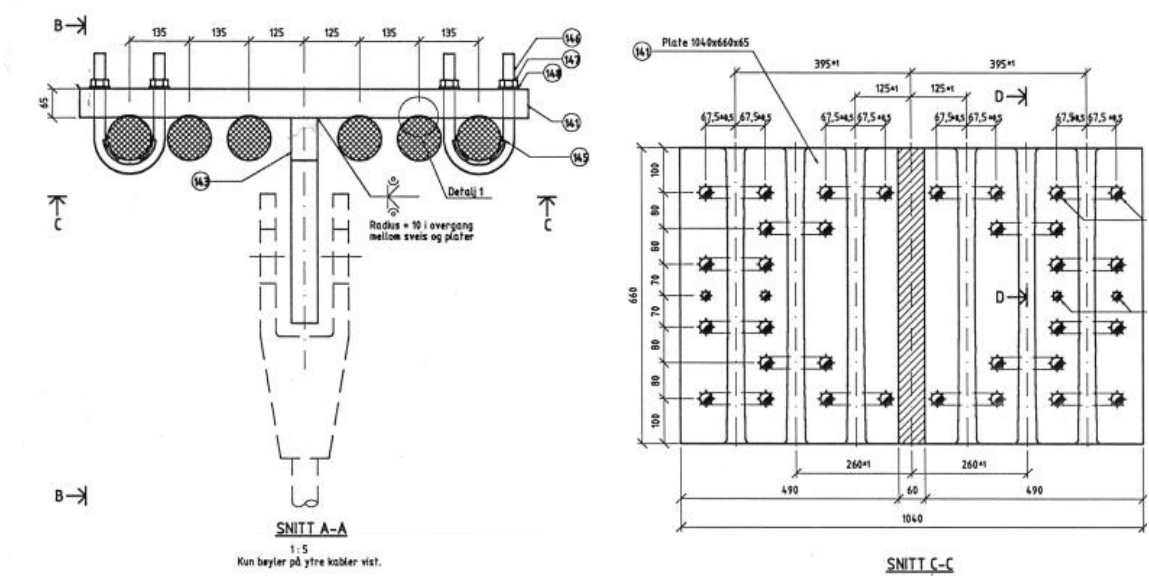


Figure 6 Connection details between the hanger and the main cable of the Lysefjord bridge

## 2.5 Anchorage

The anchor piers' task is to fix the cables to the ground, in other words, sustain the main cable's ends. Usually, in Norway as well for the case of the Lysefjord Bridge, the cables are typically anchored directly to the nearby mountain or underlying rock with the anchoring device.



Figure 7 Anchorage detail of the backstay cables on the Lysefjord bridge

Anchorage types can be classified as either gravity-type-, tunnel-type anchorages, or rock anchorages. With gravity-type, the loads from the cable are resisted with the self-weight of the anchor frame and foundation. In tunnel-type, the loads from the cable are resisted by using the shear forces of the outer circumference of the steel frame and the pressure of the plug body. Rock anchorage uses the weight, adhesion, and friction from rock wedges to resist the loads [12]. At the Lysefjord Bridge, tunnels are built underneath the anchoring points to monitor the connection.

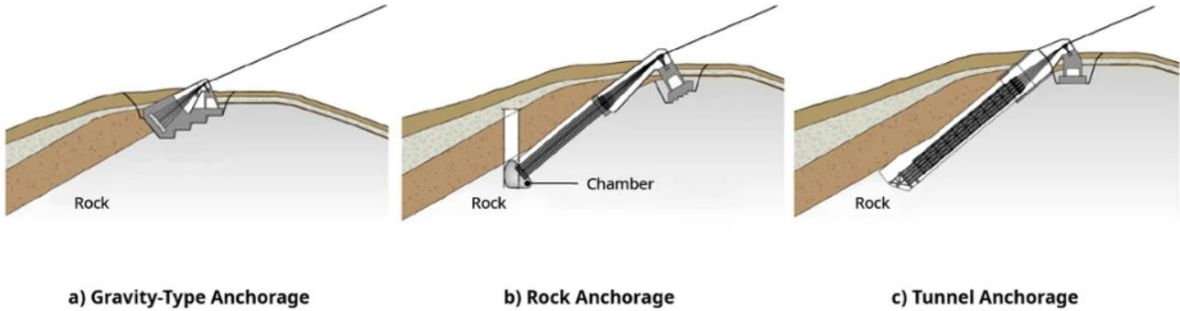


Figure 8 Anchorage types, from [12]



## 3. Field measurements

### 3.1 Cable fracture characteristics and overview

The events of a few random wire fractures after a cable-supported bridge are completed to be fully operational are not unusual. However, the magnitude of the trend and evolution of fractures in the Lysefjord Bridge after the opening was never experienced in Norway. This led to the initiative of more frequent inspection and situation reports conducted by NPRA, Det Norske Veritas (DNV), and Blom Bakke AS [2]. Three detailed inspection reports were conducted directly after the bridge opening in 1999, 2000, and 2001, where fractured samples of the outer Z-layer were extracted and tested. Throughout the testing and studying of the samples, both Visual examinations, Scanning Electron Microscope (SEM), Energy Dispersive Spectrometer (EDS), Metallographic examination, hardness measurements, and bending tests were conducted.

Based on the three reports, it was concluded that the presence of surface imperfections and defects on the Z-wires likely causes the fractures observed in the main cables. The imperfections were observed in all reports and were concluded to be a consequence of the manufacturing process, especially the rolling process of the wire. The fractures appeared to be caused by tensile overload after a reduction in the wire's cross-section due to defects and crack propagations. The effects of corrosion were not particularly discussed in these reports. Additionally, the report from the year 2000 suggests that hydrogen embrittlement may have contributed to the failure as a result of the cleaning process the wires are exposed to prior to galvanizing. The report from the year 2001 found no significant defects or damages in the received cable section other than a restricted number of minor surface imperfections in the Z-wires. Furthermore, this report indicates that fractures of brittle behavior may initiate imperfections of a specific size when the wire is exposed to bending stresses.[3], [4], [2].

The cross-section of the main cables consists of 279 wires and was spun on site. The wires are created from melted steel bars formed under a rolling process. Further on, the steel bar is formed in thin sections of wanted geometry (round or Z-shaped). This forming and cold rolling process under manufacturing could cause small surface defects of the wires, as seen in Figure 9. This process also introduces the possibility of different solidification processes, which again can cause residual stresses to appear. The reports states that the cross-sectional area is reduced by one forth in the area of defects. In addition to the reduction of cross-sectional area, the reports states that when a large number of fractures appears on a small area, the load-bearing capacity per cable is reduced by 0.4% per wire fracture. However, the construction of the cables with spun wires and Z-wires makes it so that lost capacity in the cables will be maintained after 2-3 meters so that the loss of capacity will not sum up if this distance is obtained. [3], [4], [2].

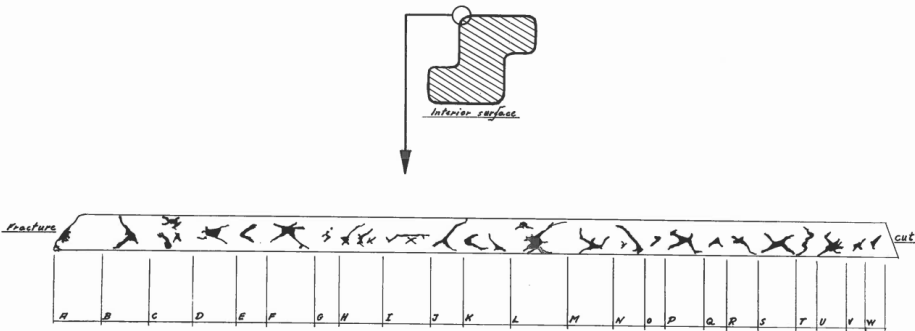


Figure 9 Test segment extracted from the cables showcasing surface defects

In addition to the reports, a SINTEF publication summary from a lecture done in 2013 involving these matters was found on SINTEF’s website. It mentions that during the bridge’s sixteen years of life at that time, approximately 1600 fractures had been registered in the outer Z-layer, following a linear development. It was believed to be caused by atomic hydrogen diffusion leading to hydrogen embrittlement. Here, it also specifies that the phenomenon is unrelated to any wire corrosion mechanism. The publication states that the surface defects, especially overlap shear defects, created during the Z-wire forming process are the root cause of the fractures [13].

## 3.2 Acoustic monitoring system

In October 2009, an acoustic monitoring system was installed by Advitam on the Lysefjord Bridge to complement the otherwise visual inspections that had been conducted since the opening of the bridge. This arrangement was included in a nine-year contract deal between the Public Road Administration and Advitam. More than 900 fractures were detected from the opening of the bridge to this date. The monitoring system, SoundPrint<sup>®</sup>, is a nondestructive test method used to detect possible wire fractures on the cables and was installed in the pursuit of a complete detection system for all the fractures that would occur. The system uses acoustic sensors distributed about the structure and a data collection device mounted at the site that collects the data and sends it to the analysis center via the Internet. By combining the visual inspections with the acoustic sensor when detecting the fractures, it was thought to be a more precise way to separate the fractures happening in the outer z-wire layer with the ones happening in the inner layers.

The monitoring system stopped operating at the end of June 2019 due to the end of the contract period, leaving the results from the last four years at a nearly negligible amount of fracture events. During these nine years, there has been some rebranding of the company and equipment. Whether it is new companies replacing the older company, a rebranding, or a buyout has been somewhat unclear when reading the reports; however, in the end, it was represented as the monitoring system EverSense<sup>®</sup> designed by SIXENSE Systems.

### **Sensor locations [14]**

To cover the main cables, 60 acoustic sensors were mounted at specific locations. Note that each main cable consists of 6 individual cables clamped together. The system allows the identification of which cable from the six is concerned for the backstays. On the main span, the system provides detection of wire breakage in longitudinal locations.

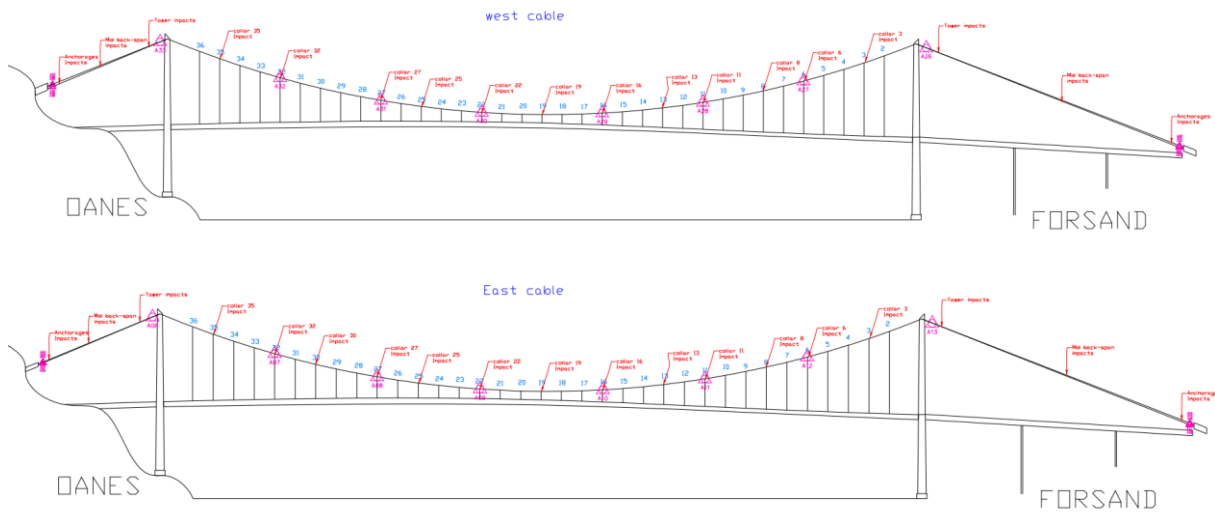


Figure 10 Main locations for acoustic sensor assembly on the West cable (top) and the East cable (bottom)

At each main cable, the following assembly of sensors was conducted:

- One sensor at each cable at a 4.5 m distance from the anchorage points.



Figure 11 Acoustic sensor assembly details on the backstay cables

- One sensor at each cable, multiplexed in one direction, at a 4.5 m distance from the saddle points at each tower on the backstay side.
- At the clamp of hanger numbers 6, 11, 16, 22, 27, and 32, six sensors are connected sideways on the clamps.

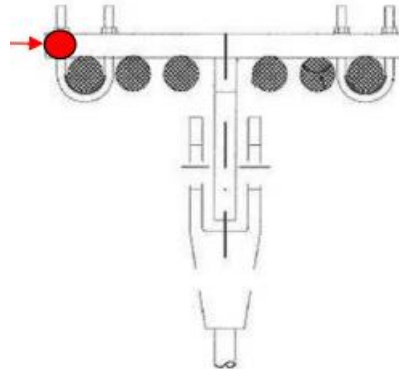
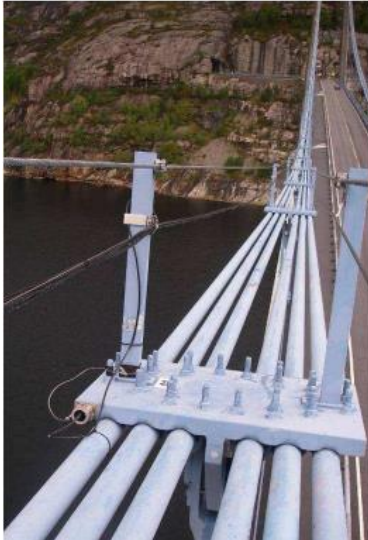


Figure 12 Acoustic sensor assembly details on the main cables

### Fracture detections and data

After the first test period, from October 2009 to April 2010, the system had been triggered 2528 times, with 75% of the events concentrated on the East cable [15]. However, only 68 events were classified in the report as probable fracture events. Further on, test results have been collected in periods of approximately three months each. The results mainly contain a summary of all events detected by the sensors, the degree of interest for each event, uptime status for the system in the given period, and other relevant cases in the test period.

A summary of the surveillance reports from 2009 to 2019 has been made and can be seen in Table 1 and Table 2. During the 2014 period, there was a change in company notation and summary content, hence the difference in table setups. In Table 2, “green events” represent events without interest, “orange events” are events of interest such as test impacts and significant activity, and the “red events” are possible wire breakages. The summary of events only considers fractures as of 2009, when the measurements started, due to a lack of detailed data on fracture events from the previous years.

Table 1 Summary of acoustic measuring events from October 2009 to September 2014

Test period	Total events	Events West cable	Events East cable	Probable fractures	System uptime
01.10.2009 - 20.04.2010	2528	634	1894	68	99,85 %
01.10.2010 - 31.12.2010	1380	518	862	12	99,66 %
01.01.2011 - 31.03.2011	2204	511	1693	20	99,80 %
01.04.2011 - 30.06.2011	4800	1748	3052	15	99,95 %
01.07.2011 - 30.09.2011	20391	9274	11117	7	98,02 %
01.10.2011 - 31.12.2011	10620	1696	8924	12	99,64 %
01.01.2012 - 31.03.2012	2187	189	1998	7	99,32 %
01.04.2012 - 30.06.2012	1177	58	1119	7	97,65 %
01.07.2012 - 30.09.2012	4143	143	4000	4	99,34 %
01.10.2012 - 31.12.2012	7920	1780	6140	43	99,58 %
01.01.2013 - 31.03.2013	1474	701	773	26	99,98 %
01.04.2013 - 30.06.2013	2487	520	1967	12	99,98 %
01.07.2013 - 30.09.2013	7285	1995	5290	5	82,89 %
01.10.2013 - 31.12.2013	11135	3307	7828	8	99,49 %
01.01.2014 - 31.03.2014	1896	1685	211	25	98,05 %
01.04.2014 - 30.06.2014	1266	1154	112	1	90,27 %
01.07.2014 - 08.09.2014	5879	5184	695	0	81,50 %

Table 2 Summary of acoustic measuring events from September 2014 to June 2019

Test period	Total events	Green event	Orange event	Red event	System uptime
09.09.2014 - 31.12.2014	125427	125366	59	2	94,48 %
01.01.2015 - 31.03.2015	3883	3883	0	0	100 %
01.04.2015 - 30.06.2015	913	913	0	0	100 %
01.07.2015 - 30.09.2015	1426	1425	0	1	100 %
01.10.2015 - 31.12.2015	4954	4954	0	0	100 %
01.01.2016 - 31.03.2016	4327	4325	0	2	99,55 %
01.04.2016 - 30.06.2016	1502	1500	0	2	68,78 %
01.07.2016 - 30.09.2016	999	999	0	0	92,90 %
01.10.2016 - 31.12.2016	1767	1766	0	1	75,60 %
01.01.2017 - 31.03.2017	1137	1137	0	0	99,86 %
01.04.2017 - 30.06.2017	703	703	0	0	92,30 %
01.07.2017 - 30.09.2017	1418	1418	0	0	99,99 %
01.10.2017 - 31.12.2017	1924	1924	0	0	100 %
01.01.2018 - 31.03.2018	964	962	0	2	94,47 %
01.04.2018 - 30.06.2018	2846	2845	1	0	69,50 %
01.07.2018 - 30.09.2018	46578	46421	157	0	99,80 %
01.10.2018 - 31.12.2018	282	282	0	0	63 %
01.01.2019 - 31.03.2019	274	271	0	3	67 %
01.04.2019 - 30.06.2019	7936	7936	0	0	100 %

Based on the given data, it is clear that the east cable has been the most triggered for events, around 65% of the total events. It is also shown that only 0.0956% of the events captured by the sensors correspond to actual wire fractures. This significant difference shows how many different activities the sensors have picked up, which may lead to a source of error when confirming wire fractures. There is also an evident seasonal affection to the events as the periods between April and September contain the lowest amounts of fracture events. January to March and October to December combined equal 75% of the total wire breakage. Another case worth noticing is the occasional drop in system uptime during some periods. As seen during e.g., July to September 2013, the system uptime dropped quite a bit which may indicate a data collection issue rather than a decrease in wire breakage. However, the frequency of wire breakage seems to have cooled at the end of 2014, which was also the beginning of the EverSense® monitoring period.

A chart showing the number of fractures at the end of each year has been created to organize the tables. Here, the trend of fractures is shown to decrease more or less each year until a sudden drop in 2015. After this, the number of fractures decreases to mostly zero in the remaining five years. The results from the last five years might indicate that the wires have come to rest and settled either by themselves, as a result of maintenance work, or by a combination of those.

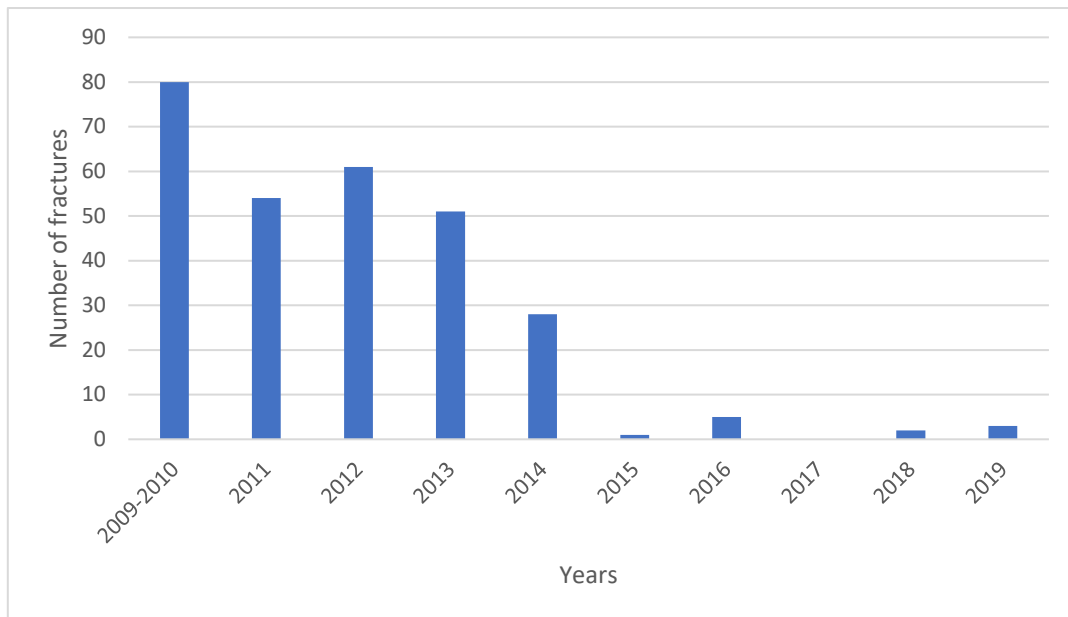


Figure 13 Yearly display of fracture events on the Lysefjord bridge collected from the acoustic measurements

Overall, the measurements from the acoustic sensors have, in general, resulted in fewer events of fractures compared to the amount registered in the years previous to this. Based on this, together with the decreasing trend from the measurements, it is safe to assume that the cable wires have settled and will not experience any sudden high number of fractures due to the production defects.

### 3.3 Ambient vibration measurements (AMV)

Since 2013, the bridge has been instrumented with structural health monitoring systems measuring wind and traffic to capture both wind flow patterns at the bridge site and the structural acceleration due to wind and traffic loads. A total of nine sonic anemometers and four pairs of three-axial accelerometers were mounted on the bridge (see Figure 14). Eight of the sonic anemometers are 3D Wind-Master Pro from Gill Instruments which can record wind velocity and sonic temperature with a sampling frequency of up to 32 Hz. The last anemometer is a weather transmitter that monitors the horizontal wind components, relative humidity, pressure, and absolute temperature with up to 4 Hz sampling frequency. In Figure 14, the sonic anemometers and weather station are shown to be mounted above the deck [16].



The tri-axial accelerometer is mounted inside the deck on the girder's west and east sides to retrieve symmetrical and asymmetrical components of horizontal, vertical, and torsional modes of the bridge's response, operating with a sampling frequency of up to 200 Hz. The lateral distance between each accelerometer pair is 7.15 m, while the distance between each hanger pair is 10.25 m. The data are collected by five acquisition units that are synchronized and aggregated into a single file, each containing 10 minutes of record that are continuously transmitted [16]. Accelerometers were also placed on the top crossbeam of the north tower but are not visualized in the figure below. The accelerometer in the girder and the tower are installed in different planes making the longitudinal direction equal to the y-direction for the bridge girder, and for the tower, the x-direction is in the bridge's longitudinal direction. This makes the lateral axis equal to the y-axis for the girder and the x-axis for the tower. In this thesis, the tri-axial accelerometers are used to export vibration data from traffic during low wind velocities.

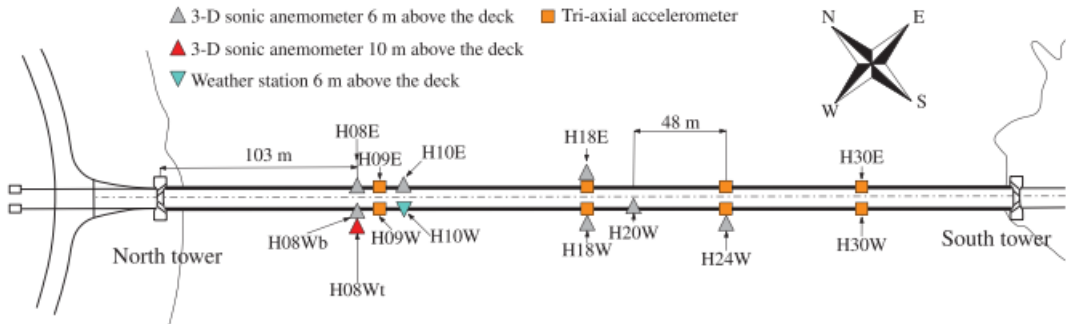


Figure 14 Overview of the monitoring system at the Lysefjord bridge including anemometers, a weather station and accelerometers, from [16]

Two MATLAB data files have been received by the thesis supervisors in terms of the bridge deck accelerations recorded in 2017. One of the datafiles consists of 10 samples of 10-minute long recordings measuring traffic-induced bridge vibrations, recorded on 07.03.2017, and will be used in this work. The response data were recorded by five pairs of accelerometers: four pairs inside the bridge deck (ref. Figure 14) and one pair located at the top of the north tower, which was deployed over a shorter period and not included in the above figure. The idea is to

implement these accelerations directly to the finite element model of the bridge at the corresponding locations. Furthermore, the response of the bridge due to these acceleration loads will be studied in different aspects. This data set originates from the research on bridge vibration induced by individual, heavy vehicles [17]. However, the exact traffic scenarios and driving patterns at the different time periods are unknown to the author. To the supervisor and author's knowledge, there are no existing studies or experiments involving the application of such real-life measured acceleration to a finite element model of a bridge in the context of traffic-induced bridge response. Further details concerning the selected data file and the application in Abaqus are discussed in Chapter 5.2.3.4.

## **4. Time-varying loads acting on the bridge**

Rogaland County in Norway is well known for its environment consisting of high and frequent wind speeds and low temperatures. In such an environment, the local environmental conditions and the related load effects must be studied in detail. The environmental factors that are most important in Norway are wind, temperature gradients, rain, and snow. Therefore, when evaluating these factors, it is crucial to have access to weather stations nearby the construction site.

The unique feature of a suspension bridge compared to other types of bridges is their high level of flexibility. This flexibility makes dynamic loading a very important and interesting aspect in engineering studies and construction, as they can result in severe responses. Wind, waves, and traffic are the most typical dynamic factors studied for bridges, as they can induce severe vibrational loads, especially for long-span flexible structures.

When collecting weather-related data to analyze towards the fracture events and in general, the weather station in Sola, Rogaland, has been used. This station, where the weather data has been regularly stored over a long period, is located around 50km west of the Lysefjord. It seemed that it exists or existed some weather stations that were closer by, but these did not seem to contain any data to extract. As mentioned, wind sensors were mounted on the bridge in 2013; however, studying the yearly fracture pattern from the acoustic measurements shows that the period before 2013 is the most interesting to investigate.

### **4.1 Wind**

Wind loads are among the governing design consideration, and understanding and accounting for different types of wind forces are essential for ensuring safety and reliable operation. Wind-

induced vibration can cause fatigue and other types of damage to critical components, leading to structural degradation and potentially compromising the safety of the bridge.

By studying the topography around the bridge, it can be observed that its location is approximately 30 km from the coastline. As a side-fjord to the Høgsfjord, the Lysefjord stretches for 40 km and spans approximately 2 km at its widest.

The mountainsides lining the Lysefjord contain an impressive steepness, with peaks reaching up to 1100m. These mountainsides form an imposing backdrop with their wall-like formations, shaping the wind dynamics in the fjord. Since the fjord is located off the coast, the bridge is shielded from the winds at the open sea. Additionally, Bergsholmen and Syngknuten, seen in Figure 15, contribute to such shielding effects from deeper inside the fjord. These features act as natural barriers, giving some protection against the direct impact of coastal winds.



Figure 15 Map of the nature surrounding the Lysefjord bridge (marked with the arrow), from [18]

The wind conditions relevant to the bridge location can be well described compactly in terms of the wind roses, which give the relative frequency of wind speeds for different wind directions. The wind rose for Sola (SN44560) from October 2009 to June 2019 can be seen in Figure 16. The wind rose displays the frequency distribution of the mean wind speeds over different directions. The wind direction is divided into 16 sectors, each covering 22.5 degrees. The wind speed is color coded and categorized by the Beaufort scale [19].

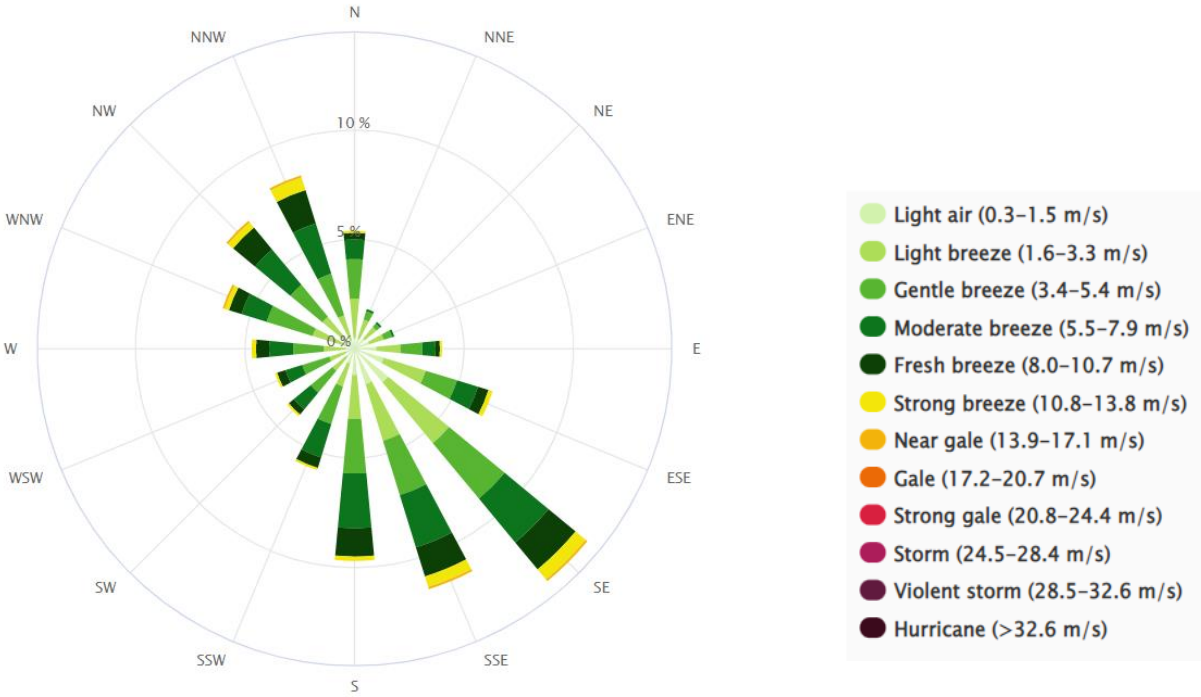


Figure 16 Wind rose collected from Sola weather station covering the entire acoustic measuring period

The wind rose displays dominant winds from the southeast, with velocities reaching between 24 m/s and 28 m/s. It is worth noting that the location of Sola is quite a flat and extensive area, providing an open way for the wind to increase and impact without much disturbance. Also, due to the relatively large distance and complex topography around the bridge in the Lysefjord, this wind rose cannot be directly translated to the location of the Lysefjord.

A wind rose has been established by studying wind data recorded by anemometers on the Lysefjord bridge, as shown in Figure 17. The wind rose is based on the 52000 samples of 10-minute duration recorded during 2015. A clear difference between the wind rose at the bridge site and that at Sola can be observed, as the fjord alignment defines the dominant wind directions at Lysefjord. At the bridge site, most of the wind speed reaches 6 m/s to 10 m/s on average during this period, with the longest impact from inside the fjord. At the maximum, it can be seen that the wind speed is reaching between 17 m/s to 21 m/s on average, providing quite large gusts of wind on the bridge. However, these measurements were conducted for one year only, unlike the wind rose from Sola that covers the entire acoustic measuring period, introducing further uncertainties regarding specific and mean results for comparisons.

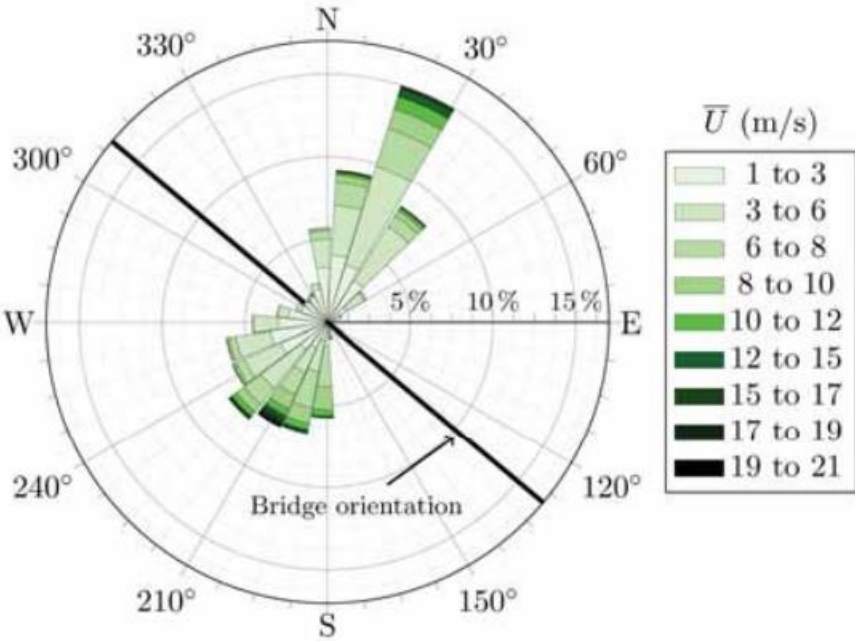


Figure 17 Wind rose from measurements done on the bridge, from [20]

## 4.2 Temperature

When materials are exposed to temperature changes, thermal- expansion and contraction follow with. These effects can further lead to changes in stress and strain of the materials. Considering these effects on top of other environmental impacts and general load cases is an

important inclusion to better understand the behavior and performance. Also, studying the history of temperature development alongside fracture patterns can lead to discovering vital events of interest.

Temperature data has been collected from the Norwegian Climate Service Center, [seklima.met.no](http://seklima.met.no), from the reference weather station in Sola, Rogaland. The temperature has been collected for each month of the acoustic monitoring period as the minimum, maximum, and mean temperatures. These data have been further calculated to the average value between each measured quarter period from the acoustic monitoring.

To investigate the relationship between temperature and fracture pattern, the most affected period of registered fracture is used as a basis, reaching from the start period of the acoustic measuring to the end of 2013. Figure 18 below displays a plot of the fracture data in Excel alongside the monthly average of the minimum, maximum, and mean temperature in the corresponding testing periods.

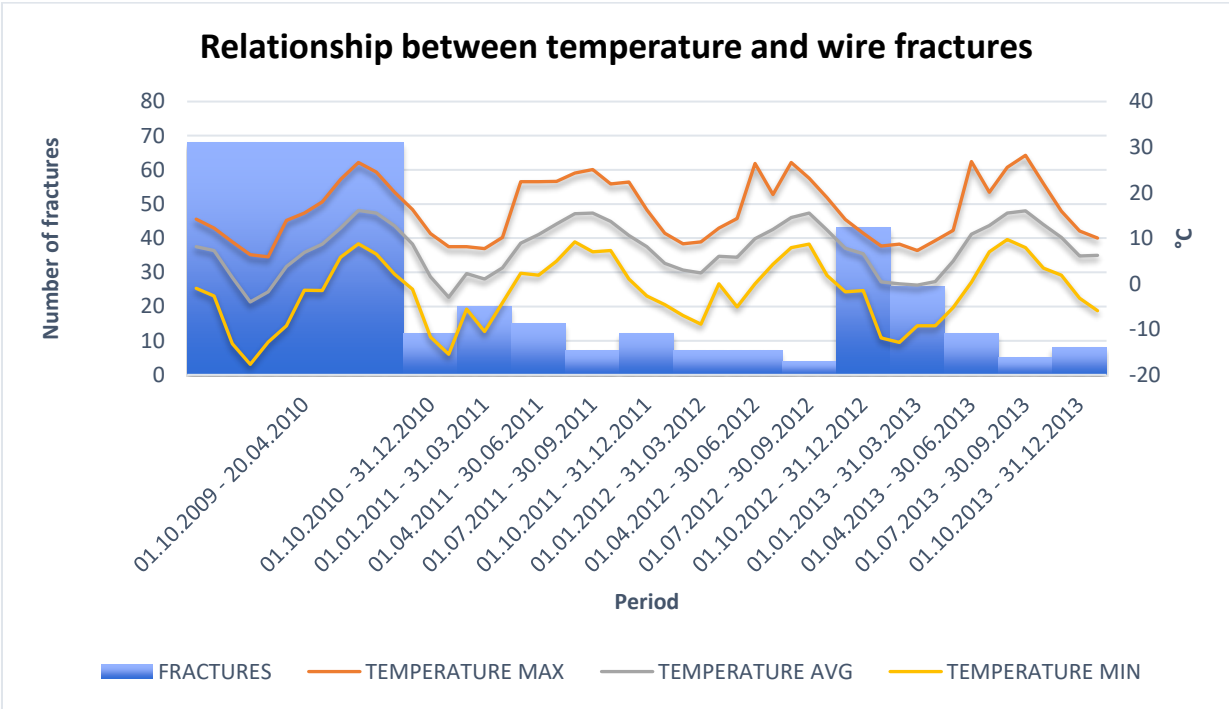


Figure 18 Relationship between average monthly temperature and periodic fracture events from 2009 to the end of 2013

From the above figure, a relationship between drops in temperature and periods of high fractures is found. This is also a natural assumption since when the temperature drops, the cables will retract, becoming more brittle and thereby being exposed to fracture. Generally, the temperature fluctuates several times above and below zero degrees, causing frequent expansion and retraction states in the material, which may cause the cable fatiguing effects. It can also be thought that within periods of cold temperatures, the sun might heat the outer layers of the cable. In contrast, the inner layers will heat more slowly, causing temperature gradients across the cross-section.

### **4.3 Traffic**

Traffic loads have a significant impact on the performance and safety of bridges. As vehicles cross the bridge, they generate dynamic loads that can cause the structure to vibrate and deform. These vibrations can lead to the development of cracks and fractures in critical components such as the main cables, which can compromise the overall structural integrity of the bridge. By analyzing how traffic loads impact these critical components, a deeper understanding of the factors contributing to the development of fractures and other types of damage can be gained.

The Lysefjord bridge experiences a good amount of lorry traffic due to several transport goods across the bridge. A concrete factory exists on the Forsand-side, and from an interview done by Tveiten, it was told that at least four lorries with loadings of 30 tons would pass the bridge daily [7]. It should be thought that the bridge is designed to withstand general lorry trafficking. However, if a substantial number of new fabrics have opened in the location since the bridge opening and hold the reasoning behind a large amount of lorry trafficking, it would be a good idea to investigate the bridge's response regarding this.

The NPRA provides a manual for bridge design to clarify standard traffic load application procedures. When a road bridge is designed for traffic loads, the loads are described with the



help of equivalent loads, i.e., simplified loads that cover the effect of certain types of heavy vehicles surrounded by a mixture of light and heavy vehicles. These are referred to as the load types V1, V2, and V3. V1 and V2 are assumed to be located within one traffic lane with a length equal to the bridge length. In the longitudinal direction of the bridge, the traffic load is positioned arbitrarily to achieve the most unfavorable load effect. In the transverse direction, however, the traffic load is placed on a case-by-case basis of unfavorable conditions. These traffic loads are mainly referred to as vertical or horizontal loads, with the horizontal components consisting of brake loads, side loads, and centrifugal-load [21].

**Load type V1, V2 and V3**

Load type V1 involves the uniformly distributed load,  $p = 9 \text{ kN/m}$ , and three-shaft loads of  $210 \text{ kN}$  with distances  $\geq 2.5 \text{ m}$  and  $\geq 6.0 \text{ m}$  (see Figure 19). Each of the shaft loads consists of two wheel loads of  $105 \text{ kN}$  separated by 2 meters.

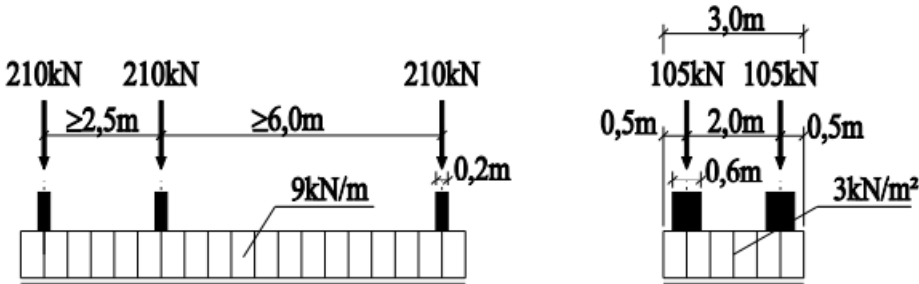


Figure 19 Load type V1. Longitudinal direction to the left and transverse direction to the right. From [21]

The load type V2 consists of one shaft load of  $260 \text{ kN}$  distributed on two wheel loads of  $130 \text{ kN}$  separated by 2 meters.

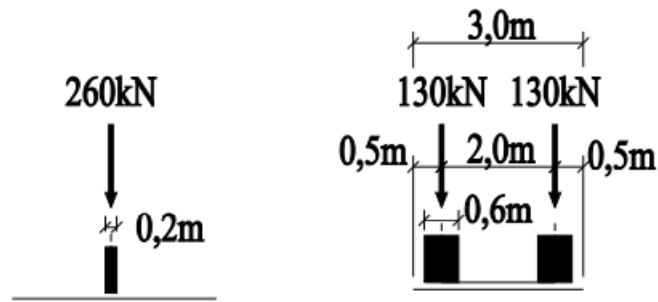


Figure 20 Load type V2. Longitudinal direction to the left and transverse direction to the right. From [21]

Load type V3 consists of only a single wheel load of 130 kN with a load surface of 0.2 m in the longitudinal direction and 0.6 m in the transversal direction. The wheel load is placed arbitrarily in the transverse direction [21].

Another important assessment to be made regarding traffic loading is its contribution to fatigue damage due to cyclic repetitive loading. Suspension bridges are designed to withstand significant amounts of traffic load. However, the repeated loading cycles may introduce wear and tear and eventually crack propagations. The bridge is equipped with many joints and connecting elements that are prone to the localized structural damage that occurs from fatigue. Both the magnitude and frequency of the loading influence the fatigue, as well as material properties and environmental conditions. Fatigue assessment will not be included in this thesis but is recommended to be investigated in further studies.

## 5. Finite Element Model

Finite element analysis (FEA), also referred to as the finite element method (FEM), is a method for numerical solutions of field problems with the requirement of determining the spatial distribution of one or more dependent variables. A field problem is mathematically described by differential equations or an integral expression. The model or member created with the finite element method gets discretized into finite elements connected by nodes to create the assembly of elements called a finite element structure. The particular arrangement of elements is called a mesh and is numerically represented by a system of algebraic equations to be solved for unknowns at nodes. The typical advantages FEA provides contrary to other numerical analysis methods is its flexibility regarding [22]:

- No geometric restrictions and being applicable to any field problems whether it's in the topic of heat transfer, stress analysis, magnetic fields etc.
- No restrictions towards material properties to isotropy, boundary conditions or loadings.
- Easily improving the approximation of results by narrowing down and grading the mesh.

Finite element software offers powerful simulation capabilities enabling the user to accurately model and analyze complex structural behavior. Abaqus and FEM-Design are two examples of widely used FE-software programs that offer static, dynamic, and nonlinear analysis capabilities. With the aid of these software tools, the user can evaluate strength, stiffness and stability of complex structural systems like a suspension bridge and further assess their response to the relevant loads such as traffic, complex wind and seismic loads.

Abaqus is an extensively used FE-software in structural analysis and will be used to model and analyze the Lysefjord bridge. Abaqus is a general-purpose software, meaning it is very flexible and applicable for most types of cases and levels of complexity. However, this flexibility also

presents itself with more detailed work in advance and a better understanding of the finite element method to obtain the correct input and output.

FEM-Design is another widely used FE-software that provides advanced simulation capabilities in the form of static, dynamic, and nonlinear analysis and is shaped with more of a structural-engineering directed user interface. This software was used to aid the Abaqus bridge model with detailed geometrical data by modeling and studying the girder cross-section in detail. Several modifications of the model have been created to obtain more realistic and real-life data regarding stiffness and mass and to better understand the actual responses of the mass rotation along the girder's axes.

To learn the Abaqus interface and to be able to model the bridge, two previous models made by Tveiten [7] and Steigen [6] have been used as a basis. Some changes have been made according to the latest model by Tveiten, which will be discussed respectively in this chapter. Further on, the Abaqus Documentation has been frequently used for general instructions, keywords and explanations. The Abaqus student version 2022 has been used which is available on Windows platforms with the limitation of structural models up to 1000 nodes.

## **5.1 FEM-Design model of the bridge girder section**

In FEM-Design, there is the option to edit or create cross-sections to add to the section library and create 3D assemblies with the member created. This editor is called Section Editor and has been used to model the girder cross-section from scratch. Based on the given cross-section drawings of the bridge girder, a model has been created with accuracy according to the given geometry and some slight simplifications.

A basic model with three different modifications has been made: the girder alone, the girder with railings and asphalt, and the girder included with the diaphragm. Neither of the modified models contains any details of hanger connections; however, these details are included in mass calculations. The output of interest collected from these models is the moment of inertia values, the cross-sectional areas, and the location of the center of gravity and shear center. Due to asymmetry in the girder cross-section, it has been found that the shear center will not collide with the center of gravity, and the product of inertia will be nonzero. However, these effects are neglected for simplicity and due to collision with other modeling approaches made in this thesis.

After finishing the girder models, the software performed a finite element calculation to extract the necessary geometrical values. The calculation proceeds with an automatic mesh generation with suitable and average element sizes [23]. The software provides a handful of results; however, only the needed values for Abaqus have been considered.

### 5.1.1 The girder

The bridge girder was first modeled with uniform properties between the diaphragms without any external mass applied. This model's purpose is to extract the bending stiffness values and other relevant cross-sectional parameters that will be used as input for the Abaqus model, together with a calculated contribution from the diaphragm.

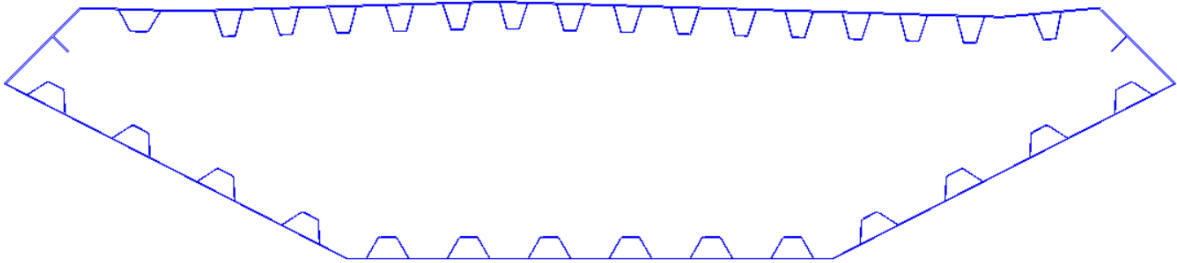
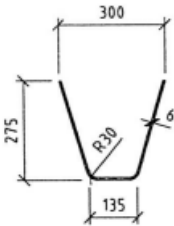


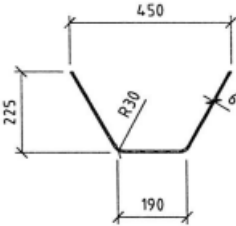
Figure 21 The girder cross-section, created in FEM-Design

The girder steel box section consists of a combination of welded and bent plates with a thickness of 8mm in the lower half and 12 mm in the upper half, starting at the slope plate that connects with the top plates. There are 30 trapezoidal stiffeners inside the steel box and two steel plates reaching 220 mm inside the box on each side. These stiffeners run continuously throughout the span length. There are two different types of trapezoidal stiffeners inside the girder:

**Type 1:**



**Type 2:**



*Figure 22 Dimensions of the trapezoidal stiffeners in the girder cross-section*

The type 1 stiffener have a spacing of 600 mm at the top plate, while the type 2 stiffener have a spacing of 1000 mm on the bottom slope sides and 850 mm horizontally on the bottom plate. Other than the difference in the geometry shown in Figure 22, they both have in common a given radius of 30mm in the plate intersection of the cross-section and 6mm thickness.

**5.1.2 Railings and asphalt**

Considering the torsional effects of the bridge girder, the external permanent masses on the girder were applied to the model to gain a more realistic picture of the mass moment of inertia. Since the cross-section consists of relatively complex geometry, it must be modeled as a homogenous material due to the limitations of the section editor module [24]. This means that the asphalt’s geometry needs to be edited to be equivalent to the steel’s density. In addition, the railing geometry has been edited for the planar view to represent the total mass along the 12m hanger span as continuous plates. This resulted in rather slim columns, which can be seen in Figure 23, as well as approximately two-thirds of a reduction in asphalt height.

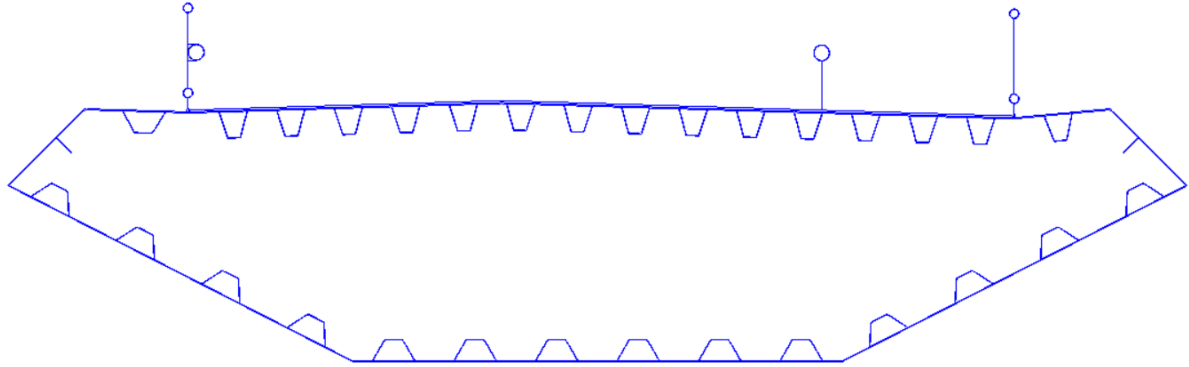


Figure 23 The girder cross-section included with modified railings and asphalt, created in FEM-Design

### 5.1.3 Diaphragm

The diaphragm is located at the position of each 4 m along the main span and under the hanger points and acts as stiffeners and support for the basic girder steel box. This model was created to obtain the stiffness contribution it provides to the girder.

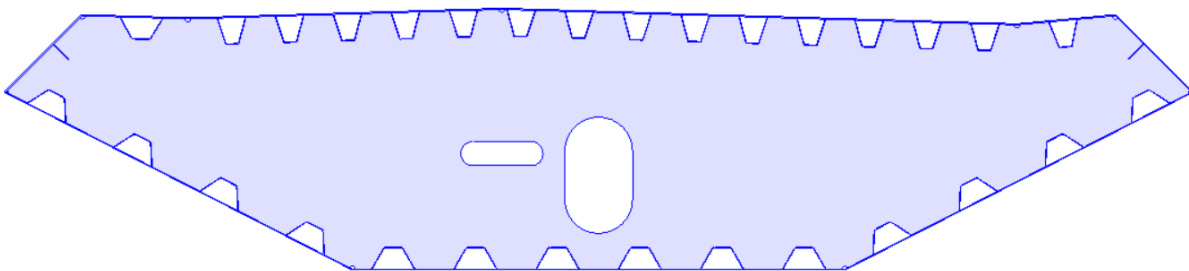


Figure 24 The girder cross-section included with the diaphragm, created in FEM-Design

### 5.1.4 Discussion

Based on these models, the following inertia values and cross-sectional areas have been calculated, shown in Table 3:

Table 3 Values of inertias from the FEM-Design analyses

<b>Stiffness data</b>				
	<b>The girder</b>	<b>Railings and asphalt</b>	<b>Diaphragm</b>	<b>Resulting values</b>
<b><math>A</math></b>	0.3824 m <sup>2</sup>	0.5792 m <sup>2</sup>	22.27 m <sup>2</sup>	0.42694 m <sup>2</sup>
<b><math>I_x</math></b>	0.4058 m <sup>4</sup>	0.5359 m <sup>4</sup>	10.49 m <sup>4</sup>	0.42678 m <sup>4</sup>
<b><math>I_y</math></b>	4.774 m <sup>4</sup>	6.204 m <sup>4</sup>	197.2 m <sup>4</sup>	5.1684 m <sup>4</sup>
<b><math>I_T</math></b>	0.9944 m <sup>4</sup>	1.209 m <sup>4</sup>	27.23 m <sup>4</sup>	1.04866 m <sup>4</sup>

The uniform girder model naturally contained the lowest stiffness and cross-sectional area. The model with railings and asphalt included showed that the cross-section has been given some increase in its stiffness and cross-sectional area. However, these increases are not representative of the girder's actual response considering traditional deflecting behavior but are used for calculating the mass moment of inertia, where externally fixed mass may play a role in the girder's response to change in rotation.

The values extracted from the diaphragm model show a very high increase in stiffness and cross-sectional area; however, the bulkhead is only 8 mm thick, and since it appears each 4 m between the hanger points, the increase in overall stiffness is relatively low. By dividing the thickness by its spacing, it is found that only an increase of 0.2% of these values is added to the previous two models. After evaluating the three models, representative values have been calculated for the use in Abaqus, shown in the "resulting values" row in the table above. These resulting values are used to define section properties for the girder itself, but when considering the mass points, which will be further explained later in this thesis, some of these values will be edited.



## 5.2 Abaqus model of the bridge

To develop an accurate and reliable computational model for the bridge, Abaqus was decided to be used, mainly due to previous existing models, which could be used as guidance and basis, and due to the possibility of carrying out dynamic analyses. The model's primary focus is to evaluate the structural behavior and response under heavy traffic loading associated with the recorded vibrations of the bridge in real-life to further assess the stress resultants and bridge motion in detail.

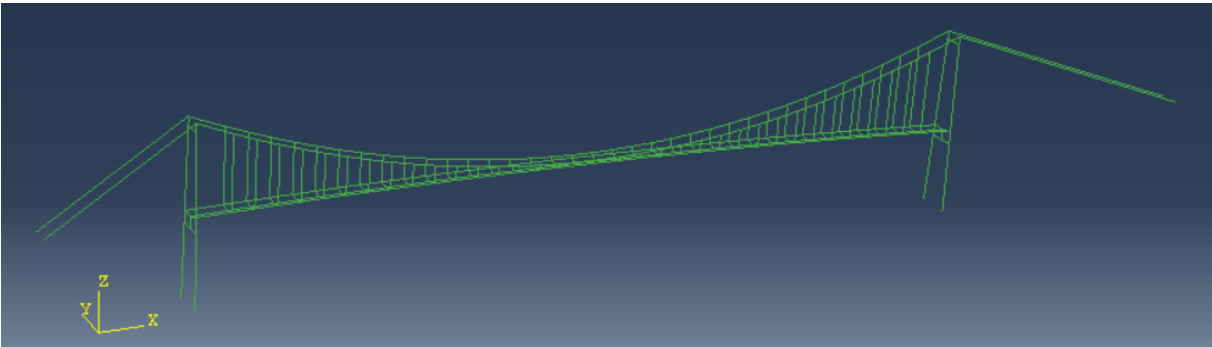


Figure 25 The bridge model created in Abaqus, Visualized in Abaqus CAE

The Abaqus model is constructed of three-dimensional beam elements directly related to the library of the finite element method, which is connected between predefined nodes. This approach was decided due to the aspects of the bridge in interest being on a global scale; hence the bridge has been modeled directly in its mesh by using the coordinates of all structural components in a reasonable discretization.

The viaducts of the bridge are not included in this model as they are considered as self-equilibrium sections of the bridge and have little secondary contribution to stiffness, considering the towers.

## 5.2.1 Global geometry

The bridge drawings have been relied upon for their projected elevations and curvature trajectory (ref. Figure 26) when collecting the global geometry of the various structural components.

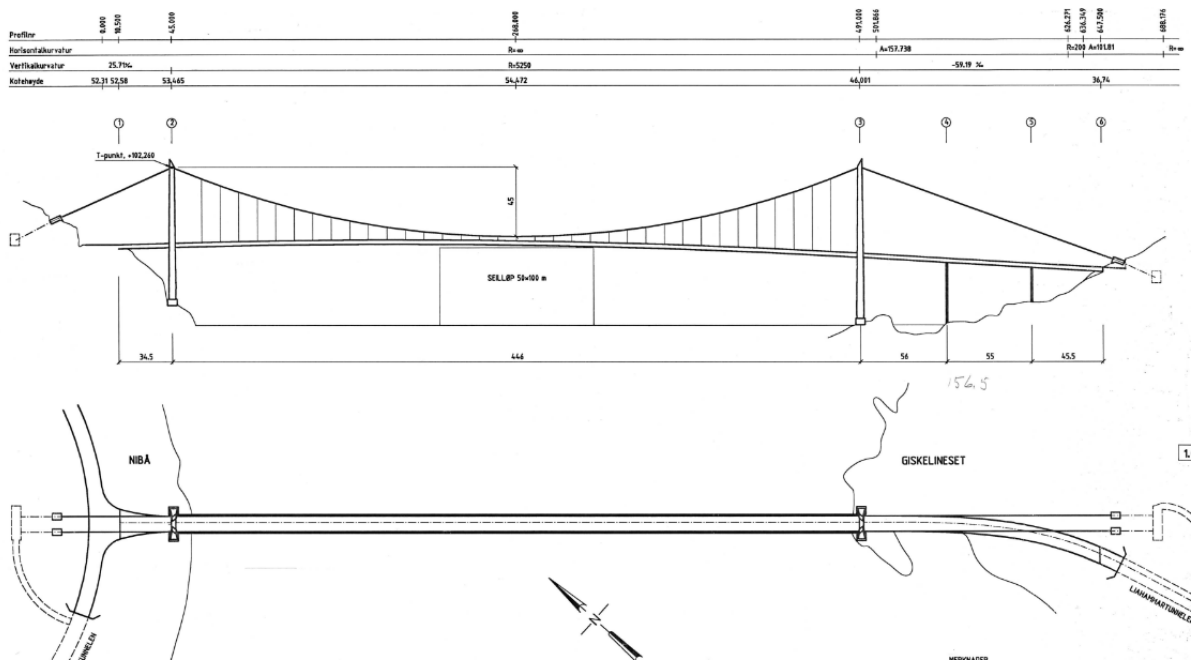


Figure 26 Plan and section of the Lysefjord bridge

Based on the given quotas, the bridge has been appointed a global right-handed, rectangular Cartesian coordinate system placing the origin in the far-left corner of the drawing. This places the majority of the structure in positive coordinates with the exception of some parts of the northern backstay cables, shown in Figure 25.

The geometry shown in Figure 26 illustrates the configuration of the structure once all the structural dead-loads have been applied. To account for the deflection at each component resulting from the dead-loads acting on the entire bridge structure as a single unit, the deflection has been calculated and adjusted at each relevant node. This methodology ensures that the structural components conform to their intended geometry following these displacements.

### 5.2.1.1 Bridge girder

The curvature of the bridge girder is considered to represent a segment of a circle with a radius of approximately 5.25 km and is placed globally unsymmetric in relation to the main span of the bridge. Within the main span, the girder has been appointed the coordinates (45, 53.465) at the north tower connection, (268, 54.472) at the middle of the span, and (491, 46.001) at the south tower connection. The coordinates are related to the elevation above sea level in the y-coordinate and the distance from the north beginning of the viaduct in the x-coordinate. The coordinates are then used to calculate the equation of the intended fictitious circle to obtain the correct coordinates for the bridge girder within the boundary area of 45 m and 491 m to cover the main span. This method differs from the one Tveiten and Steigen did as they used a parabolic curve approach with the origin in the middle of the span. Figure 27 shows the basis for the circle equation derivation, followed by the final equation for the girder. These calculations can be found in Appendix B

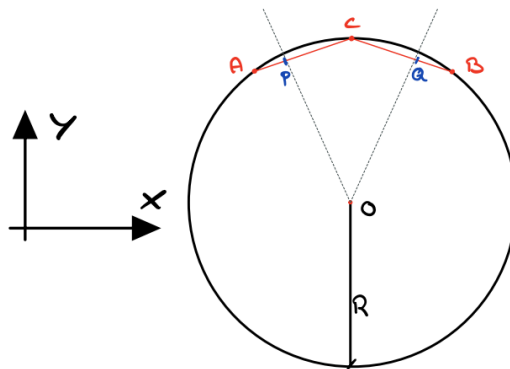


Figure 27 Basis for the circle equation derivation

$$(x - x_0)^2 + (y - y_0)^2 = r^2$$

$$\rightarrow (x - 180.208)^2 + (y + 5196.149)^2 = 27576728.452$$

$$\rightarrow y = \sqrt{-x^2 + 360.416x + 27544253.529} - 5196.149$$

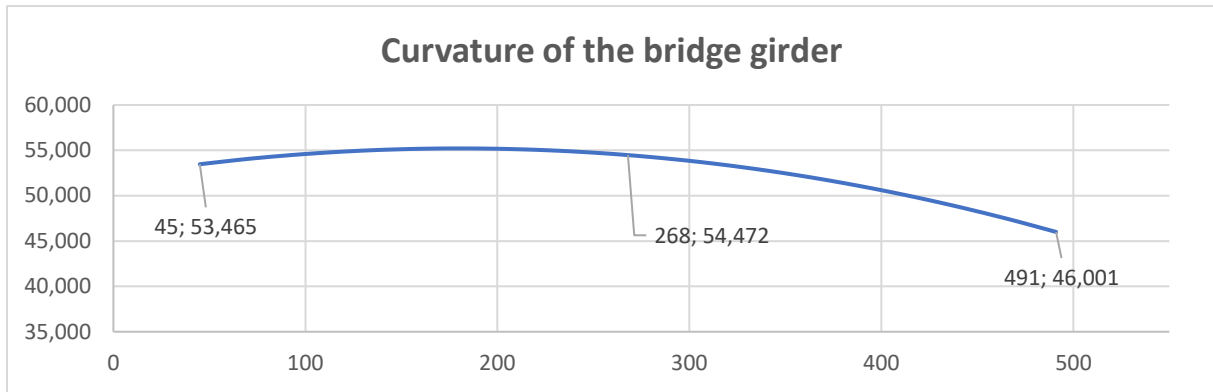


Figure 28 Diagram of the bridge girder coordinates

### 5.2.1.2 Cables

The main cables are symmetrically placed over the main span reaching 102.26 m above the sea level at each tower connection, along with a sag of 45 m in the middle of the main span. The equation for the shape of the main cable is simplified into a parabolic shape and calculated based on the given quotas.

$$y = ax^2 + bx + c$$

$$\rightarrow y = \frac{45}{49729}x^2 - \frac{90}{223}x + 102.26$$

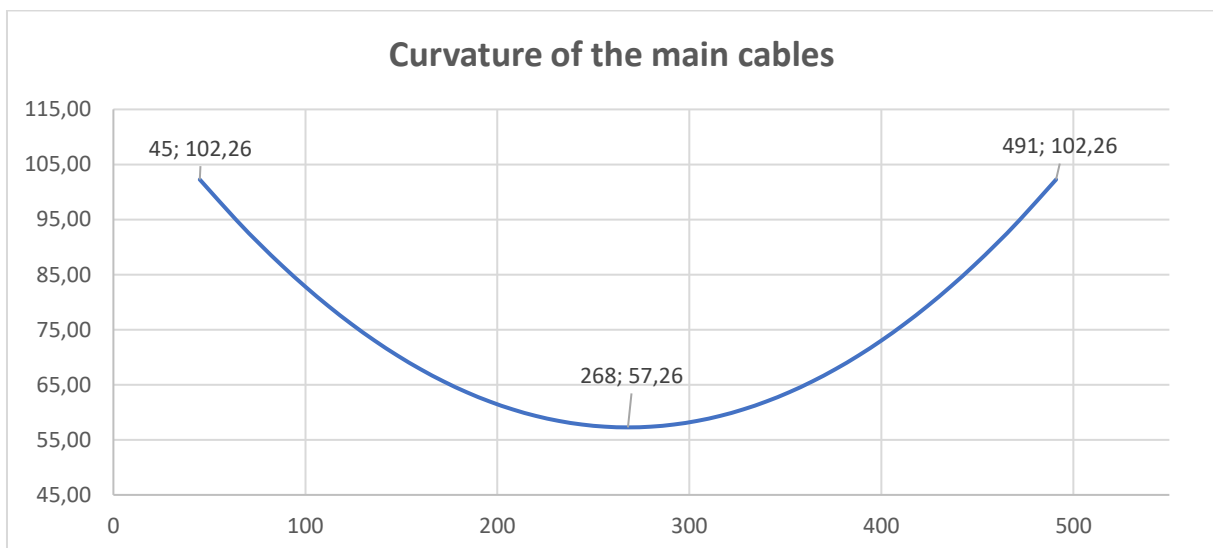


Figure 29 Diagram of the main cable coordinates

The backstay cables are anchored approximately 166 m away from the south tower and 74 m away from the north tower. These will be constructed as pure tension, straight beam strings in the model.

Figure 30 below provides a better view of the unsymmetric relation between the bridge girder and the main cable. It clearly shows that the highest point of the girder does not interact with the lowest point of the cable, which may introduce a handful of calculations on unsymmetric components for the load-transferring system. These effects are neglected in this thesis.

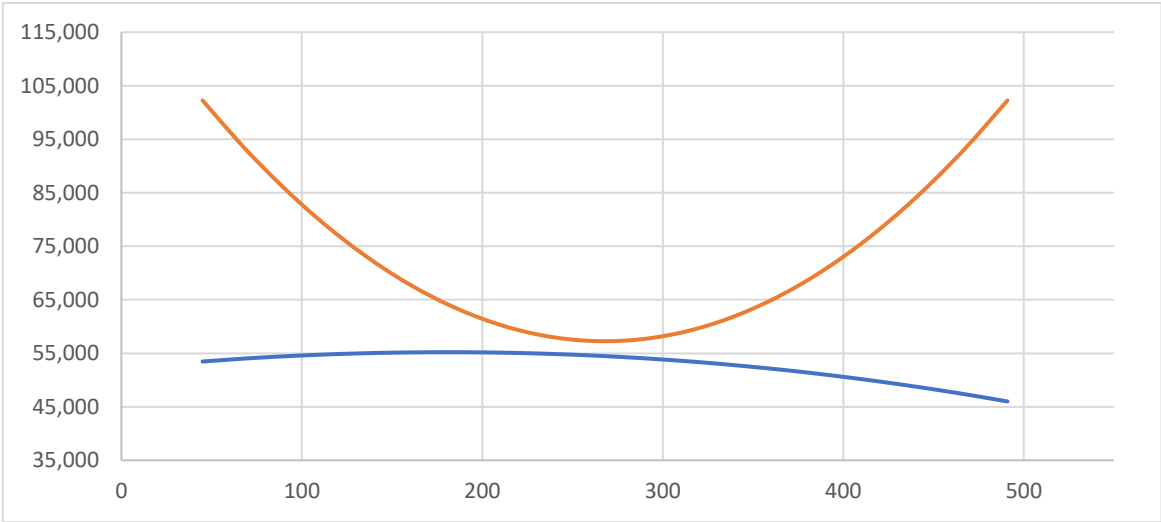


Figure 30 The shape and location of the main cable in relation to the shape and location of the bridge girder

### 5.2.1.3 Towers

The towers reach from the elevation level of +6.5 m to approximately +101 m above sea level on the north side and from +0.5 m to +101 m on the south side. The towers are tapered along their height in both directions. However, only the y-direction will obtain the tapered shape due to a string-model approach. The towers have been discretized into 30 nodes and 29 elements to capture a more detailed distribution of output response along its height and more accurate global stiffness contribution. Each tower contains two crossbeams, one supporting the girder and one close to the top ends. The crossbeams are modeled with seven nodes and six elements each. The difference in total length of the two towers is 6m, and the lower crossbeam is

approximately 7 m lower on the south side in relation to the north side. With respect to maintaining the same number of elements, this difference is considered by making the elements slightly longer on the south side until they reach the lower crossbeam. Further, the element lengths on both sides become more or less alike.

The towers play a significant role in determining the overall stiffness of the bridge at a global level. However, the cross-sectional data are taken from Tveiten's model, whereas the choice of data is assumed to have little impact on the stiffness contribution. This means that the elastic modulus is not corrected for a concrete section in stadium 2, even though load factors are not included in the loads of interest which are the service loads.

#### **5.2.1.4 The overall numbering system of nodes and elements**

The numbering system and distribution of nodes and elements are mainly collected from Tveiten and are created to correlate and easily recognize elements with belonging nodes. On the top of Figure 31, the system of nodes for the main cable and along the neutral axis on the girder is shown, with the notation "West-side node/East-side node". The elements for the cable are numbered correspondingly, starting from the north side. The backstay cables are created using node generation commands in Abaqus from the predetermined anchorage points and to the top of the main span cable. This resulted in 24 nodes on the south side and nine nodes on the north side. Additionally, the elements for the backstay cables are created similarly using element generation commands.

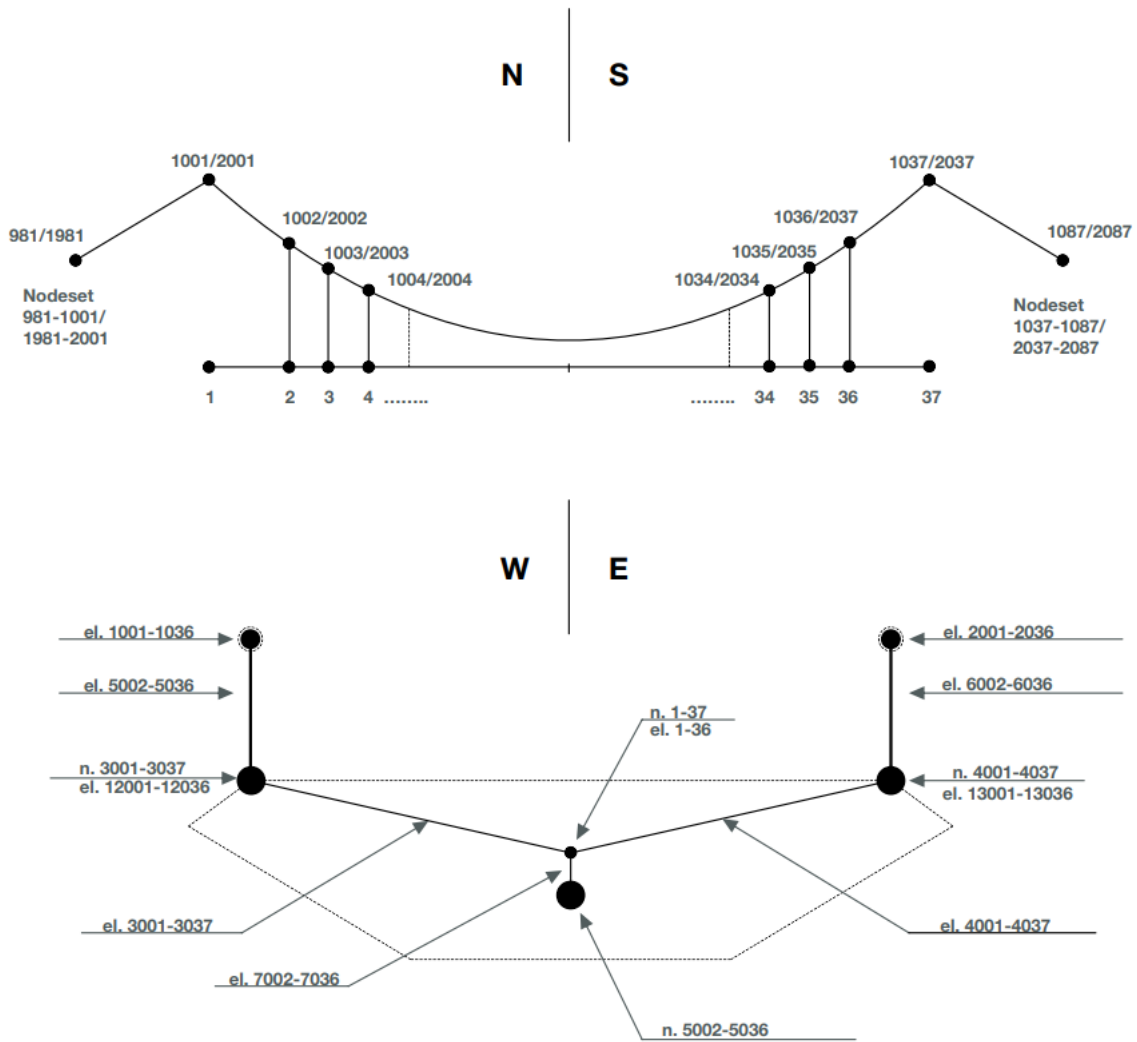


Figure 31 Numbering system and pattern for nodes and elements of the cables and girder

On the bottom of the above figure, the approach for the girder system in Abaqus, along with the hangers, are labeled with the element- and node numbering in the western and eastern parts. The bridge girder approach is explained in more detail later in chapter 5.2.2.3. The girder's neutral axis (nodes 1-37) is the center for connecting a series of dummy elements and nodes representing the girder in total. The notation of nodes and elements on the same arrow represents the numbering throughout the cross-section downwards in the plane.

The numbering of nodes for the towers is according to Figure 32, with the element numbering system following the same pattern starting from the bottom. The figure presents the north

tower, whereas the south tower follows the same system starting from 30001 and 30101 and further on. The crossbeams are shown in more detail with node and element numbering. There are also two top elements connecting the tower with the cables and a connecting element from the lower crossbeam and the bridge girder on each tower. These, and all other information about the numbering system, is presented in the input file for the Abaqus model found in Appendix E.

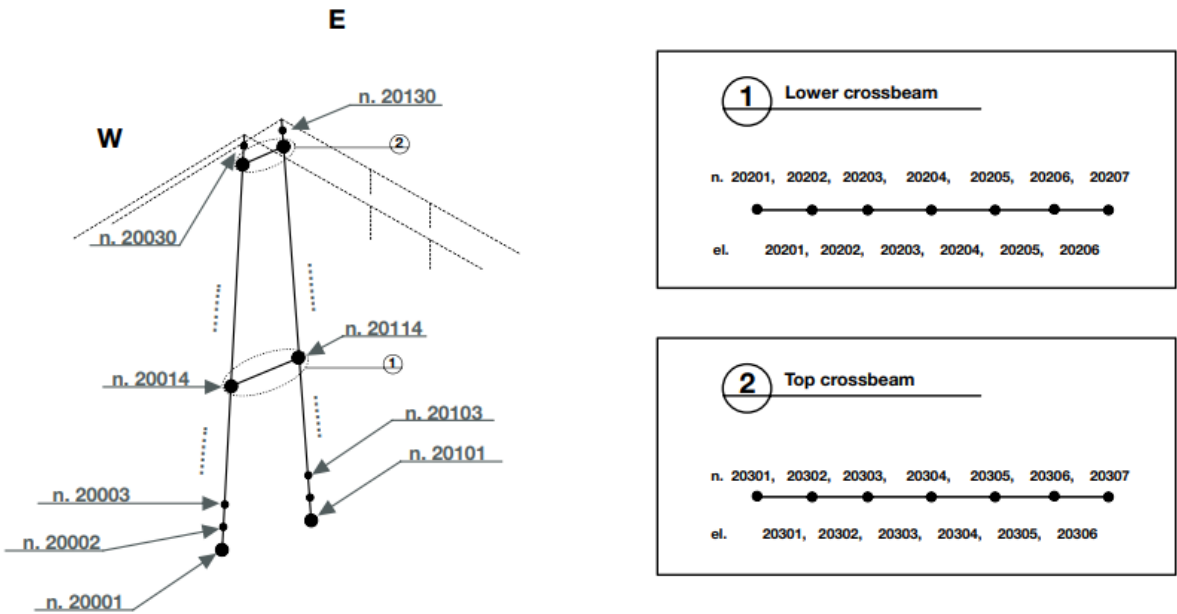


Figure 32 Numbering system and pattern for nodes of the towers

### 5.2.2 Abaqus model input

After obtaining the global geometry, it is organized in a text editor to create an input file suitable for generating the model geometry to be analyzed using Abaqus.

The Abaqus input file layout comprises two distinct sections: the Model and History data. The Model data is the section for creating and consolidating all the geometry and section assignments. On the other hand, the History data section consists of the steps, loads, and boundary conditions. Chapter 5.2.3 presents the input for the History data section developed for this project.



Both the Model data and History data sections contain keyword lines, data lines, and comment lines. The keyword and data lines are two dependent and essential components for the Abaqus input file. The keyword line specifies the command to be executed and its required and optional input information. The data lines furnish information such as coordinates, element connection and patterns, or tables of material properties. When creating the input file, the user should, to a high degree, supplement with the Abaqus documentation for descriptions and execution manuals of the keyword lines and their corresponding data lines.

### **5.2.2.1 Elements**

Abaqus offers a wide range of options for beam modeling. The beam theory is the one-dimensional approximation for the three-dimensional continuum, which directly results from slenderness assumptions that the dimensions of the cross-section are small compared to typical dimensions along the beam axis. Along this axis, deformations such as axial stretch, change in curvature and torsion is represented as stiffness along the one-dimensional line element in the three-dimensional space. The advantage of using the beam elements is that they are geometrically simple with few degrees of freedom [25].

The bridge girder comprises 36 elements with nodal locations at each hanger connection. These elements are represented as Timoshenko B31 beam elements, with the notation B31 meaning that the element is a linear beam element in space. Figure 33 shows the options for modification for this beam element. The decision behind using the Timoshenko B31 elements was mainly due to its ability to accurately capture the bending and shear deformation of thin-walled members.

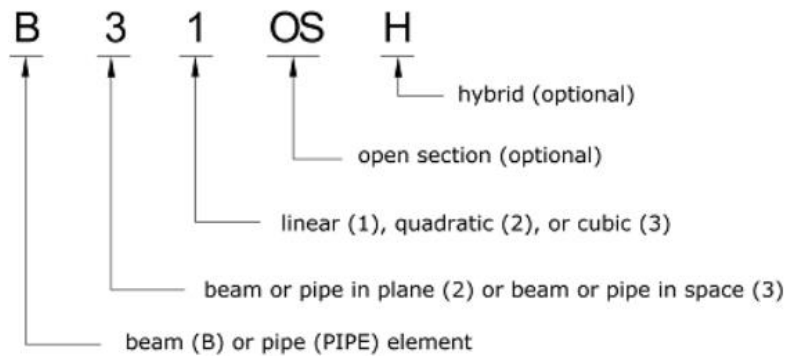


Figure 33 Options for modification of beam elements in Abaqus, from [26]

Additionally, the Timoshenko beams can be subjected to large axial strains and axial strains due to torsion are assumed to be small, making them well suited for the cable-hanger system as they should be assumed to only work by pure axial forces [26]. The cables and hangers have therefore been appointed the B31 elements as well. With this approach, the hangers have also been given small-to-none values of stiffness and density to make them act as pure load-transferring rods. The main cables retain their density but are appointed extremely low bending stiffness.

The dummy elements have been appointed the B31 elements in the same fashion as the neutral axis. However, the connecting element between the lower crossbeam and the end nodes of the girder has been given the Euler-Bernoulli beam element B33. The Euler-Bernoulli beam elements uses cubic interpolation functions, making them reasonably accurate for cases involving distributed loading along the beam.

The towers were initially modeled as the frame element FRAME3D, typically used for small-strain elastic or elastic-plastic analyses of frame-like structures of slender, initially straight beams [27]. However, it was noticed during the dynamic part of modeling and analyzing that these frame elements contain restrictions regarding the definition of damping coefficients. Due to this issue, the tower's FRAME3D elements were replaced with the Timoshenko B31 elements, which had been observed to be used in other bridge models and thereby assumed to be a sufficient approach.

### 5.2.2.2 Properties

Table 4 summarizes most of the material and geometrical properties used for the bridge model. Various stiffness values for the towers are listed in Appendix E since they vary for each element due to varying cross-sections in each element set.

Table 4 Material and geometrical properties for the bridge

<b>Globally</b>	Main span	446 m
	Sag	45 m
	Transverse cable span	10.25 m
	Distance between hangers	12 m
<b>Bridge girder</b>	Elastic modulus, $E$	$210 * 10^3 \text{ MPa}$
	Second moments of inertia, $I_x, I_y$	$0.42678 \text{ m}^4, 5.1684 \text{ m}^4$
	Torsional constant, $I_T$	$1.04866 \text{ m}^4$
	Polar moment of inertia, $J$	$5.59518 \text{ m}^4$
	Warping constant, $C_w$	$4.762 \text{ m}^6$
	Weight, $w$	$5350 \text{ kg/m}$
	Area, $A$	$0.42694 \text{ m}^2$
	Shear modulus, $G$	$80.7 * 10^3 \text{ MPa}$
	Coefficient of thermal expansion, $\alpha$	$0.00001 \text{ K}^{-1}$
<b>Main cable</b>	Elastic modulus, $E$	$180 * 10^3 \text{ MPa}$
	Second moment of inertia, $I$ (one out of 12 cables)	1% of a circle with outer diameter 97 mm
	Area, $A$ (set of 6 cables)	$0.044 \text{ m}^2$
	Shear modulus, $G$	$80.7 * 10^3 \text{ MPa}$
	Coefficient of thermal expansion, $\alpha$	$0.00001 \text{ K}^{-1}$
<b>Hangers</b>	Elastic modulus, $E$	$180 * 10^3 \text{ MPa}$
	Second moment of inertia, $I$	1% of a circle with outer diameter 48 mm
	Area, $A$	$0.0018 \text{ m}^2$
	Shear modulus, $G$	$63.077 * 10^3 \text{ MPa}$
	Coefficient of thermal expansion, $\alpha$	$0.00001 \text{ K}^{-1}$
<b>Towers</b>	Elastic modulus, $E$	$40 * 10^3 \text{ MPa}$
	Shear modulus, $G$	$16.7 * 10^3 \text{ MPa}$

### 5.2.2.3 Mass distribution

The original model for the bridge from the NPRA was created with the FE-software Alvsat. Many of the input values used in the original model have been inherited to other previous models as well as the model created in this thesis. Mainly, the masses of the girder, hangers, and main cables are collected directly from the Alvsat model. The mass of the towers is

calculated based on their geometry and discretization by Steigen. Masses calculated based on Alvsat values and Steigen’s tower calculations are listed in Table 5.

Table 5 The masses of the bridge components in focus, from [6]

Components	Mass	Total
Bridge girder (main span)	2386 tons	2750 tons
Main cable (main span)	364 tons	
Backstay cables	181 tons	181 tons
Towers	7919 tons	7919 tons
<b>Total</b>		10850 tons

The results from Abaqus give a 2% increase in total mass compared to the theoretical values from the table above, with a value of 11030 tons. The calculations of the towers could cause this difference as they are calculated based on average cross-sectional area, and perhaps Abaqus calculates each element's weight as different [6]. However, the two masses are very similar and in the same order of magnitude, which is the most important factor.

**Mass moment of inertia**

The mass moment of inertia is a critical factor to consider in order to obtain correct eigenfrequencies and displacements and is referred to as a measure of the body’s resistance to angular acceleration. The equation for the mass moment of inertia can be expressed as:

$$I_m = m \int r^2 dA$$

To rearrange this, the mass moment of inertia may be expressed as the polar moment of inertia, J, multiplied by the density of the material, ρ. This approach is slightly simplified for the case of the bridge girder because it usually applies to homogenous bodies having axial symmetry. However, the method is assumed to be appropriate due to assumptions and modeling techniques used in FEM-Design.

The polar moment of inertia describes the resistance of the body's cross-section to twisting about a reference axis, or in other words; it is the quantity used to describe resistance to torsional deformation. This is visualized in Figure 34 with a twisting motion about the dashed line representing the reference line.

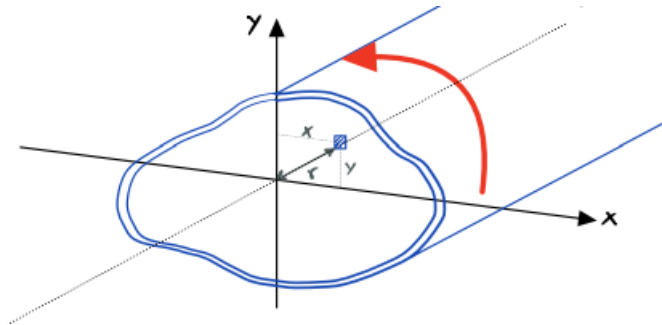


Figure 34 An arbitrary body exposed to twisting about a reference axis (dashed line) perpendicular to the plane of the cross-section

The perpendicular axis theorem states that:

$$J = \int r^2 dA = \int (x^2 + y^2) dA = I_x + I_y$$

Using this relation, the planar moments of inertia, which are used to describe resistance to deflection (bending), can describe the polar moment of inertia. Hence the reasoning behind extracting these values with accuracy from FEM-Design.

$$\begin{aligned} I_m &= \int_m r^2 dm = \int_V r^2 dV \text{ [kgm}^2\text{]} \\ &= \rho \int r^2 dA \text{ [kgm}^2\text{/m]} \end{aligned}$$

$$I_m = \rho * J = \rho * (I_x + I_y)$$

The planar moments of inertia are collected from the girder model with railings and asphalt, together with the contribution of the diaphragm. The thought process behind this approach is that the fixed masses not included in the cross-section should contribute either with or against the angular deflection. Therefore, an increase in stiffness is applied, leading to an increase in the mass moment of inertia. If the hanger connections were included in the FEM-Design models, the increase would be larger.

$$I_m = \rho * (I_x + I_y) = 7800 \frac{kg}{m^2} * (0.53587m^4 + 6.20353m^4) = 52567.367 \frac{kgm^2}{m}$$

*Contribution from the diaphragm:*

$$1619982 \frac{kgm^2}{m} * \frac{8mm}{4000mm} = 3239.964 \frac{kgm^2}{m}$$

$$\rightarrow I_m = 52567.367 + 3239.964 = 55807.331 \frac{kgm^2}{m}$$

The mass moment of inertia was initially calculated to be 82430 kgm<sup>2</sup> in Alvsat. The reasoning behind this difference can perhaps be due to the absence of specific components in the FEM-Design models when extracting the inertias. Another possibility is that the calculations behind the Alvsat values also consider the cables to contribute to mass and stiffness as a unified global system. It is concluded that a combination of both possibilities is the most likely reason behind the difference. This assumption can also be supported by TVEITEN's calculations when a value of 74544 kgm<sup>2</sup> is obtained by including the cables and 53111 kgm<sup>2</sup> is calculated when omitting them.

### **Mass point calculation**

The cross-section of the bridge girder is somewhat complex and time-consuming to model in Abaqus as well as it introduces the increased possibility of errors. In order to simplify this process, a three-point mass system is introduced. These three mass points will represent

crucial points of interest that must be considered. For this system, two of the masses will act as both nodal connections for the hangers and for the longitudinal edge of the bridge to apply transverse loading and connecting longitudinal response. The last mass point will act as a position for collective mass under the shear center, as well as the determining factor for the girder’s behavior of rotational change described by the mass moment of inertia,  $I_m$ . Overall, this arrangement is considered governing for eigenfrequencies and for static and dynamic analyses. The mass points are referred to as dummy nodes for the Abaqus input file, and their connecting elements are likewise referred to as dummy elements.

From the calculations in FEM-Design, the axes relevant to the mass distribution calculations are found and summarized in Figure 35. The axes needed are the neutral axis (N.A.), the center of mass (C.M.), and the shear center (C.S.). The neutral axis was not extracted from FEM-Design, but instead collected directly from previous models, which all seem to agree about this.

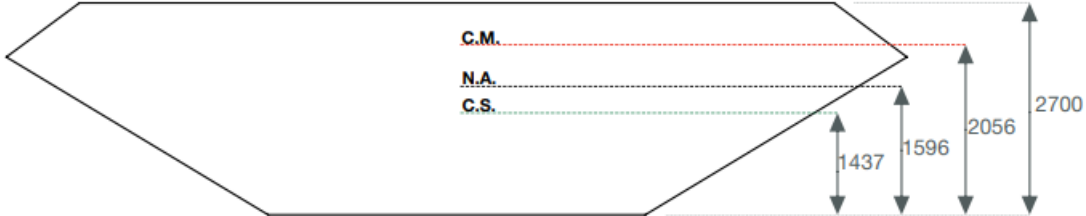


Figure 35 Location of the center of mass (CM), neutral axis (NA) and the shear center (CS) for the girder cross-section

When determining locations and magnitudes of the masses, three equations of equilibrium have been considered:

- The sum of mass moment of inertia = 0
- The sum of all masses = 0
- The mass center to be preserved.

These equations have been solved considering the shear center as the reference point, which lays 0.159 m below the neutral axis. This must also be considered in the modeling phase, as the reference nodes in Abaqus are drawn along the neutral axis.

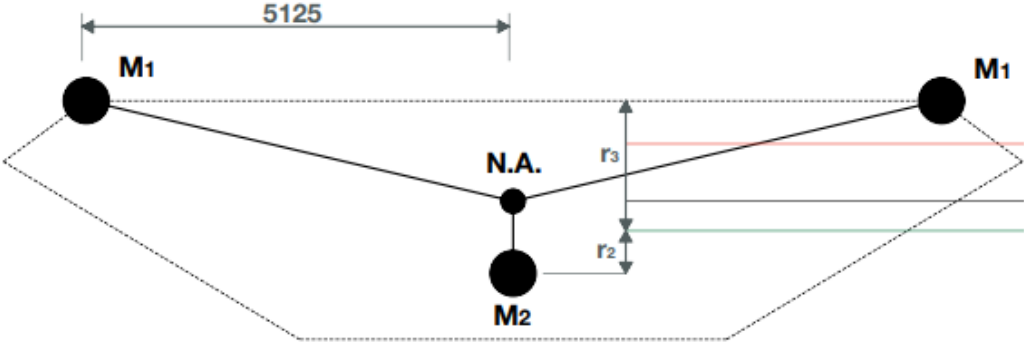


Figure 36 The "three mass-point system" approach used for girder modeling

$$M_2 r_2 + 2M_1(r_3^2 + 5.125^2) - I_m = 0$$

$$2M_1 + M_2 = 5350 \text{ kg/m}$$

$$\frac{2M_1 * 2.7 + M_2(1.437 - r_2)}{5350} = 2.056$$

The equations above display the equilibrium equations for the three mass point system calculations. The total girder mass of 5350 kg/m is a combination of the girder and half of the hanger weight, while the other hanger weight is summarized in the main cable weight. The set of equations was solved in MATLAB, presenting with  $M_1 = 998.2 \text{ kg}$ ,  $M_2 = 3353.6 \text{ kg}$  and  $r_2 = -0.236 \text{ m}$ . The results can be confirmed to be true by considering the requirement  $2M_1 + M_2 = 5350$ , which is the case for the present results as well. Should the girder's rotational response be studied further by other views or approaches, these equations can easily be edited and reused.



### **5.2.3 Abaqus history input**

As previously mentioned, the History data is the section covering all load cases and boundary conditions underlying the determined steps. Boundary conditions and structural dead loads are mainly collected from Tveiten's model, as they seemed reasonable and unchanged for the present case.

#### **5.2.3.1 Boundary conditions**

The following boundary conditions have been appointed to the model:

##### **Towers**

At the ground:

- All axes fixed for displacement.
- All axes fixed for rotation.

Connection to the cables:

- Nodes made hinged with release of all rotational degrees of freedom. This is done by the command RELEASE to release a rotational degree of freedom or a combination of rotational degree of freedoms at one or both ends.

##### **Backstay cables**

Anchorage at the ground:

- All axes fixed for displacement.
- All axes free for rotation.

##### **Girder**

The coupling between the lower cross beam and the bridge girder have been modified using the command RELEASE. Connection between girder and lower cross beam:

- North side
  - All axes fixed for displacement.
  - X-axis fixed for rotation.
  - Y- and Z-axis free for rotation.
- South side
  - X- and Z-axis modeled as a truss element connecting the cross beam and the girder for displacement.
  - Y-axis fixed for displacement.
  - X-axis fixed for rotation.
  - Y- and Z-axis free for rotation.

In addition, multi-Point constraints (MPC) commands have been used to manipulate the active degrees of freedom to connect the cross beams and the tower legs. With the aid of the command TIE, all active degrees of freedom are made equal at two nodes. These boundary conditions are considered as the fixed conditions remaining, no matter the loading situation. Therefore, they have been applied previously to the step definitions, placing them in Abaqus's initial step that remains unchanged throughout all further steps.

### **5.2.3.2 Structural dead load**

When creating steps for the structural dead loads of this bridge, the behavior of the different main components modeled needs to be considered. This means that the dead loads need to be created in a two-step fashion. Firstly, the girder, the main cables, and the towers are assigned with their belonging dead loads. Further, the backstay cables are considered alone as a continuation of the previous step to create pure tension forces that will pull the relevant weights from the first step. In this way, the bridge's structural components should be represented as they are assigned in real-life.

The main cable's dead load is considered using Abaqus's inbuilt gravity command, which calculates the loads based on the defined density of the material and the gravitational

acceleration. Since half of the hanger weight is applied to the density of the cables defined in the section properties, the gravitational acceleration is modified to gain the dead load for the cables alone. This resulted in a gravitational acceleration of  $8.5595 \text{ m/s}^2$ . The dead load of the bridge girder is presented as a distributed line load with its weight applied in Newtons due to a modified density approach in the section assignment. The towers are also applied with the gravity command to gain the dead load. Here, the gravitational acceleration remains at  $9.81 \text{ m/s}^2$  due to no modifications in the density. Like the towers, the backstay cables gain their dead load by unchanged gravity.

### **5.2.3.3 Eigenfrequencies and eigenmodes**

In order to extract eigenfrequencies with their corresponding modes in Abaqus, a frequency requested step is created by the keyword command \*FREQUENCY. When creating this step, the number of modes to be extracted must be predefined. This led to a trial-and-error methodology to obtain sufficient modes for the desired vertical, horizontal and torsional effect. Abaqus uses the Lanczos Eigensolver as default for this analysis because it has the most general capabilities [28].

The procedure of the frequency extraction in Abaqus involves the following [28]:

- Using a linear perturbation procedure.
- Performs eigenvalue extraction to calculate the natural frequencies and the corresponding mode shapes of a system.
- Includes effects from load stiffness and initial stress due to preloads and initial conditions if geometric nonlinearity is accounted for in the base state. This is so that the software can model small vibrations of a preloaded structure.
- Computes residual modes if requested.
- Solves the eigenfrequency problem only for symmetric mass and stiffness matrices; the complex eigenfrequency solver must be used if unsymmetric contributions, such as load stiffness, are required.

#### 5.2.3.4 Simulated dynamic loading: Traffic-induced acceleration

As previously mentioned, tri-axial accelerometers were installed on the bridge in 2013. From these sensors, vibrational response data from traffic has been collected and applied as a “load” input to the bridge numerical model. The data made available for this work contained a set of ten consecutive 10-minute data with a sampling frequency of 50 Hz. This results in 30 000 data samples per 10-minute period of measuring. If those data were to be applied directly in Abaqus, the analysis would end up being extremely costly in terms of computational time and storage as well as both time-consuming and troublesome to review. In light of this, a specific period with a clear traffic-induced response from all the available measurements has been extracted based on a visual inspection. This segment was then decimated by a factor of 2, i.e., reduction of sampling frequency to 25 Hz, with the help of a resample function in MATLAB. Figure 37 shows vertical acceleration at the eastern sensor inside the bridge deck at hanger number 18 (H18\_E) throughout all ten samples of the 10-minute data. This plot was used for visual inspection when deciding what segment to extract.

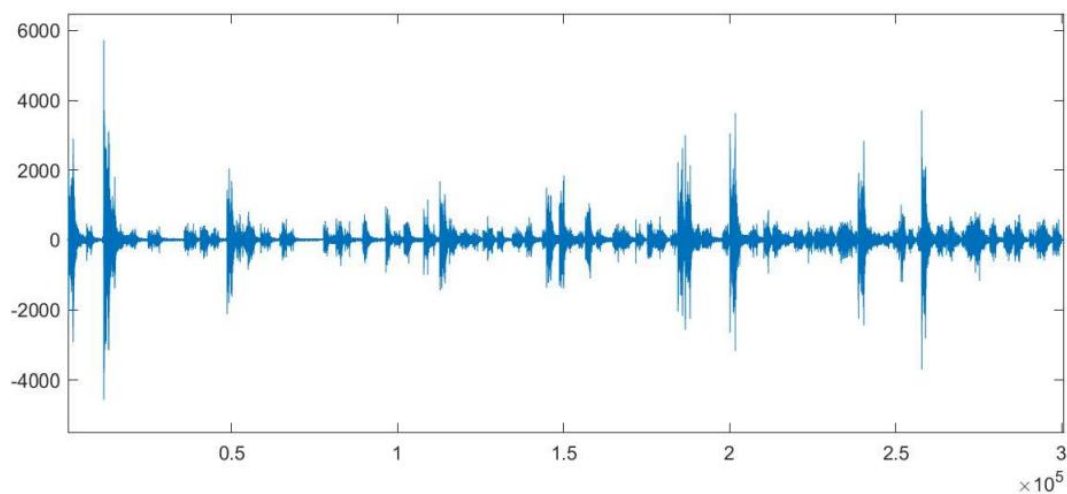


Figure 37 Vertical acceleration recorded on the H18\_E accelerometer during a 100 minutes long interval. The x-axis gives a sample number and the y-axis acceleration in  $\mu\text{g}$

As the figure above shows, there are several periods with acceleration of negligible value. These values are not desired in such an analysis as intended, since they would only create large data files with costly simulations and most probably negligible response. Therefore, it is more

efficient to limit the analysis to the shorter events with significant vibrations, triggered by some sort of heavy vehicle.

The segment extracted for this purpose is at the location between sample numbers 11001 to 16000, shown with and without the resampling in Figure 38. In Figure 37, this corresponds to the event with the largest acceleration. The resampling of the time series did however introduce some smoothing of the amplitudes, but this was considered to be of little importance for the present analysis, and the peak values remained more or less the same. It was also considered that these peaks would contribute the most to the output response. The selected time interval resulted in a series of 5000 samples, a number which was reduced to 2500 after the resampling.

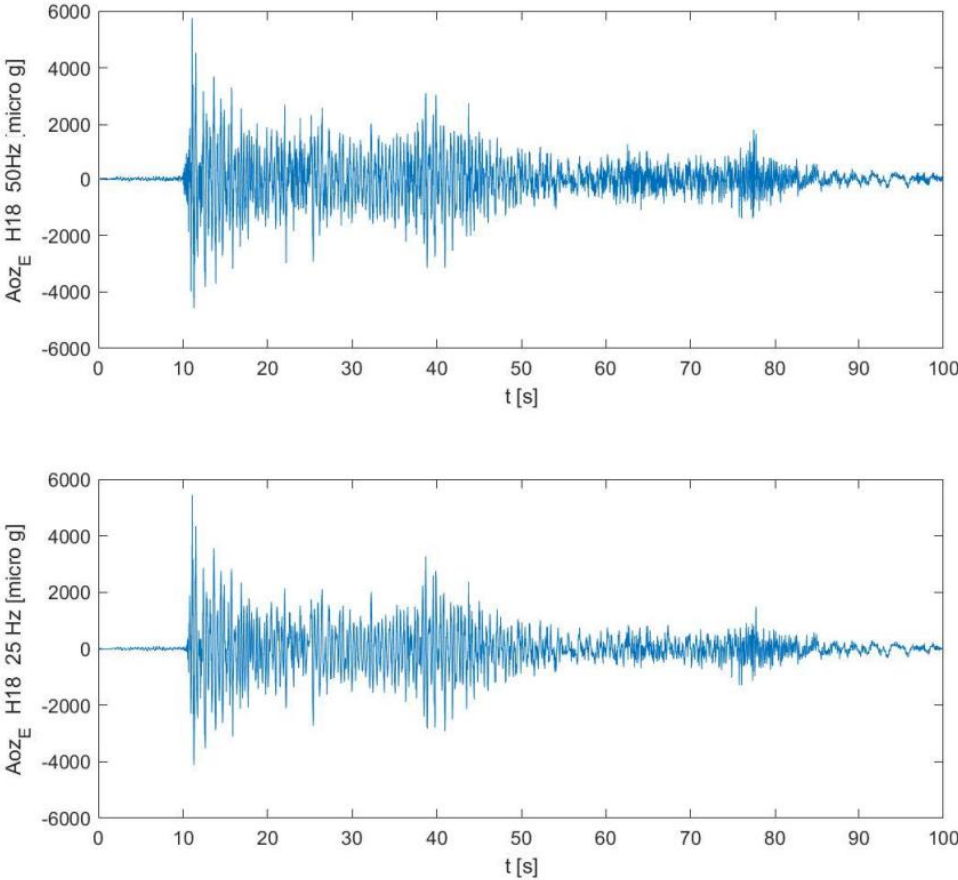


Figure 38 Vertical acceleration of the bridge deck (H18\_E) during the event selected for the present analysis, data from 07.03.2017

When applying such a load case to Abaqus, a dynamic step must be created. Abaqus offers in general two different dynamic applications: Implicit dynamic analysis and Explicit dynamic analysis. These solvers are directly based on the two different FEM dynamic analysis approaches, explicit and implicit, hence the naming.

The main difference between these two numerical solvers is that for the implicit method, equilibrium is enforced between applied loads and later generated reaction forces at every solution step, also referred to as the Newton-Raphson method [29]. Whereas for the explicit method, there is no enforcement of equilibrium. Due to this, the explicit method must be conducted by minimizing its deviation from equilibrium to almost zero by reducing the time steps to increase the number of solutions to achieve accuracy, likewise to the implicit method. The explicit solver uses a large number of very small time increments to trace the analysis solution making explicit solvers conditionally stable and will not face convergence problems like implicit solvers. The implicit method is incremental and iterative and is typically more costly in terms of computational service. The explicit method is only incremental and shows excellent cost savings over the implicit method as the size of the model increases [29].

Table 6 Main differences between Abaqus/Standard and Abaqus/Explicit, from [29]

	<i>Abaqus/Standard</i>	<i>Abaqus/Explicit</i>
<b>Element library</b>	Extensive element library	A subset of those available in Abaqus/Standard
<b>Analysis procedures</b>	General and linear perturbation	General procedures
<b>Material models</b>	Wide range of material models	Similar to those available in Abaqus/Standard
<b>Contact formulations</b>	Robust capacity	Solves even the most complex contact simulations
<b>Solution technique</b>	Stiffness based solution and unconditionally stable	Explicit integration solution and conditionally stable
<b>Disc space &amp; memory</b>	Can be large	Typically much smaller

There is normally a clear distinction when deciding whether to use the implicit method underlying the Abaqus/Standard or to use the explicit method in Abaqus/Explicit. Typically, Abaqus/Standard is more efficient when studying structure dynamics of nonlinear problems with rather smooth or slow development, such as low-frequency response, vibration, and oscillation. While as for Abaqus/Explicit, the solver is more suited for wave propagation and high-speed high-force dynamic problems such as crash analyses, blasts, and dropped objects [30], [31]. Although it would be interesting to compare the results from both dynamic analysis methods, the structure of the Abaqus model created is not in correspondence with the requirements of an explicit dynamic analysis. This was not discovered until the dynamic analysis was the remaining part of the total analysis, which led to only the creation of an implicit dynamic analysis.

The traffic-induced acceleration is applied in Abaqus to the corresponding accelerometer locations on the bridge. Figure 39 Shows these locations, i.e., at the location of hanger numbers 9, 18, 24, and 30 on the West and East side of the girder respectively, from the far-left corner. However, only the accelerations in node pairs H09, H18, and H30 have been used to apply the loading, leaving H24 open for validation of the simulations, to compare the simulated and recorded accelerations at this bridge deck location.

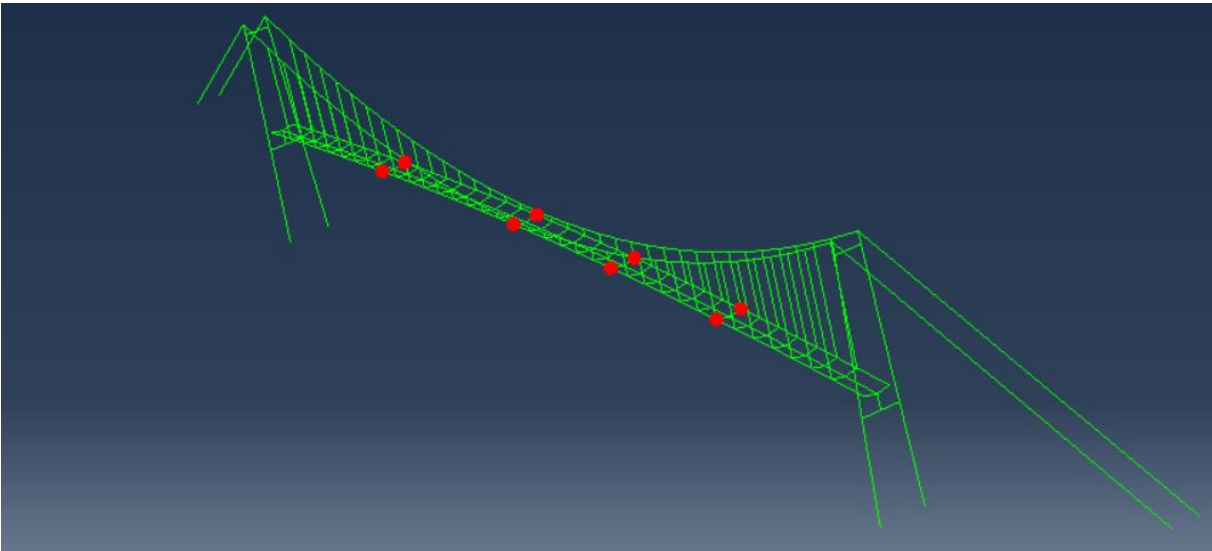


Figure 39 Accelerometer locations on the bridge girder. Displaying H09, H18, H24 and H30 respectively, starting from the top left corner (the north tower)

The acceleration data (both positive and negative) are first defined in Abaqus as so-called “amplitudes” and then used to define nonzero initial boundary conditions in the z-direction, making direct use of the recorded data. The boundary conditions can be set to as initial displacements, acceleration, or velocity for this matter, making the acceleration/angular acceleration setting well suited for this case.



# 6. Results

## 6.1 Static analysis

From the static analysis, the displacements due to dead loads are extracted and presented below. The largest deflection in the model was found in the vertical direction in the middle of the bridge, with a value of 2.842 m (ref. Figure 42 and Table 7). The maximum displacements of the towers are found at the tallest point in the longitudinal direction (Figure 40). The main cables experience their maximum displacement similar to the girder in the vertical direction (Figure 42). These displacements were also used to rearrange all structural coordinates for the bridge to settle to the intended geometry after dead loads are applied. Cable forces of interest are further discussed in Chapter 6.3.3, with comparisons between the contribution from the dead load and the traffic-induced acceleration. Figures 40 to 42 show the displacement for the whole bridge after the static analysis with a color palette describing magnitudes in meters.

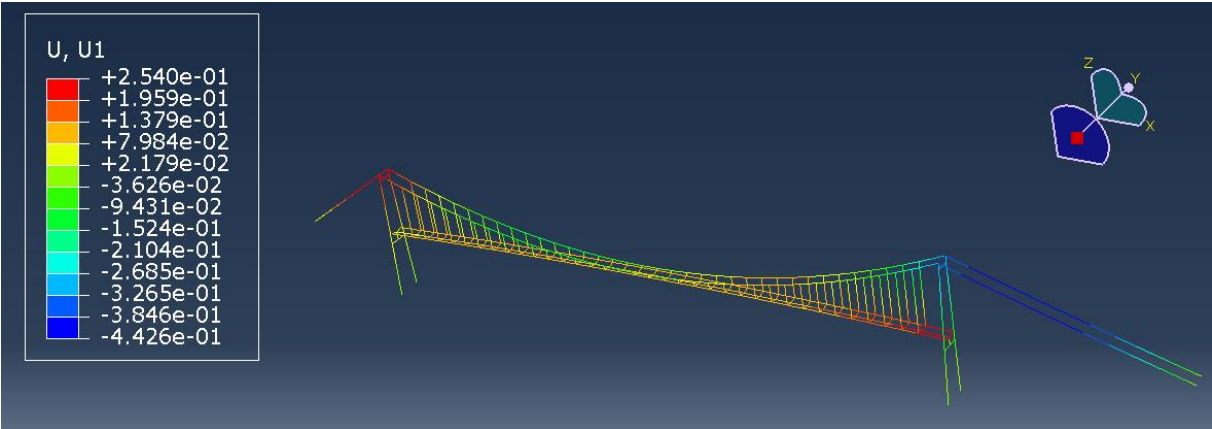


Figure 40 Simulated displacement of the bridge from static analysis in the U1 direction, i.e., x-axis along the bridge span

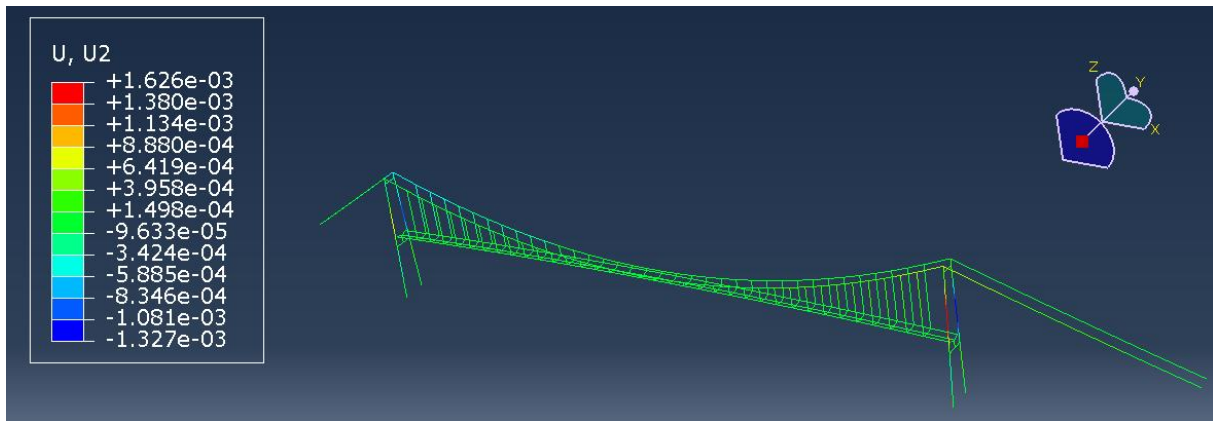


Figure 41 Simulated displacement of the bridge from static analysis in the U2 direction, i.e., y-axis transversally from the bridge span

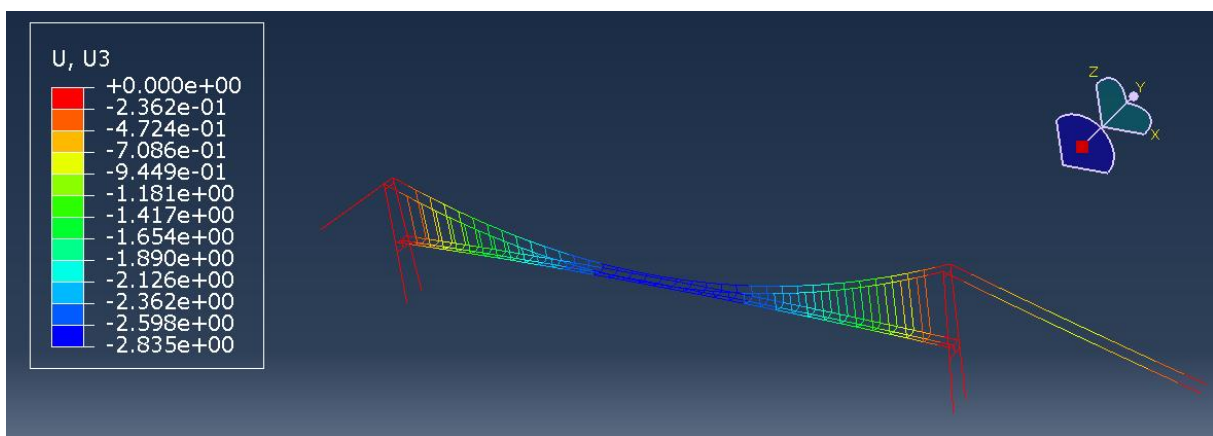


Figure 42 Simulated displacement of the bridge from static analysis in the U3 direction, i.e., z-axis vertically

The maximum displacements for the girder, tower, and cables are compared to the model created by Tveiten for verification purposes. Table 7 presents the maximum displacements and percentage of the difference between this current model and Tveiten’s model. The difference in displacements is mainly due to differences in structural coordinate relations, girder inertia magnitudes, and mass point arrangement of the idealized girder cross-section. However, the deviances are very small, and the results are in good agreement, given the differences in the two models.

Table 7 Maximum displacements from dead loads for the main structural components

<b>Component</b>	<b>ID</b>	<b>Current model</b>	<b>Tveiten</b>	<b>Difference [%]</b>
<b>Tower (U1)</b>	Node 30130	-0.333374 m	-0.326426 m	-2.1285
<b>Girder (U3)</b>	Node 19	-2.84165 m	-2.8613 m	0.6868
<b>Main cable (U3)</b>	Node 2019	-2.84527 m	-2.86202 m	0.5853

## 6.2 Eigenfrequencies

Eigenfrequencies and eigenmodes are important factors when studying dynamics. They reveal natural vibrations and movement patterns to complement the dynamic analysis. After several attempts, a total of 150 modes were made for investigation. This was mainly because Abaqus uses the center of gravity, making it hard to match the values with other models, as the created Abaqus model might not have the same number of elements in all sections. All eigenmodes and frequencies have been studied visually and numerically by partial factors to determine their main direction and relevance.

To validate the results, a comparison has been made with previous studies, found in Table 8. The extracted frequencies will be compared with the calculated values from Alvsat, Tveiten, and identified modal parameters from an operational modal analysis presented by Etienne Cheynet whose PhD thesis included extensive and detailed studies of full-scale wind and vibration data from the Lysefjord bridge. Etienne made different approaches to analyze the eigenfrequencies, where the SCI-COV will be used here. The SCI-COV derives from a time-efficient automated covariance-driven stochastic subspace identification method that was applied for six months of continuous acceleration measurements [20]. The first three modes of each horizontal and vertical type as well as the first symmetric and asymmetric torsional types have been collected for comparison and comprises of:

- Horizontal modes
  - **HS1:** 1<sup>st</sup> Horizontal symmetric mode
  - **HS2:** 2<sup>nd</sup> Horizontal symmetric mode
  - **HS3:** 3<sup>rd</sup> Horizontal symmetric mode
  
  - **HA1:** 1<sup>st</sup> Horizontal asymmetric mode
  - **HA2:** 2<sup>nd</sup> Horizontal asymmetric mode
  - **HA3:** 3<sup>rd</sup> Horizontal asymmetric mode
- Vertical modes

- **VS1:** 1<sup>st</sup> Vertical symmetric mode
- **VS2:** 2<sup>nd</sup> Vertical symmetric mode
- **VS3:** 3<sup>rd</sup> Vertical symmetric mode
  
- **VA1:** 1<sup>st</sup> Vertical asymmetric mode
- **VA2:** 2<sup>nd</sup> Vertical asymmetric mode
- **VA3:** 3<sup>rd</sup> Vertical asymmetric mode
  
- Torsional modes
  - **TS1:** 1<sup>st</sup> Torsional symmetric mode
  - **TA1:** 1<sup>st</sup> Torsional asymmetric mode

*Table 8 Comparison of eigenfrequencies from selected modes*

Modes	Frequency [Hz]	SSI-COV		Alvsat		Tveiten	
		Frequency [Hz]	Deviation [%]	Frequency [Hz]	Deviation [%]	Frequency [Hz]	Deviation [%]
HS1	0.130	0.136	4.412	0.130	0.000	0.127	-2.362
HA1	0.439	0.444	1.126	0.442	0.679	0.431	-1.856
HS2	0.555	0.577	3.813	0.557	0.359	0.533	-4.128
HA2	0.604	0.626	3.514	0.598	-1.003	0.583	-3.602
HS3	0.741	0.742	0.135	0.831	10.830	0.833	11.044
HA3	1.023	1.011	-1.187	1.002	-2.096	0.973	-5.139
VA1	0.217	0.223	2.691	0.213	-1.878	0.214	-1.402
VS1	0.303	0.294	-3.061	0.286	-5.944	0.301	-0.664
VS2	0.407	0.408	0.245	0.400	-1.750	0.406	-0.246
VA2	0.582	0.587	0.852	0.589	1.188	0.582	0.000
VS3	0.852	0.853	0.117	0.867	1.730	0.855	0.351
VA3	1.189	1.163	-2.236	1.198	0.751	1.190	0.084
TS1	1.300	1.237	-5.093	1.154	-12.652	1.086	-19.705
TA1	2.142	2.184	1.923	2.125	-0.800	1.857	-15.347

When analyzing the eigenfrequencies, several relevant symmetrical torsional modes were found. It seemed that a variation of cable motions was behind the range of these torsional modes, while the bridge girder remained more or less the same shape throughout these variations. In total there were five of these modes in a continuously increasing trend ranging from approximately 1.22 Hz to 1.31 Hz. However, modes 32 and 34 displayed the most realistic cases with a frequency of 1.297 Hz and 1.302 Hz, respectively.

The results from Table 8 show that there is generally small deviance in the comparisons. The torsional modes seem to hold the highest variation of deviance, which could be caused by the many types that were found when studying these, leading to an uncertainty of which modes that are actually compared here. For illustration purposes, each type's first asymmetrical and symmetrical modes have been extracted from Abaqus and normalized for graphing (see Figure 43).

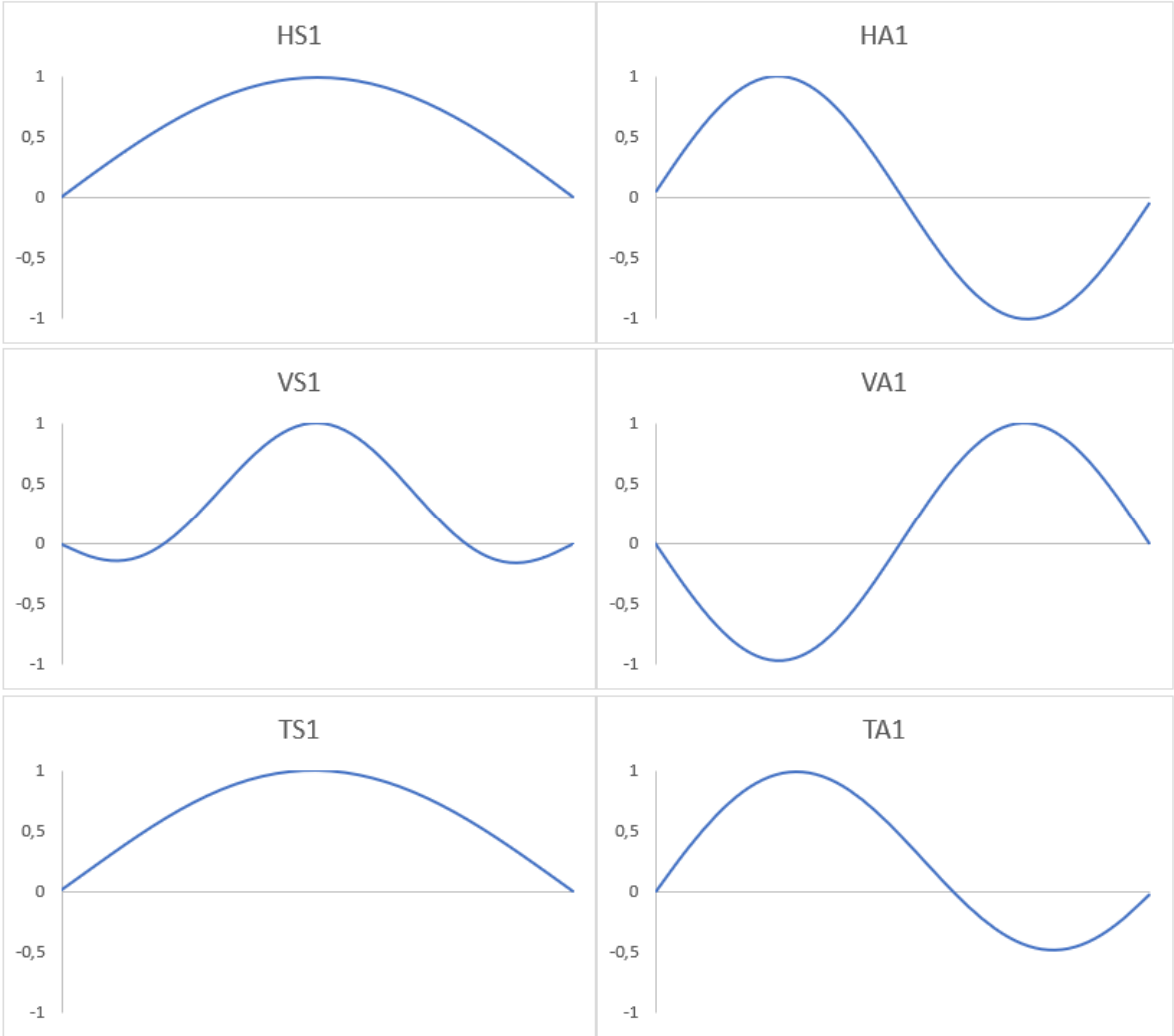


Figure 43 First asymmetrical and symmetrical modes of each type along the bridge girder

## 6.2.1 Damping

Damping is the mechanism by which vibrational energy gradually converts to heat or sound. A damper is assumed to have no mass or elasticity, and the so-called damping force can only exist when there is relative velocity between the two ends of the damper [32].

Damping is calculated based on eigenfrequencies and eigenmodes by identifying relevant mode excitations from different frequency ranges. It is often a good idea to create a response spectrum to identify which modes the vibrational data will excite.

A set of Power Spectral Density (PSD) functions made by Aronsen [33], based on similar sets of data measured on the bridge as used for this thesis, visualizes the relationship between the vertical response of the bridge girder at hanger numbers 9 and 18 and the longitudinal response of the bridge tower (Figure 44). However, it should be noted that this data is collected at a fairly higher mean wind velocity compared to the ones used for this thesis, meaning the results might have been affected by wind contributions.

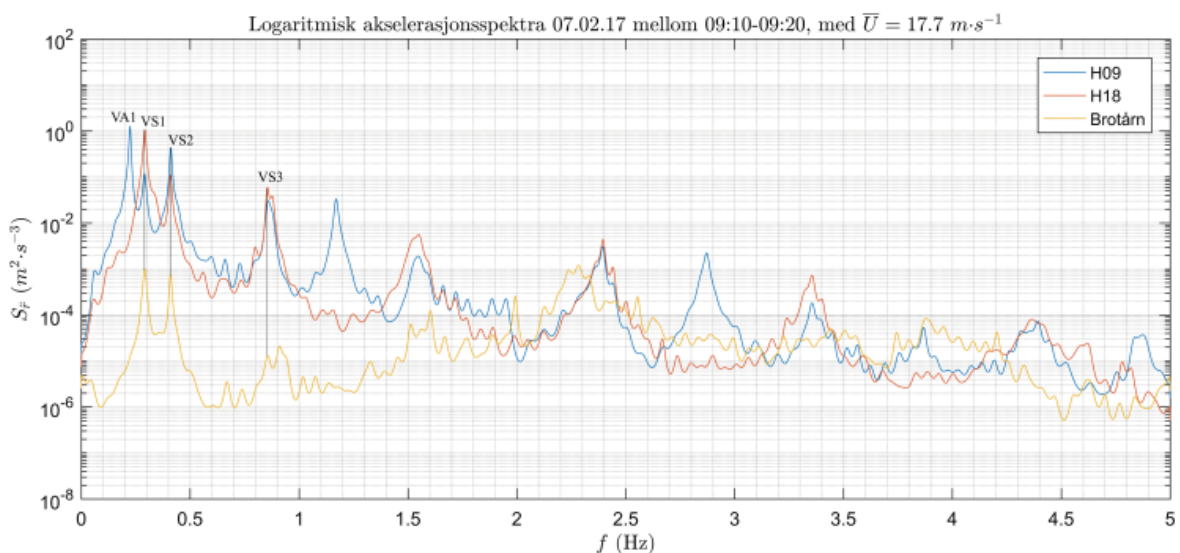


Figure 44 Logarithmically acceleration spectrums comparing the tower in the bridge-longitudinal direction and the girder's vertical response at hangers 09 and 18, from [33]

From the figure above, a clear connection can be seen between the girder's and the tower's responses, as it is mainly the symmetric vertical modes that transfer from the bridge girder to the bridge tower. This is shown where VS1, VS2, and VS3, located at approximately 0.3 Hz, 0.4 Hz, and 0.85 Hz, respectively, have shared peak excitation with the vertical response of the bridge girder. By this conclusion, the same vertical symmetrical modes used to calculate damping for the bridge girder will also be used for the towers, i.e., VS1 and VS2. However, the asymmetrical modes seem also to be a dominant factor at the bridge girder, especially VA1. However, for the sake of simplicity, only the first and second symmetrical modes will be accounted for.

Abaqus provides with the option of Rayleigh damping to model the energy dissipation in a vibrating system. To provide adequate damping to the Abaqus model, Rayleigh damping coefficients will be calculated for mode 3 (VS1) and mode 4 (VS2). Assuming the damping matrix,  $\mathbf{C}$ , is proportional to the mass and stiffness matrix,  $\mathbf{m}$  &  $\mathbf{K}$ :

$$\mathbf{C} = \alpha \mathbf{m} + \beta \mathbf{K}$$

$$\rightarrow \mathbf{X}^{(j)T} \mathbf{C} \mathbf{X}^i = \alpha \mathbf{X}^{(j)T} \mathbf{m} \mathbf{X}^i + \beta \mathbf{X}^{(j)T} * \mathbf{K} \mathbf{X}^i = 0 \quad \text{for } i \neq j$$

$$\rightarrow \mathbf{C}_i = \mathbf{X}^{(i)T} \mathbf{C} \mathbf{X}^i = \alpha \mathbf{M}_i + \beta \mathbf{K}_i$$

$$\rightarrow \xi = \frac{\mathbf{C}_i}{2\mathbf{M}_i\omega_i} = \frac{1}{2} \left( \frac{\alpha}{\omega_i} + \beta\omega_i \right)$$

$$\rightarrow \xi = \frac{\alpha}{2\omega_i} + \frac{\beta\omega_i}{2}$$

Generally,  $\alpha$  is the mass-proportional damping coefficient accounting for the portion of damping that is proportional to the mass of the system and is responsible for dissipating energy based on the velocities of the vibrating masses.  $\beta$ , however, is the stiffness-

proportional coefficient that dissipates energy based on the displacements of the vibrating system [34].

Due to the different materials in the bridge model, the critical damping ratio will vary. An exact estimation of the different critical damping ratios is somewhat challenging to achieve since the damping is generally more of an empirical measure of energy dissipation. However, an estimation of 0.5% for the steel box, 1% for the concrete towers, and around 0.1% for the cables are used and assumed to be sufficient. There has been some discussion about the critical damping of the cables if 0.1% is too small, but this estimation still stood in the end. Table 9 presents the Rayleigh damping coefficients and critical damping ratios calculated and used in the bridge model in the dynamic analysis. All coefficients are calculated based on Mode 3 and 4, i.e., VS1 and VS2.

Table 9 Rayleigh coefficients for the bridge model

	$\xi$	$\alpha$	$\beta$
Tower	1%	0.021785	0.00448964
Bridge girder	0.5%	0.01089	0.00224482
Cables	0.1%	0.0021785	0.000448964

### 6.3 Dynamic analysis

With the applied acceleration in Abaqus, one detail was noted regarding the development of deflection throughout the selected time integration. The deflection had a more or less increasing trend after the passage of a heavy vehicle, approximately halfway into the selected time-series segment. This effect could be due to an underlying trend in the selected short acceleration event. This was taken care of by detrending the considered datasets in MATLAB to remove any implicit linear trends after observing that the mean value for one of the acceleration data was nonzero at a value of around  $8E-04 \text{ ms}^{-2}$ . When the acceleration is to be double integrated into deflection, a minor constant mean value gives a displacement



contribution corresponding to:  $\frac{C}{2} * t^2$ , where C equals the mean value, i.e., a contribution growing with time. It was also detected when considering the entire data set without segment extractions, i.e., 10-minute periods in 10 sessions, that the mean value was approximately equal to zero, confirming the expected source of error. Figures 45 and 46 below show that there is no immediate noticeable difference between the acceleration before and after the detrending.

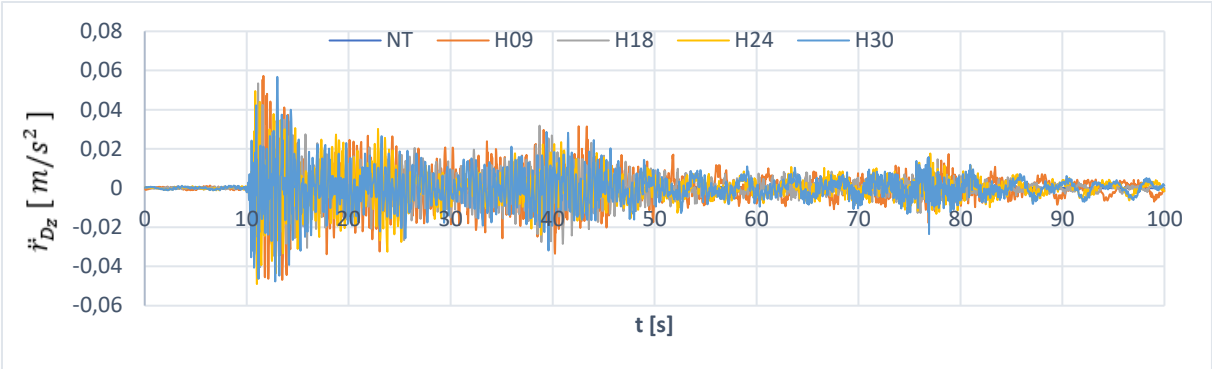


Figure 45 All sensors on the eastside of the bridge before detrending

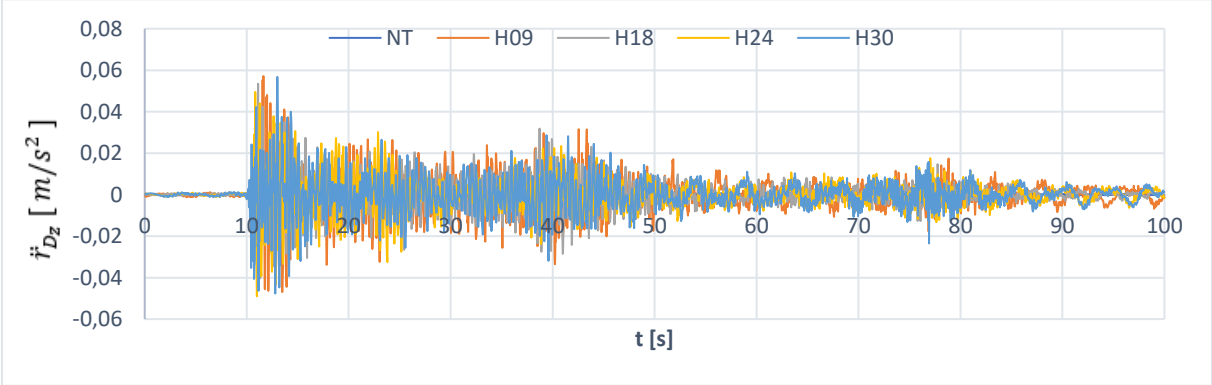


Figure 46 All sensors on the east side of the bridge after detrending

Figure 47 shows the displacements in meters as a function of time in seconds before the detrending of acceleration data in one of the nodes used to apply the accelerations. Here, the increasing trend is rather noticeable as the end displacement puts the node in a state of around 5 mm, opposite of the initial dead load at the end of the time series. In Figure 48, the displacement-time function is shown after the detrending procedure, resulting in zero

deviance between the start-displacement and end-displacement, which is the desired result for the idealized load case created here.

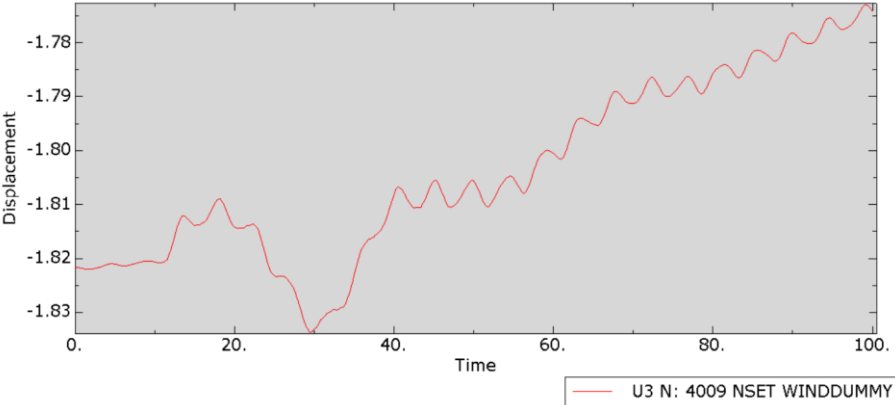


Figure 47 Displacement-time graph of node 4009 before detrending

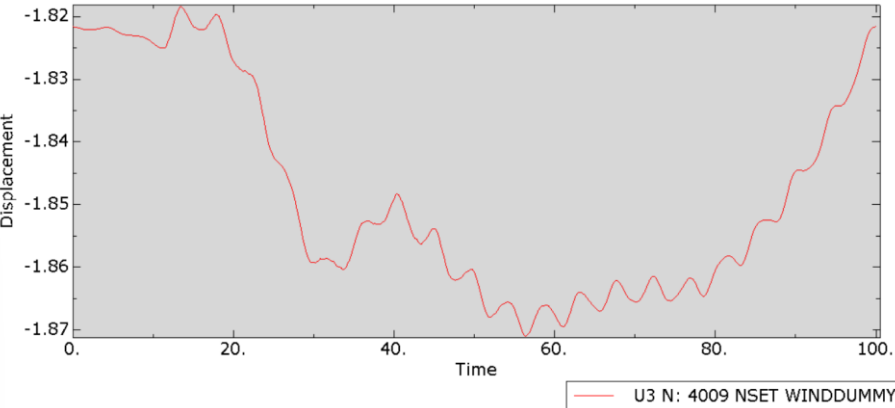
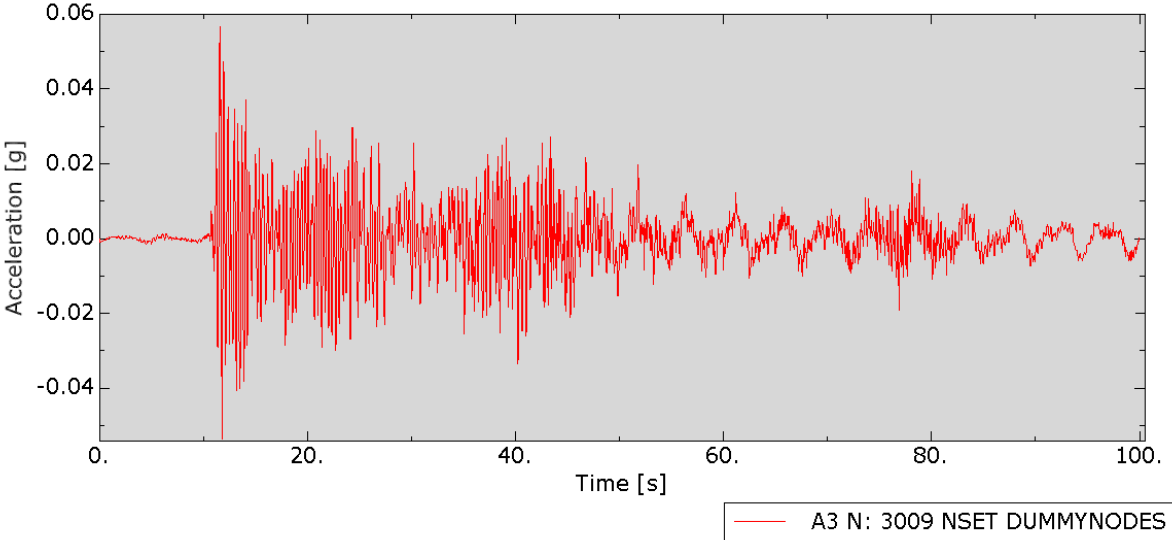


Figure 48 Displacement-time graph of node 4009 after detrending

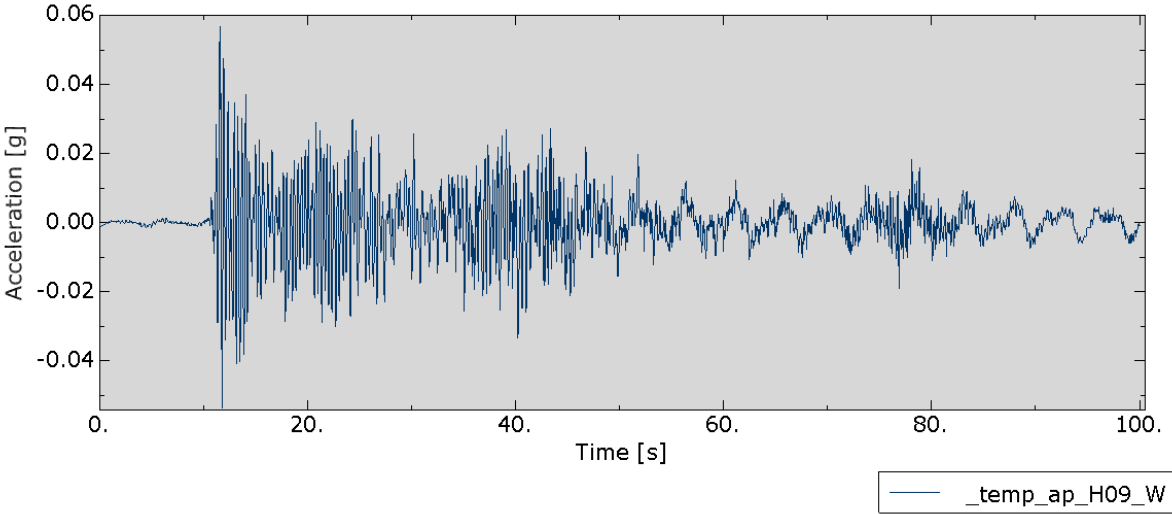
### 6.3.1 Validation of acceleration input

To validate if the input acceleration loading has been added and processed correctly, the recorded values will be checked against the output results. Abaqus offers the option of acceleration output which will be used for this control check. Respectively, nodes 3009, 4018, and 3030 will be used for the comparisons. The input acceleration is plotted by the Abaqus CAE amplitude plotter manually, while the output acceleration plot has been gained by history output in the results file by requests in the input file. Figures 49 to 54 below present these pairs of input and output acceleration.

**Node 3009**



*Figure 49 Input acceleration of hanger H09\_W*



*Figure 50 Output acceleration of hanger H09\_W*

**Node 4018**

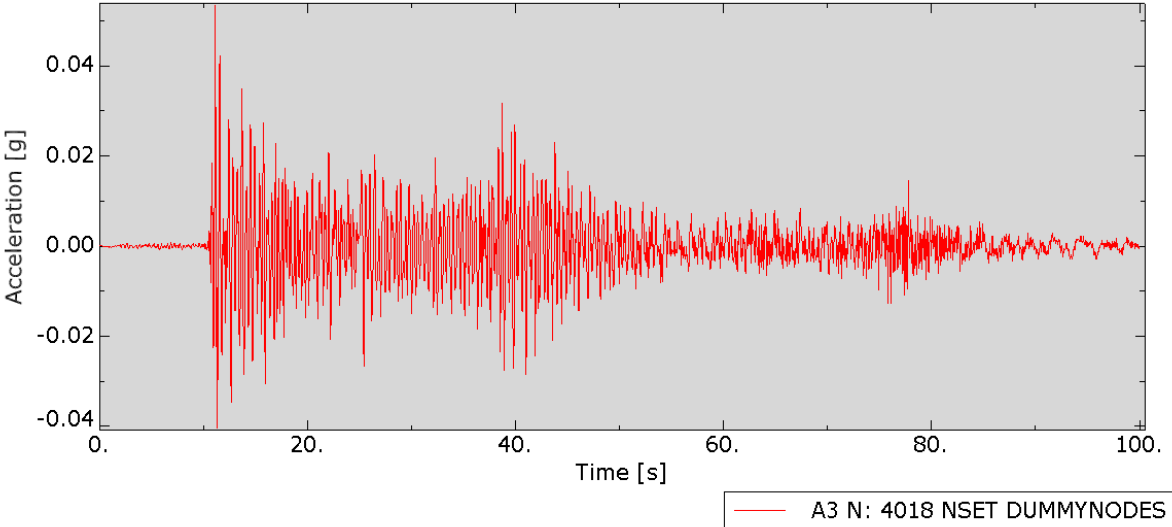


Figure 51 Input acceleration of hanger H18\_E

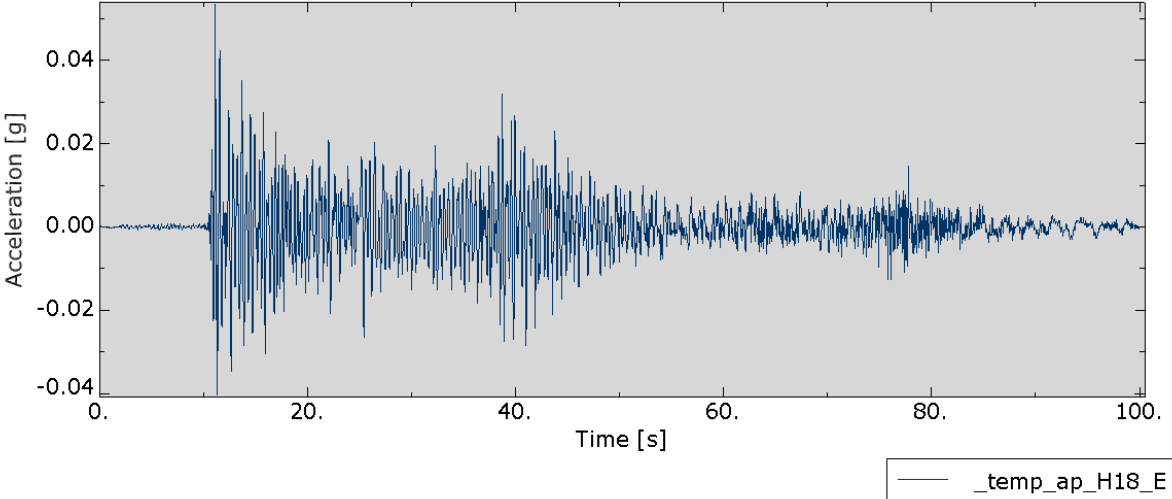
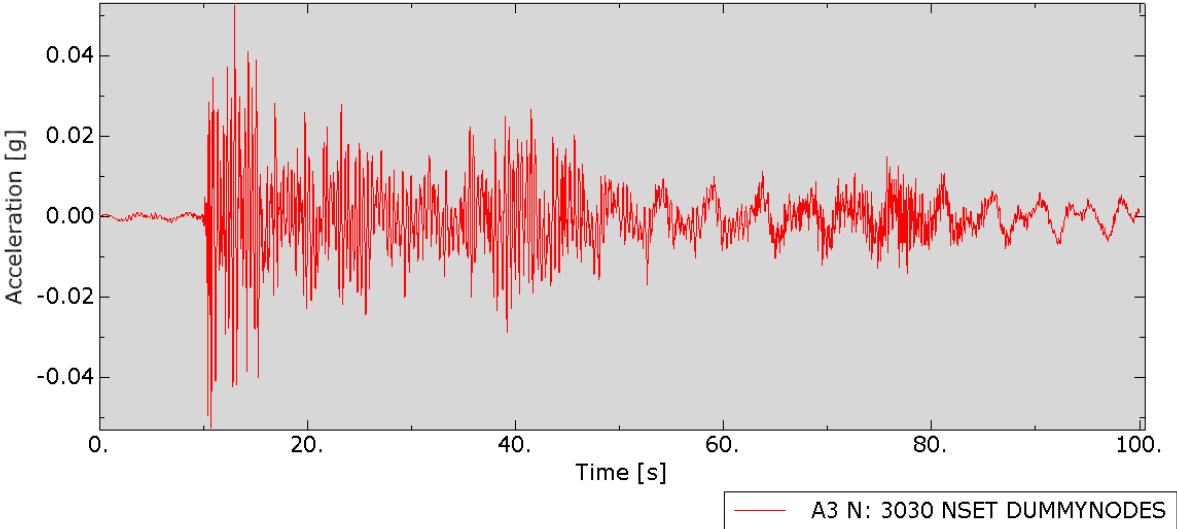
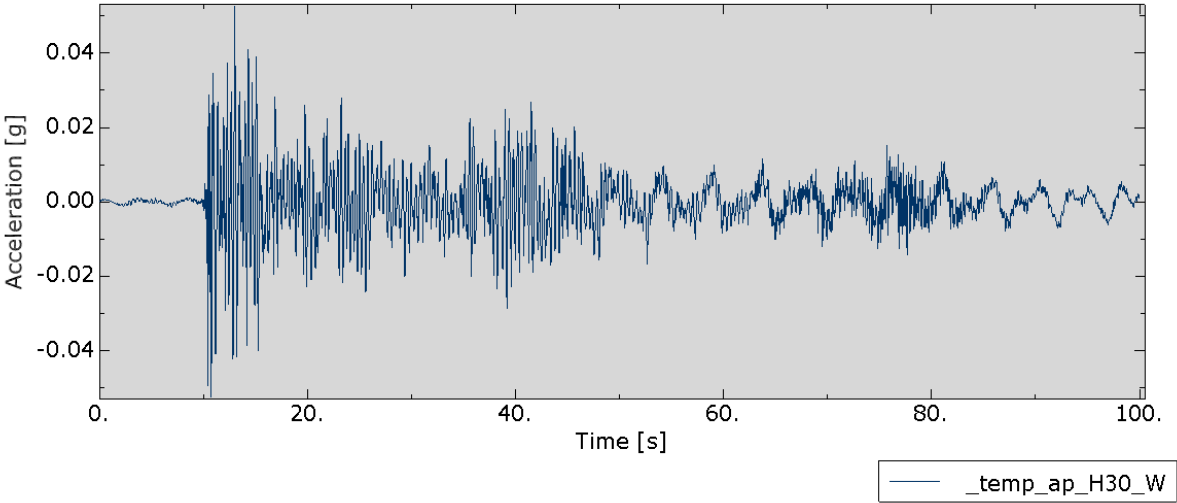


Figure 52 Output acceleration of hanger H18\_E

**Node 3030**



*Figure 53 Input acceleration of hanger H30\_W*



*Figure 54 Output acceleration of hanger H30\_W*

As the figures above show, the output acceleration is in complete correspondence with the input values. This has also been manually checked by extracting values in the result file at several random instances of time to compare with the measured values. In addition, a MATLAB script was created using integration procedures of the input acceleration charts to display the

displacement-time graphs to compare with output values, which also showed complete correspondence in displacement behavior.

### Validation of hanger number 24

The hanger number 24, corresponding to node 4024 on the eastside and node 3024 on the westside, were omitted from the input acceleration to validate Abaqus's simulated output frequency distribution to these nodes.

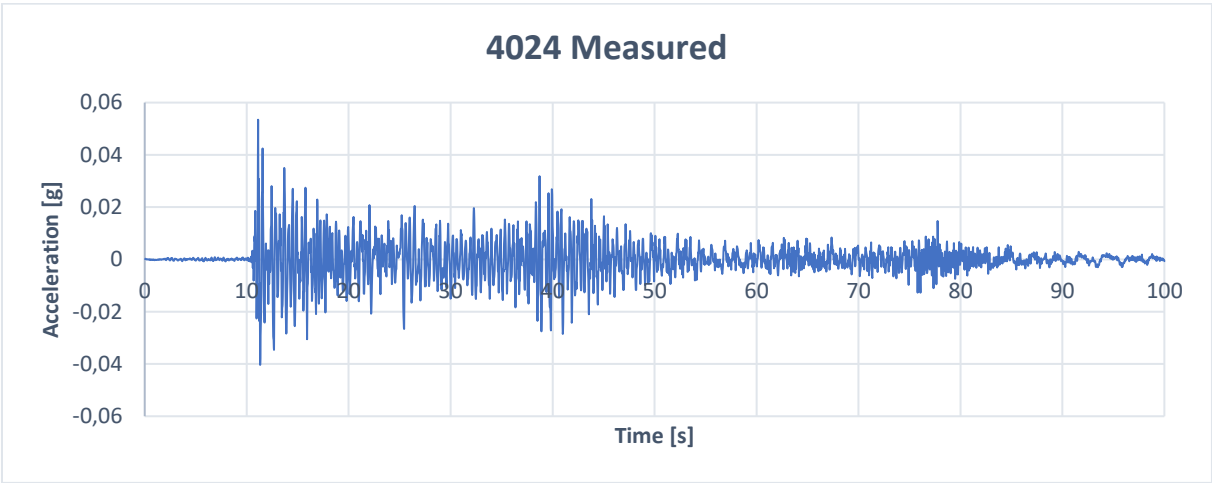


Figure 55 Measured vertical acceleration at hanger H24\_E

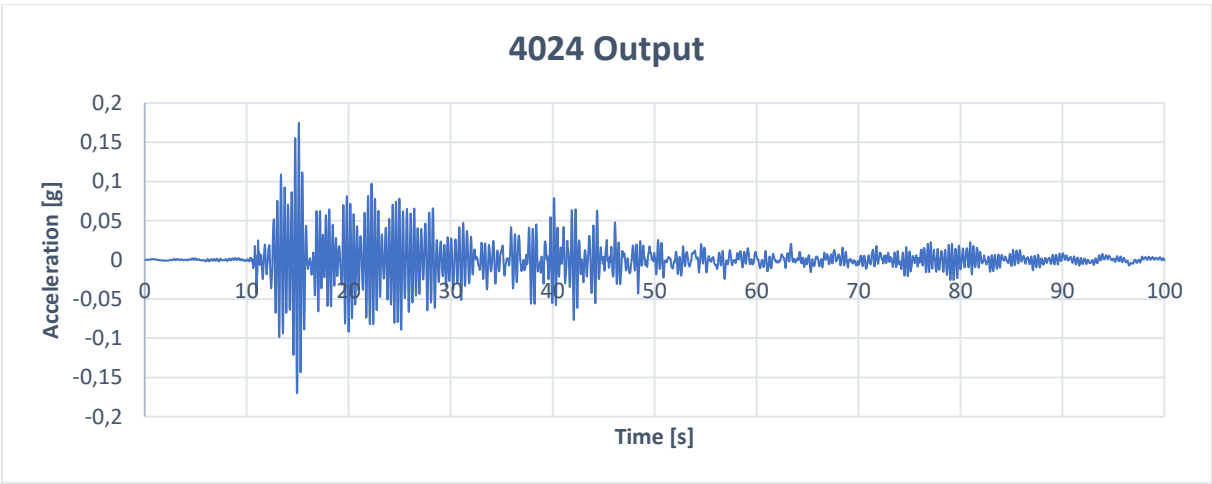


Figure 56 Simulated vertical acceleration at hanger H24\_E

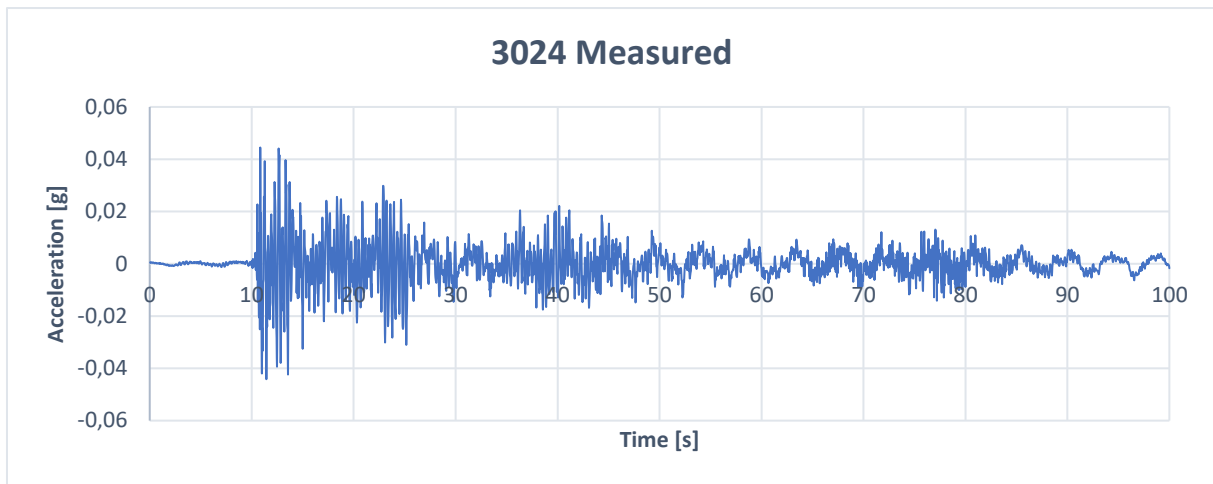


Figure 57 Measured vertical acceleration at hanger H24\_W

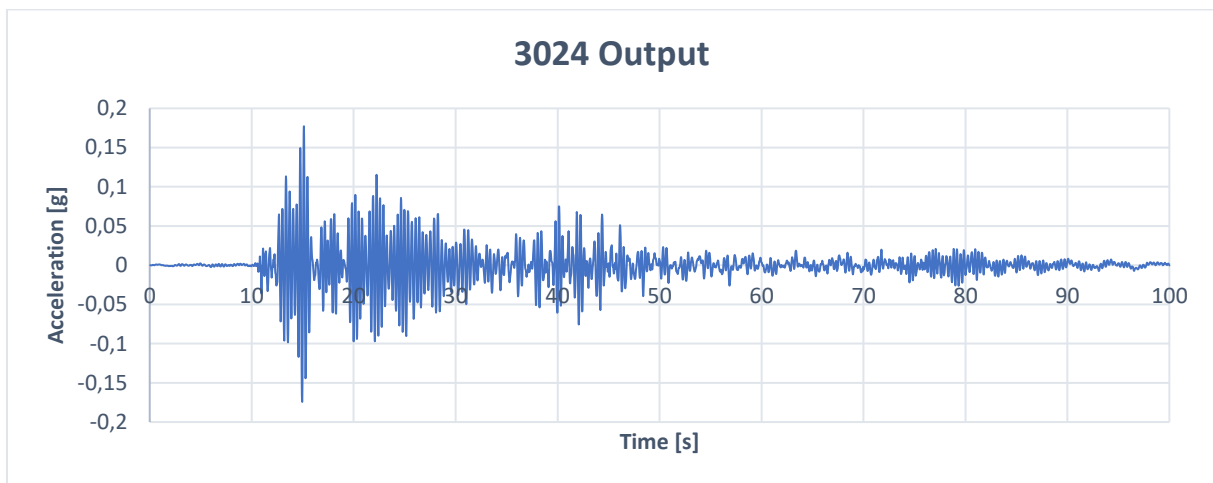


Figure 58 Simulated vertical acceleration at hanger H24\_W

The results from figures 55 to 58 shows that the magnitude of the output accelerations follows the same order of magnitude but is also different from the measurement data at the bridge. However, the location and development of the high response acceleration seem to match quite well. It should be noted that, in reality, there are several other factors that can affect the damping, such as wind and the arrangement and complexity of joints and structural components, to name some examples. Therefore, an identical output acceleration would be somewhat unrealistic, making the response development a much better factor for comparison. It became interesting to investigate possible reasons related to the damping of the bridge for the mismatch in the acceleration magnitude. Three hypothesizes are presented below.

**Hypothesis 1:**

Firstly, the increase was thought to be related to inadequately defined damping along the bridge girder. By manually investigating output acceleration along the bridge in Abaqus, a pattern of identical increases in acceleration appeared at given distances from the three load-applied node pairs. In light of this, the critical damping ratio development based on different dominant modes has been studied. To compute this, another PSD estimate of the traffic-induced vertical acceleration response, originating from [17], will be investigated (See Figure 59). This PSD is also based on the same recordings used for this thesis.

Mode	Identified frequency [Hz]	Mode	Identified frequency [Hz]
VA1	0.22	TA1	1.24
VS1	0.29	TS1	2.19
VS2	0.41	TS2	3.24
VA2	0.59	TA2	3.96
VS3	0.85	TS3	4.29
VA3	1.16	TA3	4.78

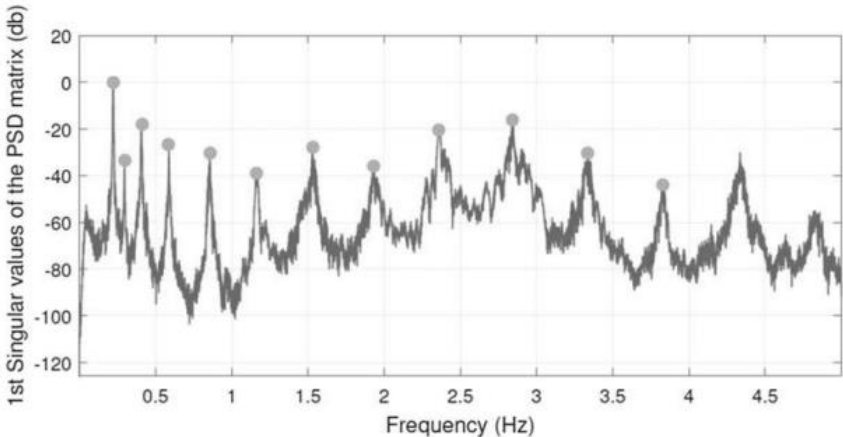


Figure 59 PSD estimate of a traffic induced vertical acceleration response with identified "peak-picking" frequency values, from [17]

From Figure 59, the asymmetrical modes are shown to be quite dominant similar to the previous PSD from Figure 44. The figure also shows that the traffic acceleration excites quite a broad range of frequencies. Due to this, the vertical modes and the first symmetric and asymmetric torsional modes will be used to investigate the critical damping development, shown in Table 10.



Table 10 Development of critical damping ratio based on different modes

ID	$\omega$ [rad/s]	$\alpha$	$\beta$	Critical damping, $\xi$
VA1	1.361252097	0.01089	0.00224482	0.005527877
VA2	3.654426238	0.01089	0.00224482	0.005591739
VS3	5.355472997	0.01089	0.00224482	0.007027753
VA3	7.466309101	0.01089	0.00224482	0.009109536
TS1	8.168140899	0.01089	0.00224482	0.009834617
TA1	13.45795461	0.01089	0.00224482	0.015509936

Table 10 shows that the critical damping ratio increases as the eigenfrequencies of the modes increase. This would mean that the bridge deck has sufficient damping based on the calculations done in Chapter 6.2.1. It can also be seen that VA1, which was denoted as quite dominant, does not give any significant large increase in damping but could however contribute to a corresponding linear increase in the rest of the modes if it were to be used for the Rayleigh coefficients.

It was also noted during test calculations of new Rayleigh coefficients based on the different sets of modes shown above that the alpha-coefficient became descending while the beta-coefficient came to be the dominant factor. This makes sense since when the angular frequency increases, the mass-proportional coefficient (alpha) decreases while the stiffness-proportional coefficient (beta) increases, also shown in the schematic below (Figure 60) showcasing the relationship between these coefficients and their intended combined curve.

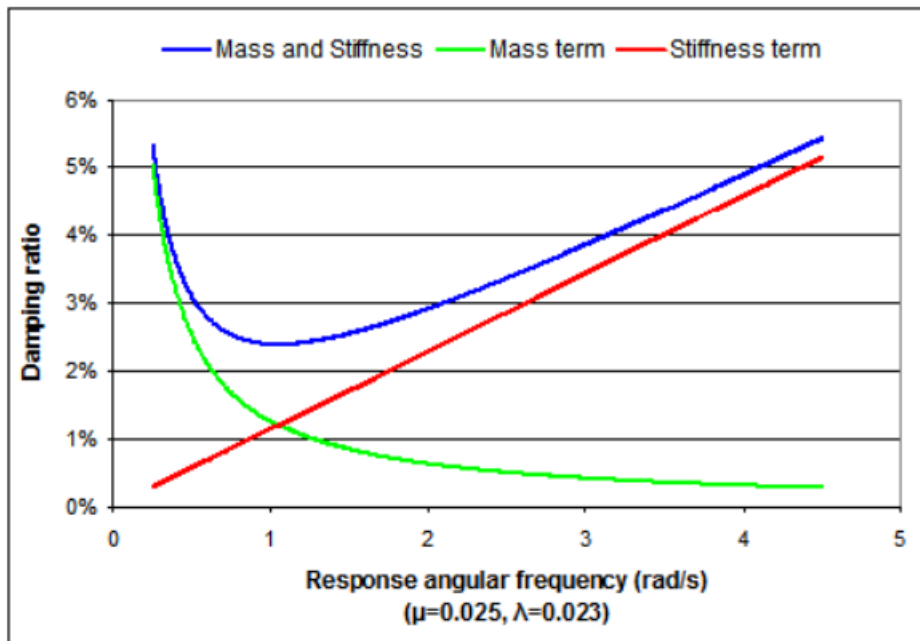


Figure 60 Variation of damping ratio with frequency, from [34]

### Hypothesis 2:

Due to the section assignment approaches for the bridge girder, the girder elements do not contain any density, and only the elements connecting the neutral axis have real values of stiffness. The damping coefficients are applied to the elements in the neutral axis, while the dummy elements have been appointed extremely high stiffness values to act as a structural extension of the neutral axis. It is not clear to what degree the masses applied outside of the neutral axis, via dummy elements, is effectively carried out by the key girder elements, i.e., whether the lack of mass in the elements could be a source of this problem involving output acceleration in H24 if the mass component in the equation of the damping matrix loses its value, i.e.,

$$C = \alpha m + \beta K = \beta K$$

Meaning that only the stiffness-proportional component,  $\beta$ , contributes to the damping. This is, however, only an untested theory here. The idea would be to only consider the red (stiffness-proportional) graph in Figure 60 and calculate approximate locations for the modes used in calculating coefficients.

### **Hypothesis 3:**

The last investigation involved editing the damping state of the cables since the critical damping ratio for these was of some discussion in the beginning. The cables were given an increased critical damping ratio to 0.5%, 1%, and 0.5% with the modes VS1 and VA1 for new Rayleigh coefficients. The results here showed some improvement, but on a very small scale, and thereby concluded not to be a decisive factor.

Overall, the exact and dominant reason behind the mismatch in acceleration is of uncertainty. This could be part of a small subject for further studies with a deeper investigation of the effectively modeled damping, e.g., by studying a simulated free decay response of the bridge.

### **6.3.2 Displacement response of the girder**

A short analysis of the displacement due to the recorded accelerations as input is made by studying the history response with a focus on the underlying vehicle behavior, contributing modes/eigen periods, and the torsional response of the bridge girder. The traffic scenario from this extracted series of acceleration is unknown to the author and will hopefully be better clarified in this chapter. Generally, in Abaqus, the previous steps will appear with the initial response as an addition to the load step in focus. Therefore, a manual procedure of isolating the displacement caused by the acceleration loading without any initial dead load values has been executed.

Firstly, the displacement of the load-applied nodes will be studied. Here, the three nodes along the bridge on the east side will be displayed (figures 61 to 63), as the west side rows of nodes showcase more or less the same response to their equivalent pair with some small variations in magnitude. These variations will be used to study the torsional response of the girder. Secondly, the displacement history along the neutral axis in the longitudinal direction has been arranged and plotted manually to gain a clear view of the girder's response on a global scale.

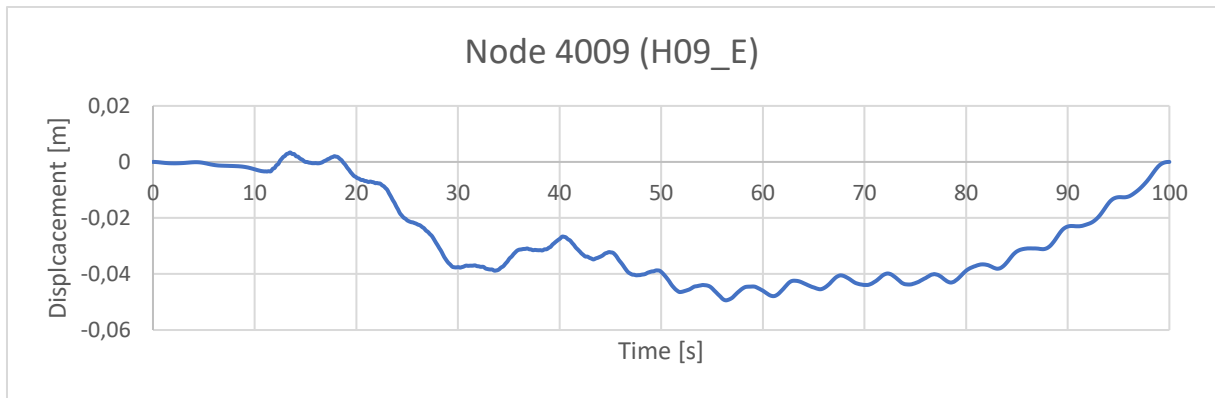


Figure 61 Displacement of node 4009 as a function of time due to traffic induced vibration

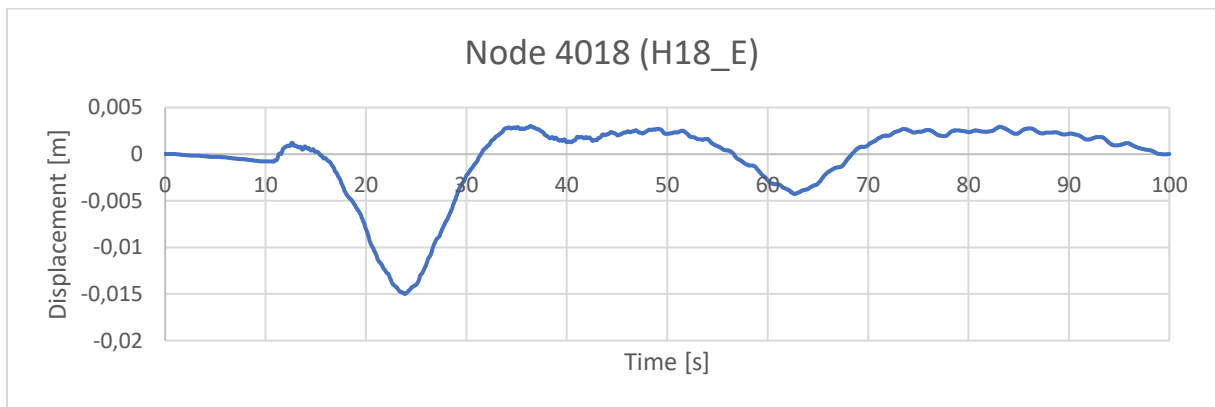


Figure 62 Displacement of node 4018 as a function of time due to traffic induced vibration

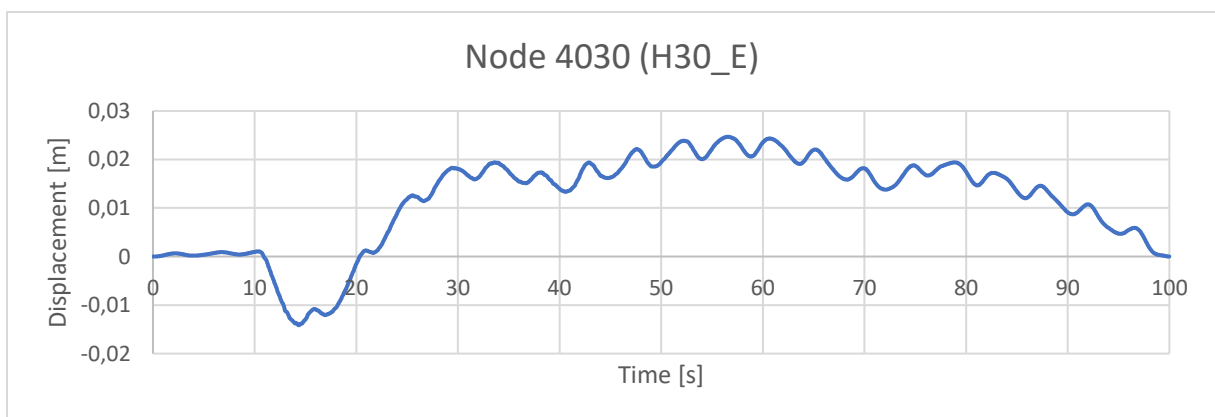


Figure 63 Displacement of node 4030 as a function of time due to traffic induced vibration

By studying the displacements at nodes 4009 and 4030, it shows that the graphs are almost identical but mirrored. These nodes are located approximately the same distance from their closest tower and from the center of the bridge's main span. The mirrored similarity could indicate that when vehicles travel from one node to the other, the nodes reveal equal responses but in opposite directions due to the vehicle's path.

The more extended response after 40 seconds in nodes 4009 and 4030 has been identified with associated modes in Table 11. The results here show again how the asymmetrical vertical modes are excited by the girder's response, similar to the conclusion in the validation of H24.

*Table 11 Associated modes for intervals of displacement*

<b>Node number</b>	<b>Time interval</b>	<b>Amplitudes</b>	<b><math>T_d</math></b>	<b>Associated mode</b>
4009	40s-80s	9	4.444	2 <sup>nd</sup> mode – VA1
4030	40s-80s	9	4.444	2 <sup>nd</sup> mode – VA1

In the periods between 10s to 34s approximately, node displacement begins to develop in the downwards (negative) direction. It starts in node 4030 with a peak after around 13s and further develops with peaks in node 4018 and 4009 after approximately 24s and 34s, respectively. This would indicate that one or several vehicles were driving from the south side of the bridge towards the north side in this period. The vehicle is directly over the nodes at the time of their respective peaks in this period. The distance between node 4009 and 4030 is 252m, meaning that the vehicle would have been driving at a velocity of approximately 45 km/h during this distance. In node 4009, an upwards spike that starts at around 35s and ends after 40s could represent the vehicle leaving the bridge. In addition, at 56s in node 4009, a new peak appears. This peak transfers to node 4018 at 63s and node 4030 at 71 seconds. This could represent another vehicle arriving from the north side of to bridge at a velocity of approximately 60 km/h.

In node 4018, which is approximately at the center of the main span, the deformation excited by vehicles leaving the node location is much shorter in duration than the nodes further out towards the ends of the bridge. It would make sense with these effects at this location when considering the asymmetrical vertical mode VA1 again, whereas the center of the bridge functions as the distinction between positive and negative displacement (ref. Figure 43). This is also clearly shown in the global displacement plots below (figures 64 and 65), where this center location acts as the joint location of distinction.

In the figures below, the displacement is plotted on a global scale along the neutral axis of the girder in time intervals of 10 seconds. These values were exported from the Abaqus result file and processed manually to isolate and represent time-varying longitudinal responses.

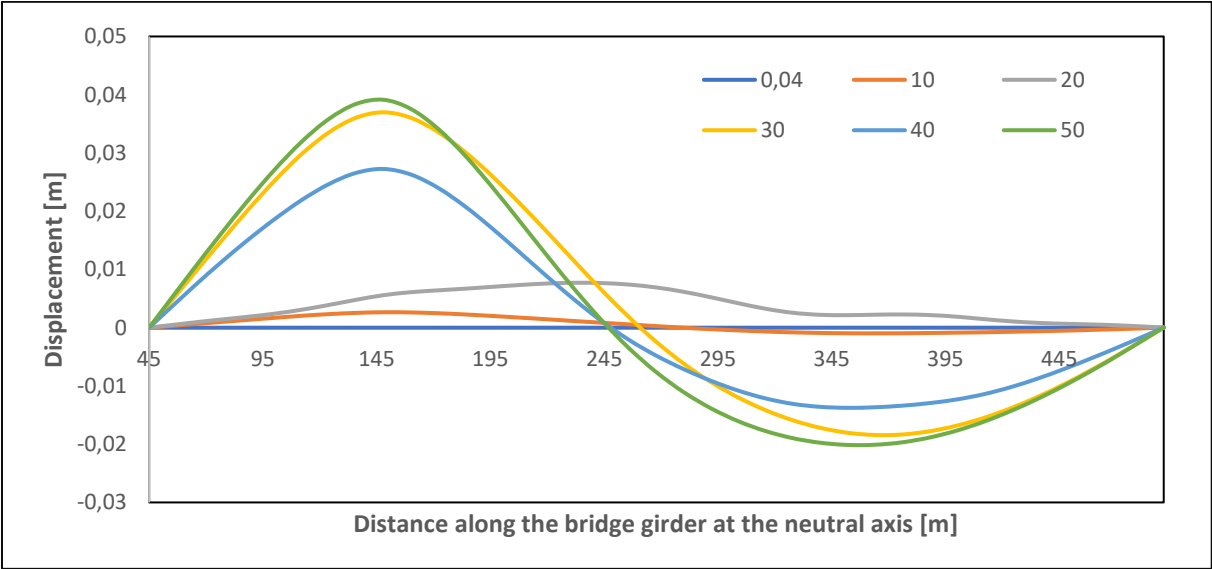


Figure 64 Global displacement along the nodes in the neutral axis. Illustrated with response during the listed time marks

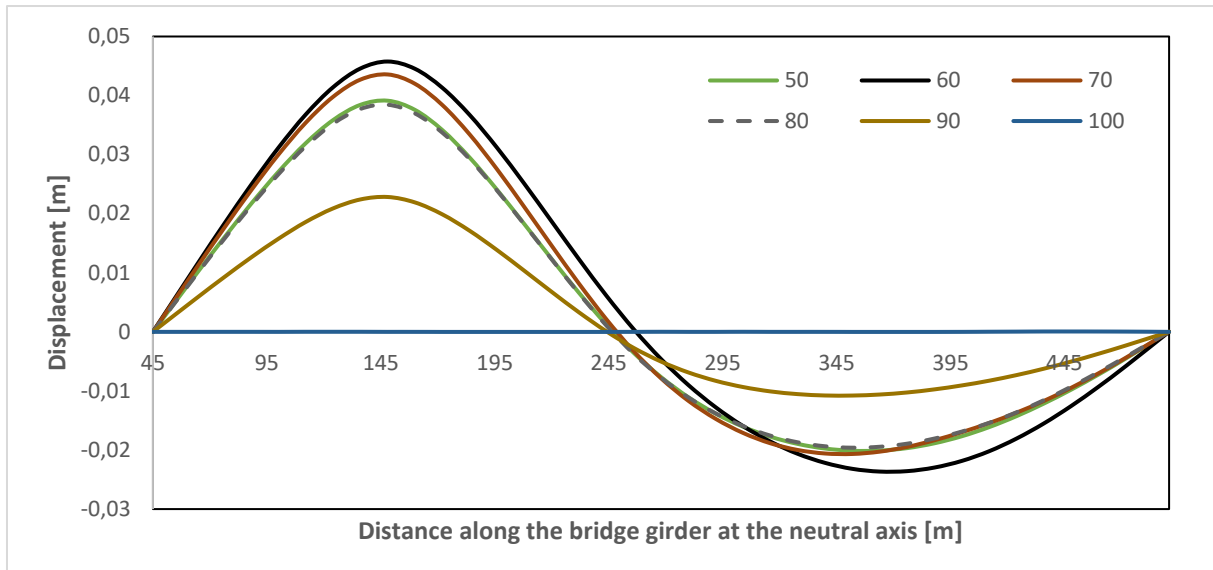


Figure 65 Global displacement along the nodes in the neutral axis. Illustrated with response during the listed time marks

Based on the global displacement response from the figures 64 and 65, it shows that at the time of 60 seconds, the bridge experiences peak displacements of nearly 0.05 m due to the traffic. This can be traced back to the nodal displacements shown earlier, where a new vehicle was detected on the top of already present deformation, creating peak conditions at both sides of the bridge in opposite directions. Other than that, the development of displacements appears as expected based on the studies of nodal displacements.

### Torsional response

When regular traffic emerges on the bridge, there will be differences in the loading situation on each side of the two traffic lanes. Perhaps a truck is driving on one of the lanes while a much smaller vehicle is driving on the other side, creating different displacement scenarios transversally. This difference causes torsion along the bridge girder and is important to investigate.

By creating an Excel sheet calculating the difference in vertical displacement on each dummy node, torsional effects are plotted and displayed in figures 66 and 67. Here, dummy node pairs 5 to 15 are used as visualization. This result is also verified by Abaqus's output of rotational

displacement about the longitudinal direction of the bridge, UR1, shown further below in figures 68 to 70. The rotational displacement in radians in Figure 67 is found by dividing the values of Figure 66 by the hanger-pair span, i.e., 10.25 m.

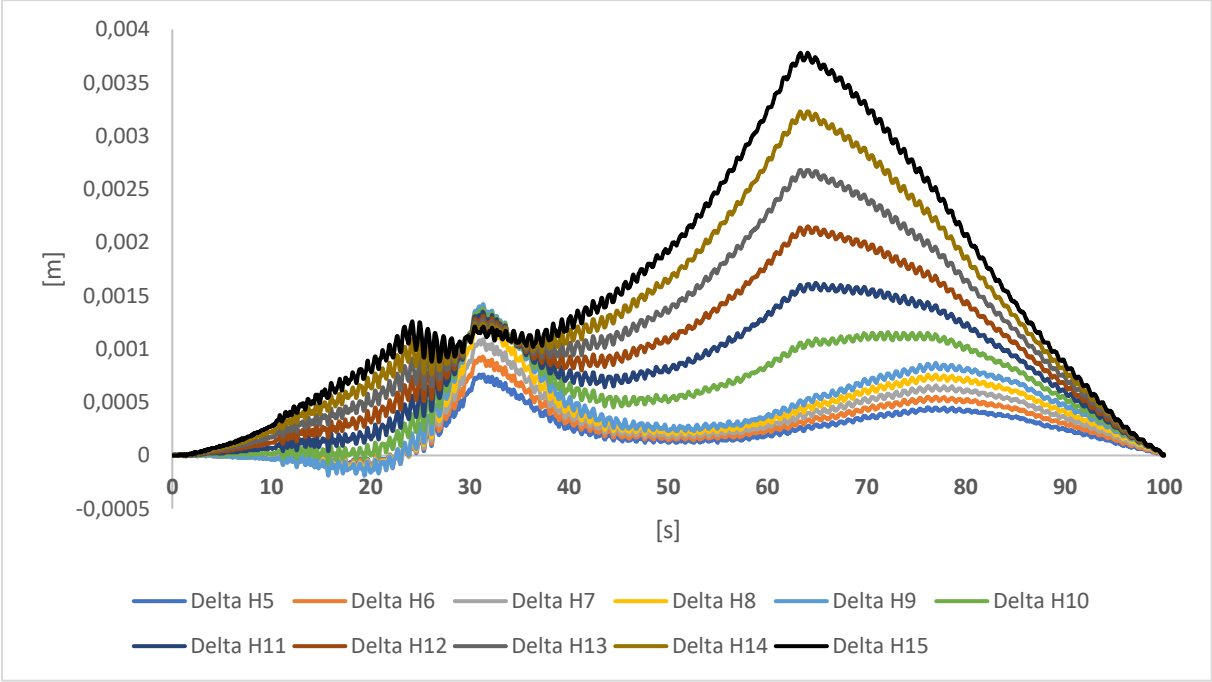


Figure 66 Torsional response expressed in terms of the difference in vertical displacement of the deck at two hanger planes, for the node pairs from 5 to 15



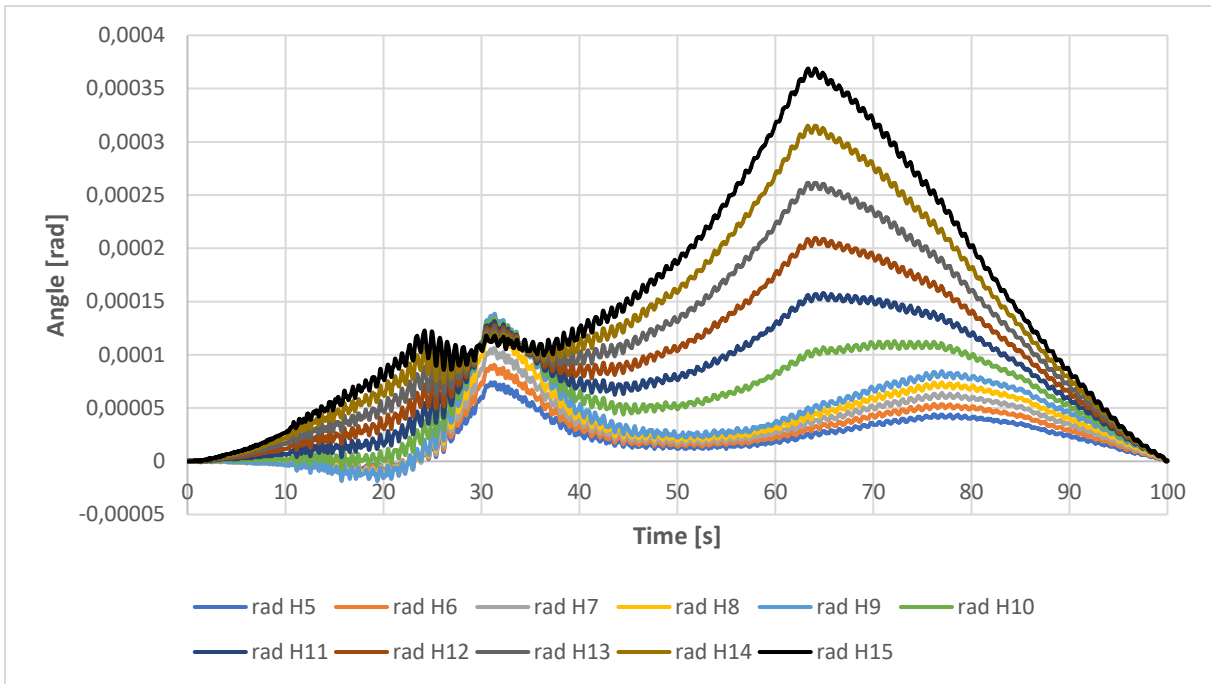


Figure 67 Torsional response in radians calculated for the node pairs from 5 to 15

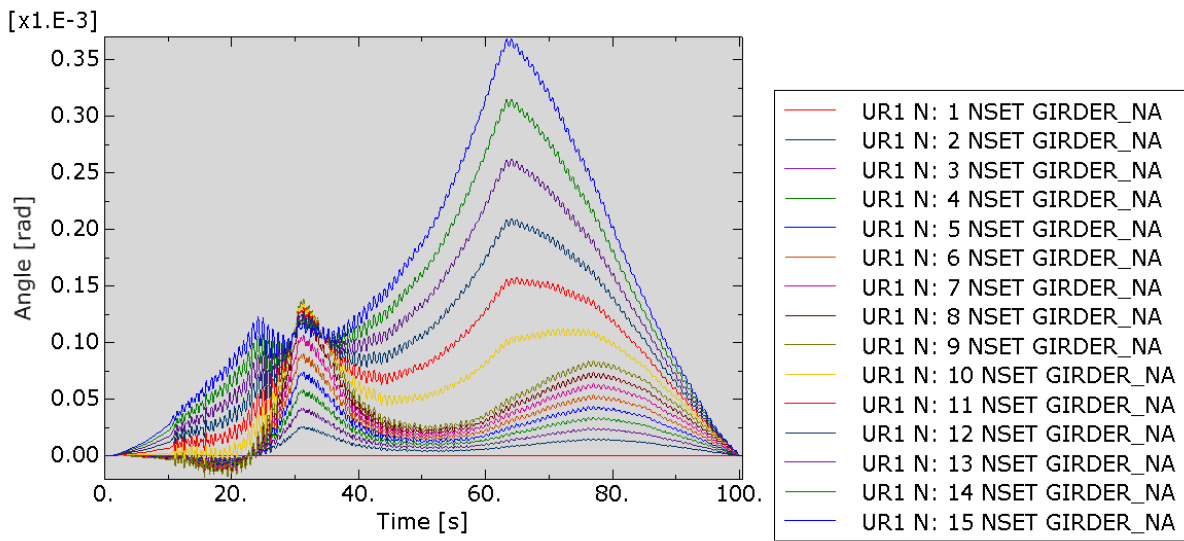


Figure 68 Simulated time-history of torsional response of the bridge girder for node 1 to 15 in radians

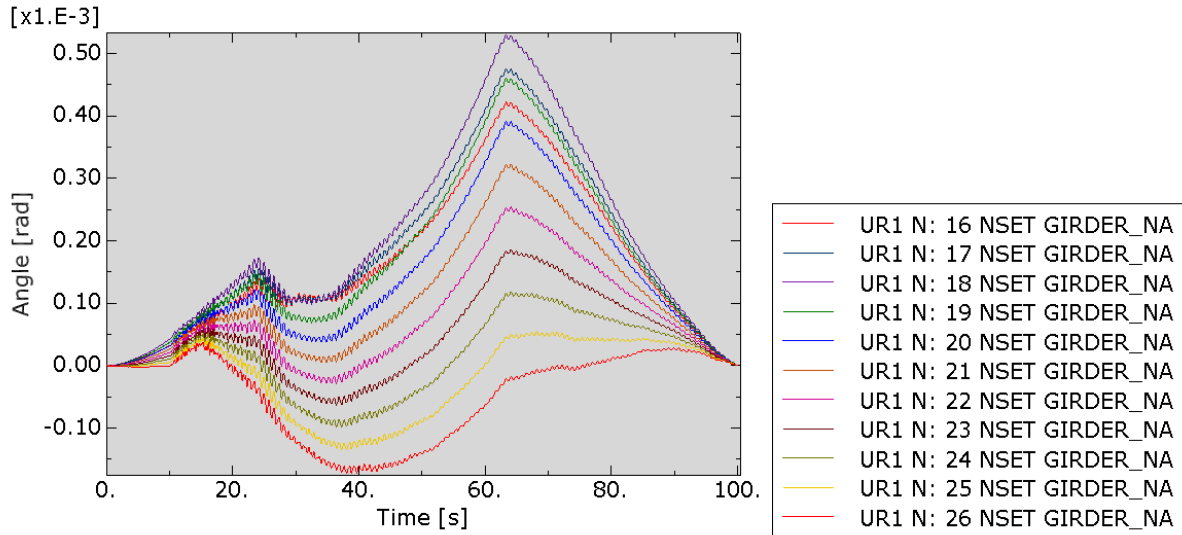


Figure 69 Simulated time-history of torsional response of the bridge girder for node 16 to 26 in radians

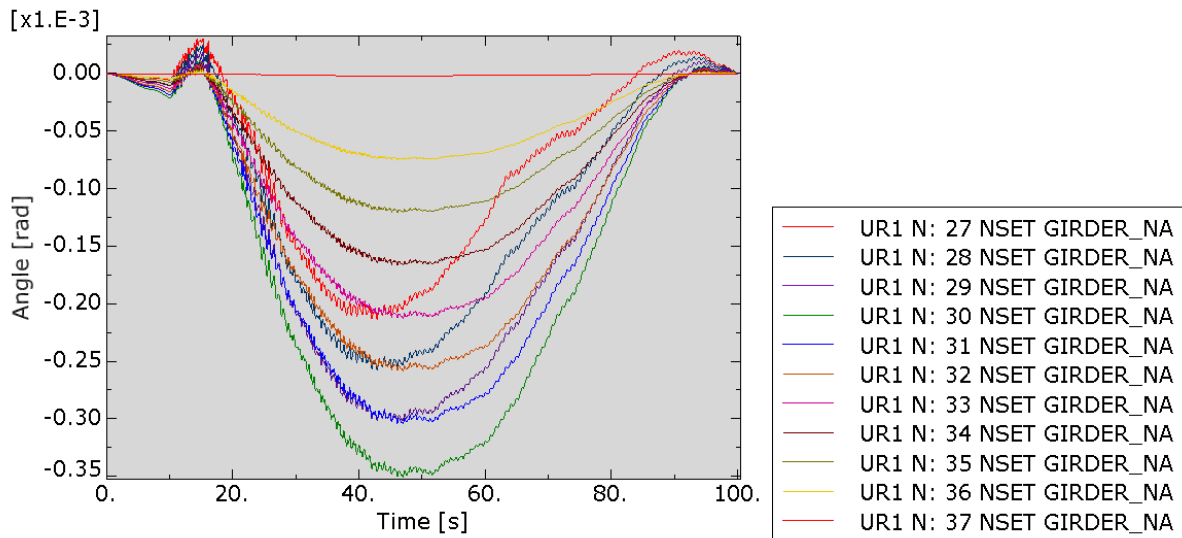


Figure 70 Simulated time-history of torsional response of the bridge girder for node 27 to 37 in radians

From Figure 68 to 70, the torsional response is visualized in different directions in the same fashion as the displacement found in the nodes and along the bridge girder. From the results in Abaqus, it was found that the center of the bridge, at node 18, holds the largest torsional response with an angle of 0.00053 radians. This corresponds to 5.43 mm in difference of vertical displacement at each side of the girder. This peak appears at approximately the 65s mark of the time series and can be seen in Figure 69. The nodes after node 18 show a

descending trend until the torsion is displayed in negative radians in Figure 70, meaning twisting motion in the opposite direction.

### 6.3.3 Cable forces

To compare the axial forces in the cable elements from the Abaqus analysis, a simplified calculation of the main cable’s tension force due to dead load only is made. The simplified calculation was found at [35], using the assumption of the sag-to-length ratio being within 0.1, which is close enough for this case with a ratio of 0.10089.

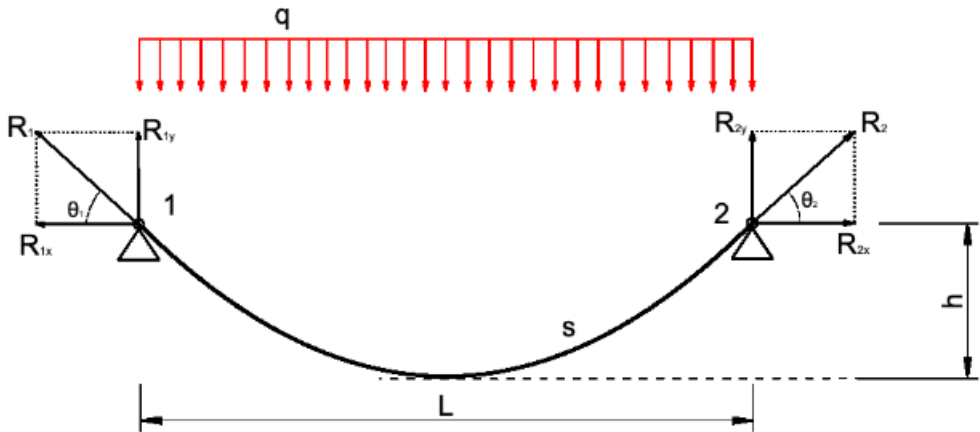


Figure 71 Uniformly loaded cable, from [35]

- $w_{girder(+)} = 5350 \frac{kg}{m} = 52483.5 \frac{N}{m}$
- $w_{cable(+)} = 408 \frac{kg}{m} = 4002.5 \frac{N}{m}$
- $q = w_{girder(+)} + w_{cable(+)}$
- $L = 446m$
- $h = 45m$

The equation accounts for the parameters: the value of distributed dead load, the horizontal length of main span, and the sag of the main cable. The weight of “girder (+)” includes the girder dead load and half of the hanger weight. The weight of “cable (+)” includes the main

cable dead load and the other half of the hanger weight. It is worth noting that this method assumes constant inclination, which is not really the case due to the hanger cables.

$$R_{1x} = R_{2x} = \frac{qL^2}{8h} = 31211025.5 \text{ N}$$

$$R_{1y} = R_{2y} = \frac{qL}{2} = 12596378 \text{ N}$$

$$R_1 = R_2 = \sqrt{R_x^2 + R_y^2} = 33.657 * 10^6 \text{ N}$$

Per cable plane makes  $R_1/2 = R_2/2 = 16.8285 * 10^6 \text{ N}$ . With this assumption, the main cables are exposed to an average tensional force of 16828.5 kN per cable plan. This will be used to compare the results from Abaqus to identify either reasonable or unreasonable values. Note that the Abaqus results should be higher at the maximums and varying in magnitude depending on which cable element is in focus due to the application of traffic acceleration and correct inclination at each hanger point.

From Abaqus, the following cable element forces have been requested:

- SF1: Axial force
- SF2: Transverse shear force in local 2-direction
- SF3: Transverse shear force in local 1-direction
- SM1: Bending moment 1-axis
- SM2: Bending moment 2-axis
- SM3: Twisting moment about the beam axis

Figures 72 to 74 below display the axial force in one cable element pair at a time. Due to the inclination and larger loading area at the two ends of the bridge, the positions closest to the

towers and in the middle of the span are interesting to investigate. These locations should host the maximums and minimums values of tensional axial force, respectively. For the midspan elements, numbers 2018/1018 and 2019/1019 share this location. However, visual inspection in Abaqus shows that element 2019/1019 holds the lowest axial force and will therefore be presented here.

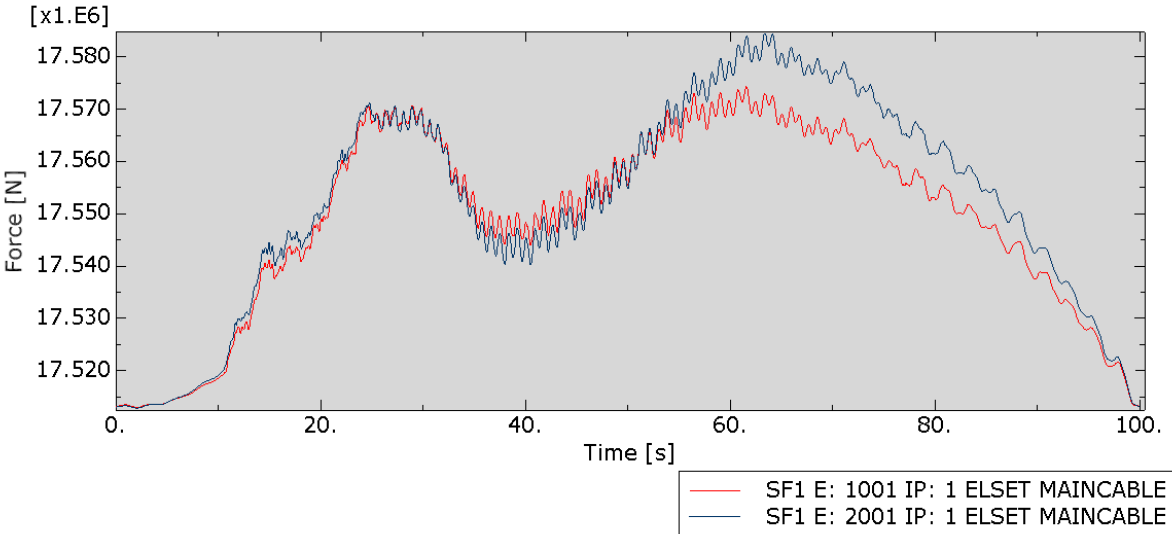


Figure 72 Output axial force in the main cable elements 1001 and 2001, located closest to the north tower

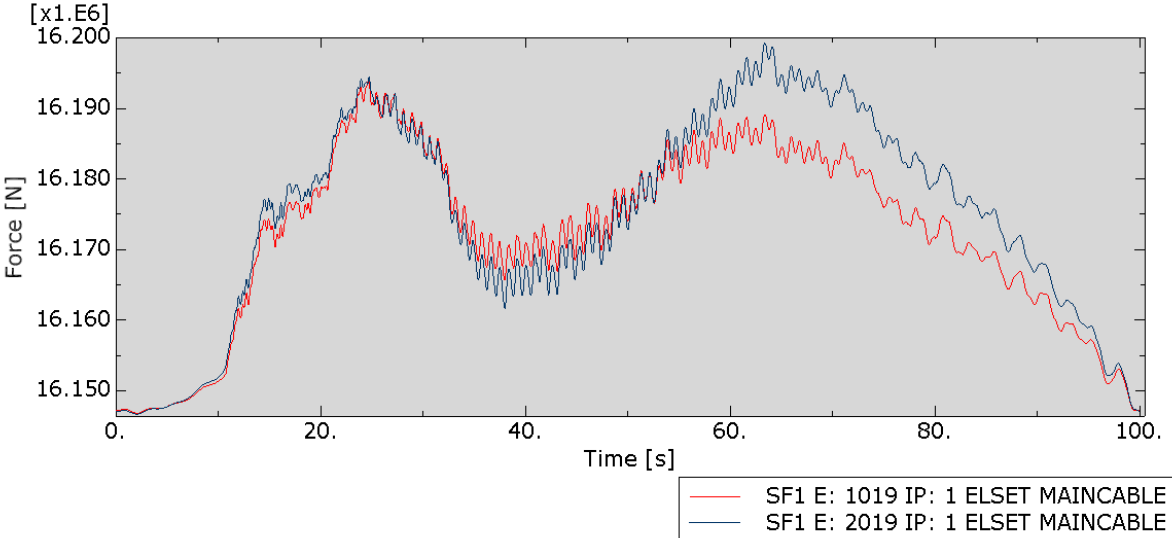


Figure 73 Output axial force in the main cable elements 1019 and 2019, located at the middle of the main span

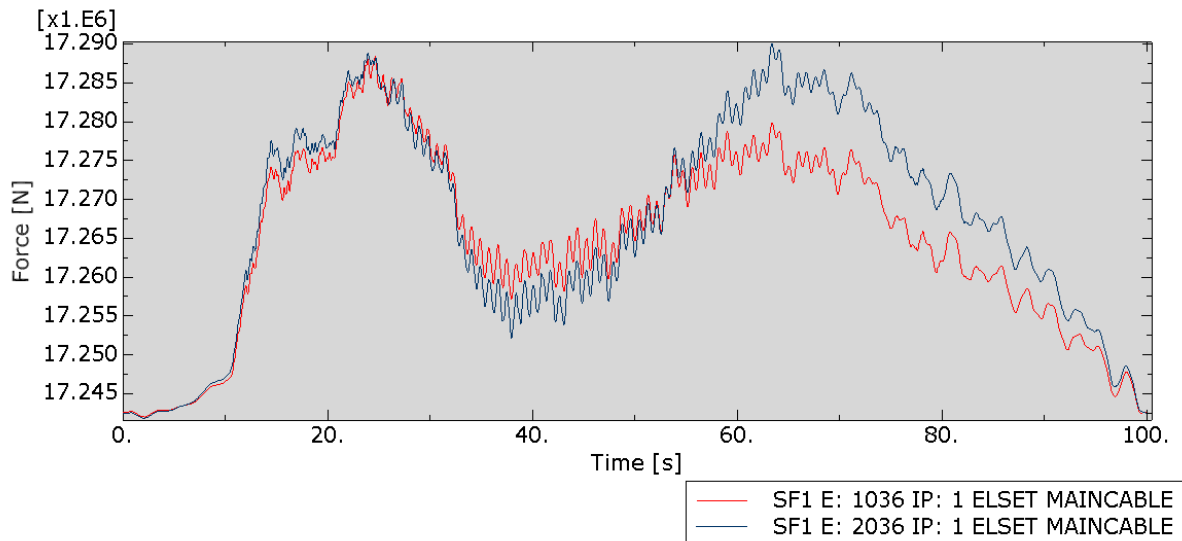


Figure 74 Output axial force in the main cable elements 1036 and 2036, located closest to the south tower

The results show that the eastside cable (ID 2001-2036) is generally experiencing higher tensile forces in the peak locations compared to the westside cable. This has also been confirmed in the deflection graphs as the eastside nodes in the middle of the span experience slightly higher vertical deformations in the corresponding periods. The maximum values fluctuate between 16199 kN and 17584 kN, making them reasonable compared to the simplified calculation of the axial forces. The forces also fluctuate in the same arrangement as the deflection plots shown earlier, meaning good relation and response to the applied traffic accelerations.

Another mode-identification can be investigated based on the axial force. By studying the same time period, i.e., between 40s and 80s, a total of 42 amplitudes is registered. This results in a damped period of 0.95s, or approximately 1.05 Hz. This means that the cable gets excited by the area involving the asymmetrical vertical modes, the symmetric torsional mode, and the third asymmetrical horizontal mode in this period (ref. Table 8).

From the static analysis, the previous axial forces can be obtained. Further, the forces from figures 72 to 74 can be subtracted from the static forces to isolate the traffic-induced forces only. This has been done for the maximum and minimum values of axial forces (Table 12).

Table 12 Contribution from static and dynamic analysis in relation to axial force

Node ID	Dynamic analysis [kN]	Static analysis [kN]	Isolated value [kN]	Increase [%]
1001	17574.3	17513.2	61.1	0.35
2001	17584.4	17513	71.4	0.41
1019	16194.1	16147.2	46.9	0.29
2019	16199.3	16147.1	52.2	0.32
1036	17288.4	17242.6	45.8	0.27
2036	17290	17242.4	47.6	0.28

From Table 12, the traffic-induced acceleration is shown to produce very little axial force in the main cables in comparison to the dead loads. From Tveiten's analysis, vertical loads were applied according to the design traffic loading for a lorry running on the east side of the bridge. Based on those results, not more than a 3% increase in cable forces compared to the dead loads was found. With respect to the design loads including several safety factors, and uses conservative load magnitudes and traffic scenarios, the results from Table 11, with a maximum of 0.41% increase, show quite sensible numbers [7].

### Bending moment and shear

Due to the material properties used for the cables (very low stiffness), they should not attract any significant bending moments. Only SM1 showed values of some interest, while the rest of the moments were negligible. In addition, as done for the axial forces, an isolating calculation is done for these results to identify how much the traffic-induced acceleration has contributed.

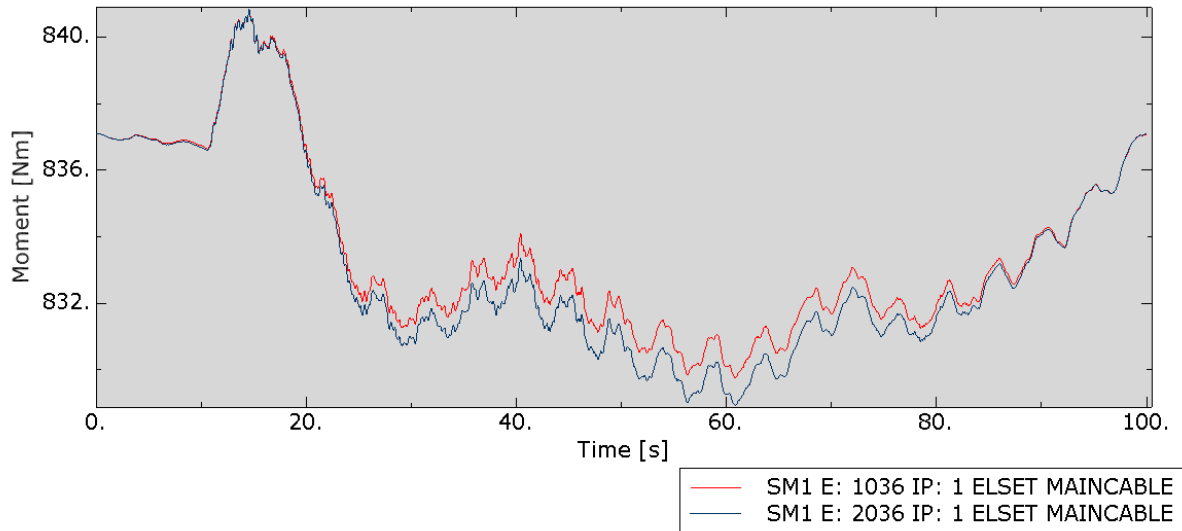


Figure 75 Maximum bending moments for elements 1036 and 2036

The maximum values of bending moments found along all cable elements were closest to the towers on the south side of the bridge, i.e., element numbers 1036 and 2036. A value of nearly 841 Nm appears after about 15 seconds in the time series, see Figure 75. This is a very small bending moment covering a cable element of over 20 meters in length. It was also found by investigating the results from the static analysis that over 99% of the bending moments originate from the dead loads (ref. Table 13), which was also the case for the axial forces.

Table 13 Contribution from static and dynamic analysis in relation to bending moments

Node ID	Dynamic analysis [Nm]	Static analysis [Nm]	Isolated value [Nm]	Increase [%]
1036	840.832	837.082	3.75	0.45
2036	840.831	837.083	3.748	0.45

The shear force corresponding to these bending moments, i.e., transverse shear force in the local 2-direction (SF2), was found to be in a similar fashion of small values (ref. Figure 76). Also, after isolating the values from the dead loads, it shows that only 1 Newton of the response originates from the dynamic loading.



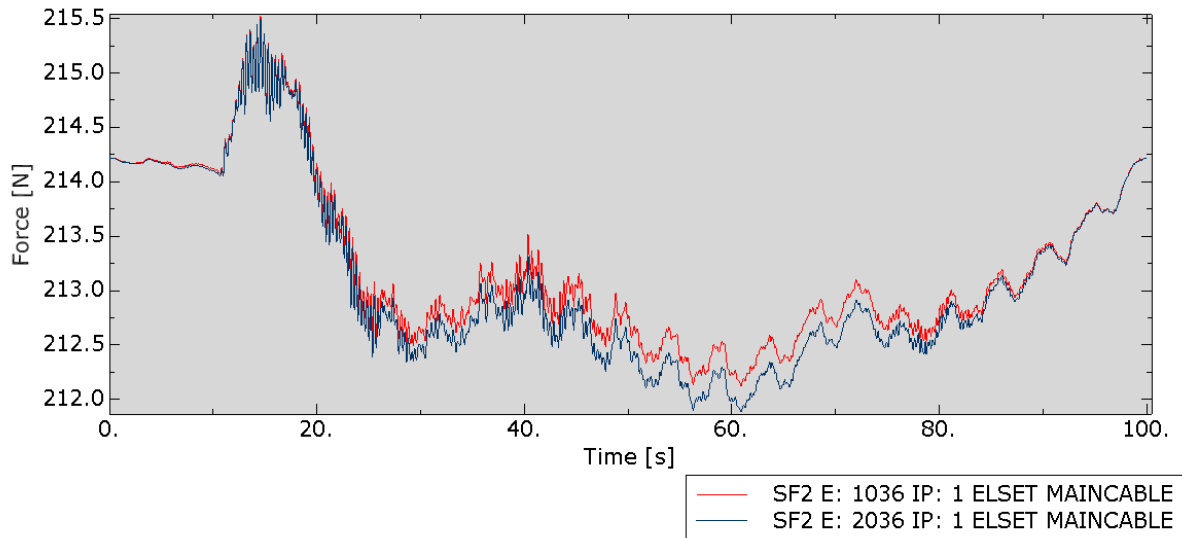


Figure 76 Maximum shear force of the main cable, located at elements 1036 and 2036

## 6.4 Discussion

Throughout the development of the Abaqus model and during the analyses, some obstacles and challenges were encountered, particularly during the dynamic modeling and analysis phase. Extensive periods and multiple attempts were spent in order to find a correct approach to perform a dynamic analysis that aligned with the desired specifications. The main challenge was achieving the modifications of the time steps and increments in such a way that the entire time series could be examined in intervals corresponding to the sampling frequency. Various dynamic analysis approaches offered by Abaqus were explored, ultimately leading to the application of a direct dynamic analysis. This type of dynamic analysis gives the user complete control of the incremental views and calculations by appropriately configuring the time steps, number of increments, and increment size in the input file.

The idea of omitting the H24 node pair for validation purposes did not result as satisfying as expected, as the simulated acceleration showed greater magnitudes compared to the recorded data. Efforts involving modification and recalculation of the damping coefficients were performed, but no significant breakthroughs were achieved. The obtained eigenfrequencies and eigenmodes showed a good correlation with previous measured and calculated results. These results laid the foundation for the calculation of Rayleigh's damping coefficients. However, there remains an opportunity for deeper studies into the relationship between specific structural components by involving Power Spectral Density functions. This additional analysis would enable a more comprehensive understanding of the damping distribution across the entire bridge. Through this, a better understanding of the error encountered during the H24 validation could be achieved.

The displacement results of the bridge girder revealed that the traffic-induced acceleration resulted in minimal deformations, especially when compared to the effects of dead loads. The nodal displacement exhibited distinct patterns associated with vehicle behavior and characteristics, including velocity, direction, arrival, and departure times. It should also be possible to estimate the weight of the vehicle based on these graphs and some additional calculations; however, this aspect was not explored due to limitations and time constraints. Further investigation into the displacement pattern along the girder length, on a global scale, showed correspondence with the nodal displacements. Both the nodal- and global displacement graphs displayed peak values occurring after approximately 60-65 seconds. Additionally, the torsional response of the bridge girder showed to be non-concerning, with the maximum rotation at the middle of the bridge span, measuring 0.00053 radians, equivalent to  $0.03^\circ$  or a vertical displacement difference of 5.43 mm on each side of the girder. The displacement plots and the torsional response plots corresponded with peak values after 60-65 seconds, strengthening the assumption of a correct dynamic analysis.

Forces in the main cable have been examined by extracting axial force, bending moments, and shear force. Among the cable elements closest to the north tower, a maximum axial force of 17584.4 kN was found. However, only 0.41% of this force came from the traffic-induced acceleration. A simplified calculation of the axial force in the main cables was made to confirm if the results were reasonable. The simplified calculation resulted in a value of  $16828.5 * 10^3 \text{ kN}$  per cable plane. The simulated results from Abaqus showed a fluctuating maximum between 16199 kN and 17584 kN, meaning the simplified estimate and the Abaqus result corresponded well. Despite the expectation that the cables should not attract significant bending moments and shear forces, these factors were still evaluated. The static and dynamic analyses revealed minimal bending moments and shear forces, with contribution from the dynamic loading below 1%.

The cable forces resulting from the traffic-induced vibrations contribute to repetitive impact, as seen in most of this chapter's force- and displacement graphs. Therefore, identifying which parts of the bridge experience significant magnitude and duration of response are vital to better understand fatigue due to stress localizations. Furthermore, even though the response from the vibrational loading is small, the entire bridge is affected, leading to a much larger response since the masses of the structure are inflicted with movement. Notably, it has been observed that during specific time periods, particularly just after 20 seconds and 60 seconds, there are instances of significant and prolonged displacement and axial force responses. These responses would be interesting to investigate further in relation to fatigue and in combination with other load effects, such as wind and temperature.

## 7. Conclusion

This thesis presents a dynamic analysis of the Lysefjord bridge conducted with the aid of a finite element model of the bridge created in Abaqus. Previous models developed by Tveiten [7] and Steigen [6] have been used as guidance when creating the Abaqus model. This work in this thesis is in the category of reviewing structural integrity and serves as research material given that the bridge has already existed for 25 years, and the dynamic loading originates from real-life recorded traffic-induced acceleration of the bridge deck. The traffic-induced acceleration is collected at the bridge site from accelerometers mounted inside the bridge girder and is processed for a computationally efficient finite element method analysis.

Since the opening of the bridge in 1997, and in the first 16 years in particular, the bridge experienced an abnormal trend of wire fractures in the main cable following an almost linear pattern of increase in fractures. These fractures have been studied in detail by the NPRA and DNV, and later measured by acoustic sensors between 2009 and 2019. The results from these studies and measurements have been presented, and several reliable parties seem to agree on the reasoning behind the unusual number of fractures. The acoustic measurement data have been processed in several different ways to visualize relationships between trends, temperature, seasons, and technical dependencies. Time-varying loads acting on the bridge are briefly discussed, and weather data from weather stations on Sola and the bridge site have been collected to present the temperature and wind history. Suspension bridges and the Lysefjord bridge have been discussed, with their main structural components presented stepwise.

The finite element method is briefly defined, followed by a detailed description of the structure of the FEM-Design model and the Abaqus model. Three different modifications of the FEM-Design model were created to provide accurate bending stiffness contributions to cover the mass moment of inertia calculations and the general response of the girder. Arranging the dynamic loading step in the desired manner has presented several challenges, yet, in the end, a satisfying arrangement was achieved. The dynamic loading step is created as a direct dynamic step, giving the author direct control of increment and timestep measures to review and plot the results by each applied amplitude with corresponding time increments. A total of six sets of acceleration data, consisting of 2500 amplitudes with a 25Hz sampling frequency each, have been applied in the model at the corresponding accelerometer locations as nonzero acceleration/angular acceleration boundary conditions.

A comprehensive inspection and calculation of the eigenfrequencies and eigenmodes have been carried out, demonstrating a good correlation with previous findings. An investigation of the damping of the bridge has been thoroughly investigated due to a mismatch between simulated acceleration and recorded acceleration on the H24 test node-pair. Specific modes were identified for damping analysis, along with an exploration of the relationship between tower response and girder response. Modifications to the Rayleigh damping were studied and discussed based on Power Spectral Density functions of the vehicle-induced vibration.

Nodal and global displacements due to the dynamic loading are presented and analyzed. The nodal displacements are used to identify vehicle patterns and activity. Torsional activities of the bridge deck are plotted and discussed based on timewise heterogeneous load cases on each traffic lane. Cable forces are extracted, compared to theoretical values, and isolated to review the dynamic contributions. The results from the cable forces show that the dynamic loading has an extremely low contribution compared to the static dead load. The axial loading of the cable is compared to a design traffic load analysis from Tveiten. There, it was found that the traffic contributes to around 3% of the total axial loading; meanwhile, only 0.41% was found in this thesis. Some of the differences here most likely originate from safety factors and conservative magnitudes from the design loads.

The analysis of displacements confirmed that the traffic-induced acceleration had minimal impact on deformations compared to the dead loads. Nodal displacements have provided information concerning vehicle behavior, while displacement patterns along the girder length aligned with the nodal displacements. The torsional response of the bridge girder was not of concerning matter, with minimal values of rotational displacement. The study of cable forces revealed maximum axial forces in the elements closest to the north tower, with the traffic-induced acceleration contributing only a tiny percentage. Bending moments and shear forces were minimal in both the static and dynamic analysis. To conclude, the work carried out in this thesis has contributed to a successful investigation of the dynamic behavior of the Lysefjord bridge, providing valuable insights and an experience in such type of analysis. The findings lay the groundwork for further studies regarding this matter.

## 8. Further recommendations

Expanding upon this study by combining recorded wind data with the recorded traffic-induced acceleration would be an interesting extension that can increase the understanding of the bridge's dynamic behavior. Additionally, conducting fatigue assessments by examining the frequency excitation on components experiencing localized stress, such as joints connecting the main cables and girder with the hangers and the towers, would provide valuable insights.

Further exploration of the different dynamic approaches offered by Abaqus to determine the most suitable one is of great interest to achieve more detailed results. Subroutines are widely used in Abaqus, and it would be worthwhile to compare this with the regular implicit and explicit approaches. Additionally, applying influence lines to simulate moving traffic along the length of the girder could distribute the vibration differently from the approach employed in this thesis. Numerous possibilities exist for the application approaches of acceleration to be further investigated. Several software options exist that are more applicable for bridge design, which inhabits specialized finite element approaches and interfaces. Creating a new bridge model in one of these software programs to compare with the Abaqus model would be beneficial. Since Abaqus is a general-purpose software, the bridge design may be influenced by certain simplifications and unconventional approaches, which could be avoided by using a software designed for bridge-related analyses.

The complete dataset of recorded acceleration contains several other instances of heavy vehicle scenarios to be analyzed. For example, between sample number 185000 and 190000 (ref. Figure 37), two heavy vehicles seem to occur in quick succession. Examining the response of these instances and comparing them with the results from this thesis could provide better insight into specific events' reasons. Due to the time limitations within this thesis, some of the results have only been briefly discussed, leaving plenty of exciting events left for further studies.

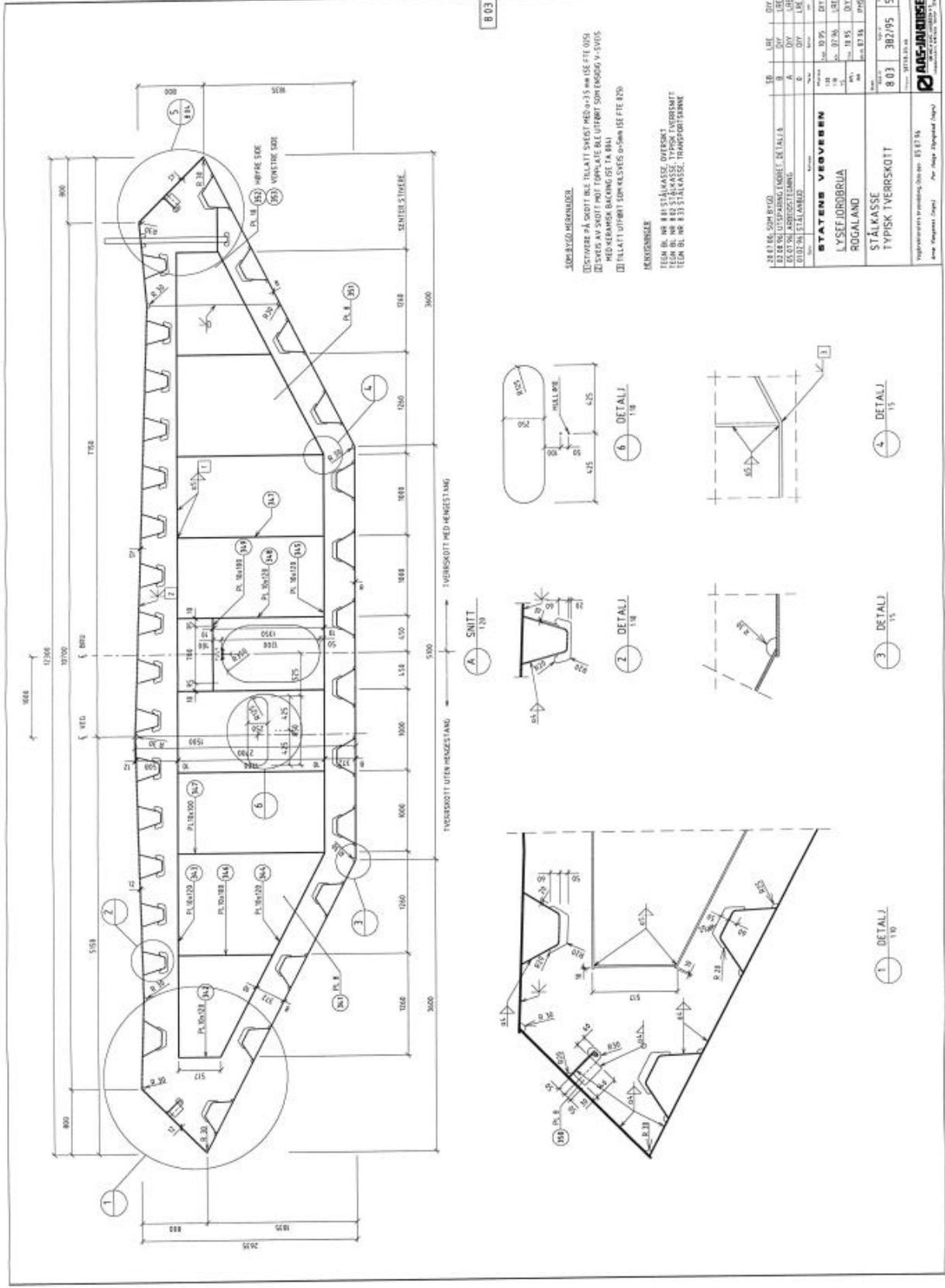
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 3) FILLATT UTPÅRT SUPPÅLVES 0-5mm ISE FTE 826

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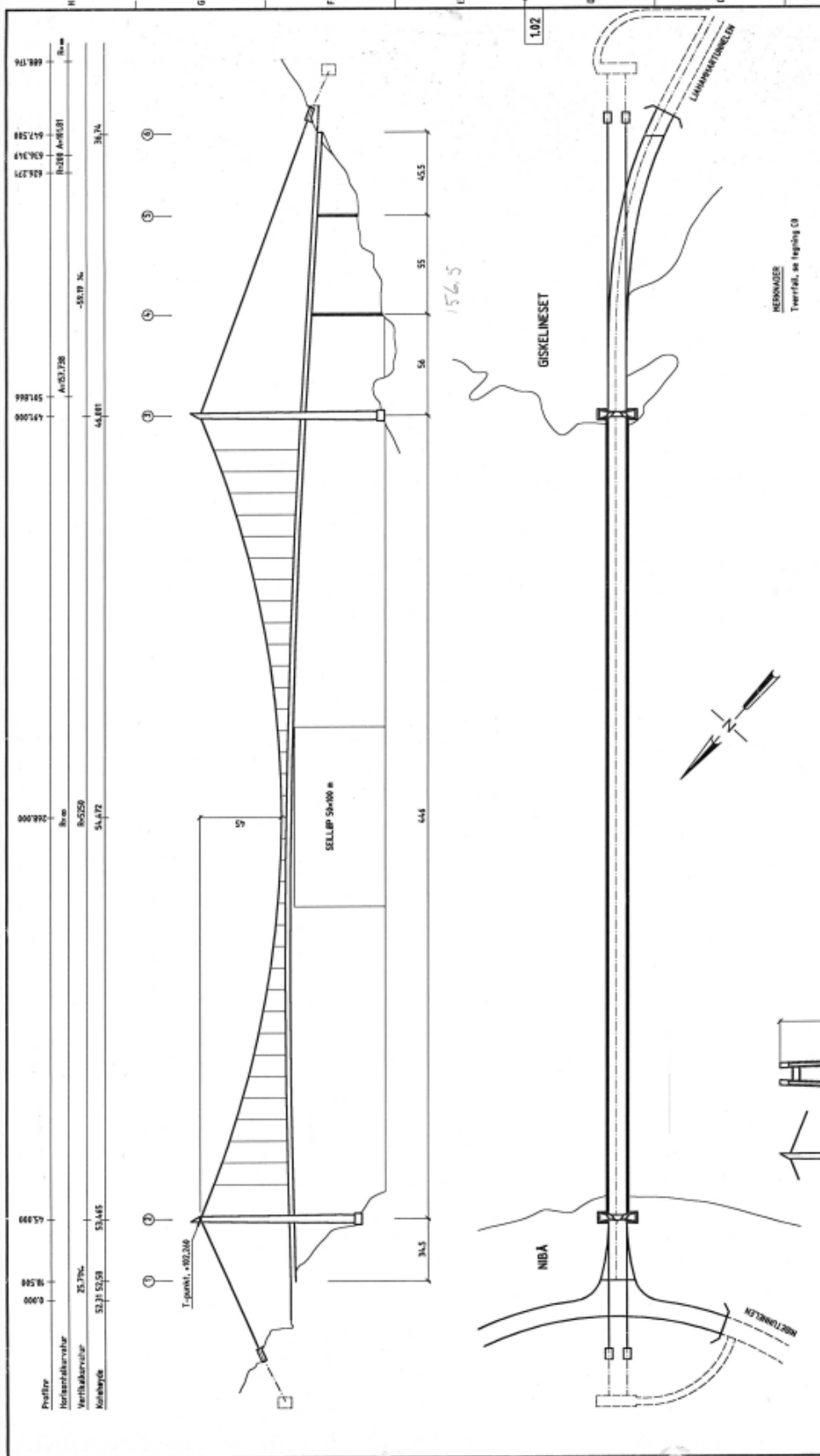
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 STÅLKASSE  
 TYPISK TVERRSKOTT  
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 Tegningens forfatter: (Støt 1) E 17/14  
 Tegningens dato: 14/10/99  
 Tegningens tittel: (Støt 1) Støt 1









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18.07.75	Parasolte ved sønn 18m	A	PMS	
17.05.75	Arbeid		PMS	

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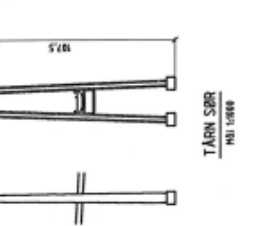
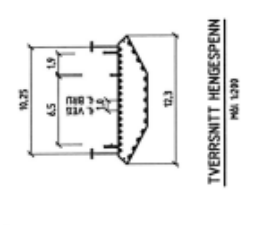
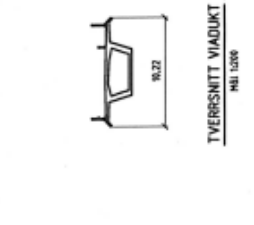
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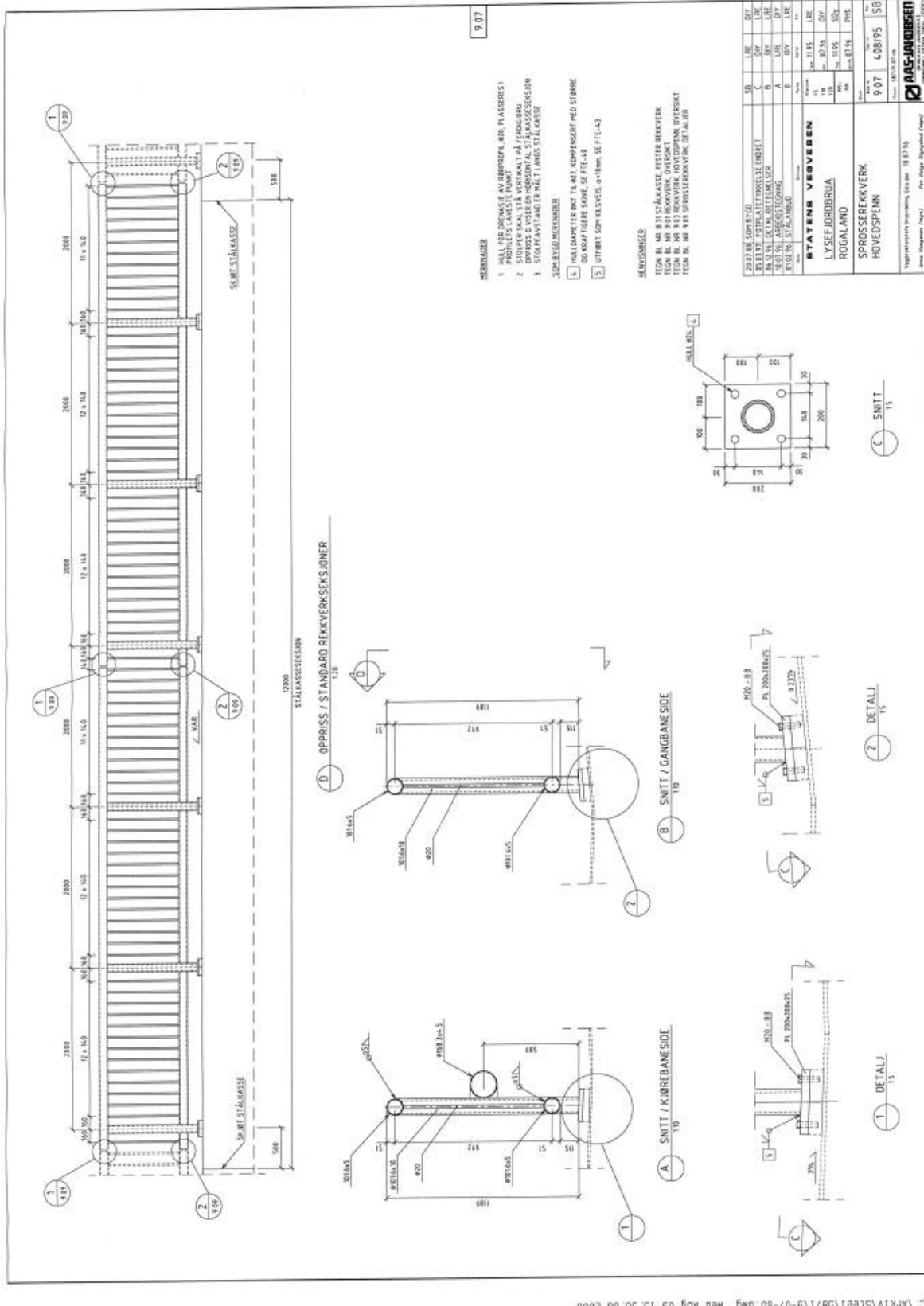
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Anne Vangnes tegner





- 9 07**
- REINFORSE**
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- SOM BYGGES HENHOLDE**
- L HULLDAMPETER BENT FLAKET, KOMPENSERT MED STØRRE OG KOMPENSERT SVIK, SE FTE-14
  - S UTFØRT SOM KLEVED, A RILM, SE FTE-43

**HENDSOMMER**

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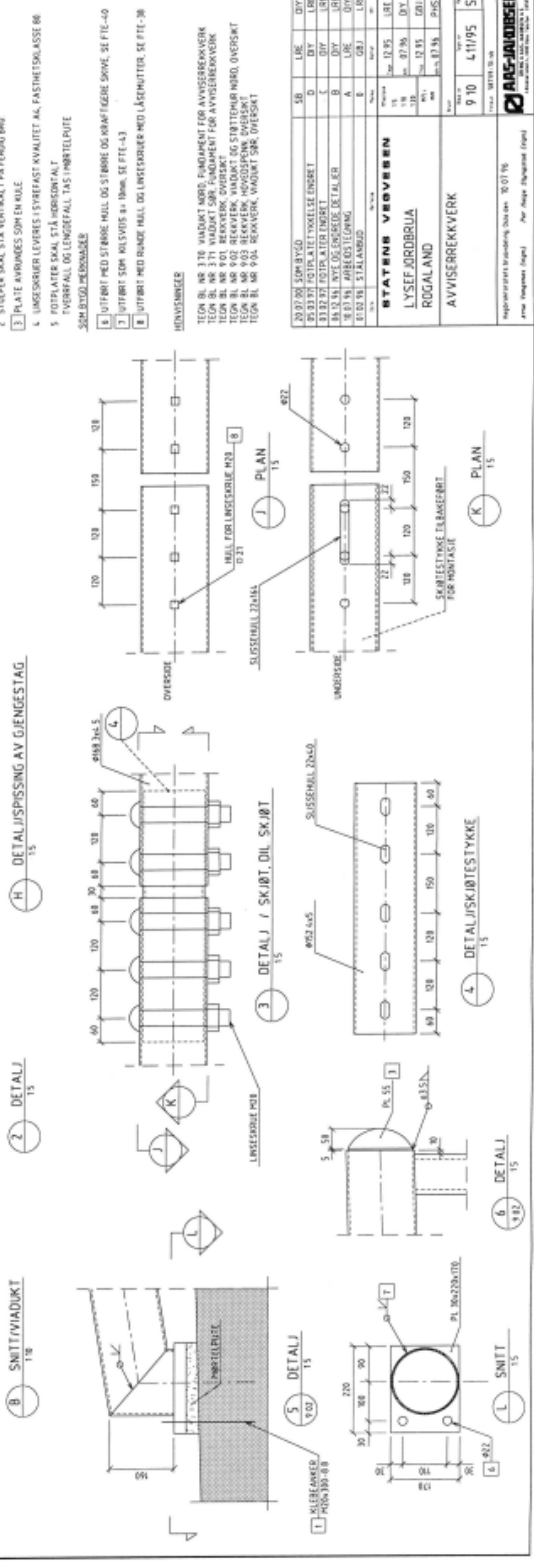
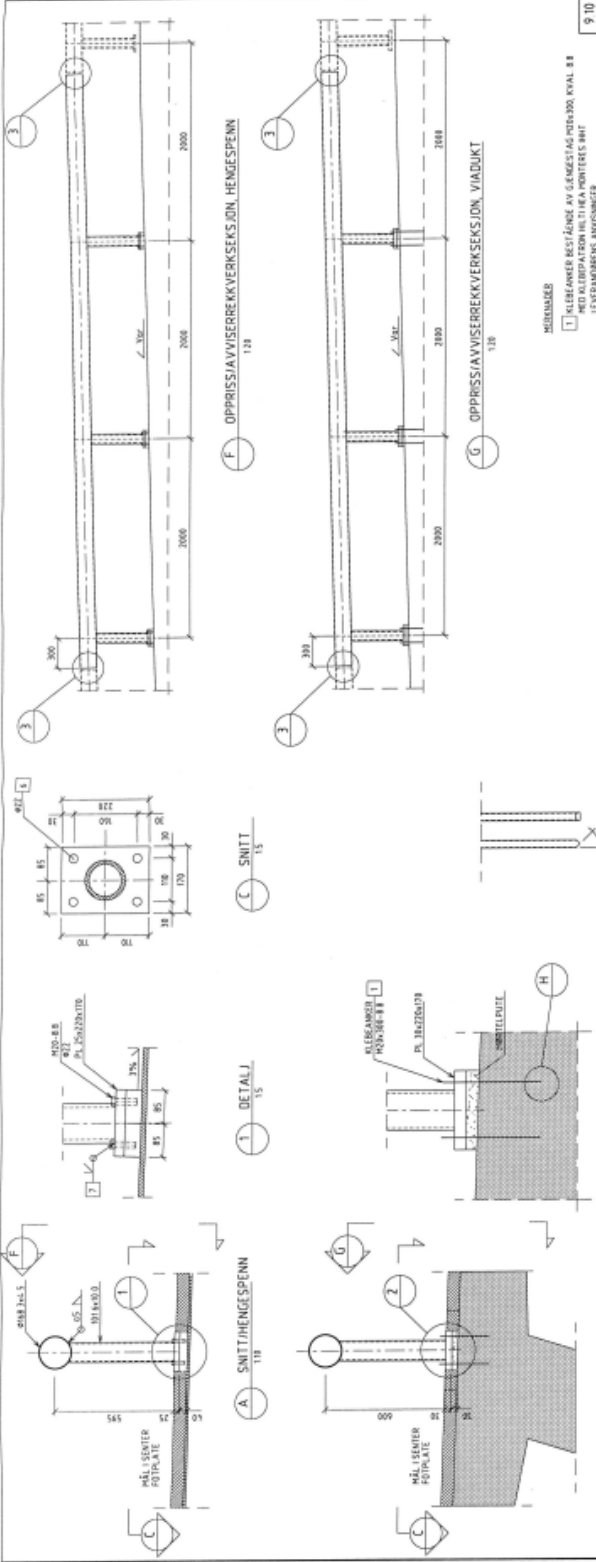
  

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- 5) FØTTEPLATE PÅL, STA. HØRSDRIFAL
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- 7) UTTRIKT SOM ALVITIS 4+ (NÅM, SE FTE-43)
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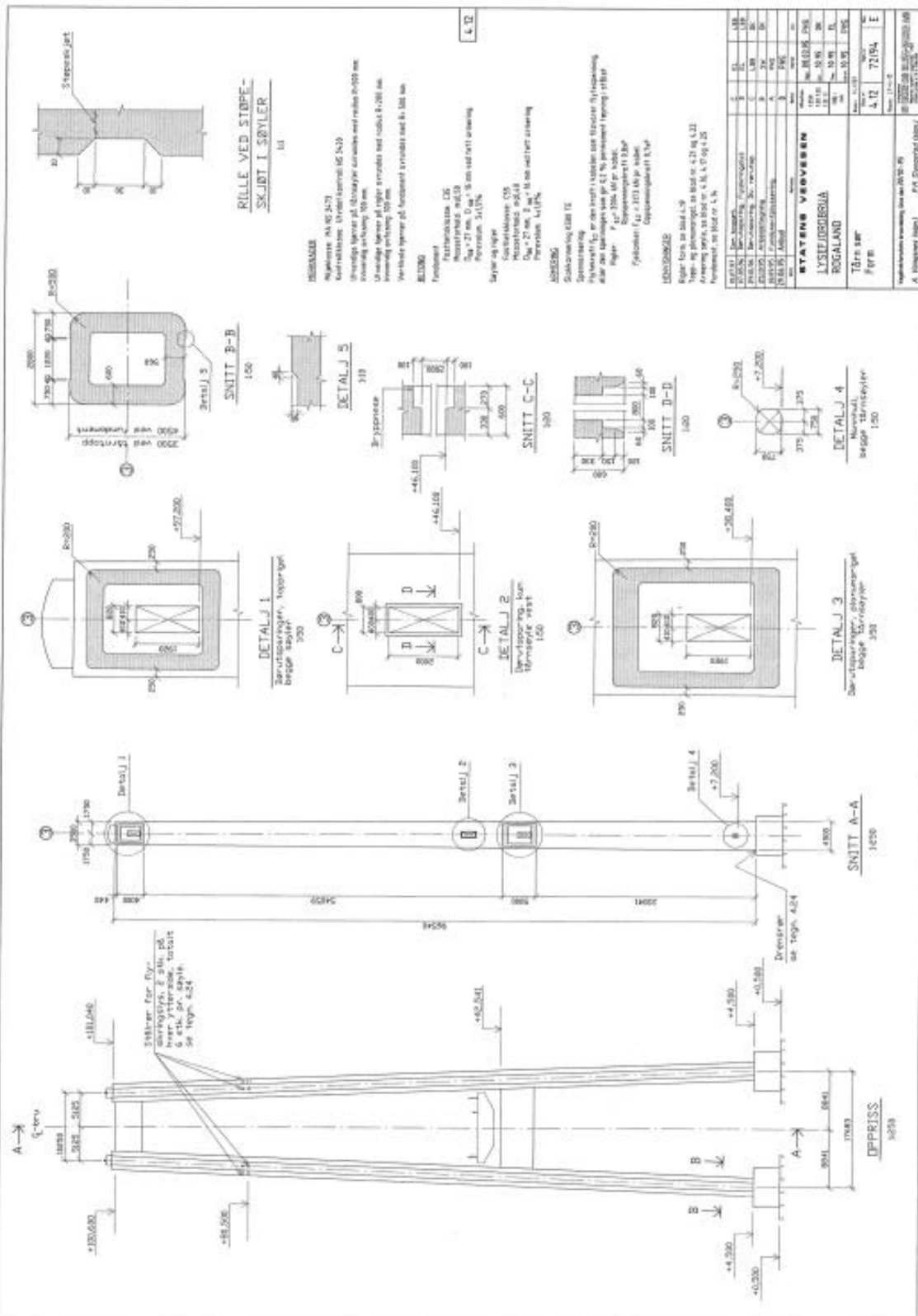
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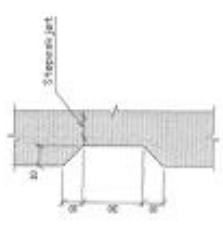
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 RØDALAND  
 AVVISERREKKEVERK

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 Prosjektleder: [Name]  
 Prosjektansvarlig: [Name]



RILLE VED STØPPE-  
SKJØT I SPYLER



**REKVISITER**  
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Byggherrens navn: Lyste Jørgensen  
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 Byggherrens telefon: 021 23 45 67  
 Byggherrens e-post: lyste.jorgensen@lyste.no  
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## APPENDIX B: GEOMETRY OF BRIDGE GIRDER AND MAIN CABLE

### Equation for bridge deck

$$\begin{aligned}
 A_x &:= 45 & C_x &:= 268 & B_x &:= 491 \\
 A_y &:= 53.465 & C_y &:= 54.472 & B_y &:= 46.001 \\
 X_p &:= \frac{A_x + C_x}{2} & X_Q &:= \frac{B_x + C_x}{2} \\
 Y_p &:= \frac{A_y + C_y}{2} & Y_Q &:= \frac{B_y + C_y}{2} \\
 m_{AC} &:= \frac{C_y - A_y}{C_x - A_x} & m_{OP} &:= \frac{-1}{m_{AC}} \\
 m_{CB} &:= \frac{B_y - C_y}{B_x - C_x} & m_{OQ} &:= \frac{-1}{m_{CB}} \\
 y_o &:= \frac{\left( \frac{-m_{OP} \cdot Y_Q + X_Q \cdot m_{OP} - m_{OP} \cdot X_p + Y_p}{m_{OQ}} \right)}{\left( 1 - \frac{m_{OP}}{m_{OQ}} \right)} = -5196.149 \\
 x_o &:= \frac{y_o - Y_Q}{m_{OQ}} + X_Q = 180.208
 \end{aligned}$$

$$r := \sqrt{(A_x - x_o)^2 + (A_y - y_o)^2} = 5251.355$$

$$r^2 = 27576728.452$$

$$(x - x_o)^2 + (y - y_o)^2 = ? r^2$$

$$(x - 180.208)^2 + (y + 5196.149)^2 = ? 27576728.452$$

$$k := r^2 - 180.208 \cdot 180.208 = 27544253.529$$

$$2 \cdot 180.208 = 360.416$$

$$f(x) := \sqrt{-x^2} + 2 \cdot 180.208 x + k - 5196.149$$

$$f(45) = 53.465$$

$$f(64) = 53.92$$

$$f(76) = 54.172$$

$$f(88) = 54.396$$

$$f(100) = 54.593$$

$$f(112) = 54.763$$

$$f(124) = 54.905$$

$$f(136) = 55.02$$

$$f(148) = 55.107$$

$$f(160) = 55.167$$

$$f(172) = 55.2$$

$$f(184) = 55.205$$

$$f(196) = 55.182$$

$$f(208) = 55.132$$

$$f(220) = 55.055$$

$$f(232) = 54.951$$

$$f(244) = 54.818$$

$$f(256) = 54.659$$

$$f(268) = 54.472$$

$$f(280) = 54.258$$

$$f(292) = 54.016$$

$$f(304) = 53.747$$

$$f(316) = 53.45$$

$$f(328) = 53.126$$

$$f(340) = 52.774$$

$$f(352) = 52.395$$

$$f(364) = 51.989$$

$$f(376) = 51.555$$

$$f(388) = 51.093$$

$$f(400) = 50.604$$

$$f(412) = 50.088$$

$$f(424) = 49.544$$

$$f(436) = 48.972$$

$$f(448) = 48.373$$

$$f(460) = 47.747$$

$$f(472) = 47.093$$

$$f(491) = 46.001$$

2x19m in the ends and  
34x12m otherwise

Created with Mathcad Express. See [www.mathcad.com](http://www.mathcad.com) for more information.

## **NODES MAIN CABLE**

$$f(x) := \frac{45}{49729} x^2 - \frac{90}{223} x + 102.26$$

$f(0) = 102.26$   
 $f(19) = 94.9185091194273$   
 $f(31) = 90.6184025417764$   
 $f(43) = 86.5789084839832$   
 $f(55) = 82.8000269460476$   
 $f(67) = 79.2817579279696$   
 $f(79) = 76.0241014297493$   
 $f(91) = 73.0270574513865$   
 $f(103) = 70.2906259928814$   
 $f(115) = 67.814807054234$   
 $f(127) = 65.5996006354441$   
 $f(139) = 63.6450067365119$   
 $f(151) = 61.9510253574373$   
 $f(163) = 60.5176564982204$   
 $f(175) = 59.344900158861$   
 $f(187) = 58.4327563393593$   
 $f(199) = 57.7812250397153$   
 $f(211) = 57.3903062599288$   
 $f(223) = 57.26$   
 $f(235) = 57.3903062599288$   
 $f(247) = 57.7812250397153$   
 $f(259) = 58.4327563393593$   
 $f(271) = 59.344900158861$   
 $f(283) = 60.5176564982204$   
 $f(295) = 61.9510253574373$   
 $f(307) = 63.6450067365119$   
 $f(319) = 65.5996006354441$   
 $f(331) = 67.814807054234$   
 $f(343) = 70.2906259928814$   
 $f(355) = 73.0270574513865$   
 $f(367) = 76.0241014297493$   
 $f(379) = 79.2817579279696$   
 $f(391) = 82.8000269460476$   
 $f(403) = 86.5789084839832$   
 $f(415) = 90.6184025417764$   
 $f(427) = 94.9185091194273$   
 $f(446) = 102.26$

Created with Mathcad Express. See [www.mathcad.com](http://www.mathcad.com) for more information.

## APPENDIX C: MATLAB SCRIPTS

### Acceleration data and processing:

```
load('BasicData2017_03_07.mat')

% limiting what time-series I want to study
N=11001:16000;
Az=Aoz_E(:,N);
t1=1/50*[0:length(Az)-1];

N=11001:16000;
Az2=Aoz_W(:,N);
t2=1/50*[0:length(Az2)-1];

% Resample from 50 Hz to 25 Hz, i.e. with a factor 2
AzR1=resample(Az',1,2);
tR=1/25*[0:length(AzR1)-1]; % Associated time, with a time-step 0.04 s

AzR2=resample(Az2',1,2);
tR2=1/25*[0:length(AzR2)-1];

A_E=detrend((AzR1));
A_W=detrend((AzR2));

T=table([A_E],[A_W])
writetable(T,'Acc_11001_16000.xls')
```

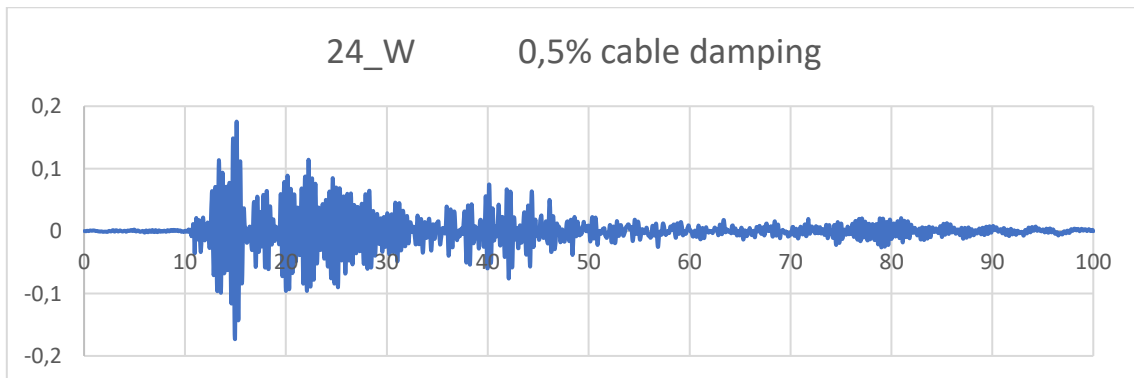
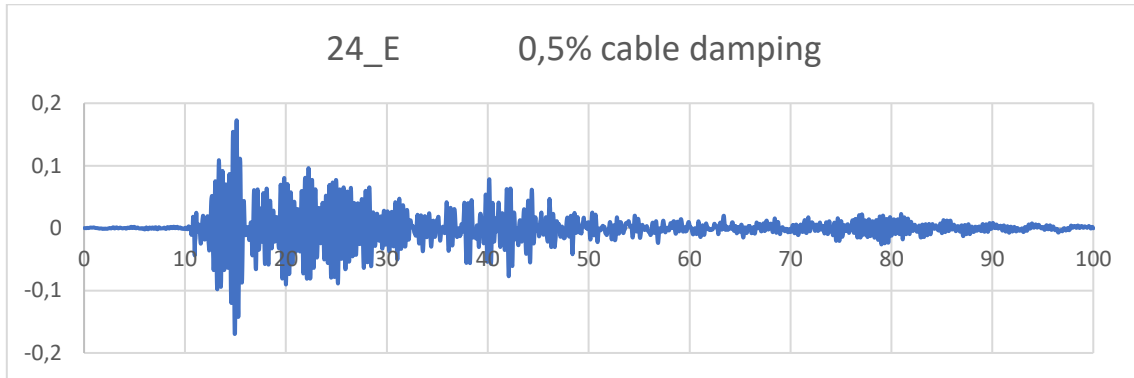
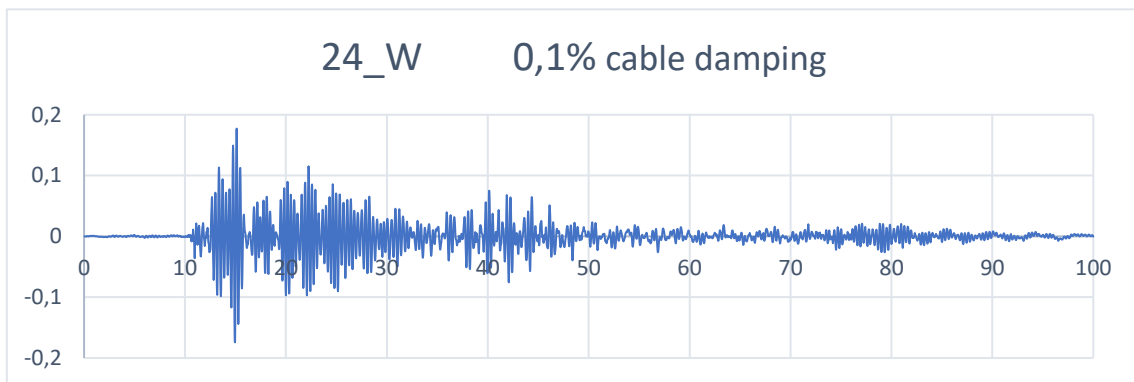
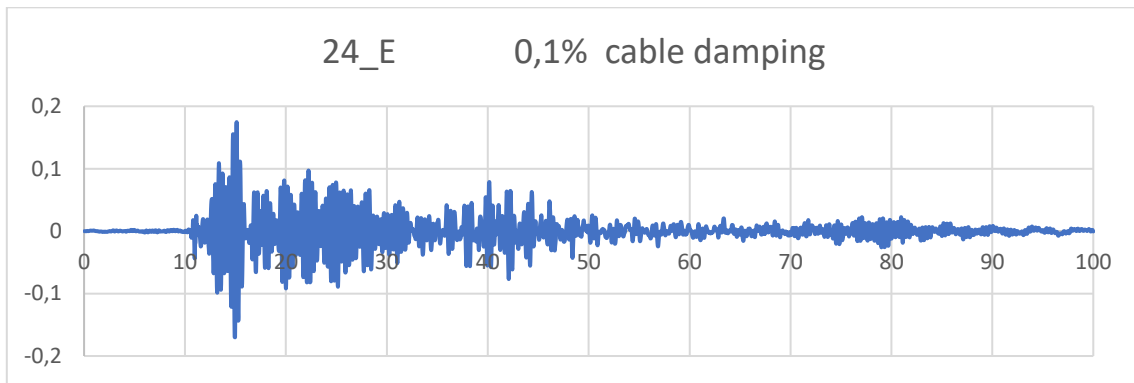
### Mass point calculations:

```
syms M_1 M_2 r
eq1 = M_2*r.^2 + 2*M_1*(1.263^2 + 5.125^2) - 55807.331;
eq2 = 2*M_1 + M_2 - 5350;
eq3 = ((2*2.7*M_1 + M_2*(1.437-r))/5350)-2.056;

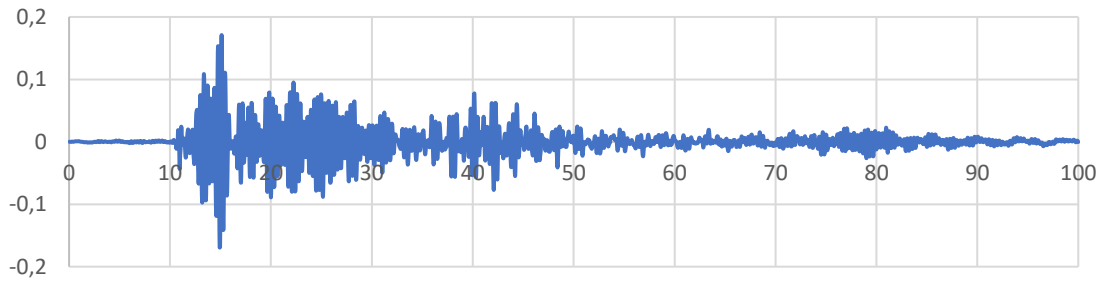
eqs = [eq1,eq2, eq3];
[M_1,M_2,r] = vpasolve(eqs, [M_1,M_2,r])

M=2*M_1(2)+M_2(2)
```

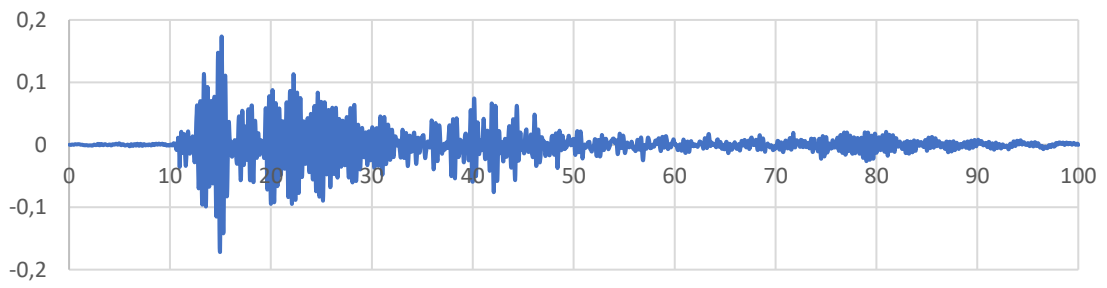
## APPENDIX D: DAMPING MODIFICATION



24\_E 1% cable damping



24\_W 1% cable damping





## APPENDIX E: INPUT FILES

### File LYSEFJORD\_B31.inp

\*HEADING

Lysefjordbroen

\*\* Author: Marcus Abbedissen

\*\* Date: 12.02.2023

\*\*

\*PREPRINT, , HISTORY=YES

\*\*

\*NODE

\*\*

\*\*\*\*\* Nodes for the stiffening girder/deck (NA)

\*\*

1	,	44.9513423	,	0	,	53.46975951
2	,	64.000	,	0	,	54.292624
3	,	76.000	,	0	,	54.766754
4	,	88.000	,	0	,	55.203235
5	,	100.00	,	0	,	55.60247
6	,	112.00	,	0	,	55.96398
7	,	124.00	,	0	,	56.28633
8	,	136.00	,	0	,	56.57014
9	,	148.00	,	0	,	56.81403
10	,	160.00	,	0	,	57.01866
11	,	172.00	,	0	,	57.18368
12	,	184.00	,	0	,	57.30775
13	,	196.00	,	0	,	57.39053
14	,	208.00	,	0	,	57.43267
15	,	220.00	,	0	,	57.43381
16	,	232.00	,	0	,	57.3936
17	,	244.00	,	0	,	57.31066
18	,	256.00	,	0	,	57.18463
19	,	268.00	,	0	,	57.01612
20	,	280.00	,	0	,	56.80477
21	,	292.00	,	0	,	56.5493
22	,	304.00	,	0	,	56.2505
23	,	316.00	,	0	,	55.90725
24	,	328.00	,	0	,	55.52052
25	,	340.00	,	0	,	55.08933
26	,	352.00	,	0	,	54.61478
27	,	364.00	,	0	,	54.097
28	,	376.00	,	0	,	53.53517
29	,	388.00	,	0	,	52.92952
30	,	400.00	,	0	,	52.28134
31	,	412.00	,	0	,	51.59095
32	,	424.00	,	0	,	50.85772
33	,	436.00	,	0	,	50.08313
34	,	448.00	,	0	,	49.266687
35	,	460.00	,	0	,	48.40905
36	,	472.00	,	0	,	47.51207
37	,	491.0612823	,	0	,	46.0146308

\*\*

\*\*\*\*\* Anchor bolt north-west

\*\*

981	,	-28.6797	,	5.1250	,	53.4647
-----	---	----------	---	--------	---	---------

\*\*

\*\*\*\*\* Nodes for the Main cables

\*\*\*\*\* WEST CABLE

\*\*

1001	,	44.786528	,	5.1250	,	102.2687784
1002	,	64.0000	,	5.1250	,	95.25627312
1003	,	76.0000	,	5.1250	,	91.17968854
1004	,	88.0000	,	5.1250	,	87.35639948
1005	,	100.000	,	5.1250	,	83.78365295
1006	,	112.000	,	5.1250	,	80.46063793
1007	,	124.000	,	5.1250	,	77.38689143
1008	,	136.000	,	5.1250	,	74.56200745
1009	,	148.000	,	5.1250	,	71.98561599
1010	,	160.000	,	5.1250	,	69.65735705
1011	,	172.000	,	5.1250	,	67.57688064
1012	,	184.000	,	5.1250	,	65.74384674
1013	,	196.000	,	5.1250	,	64.15791536
1014	,	208.000	,	5.1250	,	62.8187065
1015	,	220.000	,	5.1250	,	61.72582016
1016	,	232.000	,	5.1250	,	60.87870634
1017	,	244.000	,	5.1250	,	60.27666504
1018	,	256.000	,	5.1250	,	59.91897626
1019	,	268.000	,	5.1250	,	59.80549
1020	,	280.000	,	5.1250	,	59.93663626
1021	,	292.000	,	5.1250	,	60.31257504
1022	,	304.000	,	5.1250	,	60.93299634
1023	,	316.000	,	5.1250	,	61.79757016
1024	,	328.000	,	5.1250	,	62.9061065
1025	,	340.000	,	5.1250	,	64.25856536
1026	,	352.000	,	5.1250	,	65.85499674
1027	,	364.000	,	5.1250	,	67.69553064
1028	,	376.000	,	5.1250	,	69.78035705
1029	,	388.000	,	5.1250	,	72.10970599
1030	,	400.000	,	5.1250	,	74.68386745
1031	,	412.000	,	5.1250	,	77.50319143
1032	,	424.000	,	5.1250	,	80.56807793
1033	,	436.000	,	5.1250	,	83.87901695
1034	,	448.000	,	5.1250	,	87.43664648
1035	,	460.000	,	5.1250	,	91.24232954
1036	,	472.000	,	5.1250	,	95.30127512
1037	,	491.308929	,	5.1250	,	102.2671496

\*\*

\*\*\*\*\* Anchor bolt south-west

\*\*

1087	,	657.7097	,	5.1250	,	46.0009
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\*\*

\*\*\*\*\* Anchor bolt north-east

\*\*

1981	,	-28.96797	,	-5.125	,	53.4647
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\*\*

\*\*\*\*\* EAST CABLE

\*\*

2001	,	44.786528	,	-5.1250	,	102.2687784
2002	,	64.0000	,	-5.1250	,	95.25627312
2003	,	76.0000	,	-5.1250	,	91.17968854
2004	,	88.0000	,	-5.1250	,	87.35639948
2005	,	100.000	,	-5.1250	,	83.78365295
2006	,	112.000	,	-5.1250	,	80.46063793

2007	,	124.000	,	-5.1250	,	77.38689143
2008	,	136.000	,	-5.1250	,	74.56200745
2009	,	148.000	,	-5.1250	,	71.98561599
2010	,	160.000	,	-5.1250	,	69.65735705
2011	,	172.000	,	-5.1250	,	67.57688064
2012	,	184.000	,	-5.1250	,	65.74384674
2013	,	196.000	,	-5.1250	,	64.15791536
2014	,	208.000	,	-5.1250	,	62.8187065
2015	,	220.000	,	-5.1250	,	61.72582016
2016	,	232.000	,	-5.1250	,	60.87870634
2017	,	244.000	,	-5.1250	,	60.27666504
2018	,	256.000	,	-5.1250	,	59.91897626
2019	,	268.000	,	-5.1250	,	59.80549
2020	,	280.000	,	-5.1250	,	59.93663626
2021	,	292.000	,	-5.1250	,	60.31257504
2022	,	304.000	,	-5.1250	,	60.93299634
2023	,	316.000	,	-5.1250	,	61.79757016
2024	,	328.000	,	-5.1250	,	62.9061065
2025	,	340.000	,	-5.1250	,	64.25856536
2026	,	352.000	,	-5.1250	,	65.85499674
2027	,	364.000	,	-5.1250	,	67.69553064
2028	,	376.000	,	-5.1250	,	69.78035705
2029	,	388.000	,	-5.1250	,	72.10970599
2030	,	400.000	,	-5.1250	,	74.68386745
2031	,	412.000	,	-5.1250	,	77.50319143
2032	,	424.000	,	-5.1250	,	80.56807793
2033	,	436.000	,	-5.1250	,	83.87901695
2034	,	448.000	,	-5.1250	,	87.43664648
2035	,	460.000	,	-5.1250	,	91.24232954
2036	,	472.000	,	-5.1250	,	95.30127512
2037	,	491.308929	,	-5.1250	,	102.2671496

\*\*

\*\*\*\*\* Anchor , bolt south-east

\*\*

2087	,	657.7097	,	-5.125	,	46.0009
------	---	----------	---	--------	---	---------

\*\*

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\*\*\*\*\* Nodes for masspoints/hangerpoints, (1.104m above NA + correction for displ), for the dummy elements

\*\*

3001	,	45	,	5.1250	,	54.57591844
3002	,	64	,	5.1250	,	55.396822
3003	,	76	,	5.1250	,	55.870936
3004	,	88	,	5.1250	,	56.3074
3005	,	100	,	5.1250	,	56.70662
3006	,	112	,	5.1250	,	57.06811
3007	,	124	,	5.1250	,	57.39045
3008	,	136	,	5.1250	,	57.67424
3009	,	148	,	5.1250	,	57.91812
3010	,	160	,	5.1250	,	58.12273
3011	,	172	,	5.1250	,	58.28774
3012	,	184	,	5.1250	,	58.4118
3013	,	196	,	5.1250	,	58.49457
3014	,	208	,	5.1250	,	58.5367
3015	,	220	,	5.1250	,	58.53783
3016	,	232	,	5.1250	,	58.49761
3017	,	244	,	5.1250	,	58.41467
3018	,	256	,	5.1250	,	58.28863

3019	,	268	,	5.1250	,	58.12012
3020	,	280	,	5.1250	,	57.90877
3021	,	292	,	5.1250	,	57.65331
3022	,	304	,	5.1250	,	57.35451
3023	,	316	,	5.1250	,	57.01126
3024	,	328	,	5.1250	,	56.62454
3025	,	340	,	5.1250	,	56.19336
3026	,	352	,	5.1250	,	55.71882
3027	,	364	,	5.1250	,	55.20105
3028	,	376	,	5.1250	,	54.63924
3029	,	388	,	5.1250	,	54.03361
3030	,	400	,	5.1250	,	53.38545
3031	,	412	,	5.1250	,	52.69507
3032	,	424	,	5.1250	,	51.96187
3033	,	436	,	5.1250	,	51.1873
3034	,	448	,	5.1250	,	50.370879
3035	,	460	,	5.1250	,	49.513265
3036	,	472	,	5.1250	,	48.616307
3037	,	491	,	5.1250	,	47.1216748

\*\*

4001	,	45	,	-5.125	,	54.57591844
4002	,	64	,	-5.125	,	55.396822
4003	,	76	,	-5.125	,	55.870936
4004	,	88	,	-5.125	,	56.3074
4005	,	100	,	-5.125	,	56.70662
4006	,	112	,	-5.125	,	57.06811
4007	,	124	,	-5.125	,	57.39045
4008	,	136	,	-5.125	,	57.67424
4009	,	148	,	-5.125	,	57.91812
4010	,	160	,	-5.125	,	58.12273
4011	,	172	,	-5.125	,	58.28774
4012	,	184	,	-5.125	,	58.4118
4013	,	196	,	-5.125	,	58.49457
4014	,	208	,	-5.125	,	58.5367
4015	,	220	,	-5.125	,	58.53783
4016	,	232	,	-5.125	,	58.49761
4017	,	244	,	-5.125	,	58.41467
4018	,	256	,	-5.125	,	58.28863
4019	,	268	,	-5.125	,	58.12012
4020	,	280	,	-5.125	,	57.90877
4021	,	292	,	-5.125	,	57.65331
4022	,	304	,	-5.125	,	57.35451
4023	,	316	,	-5.125	,	57.01126
4024	,	328	,	-5.125	,	56.62454
4025	,	340	,	-5.125	,	56.19336
4026	,	352	,	-5.125	,	55.71882
4027	,	364	,	-5.125	,	55.20105
4028	,	376	,	-5.125	,	54.63924
4029	,	388	,	-5.125	,	54.03361
4030	,	400	,	-5.125	,	53.38545
4031	,	412	,	-5.125	,	52.69507
4032	,	424	,	-5.125	,	51.96187
4033	,	436	,	-5.125	,	51.1873
4034	,	448	,	-5.125	,	50.370879
4035	,	460	,	-5.125	,	49.513265
4036	,	472	,	-5.125	,	48.616307
4037	,	491	,	-5.125	,	47.1216748

\*\*

\*\*\*\*\* Fictitious mass point under the bridge girder  
(moment of inertia), (0.236+0.159=0.395)m below NA,  
\*\*\*\*\* Exclude start & end points due to different  
supportsystem.

\*\*

5002	,	64	,	0	,	53.897624
5003	,	76	,	0	,	54.371754
5004	,	88	,	0	,	54.808235
5005	,	100	,	0	,	55.20747
5006	,	112	,	0	,	55.56898
5007	,	124	,	0	,	55.89133
5008	,	136	,	0	,	56.17514
5009	,	148	,	0	,	56.41903
5010	,	160	,	0	,	56.62366
5011	,	172	,	0	,	56.78868
5012	,	184	,	0	,	56.91275
5013	,	196	,	0	,	56.99553
5014	,	208	,	0	,	57.03767
5015	,	220	,	0	,	57.03881
5016	,	232	,	0	,	56.9986
5017	,	244	,	0	,	56.91566
5018	,	256	,	0	,	56.78963
5019	,	268	,	0	,	56.62112
5020	,	280	,	0	,	56.40977
5021	,	292	,	0	,	56.1543
5022	,	304	,	0	,	55.8555
5023	,	316	,	0	,	55.51225
5024	,	328	,	0	,	55.12552
5025	,	340	,	0	,	54.69433
5026	,	352	,	0	,	54.21978
5027	,	364	,	0	,	53.702
5028	,	376	,	0	,	53.14017
5029	,	388	,	0	,	52.53452
5030	,	400	,	0	,	51.88634
5031	,	412	,	0	,	51.19595
5032	,	424	,	0	,	50.46272
5033	,	436	,	0	,	49.68813
5034	,	448	,	0	,	48.871687
5035	,	460	,	0	,	48.01405
5036	,	472	,	0	,	47.11707

\*\*  
\*\*\*\*\* Nodes for the towers

\*\* North tower, West foot

\*\*

20001	,	45.00000	,	9.15100	,	6.5000
20002	,	44.99958811	,	9.01220	,	10.5000
20003	,	44.99864553	,	8.87340	,	13.5833
20004	,	44.99709093	,	8.73460	,	16.6667
20005	,	44.99493929	,	8.59580	,	19.7500
20006	,	44.99220583	,	8.45700	,	22.8333
20007	,	44.9889061	,	8.31820	,	25.9167
20008	,	44.9850562	,	8.17940	,	29.0000
20009	,	44.9806725	,	8.04060	,	32.0833
20010	,	44.9757716	,	7.90180	,	35.1667
20011	,	44.9703707	,	7.76300	,	38.2500
20012	,	44.9644871	,	7.62420	,	41.3333
20013	,	44.9581382	,	7.48540	,	44.4167
20014	,	44.9513423	,	7.34660	,	47.5000

20015 ,	44.9434733 ,	7.20780 ,	50.8463
20016 ,	44.9351132 ,	7.06900 ,	54.1925
20017 ,	44.9262961 ,	6.93020 ,	57.5388
20018 ,	44.9170579 ,	6.79140 ,	60.8850
20019 ,	44.9074334 ,	6.65260 ,	64.2313
20020 ,	44.897459 ,	6.51380 ,	67.5775
20021 ,	44.887171 ,	6.37500 ,	70.9237
20022 ,	44.876604 ,	6.23620 ,	74.2700
20023 ,	44.865797 ,	6.09740 ,	77.6163
20024 ,	44.854786 ,	5.95860 ,	80.9625
20025 ,	44.843608 ,	5.81980 ,	84.3088
20026 ,	44.8323 ,	5.68100 ,	87.6550
20027 ,	44.8209 ,	5.54220 ,	91.0013
20028 ,	44.809447 ,	5.40340 ,	94.3475
20029 ,	44.797977 ,	5.26460 ,	97.6938
20030 ,	44.786528 ,	5.12580 ,	101.0400

\*\*

\*\* North tower, East foot

\*\*

20101 ,	45.00000 ,	-9.15100 ,	6.5000
20102 ,	44.99958811 ,	-9.01220 ,	10.5000
20103 ,	44.99864553 ,	-8.87340 ,	13.5833
20104 ,	44.99709093 ,	-8.73460 ,	16.6667
20105 ,	44.99493929 ,	-8.59580 ,	19.7500
20106 ,	44.99220583 ,	-8.45700 ,	22.8333
20107 ,	44.9889061 ,	-8.31820 ,	25.9167
20108 ,	44.9850562 ,	-8.17940 ,	29.0000
20109 ,	44.9806725 ,	-8.04060 ,	32.0833
20110 ,	44.9757716 ,	-7.90180 ,	35.1667
20111 ,	44.9703707 ,	-7.76300 ,	38.2500
20112 ,	44.9644871 ,	-7.62420 ,	41.3333
20113 ,	44.9581382 ,	-7.48540 ,	44.4167
20114 ,	44.9513423 ,	-7.34660 ,	47.5000
20115 ,	44.9434733 ,	-7.20780 ,	50.8463
20116 ,	44.9351132 ,	-7.06900 ,	54.1925
20117 ,	44.9262961 ,	-6.93020 ,	57.5388
20118 ,	44.9170579 ,	-6.79140 ,	60.8850
20119 ,	44.9074334 ,	-6.65260 ,	64.2313
20120 ,	44.897459 ,	-6.51380 ,	67.5775
20121 ,	44.887171 ,	-6.37500 ,	70.9237
20122 ,	44.876604 ,	-6.23620 ,	74.2700
20123 ,	44.865797 ,	-6.09740 ,	77.6163
20124 ,	44.854786 ,	-5.95860 ,	80.9625
20125 ,	44.843608 ,	-5.81980 ,	84.3088
20126 ,	44.8323 ,	-5.68100 ,	87.6550
20127 ,	44.8209 ,	-5.54220 ,	91.0013
20128 ,	44.809447 ,	-5.40340 ,	94.3475
20129 ,	44.797977 ,	-5.26460 ,	97.6938
20130 ,	44.786528 ,	-5.12580 ,	101.0400

\*\*

\*\* Crossbeam under the bridge girder, North tower

\*\*

20201 ,	44.9513423 ,	7.34660 ,	47.5
20202 ,	44.9513423 ,	4.99030 ,	47.5
20203 ,	44.9513423 ,	2.49520 ,	47.5
20204 ,	44.9513423 ,	0.00010 ,	47.5
20205 ,	44.9513423 ,	-2.4950 ,	47.5
20206 ,	44.9513423 ,	-4.9901 ,	47.5

20207 ,	44.9513423 ,	-7.3466 ,	47.5
**			
** Crossbeam top, North tower			
**			
20301 ,	44.797977 ,	5.26460 ,	97.6938
20302 ,	44.797977 ,	3.50970 ,	97.6938
20303 ,	44.797977 ,	1.75480 ,	97.6938
20304 ,	44.797977 ,	-0.0001 ,	97.6938
20305 ,	44.797977 ,	-1.7550 ,	97.6938
20306 ,	44.797977 ,	-3.5099 ,	97.6938
20307 ,	44.797977 ,	-5.2646 ,	97.6938
**			
** South tower, West foot			
**			
30001 ,	491.0000 ,	9.15100 ,	0.5000
30002 ,	491.000571 ,	9.01220 ,	4.5000
30003 ,	491.0021465 ,	8.87340 ,	8.0541
30004 ,	491.0048308 ,	8.73460 ,	11.6082
30005 ,	491.0085842 ,	8.59580 ,	15.1623
30006 ,	491.0133661 ,	8.45700 ,	18.7164
30007 ,	491.0191351 ,	8.31820 ,	22.2705
30008 ,	491.0258489 ,	8.17940 ,	25.8246
30009 ,	491.0334645 ,	8.04060 ,	29.3787
30010 ,	491.0419383 ,	7.90180 ,	32.9328
30011 ,	491.0512259 ,	7.76300 ,	36.4869
30012 ,	491.0612823 ,	7.62420 ,	40.0410
30013 ,	491.0715459 ,	7.48540 ,	43.4298
30014 ,	491.0824303 ,	7.34660 ,	46.8187
30015 ,	491.0938968 ,	7.20780 ,	50.2075
30016 ,	491.105907 ,	7.06900 ,	53.5963
30017 ,	491.118423 ,	6.93020 ,	56.9852
30018 ,	491.131404 ,	6.79140 ,	60.3740
30019 ,	491.144813 ,	6.65260 ,	63.7628
30020 ,	491.158608 ,	6.51380 ,	67.1517
30021 ,	491.17275 ,	6.37500 ,	70.5405
30022 ,	491.187198 ,	6.23620 ,	73.9293
30023 ,	491.201914 ,	6.09740 ,	77.3182
30024 ,	491.216854 ,	5.95860 ,	80.7070
30025 ,	491.231979 ,	5.81980 ,	84.0958
30026 ,	491.247248 ,	5.68100 ,	87.4847
30027 ,	491.262618 ,	5.54220 ,	90.8735
30028 ,	491.27805 ,	5.40340 ,	94.2623
30029 ,	491.293501 ,	5.26460 ,	97.6512
30030 ,	491.308929 ,	5.12580 ,	101.0400
**			
** South tower, East foot			
**			
30101 ,	491.0000 ,	-9.15100 ,	0.5000
30102 ,	491.000571 ,	-9.01220 ,	4.5000
30103 ,	491.0021465 ,	-8.87340 ,	8.0541
30104 ,	491.0048308 ,	-8.73460 ,	11.6082
30105 ,	491.0085842 ,	-8.59580 ,	15.1623
30106 ,	491.0133661 ,	-8.45700 ,	18.7164
30107 ,	491.0191351 ,	-8.31820 ,	22.2705
30108 ,	491.0258489 ,	-8.17940 ,	25.8246
30109 ,	491.0334645 ,	-8.04060 ,	29.3787
30110 ,	491.0419383 ,	-7.90180 ,	32.9328
30111 ,	491.0512259 ,	-7.76300 ,	36.4869

```

30112 ,      491.0612823 ,      -7.62420 ,      40.0410
30113 ,      491.0715459 ,      -7.48540 ,      43.4298
30114 ,      491.0824303 ,      -7.34660 ,      46.8187
30115 ,      491.0938968 ,      -7.20780 ,      50.2075
30116 ,      491.105907 ,      -7.06900 ,      53.5963
30117 ,      491.118423 ,      -6.93020 ,      56.9852
30118 ,      491.131404 ,      -6.79140 ,      60.3740
30119 ,      491.144813 ,      -6.65260 ,      63.7628
30120 ,      491.158608 ,      -6.51380 ,      67.1517
30121 ,      491.17275 ,      -6.37500 ,      70.5405
30122 ,      491.187198 ,      -6.23620 ,      73.9293
30123 ,      491.201914 ,      -6.09740 ,      77.3182
30124 ,      491.216854 ,      -5.95860 ,      80.7070
30125 ,      491.231979 ,      -5.81980 ,      84.0958
30126 ,      491.247248 ,      -5.68100 ,      87.4847
30127 ,      491.262618 ,      -5.54220 ,      90.8735
30128 ,      491.27805 ,      -5.40340 ,      94.2623
30129 ,      491.293501 ,      -5.26460 ,      97.6512
30130 ,      491.308929 ,      -5.12580 ,      101.0400

```

\*\*

\*\* Crossbar under the bridge girder, South tower

\*\*

```

30201 ,      491.0612823 ,      7.62420 ,      40.0410
30202 ,      491.0612823 ,      5.17540 ,      40.0410
30203 ,      491.0612823 ,      2.58770 ,      40.0410
30204 ,      491.0612823 ,      0.00010 ,      40.0410
30205 ,      491.0612823 ,      -2.5877 ,      40.0410
30206 ,      491.0612823 ,      -4.1754 ,      40.0410
30207 ,      491.0612823 ,      -7.6242 ,      40.0410

```

\*\*

\*\* Crossbar top, South tower

\*\*

```

30301 ,      491.293501 ,      5.26460 ,      97.6512
30302 ,      491.293501 ,      3.50970 ,      97.6512
30303 ,      491.293501 ,      1.75480 ,      97.6512
30304 ,      491.293501 ,      -0.0001 ,      97.6512
30305 ,      491.293501 ,      -1.7550 ,      97.6512
30306 ,      491.293501 ,      -3.5099 ,      97.6512
30307 ,      491.293501 ,      -5.2646 ,      97.6512

```

\*\*

\*\*

\*\*

\*\*\*\*\* Generating nodes from anchorpoint to main cable top, ca.7m apart, generating each 2nd. node number.

\*NGEN, NSET=BACKSTAY

```

981 ,      1001 ,      2
1981 ,      2001 ,      2
1037 ,      1087 ,      2
2037 ,      2087 ,      2

```

\*\*

\*\*

\*\*

\*\*

```

*****
*****
*****
*****
*****

```

ELEMENTS



\*\*  
\*\*  
\*\*  
\*\*

\*\*\*\*\* Elements for deck along longitudinal axis at NA

\*ELEMENT, TYPE=B31, ELSET=GIRDER

1	,	1	,	2
2	,	2	,	3
3	,	3	,	4
4	,	4	,	5
5	,	5	,	6
6	,	6	,	7
7	,	7	,	8
8	,	8	,	9
9	,	9	,	10
10	,	10	,	11
11	,	11	,	12
12	,	12	,	13
13	,	13	,	14
14	,	14	,	15
15	,	15	,	16
16	,	16	,	17
17	,	17	,	18
18	,	18	,	19
19	,	19	,	20
20	,	20	,	21
21	,	21	,	22
22	,	22	,	23
23	,	23	,	24
24	,	24	,	25
25	,	25	,	26
26	,	26	,	27
27	,	27	,	28
28	,	28	,	29
29	,	29	,	30
30	,	30	,	31
31	,	31	,	32
32	,	32	,	33
33	,	33	,	34
34	,	34	,	35
35	,	35	,	36
36	,	36	,	37

\*\*  
\*\*

\*\*\*\*\* Elements for both main cables

\*\*

\*ELEMENT, TYPE=B31, ELSET=MAINCABLE

1001	,	1001	,	1002
1002	,	1002	,	1003
1003	,	1003	,	1004
1004	,	1004	,	1005
1005	,	1005	,	1006
1006	,	1006	,	1007
1007	,	1007	,	1008
1008	,	1008	,	1009
1009	,	1009	,	1010
1010	,	1010	,	1011
1011	,	1011	,	1012

1012	,	1012	,	1013
1013	,	1013	,	1014
1014	,	1014	,	1015
1015	,	1015	,	1016
1016	,	1016	,	1017
1017	,	1017	,	1018
1018	,	1018	,	1019
1019	,	1019	,	1020
1020	,	1020	,	1021
1021	,	1021	,	1022
1022	,	1022	,	1023
1023	,	1023	,	1024
1024	,	1024	,	1025
1025	,	1025	,	1026
1026	,	1026	,	1027
1027	,	1027	,	1028
1028	,	1028	,	1029
1029	,	1029	,	1030
1030	,	1030	,	1031
1031	,	1031	,	1032
1032	,	1032	,	1033
1033	,	1033	,	1034
1034	,	1034	,	1035
1035	,	1035	,	1036
1036	,	1036	,	1037

\*\*

2001	,	2001	,	2002
2002	,	2002	,	2003
2003	,	2003	,	2004
2004	,	2004	,	2005
2005	,	2005	,	2006
2006	,	2006	,	2007
2007	,	2007	,	2008
2008	,	2008	,	2009
2009	,	2009	,	2010
2010	,	2010	,	2011
2011	,	2011	,	2012
2012	,	2012	,	2013
2013	,	2013	,	2014
2014	,	2014	,	2015
2015	,	2015	,	2016
2016	,	2016	,	2017
2017	,	2017	,	2018
2018	,	2018	,	2019
2019	,	2019	,	2020
2020	,	2020	,	2021
2021	,	2021	,	2022
2022	,	2022	,	2023
2023	,	2023	,	2024
2024	,	2024	,	2025
2025	,	2025	,	2026
2026	,	2026	,	2027
2027	,	2027	,	2028
2028	,	2028	,	2029
2029	,	2029	,	2030
2030	,	2030	,	2031
2031	,	2031	,	2032
2032	,	2032	,	2033

```
2033 ,      2033 ,      2034
2034 ,      2034 ,      2035
2035 ,      2035 ,      2036
2036 ,      2036 ,      2037
```

```
**
**
```

```
***** Elements for anchor points/BACKSTAY
```

```
**
```

```
*ELEMENT , TYPE = B31 , ELSET=BACKSTAYCABLE
```

```
981 ,      981 ,      983
1037 ,     1037 ,     1039
1981 ,     1981 ,     1983
2037 ,     2037 ,     2039
```

```
*ELGEN , ELSET=BACKSTAYCABLE
```

```
981 ,      10 ,      2 ,      2
1037 ,     25 ,      2 ,      2
1981 ,     10 ,      2 ,      2
2037 ,     25 ,      2 ,      2
```

```
**
```

```
***** Elements for dummy
```

```
***** Dummy1 and Dummy2 = connecting Hangerpoints to
```

```
NA
```

```
***** Dummy3 = Connecting fictitious mass point with NA
```

```
***** Dummy4 = Connecting masspoints to eachother to  
create longitudinal structure for the edges
```

```
**
```

```
*ELEMENT , TYPE = B31 , ELSET=DUMMY1
```

```
3001 ,      1 ,      3001
3002 ,      2 ,      3002
3003 ,      3 ,      3003
3004 ,      4 ,      3004
3005 ,      5 ,      3005
3006 ,      6 ,      3006
3007 ,      7 ,      3007
3008 ,      8 ,      3008
3009 ,      9 ,      3009
3010 ,     10 ,      3010
3011 ,     11 ,      3011
3012 ,     12 ,      3012
3013 ,     13 ,      3013
3014 ,     14 ,      3014
3015 ,     15 ,      3015
3016 ,     16 ,      3016
3017 ,     17 ,      3017
3018 ,     18 ,      3018
3019 ,     19 ,      3019
3020 ,     20 ,      3020
3021 ,     21 ,      3021
3022 ,     22 ,      3022
3023 ,     23 ,      3023
3024 ,     24 ,      3024
3025 ,     25 ,      3025
3026 ,     26 ,      3026
3027 ,     27 ,      3027
3028 ,     28 ,      3028
3029 ,     29 ,      3029
3030 ,     30 ,      3030
3031 ,     31 ,      3031
```

```
3032 , 32 , 3032
3033 , 33 , 3033
3034 , 34 , 3034
3035 , 35 , 3035
3036 , 36 , 3036
3037 , 37 , 3037
```

\*\*

\*\*

\*\*

```
*ELEMENT , TYPE = B31 , ELSET=DUMMY2
```

```
4001 , 1 , 4001
4002 , 2 , 4002
4003 , 3 , 4003
4004 , 4 , 4004
4005 , 5 , 4005
4006 , 6 , 4006
4007 , 7 , 4007
4008 , 8 , 4008
4009 , 9 , 4009
4010 , 10 , 4010
4011 , 11 , 4011
4012 , 12 , 4012
4013 , 13 , 4013
4014 , 14 , 4014
4015 , 15 , 4015
4016 , 16 , 4016
4017 , 17 , 4017
4018 , 18 , 4018
4019 , 19 , 4019
4020 , 20 , 4020
4021 , 21 , 4021
4022 , 22 , 4022
4023 , 23 , 4023
4024 , 24 , 4024
4025 , 25 , 4025
4026 , 26 , 4026
4027 , 27 , 4027
4028 , 28 , 4028
4029 , 29 , 4029
4030 , 30 , 4030
4031 , 31 , 4031
4032 , 32 , 4032
4033 , 33 , 4033
4034 , 34 , 4034
4035 , 35 , 4035
4036 , 36 , 4036
4037 , 37 , 4037
```

\*\*

\*\*

```
***** Elements for the hangers
```

\*\*

```
*ELEMENT , TYPE = B31 , ELSET=HANGERS
```

```
5002 , 3002 , 1002
5003 , 3003 , 1003
5004 , 3004 , 1004
5005 , 3005 , 1005
5006 , 3006 , 1006
5007 , 3007 , 1007
```

5008	,	3008	,	1008
5009	,	3009	,	1009
5010	,	3010	,	1010
5011	,	3011	,	1011
5012	,	3012	,	1012
5013	,	3013	,	1013
5014	,	3014	,	1014
5015	,	3015	,	1015
5016	,	3016	,	1016
5017	,	3017	,	1017
5018	,	3018	,	1018
5019	,	3019	,	1019
5020	,	3020	,	1020
5021	,	3021	,	1021
5022	,	3022	,	1022
5023	,	3023	,	1023
5024	,	3024	,	1024
5025	,	3025	,	1025
5026	,	3026	,	1026
5027	,	3027	,	1027
5028	,	3028	,	1028
5029	,	3029	,	1029
5030	,	3030	,	1030
5031	,	3031	,	1031
5032	,	3032	,	1032
5033	,	3033	,	1033
5034	,	3034	,	1034
5035	,	3035	,	1035
5036	,	3036	,	1036

\*\*

6002	,	4002	,	2002
6003	,	4003	,	2003
6004	,	4004	,	2004
6005	,	4005	,	2005
6006	,	4006	,	2006
6007	,	4007	,	2007
6008	,	4008	,	2008
6009	,	4009	,	2009
6010	,	4010	,	2010
6011	,	4011	,	2011
6012	,	4012	,	2012
6013	,	4013	,	2013
6014	,	4014	,	2014
6015	,	4015	,	2015
6016	,	4016	,	2016
6017	,	4017	,	2017
6018	,	4018	,	2018
6019	,	4019	,	2019
6020	,	4020	,	2020
6021	,	4021	,	2021
6022	,	4022	,	2022
6023	,	4023	,	2023
6024	,	4024	,	2024
6025	,	4025	,	2025
6026	,	4026	,	2026
6027	,	4027	,	2027
6028	,	4028	,	2028
6029	,	4029	,	2029

```
6030 ,      4030 ,      2030
6031 ,      4031 ,      2031
6032 ,      4032 ,      2032
6033 ,      4033 ,      2033
6034 ,      4034 ,      2034
6035 ,      4035 ,      2035
6036 ,      4036 ,      2036
```

\*\*

\*\*

```
*ELEMENT , TYPE = B31 , ELSET=DUMMY3
```

```
7002 ,      2      ,      5002
7003 ,      3      ,      5003
7004 ,      4      ,      5004
7005 ,      5      ,      5005
7006 ,      6      ,      5006
7007 ,      7      ,      5007
7008 ,      8      ,      5008
7009 ,      9      ,      5009
7010 ,     10     ,      5010
7011 ,     11     ,      5011
7012 ,     12     ,      5012
7013 ,     13     ,      5013
7014 ,     14     ,      5014
7015 ,     15     ,      5015
7016 ,     16     ,      5016
7017 ,     17     ,      5017
7018 ,     18     ,      5018
7019 ,     19     ,      5019
7020 ,     20     ,      5020
7021 ,     21     ,      5021
7022 ,     22     ,      5022
7023 ,     23     ,      5023
7024 ,     24     ,      5024
7025 ,     25     ,      5025
7026 ,     26     ,      5026
7027 ,     27     ,      5027
7028 ,     28     ,      5028
7029 ,     29     ,      5029
7030 ,     30     ,      5030
7031 ,     31     ,      5031
7032 ,     32     ,      5032
7033 ,     33     ,      5033
7034 ,     34     ,      5034
7035 ,     35     ,      5035
7036 ,     36     ,      5036
```

\*\*

\*\*

```
*ELEMENT , TYPE = B31 , ELSET=DUMMY4
```

```
12001 ,     3001  ,      3002
12002 ,     3002  ,      3003
12003 ,     3003  ,      3004
12004 ,     3004  ,      3005
12005 ,     3005  ,      3006
12006 ,     3006  ,      3007
12007 ,     3007  ,      3008
12008 ,     3008  ,      3009
12009 ,     3009  ,      3010
12010 ,     3010  ,      3011
```

12011 ,	3011 ,	3012
12012 ,	3012 ,	3013
12013 ,	3013 ,	3014
12014 ,	3014 ,	3015
12015 ,	3015 ,	3016
12016 ,	3016 ,	3017
12017 ,	3017 ,	3018
12018 ,	3018 ,	3019
12019 ,	3019 ,	3020
12020 ,	3020 ,	3021
12021 ,	3021 ,	3022
12022 ,	3022 ,	3023
12023 ,	3023 ,	3024
12024 ,	3024 ,	3025
12025 ,	3025 ,	3026
12026 ,	3026 ,	3027
12027 ,	3027 ,	3028
12028 ,	3028 ,	3029
12029 ,	3029 ,	3030
12030 ,	3030 ,	3031
12031 ,	3031 ,	3032
12032 ,	3032 ,	3033
12033 ,	3033 ,	3034
12034 ,	3034 ,	3035
12035 ,	3035 ,	3036
12036 ,	3036 ,	3037

\*\*

13001 ,	4001 ,	4002
13002 ,	4002 ,	4003
13003 ,	4003 ,	4004
13004 ,	4004 ,	4005
13005 ,	4005 ,	4006
13006 ,	4006 ,	4007
13007 ,	4007 ,	4008
13008 ,	4008 ,	4009
13009 ,	4009 ,	4010
13010 ,	4010 ,	4011
13011 ,	4011 ,	4012
13012 ,	4012 ,	4013
13013 ,	4013 ,	4014
13014 ,	4014 ,	4015
13015 ,	4015 ,	4016
13016 ,	4016 ,	4017
13017 ,	4017 ,	4018
13018 ,	4018 ,	4019
13019 ,	4019 ,	4020
13020 ,	4020 ,	4021
13021 ,	4021 ,	4022
13022 ,	4022 ,	4023
13023 ,	4023 ,	4024
13024 ,	4024 ,	4025
13025 ,	4025 ,	4026
13026 ,	4026 ,	4027
13027 ,	4027 ,	4028
13028 ,	4028 ,	4029
13029 ,	4029 ,	4030
13030 ,	4030 ,	4031
13031 ,	4031 ,	4032

```

13032 ,      4032 ,      4033
13033 ,      4033 ,      4034
13034 ,      4034 ,      4035
13035 ,      4035 ,      4036
13036 ,      4036 ,      4037
**
**
**
**
***** Elements for the towers
** North tower
**
*ELEMENT, TYPE=B31,ELSET=NIVA1
20001 ,      20001 ,      20002
20101 ,      20101 ,      20102
*ELEMENT, TYPE=B31,ELSET=NIVA2
20002 ,      20002 ,      20003
20102 ,      20102 ,      20103
*ELEMENT, TYPE=B31,ELSET=NIVA3
20003 ,      20003 ,      20004
20103 ,      20103 ,      20104
*ELEMENT, TYPE=B31,ELSET=NIVA4
20004 ,      20004 ,      20005
20104 ,      20104 ,      20105
*ELEMENT, TYPE=B31,ELSET=NIVA5
20005 ,      20005 ,      20006
20105 ,      20105 ,      20106
*ELEMENT, TYPE=B31,ELSET=NIVA6
20006 ,      20006 ,      20007
20106 ,      20106 ,      20107
*ELEMENT, TYPE=B31,ELSET=NIVA7
20007 ,      20007 ,      20008
20107 ,      20107 ,      20108
*ELEMENT, TYPE=B31,ELSET=NIVA8
20008 ,      20008 ,      20009
20108 ,      20108 ,      20109
*ELEMENT, TYPE=B31,ELSET=NIVA9
20009 ,      20009 ,      20010
20109 ,      20109 ,      20110
*ELEMENT, TYPE=B31,ELSET=NIVA10
20010 ,      20010 ,      20011
20110 ,      20110 ,      20111
*ELEMENT, TYPE=B31,ELSET=NIVA11
20011 ,      20011 ,      20012
20111 ,      20111 ,      20112
*ELEMENT, TYPE=B31,ELSET=NIVA12
20012 ,      20012 ,      20013
20112 ,      20112 ,      20113
*ELEMENT, TYPE=B31,ELSET=NIVA13
20013 ,      20013 ,      20014
20113 ,      20113 ,      20114
*ELEMENT, TYPE=B31,ELSET=NIVA14
20014 ,      20014 ,      20015
20114 ,      20114 ,      20115
*ELEMENT, TYPE=B31,ELSET=NIVA15
20015 ,      20015 ,      20016
20115 ,      20115 ,      20116
*ELEMENT, TYPE=B31,ELSET=NIVA16

```



20016 , 20016 , 20017  
20116 , 20116 , 20117  
\*ELEMENT, TYPE=B31,ELSET=NIVA17  
20017 , 20017 , 20018  
20117 , 20117 , 20118  
\*ELEMENT, TYPE=B31,ELSET=NIVA18  
20018 , 20018 , 20019  
20118 , 20118 , 20119  
\*ELEMENT, TYPE=B31,ELSET=NIVA19  
20019 , 20019 , 20020  
20119 , 20119 , 20120  
\*ELEMENT, TYPE=B31,ELSET=NIVA20  
20020 , 20020 , 20021  
20120 , 20120 , 20121  
\*ELEMENT, TYPE=B31,ELSET=NIVA21  
20021 , 20021 , 20022  
20121 , 20121 , 20122  
\*ELEMENT, TYPE=B31,ELSET=NIVA22  
20022 , 20022 , 20023  
20122 , 20122 , 20123  
\*ELEMENT, TYPE=B31,ELSET=NIVA23  
20023 , 20023 , 20024  
20123 , 20123 , 20124  
\*ELEMENT, TYPE=B31,ELSET=NIVA24  
20024 , 20024 , 20025  
20124 , 20124 , 20125  
\*ELEMENT, TYPE=B31,ELSET=NIVA25  
20025 , 20025 , 20026  
20125 , 20125 , 20126  
\*ELEMENT, TYPE=B31,ELSET=NIVA26  
20026 , 20026 , 20027  
20126 , 20126 , 20127  
\*ELEMENT, TYPE=B31,ELSET=NIVA27  
20027 , 20027 , 20028  
20127 , 20127 , 20128  
\*ELEMENT, TYPE=B31,ELSET=NIVA28  
20028 , 20028 , 20029  
20128 , 20128 , 20129  
\*ELEMENT, TYPE=B31,ELSET=NIVA29  
20029 , 20029 , 20030  
20129 , 20129 , 20130  
\*ELEMENT, TYPE=B31,ELSET=NIVA30  
20030 , 20030 , 1001  
20130 , 20130 , 2001  
\*ELEMENT, TYPE=B31,ELSET=CROSSBEAM1  
20201 , 20201 , 20202  
20202 , 20202 , 20203  
20203 , 20203 , 20204  
20204 , 20204 , 20205  
20205 , 20205 , 20206  
20206 , 20206 , 20207  
\*ELEMENT, TYPE=B31,ELSET=CROSSBEAM2  
20301 , 20301 , 20302  
20302 , 20302 , 20303  
20303 , 20303 , 20304  
20304 , 20304 , 20305  
20305 , 20305 , 20306  
20306 , 20306 , 20307

```
**
**
*****
** South tower
**
*ELEMENT, TYPE=B31,ELSET=NIVA1
30001 ,      30001 ,      30002
30101 ,      30101 ,      30102
*ELEMENT, TYPE=B31,ELSET=NIVA2
30002 ,      30002 ,      30003
30102 ,      30102 ,      30103
*ELEMENT, TYPE=B31,ELSET=NIVA3
30003 ,      30003 ,      30004
30103 ,      30103 ,      30104
*ELEMENT, TYPE=B31,ELSET=NIVA4
30004 ,      30004 ,      30005
30104 ,      30104 ,      30105
*ELEMENT, TYPE=B31,ELSET=NIVA5
30005 ,      30005 ,      30006
30105 ,      30105 ,      30106
*ELEMENT, TYPE=B31,ELSET=NIVA6
30006 ,      30006 ,      30007
30106 ,      30106 ,      30107
*ELEMENT, TYPE=B31,ELSET=NIVA7
30007 ,      30007 ,      30008
30107 ,      30107 ,      30108
*ELEMENT, TYPE=B31,ELSET=NIVA8
30008 ,      30008 ,      30009
30108 ,      30108 ,      30109
*ELEMENT, TYPE=B31,ELSET=NIVA9
30009 ,      30009 ,      30010
30109 ,      30109 ,      30110
*ELEMENT, TYPE=B31,ELSET=NIVA10
30010 ,      30010 ,      30011
30110 ,      30110 ,      30111
*ELEMENT, TYPE=B31,ELSET=NIVA11
30011 ,      30011 ,      30012
30111 ,      30111 ,      30112
*ELEMENT, TYPE=B31,ELSET=NIVA12
30012 ,      30012 ,      30013
30112 ,      30112 ,      30113
*ELEMENT, TYPE=B31,ELSET=NIVA13
30013 ,      30013 ,      30014
30113 ,      30113 ,      30114
*ELEMENT, TYPE=B31,ELSET=NIVA14
30014 ,      30014 ,      30015
30114 ,      30114 ,      30115
*ELEMENT, TYPE=B31,ELSET=NIVA15
30015 ,      30015 ,      30016
30115 ,      30115 ,      30116
*ELEMENT, TYPE=B31,ELSET=NIVA16
30016 ,      30016 ,      30017
30116 ,      30116 ,      30117
*ELEMENT, TYPE=B31,ELSET=NIVA17
30017 ,      30017 ,      30018
30117 ,      30117 ,      30118
*ELEMENT, TYPE=B31,ELSET=NIVA18
30018 ,      30018 ,      30019
```

```

30118 ,      30118 ,      30119
*ELEMENT, TYPE=B31,ELSET=NIVA19
30019 ,      30019 ,      30020
30119 ,      30119 ,      30120
*ELEMENT, TYPE=B31,ELSET=NIVA20
30020 ,      30020 ,      30021
30120 ,      30120 ,      30121
*ELEMENT, TYPE=B31,ELSET=NIVA21
30021 ,      30021 ,      30022
30121 ,      30121 ,      30122
*ELEMENT, TYPE=B31,ELSET=NIVA22
30022 ,      30022 ,      30023
30122 ,      30122 ,      30123
*ELEMENT, TYPE=B31,ELSET=NIVA23
30023 ,      30023 ,      30024
30123 ,      30123 ,      30124
*ELEMENT, TYPE=B31,ELSET=NIVA24
30024 ,      30024 ,      30025
30124 ,      30124 ,      30125
*ELEMENT, TYPE=B31,ELSET=NIVA25
30025 ,      30025 ,      30026
30125 ,      30125 ,      30126
*ELEMENT, TYPE=B31,ELSET=NIVA26
30026 ,      30026 ,      30027
30126 ,      30126 ,      30127
*ELEMENT, TYPE=B31,ELSET=NIVA27
30027 ,      30027 ,      30028
30127 ,      30127 ,      30128
*ELEMENT, TYPE=B31,ELSET=NIVA28
30028 ,      30028 ,      30029
30128 ,      30128 ,      30129
*ELEMENT, TYPE=B31,ELSET=NIVA29
30029 ,      30029 ,      30030
30129 ,      30129 ,      30130
*ELEMENT, TYPE=B31,ELSET=NIVA30
30030 ,      30030 ,      1037
30130 ,      30130 ,      2037
*ELEMENT, TYPE=B31,ELSET=CROSSBEAM1
30201 ,      30201 ,      30202
30202 ,      30202 ,      30203
30203 ,      30203 ,      30204
30204 ,      30204 ,      30205
30205 ,      30205 ,      30206
30206 ,      30206 ,      30207
*ELEMENT, TYPE=B31,ELSET=CROSSBEAM2
30301 ,      30301 ,      30302
30302 ,      30302 ,      30303
30303 ,      30303 ,      30304
30304 ,      30304 ,      30305
30305 ,      30305 ,      30306
30306 ,      30306 ,      30307
**
**
**
**
*****
*****
*****

```

MASS ELEMENTS

```
*****
*****
**
**
**
**
```

```
***** Mass points for the hangerpoints on the girder
(Exclude start and end points)
```

```
**
*ELEMENT , TYPE = MASS , ELSET=SIDE
```

```
18002 , 3002
18003 , 3003
18004 , 3004
18005 , 3005
18006 , 3006
18007 , 3007
18008 , 3008
18009 , 3009
18010 , 3010
18011 , 3011
18012 , 3012
18013 , 3013
18014 , 3014
18015 , 3015
18016 , 3016
18017 , 3017
18018 , 3018
18019 , 3019
18020 , 3020
18021 , 3021
18022 , 3022
18023 , 3023
18024 , 3024
18025 , 3025
18026 , 3026
18027 , 3027
18028 , 3028
18029 , 3029
18030 , 3030
18031 , 3031
18032 , 3032
18033 , 3033
18034 , 3034
18035 , 3035
18036 , 3036
```

```
**
19002 , 4002
19003 , 4003
19004 , 4004
19005 , 4005
19006 , 4006
19007 , 4007
19008 , 4008
19009 , 4009
19010 , 4010
19011 , 4011
19012 , 4012
19013 , 4013
```

```
19014 ,      4014
19015 ,      4015
19016 ,      4016
19017 ,      4017
19018 ,      4018
19019 ,      4019
19020 ,      4020
19021 ,      4021
19022 ,      4022
19023 ,      4023
19024 ,      4024
19025 ,      4025
19026 ,      4026
19027 ,      4027
19028 ,      4028
19029 ,      4029
19030 ,      4030
19031 ,      4031
19032 ,      4032
19033 ,      4033
19034 ,      4034
19035 ,      4035
19036 ,      4036
```

```
**
**
**
```

```
***** Mass point for fictitious point on girder under
NA (Exclude start and end points, which is already done in the nodes)
**
```

```
*ELEMENT , TYPE = MASS , ELSET=UNDER
10002 ,      5002
10003 ,      5003
10004 ,      5004
10005 ,      5005
10006 ,      5006
10007 ,      5007
10008 ,      5008
10009 ,      5009
10010 ,      5010
10011 ,      5011
10012 ,      5012
10013 ,      5013
10014 ,      5014
10015 ,      5015
10016 ,      5016
10017 ,      5017
10018 ,      5018
10019 ,      5019
10020 ,      5020
10021 ,      5021
10022 ,      5022
10023 ,      5023
10024 ,      5024
10025 ,      5025
10026 ,      5026
10027 ,      5027
10028 ,      5028
10029 ,      5029
```

10030 , 5030  
10031 , 5031  
10032 , 5032  
10033 , 5033  
10034 , 5034  
10035 , 5035  
10036 , 5036

\*\*  
\*\*  
\*\*

\*\*\*\*\* Mass point for the two ends on the girders NA

\*\*

\*ELEMENT , TYPE = MASS , ELSET=END

11001 , 1  
11037 , 37

\*\*  
\*\*  
\*\*  
\*\*

\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*

DUMMY ELEMENTS

(TOWER)

\*\*\*\*\*  
\*\*\*\*\*

\*\*  
\*\*  
\*\*  
\*\*

\*\*\*\*\* Dummy gripping of tower leg

\*\*

\*ELEMENT, TYPE=SPRING1, ELSET=TOWERLEG

29001 , 2001  
29101 , 20101  
39001 , 30001  
39101 , 30101

\*\*  
\*\*  
\*\*

\*\*\*\*\* Element between crossbeam and girder

\*\*

\*ELEMENT, TYPE=B33, ELSET=BOUNDARY

29501 , 20204 , 1  
39501 , 30204 , 37

\*\*  
\*\*  
\*\*  
\*\*

\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*

RIGIDITY, ASSIGNING

SECTION

\*\*\*\*\*  
\*\*\*\*\*

\*\*  
\*\*  
\*\*  
\*\*

```

***** Section for Bridge girder
**
***** Sjekke med Ixy, prøve med og uten.
**
***** A, Ix, Ixy, Iy, TORSIONAL CONSTANT (I_T),
SECTORIAL MOMENT, WARPING CONSTANT
**
*BEAM GENERAL SECTION, ELSET=GIRDER, SECTION=GENERAL, DENSITY=0.001
0.42694 , 0.42678 , 0.0 , 5.1684 , 1.04866
, , 4.762
0,1,0
210000E6 , 80700E6 , 0.00001
**
** USE alpha and beta from the VS1 and VS2
*DAMPING, ALPHA=0.01089 , BETA=0.00224482
*SHEAR CENTER
0 , -0.159
**
**
***** Section for Coupling elements, very rigid, no
mass
**
*BEAM GENERAL SECTION, ELSET=DUMMY1, SECTION=GENERAL, DENSITY=1E-12
1000 , 1000 , 0.0 , 1000 , 1000 , ,
0,1,0
210000E6 , 80700E6 , 0.00001
*BEAM GENERAL SECTION, ELSET=DUMMY2, SECTION=GENERAL, DENSITY=1E-12
1000 , 1000 , 0.0 , 1000 , 1000 , ,
0,1,0
210000E6 , 80700E6 , 0.00001
*BEAM GENERAL SECTION, ELSET=DUMMY3, SECTION=GENERAL, DENSITY=1E-12
1000 , 1000 , 0.0 , 1000 , 1000 , ,
0,1,0
210000E6 , 80700E6 , 0.00001
**
**
**
***** Section for windmoment/dummy connector, low
rigidity, no mass
**
*BEAM GENERAL SECTION, ELSET=DUMMY4, SECTION=GENERAL, DENSITY=0.00001
0.42694 , 0.42678 , 0.0 , 5.1684 , 1.04866
, , 4.762
0,1,0
210000E1 , 80700E1 , 0.00001
**
**
**
***** Section for the Hangers
**
*BEAM GENERAL SECTION, ELSET=HANGERS, SECTION=GENERAL, DENSITY=0.001
0.0018 , 2.6E-9 , 0 , 2.6E-9 , 5.2E-9
1,1,0
180000E6 , 63077E6 , 0.00001
**
**
**
*DAMPING, ALPHA=0.0021785 , BETA=0.000448964

```

```

**
**
***** Section for MainCable
**
** Cable weight+hangerlink+half the hanger = 408 kg/m
**
** RHO = 408kg/m / 0.05 m2 = 8160 kg/m3
**
*BEAM GENERAL SECTION, ELSET=MAINCABLE, SECTION=GENERAL, DENSITY=8160
0.05 , 0.0000026 , 0 , 0.0000026 , 0.0000052
0,1,0
180000E6 , 80700E6 , 0.00001
**
**
*DAMPING, ALPHA=0.0021785 , BETA=0.000448964
**
**
***** Section for Cable in backstay
**
** Cable weight = 356 kg/m
**
** RHO = 356kg/m / 0.05m2 = 7120
**
*BEAM GENERAL SECTION, ELSET=BACKSTAYCABLE,
SECTION=GENERAL,DENSITY=7120
0.05 , 0.0000026 , 0 , 0.0000026 , 0.0000052
0,1,0
180000E6 , 80700E6 , 0.00001
**
**
*DAMPING, ALPHA=0.0021785 , BETA=0.000448964
**
**
***** Section for towers, from foundation to top
**
*BEAM GENERAL SECTION,ELSET=NIVA1,SECTION=GENERAL,DENSITY=2500
12.4264 , 8.1186 , 0.1 , 20.3957 , 30
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA2,SECTION=GENERAL,DENSITY=2500
7.1176 , 6.9440 , 0.1 , 15.5673 , 16.2382
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA3,SECTION=GENERAL,DENSITY=2500
7.0759 , 6.8366 , 0.1 , 15.5169 , 16.0630
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA4,SECTION=GENERAL,DENSITY=2500

```



```

7.0342      ,      6.7293      ,      0.1      ,      15.4664      ,      15.8878
1      ,      0      ,      0
4E+10      ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA5,SECTION=GENERAL,DENSITY=2500
6.9925      ,      6.6219      ,      0.1      ,      15.4160      ,      15.7127
1      ,      0      ,      0
4E+10      ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA6,SECTION=GENERAL,DENSITY=2500
6.9508      ,      6.5145      ,      0.1      ,      15.3655      ,      15.5375
1      ,      0      ,      0
4E+10      ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA7,SECTION=GENERAL,DENSITY=2500
6.9092      ,      6.4072      ,      0.1      ,      15.3151      ,      15.3623
1      ,      0      ,      0
4E+10      ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA8,SECTION=GENERAL,DENSITY=2500
6.8675      ,      6.2998      ,      0.1      ,      15.2646      ,      15.1871
1      ,      0      ,      0
4E+10      ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA9,SECTION=GENERAL,DENSITY=2500
6.8258      ,      6.1925      ,      0.1      ,      15.2142      ,      15.0119
1      ,      0      ,      0
4E+10      ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA10,SECTION=GENERAL,DENSITY=2500
6.7841      ,      6.0851      ,      0.1      ,      15.1637      ,      14.8367
1      ,      0      ,      0
4E+10      ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA11,SECTION=GENERAL,DENSITY=2500
6.7424      ,      5.9777      ,      0.1      ,      15.1133      ,      14.6616
1      ,      0      ,      0
4E+10      ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA12,SECTION=GENERAL,DENSITY=2500
6.7007      ,      5.8704      ,      0.1      ,      15.0629      ,      14.4864
1      ,      0      ,      0

```

```

4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA13,SECTION=GENERAL,DENSITY=2500
6.6590      ,      5.7630      ,      0.1      ,      15.0124      ,      14.3112
1      ,      0      ,      0
4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA14,SECTION=GENERAL,DENSITY=2500
6.6173      ,      5.6556      ,      0.1      ,      14.9620      ,      14.1360
1      ,      0      ,      0
4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA15,SECTION=GENERAL,DENSITY=2500
6.5756      ,      5.5483      ,      0.1      ,      14.9115      ,      13.9608
1      ,      0      ,      0
4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA16,SECTION=GENERAL,DENSITY=2500
6.5340      ,      5.4409      ,      0.1      ,      14.8611      ,      13.7857
1      ,      0      ,      0
4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA17,SECTION=GENERAL,DENSITY=2500
6.4923      ,      5.3336      ,      0.1      ,      14.8106      ,      13.6105
1      ,      0      ,      0
4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA18,SECTION=GENERAL,DENSITY=2500
6.4506      ,      5.2262      ,      0.1      ,      14.7602      ,      13.4353
1      ,      0      ,      0
4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA19,SECTION=GENERAL,DENSITY=2500
6.4089      ,      5.1188      ,      0.1      ,      14.7097      ,      13.2601
1      ,      0      ,      0
4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA20,SECTION=GENERAL,DENSITY=2500
6.3672      ,      5.0115      ,      0.1      ,      14.6593      ,      13.0849
1      ,      0      ,      0
4E+10 ,      1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964

```

```

**
**
*BEAM GENERAL SECTION,ELSET=NIVA21,SECTION=GENERAL,DENSITY=2500
6.3255 , 4.9041 , 0.1 , 14.6089 , 12.9098
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA22,SECTION=GENERAL,DENSITY=2500
6.2838 , 4.7967 , 0.1 , 14.5584 , 12.7346
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA23,SECTION=GENERAL,DENSITY=2500
6.2421 , 4.6894 , 0.1 , 14.5080 , 12.5594
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA24,SECTION=GENERAL,DENSITY=2500
6.2004 , 4.5820 , 0.1 , 14.4575 , 12.3842
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA25,SECTION=GENERAL,DENSITY=2500
6.1588 , 4.4747 , 0.1 , 14.4071 , 12.2090
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA26,SECTION=GENERAL,DENSITY=2500
6.1171 , 4.3673 , 0.1 , 14.3566 , 12.0338
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA27,SECTION=GENERAL,DENSITY=2500
6.0754 , 4.2599 , 0.1 , 14.3062 , 11.8587
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**
*BEAM GENERAL SECTION,ELSET=NIVA28,SECTION=GENERAL,DENSITY=2500
6.0337 , 4.1526 , 0.1 , 14.2557 , 11.6835
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
**

```

```

*BEAM GENERAL SECTION,ELSET=NIVA29,SECTION=GENERAL,DENSITY=2500
5.9920 , 4.0452 , 0.10 , 14.2053 , 11.5083
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
** What happens if I change the beam direction n1 to go to global Y dir
** This removes the warning previously seen
** JT
*BEAM GENERAL SECTION,ELSET=NIVA30,DENSITY=0.00001
**5.9503 , 3.9378 , 0.10 , 14.1549 , 11.3331
**1 , 0 , 0
5.9503 , 14.1549 , 0.10 , 3.9378 , 11.3331
0 , 1 , 0
4E+10 , 1.67E+10
**
**
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**DAMPING, BETA=0.0053
**
*BEAM GENERAL SECTION,ELSET=CROSSBEAM1,SECTION=GENERAL,DENSITY=2500
13.36 , 12.7516 , 0.10 , 27.8824 , 18.6318
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
*BEAM GENERAL SECTION,ELSET=CROSSBEAM2,SECTION=GENERAL,DENSITY=2500
9.28 , 6.2500 , 0.10 , 12.2500 , 10.2875
1 , 0 , 0
4E+10 , 1.67E+10
*DAMPING, ALPHA=0.021785 , BETA=0.00448964
**
***** Dummy boundary towerleg, infinite rigid torsion
spring
**
*SPRING, ELSET=TOWERLEG
6
1E+12
**
***** Coupling between the cross beam under the
bridge girder and the bridge girder, rigid element
**
**
*BEAM GENERAL SECTION,ELSET=BOUNDARY,DENSITY=0.000001
100 , 1000 , 0 , 1000 , 1000
1 , 0 , 0
4E+10 , 1.67E+10
**
**
**
**
*****
*****
*****
ASSIGNING LUMPED
MASS
*****
*****
**

```

```

**
**
**
***** Lumped mass along the bridge girder, fitted
such that  $I_m=55807.331$ 
***** "SIDE" = 998.2 x 12 meter between hangers =
11978.4 , "UNDER" = 3353.6 x 12 = 40243.2
**
*MASS, ELSET=SIDE
11978.4
**
*MASS, ELSET=UNDER
40243.2
**
***** Weight of the last 9,5 meters have to be added
as a lumped mass at the end of the bridge girder (9,5x5350=50825)
*MASS, ELSET=END
50825
**
**
**
*****
*****
***** BOUNDARY CONDITIONS
*****
**
**
**
**
**
***** Nodes for boundary conditions
**
*NSET,NSET=ROCK
981 , 1087 , 1981 , 2087
**
*NSET,NSET=TOWER
1001 , 1037 , 2001 , 2037
**
*NSET,NSET=BOUNDARY
1 , 37
**
*NSET,NSET=ALL,GENERATE
1 , 50000 , 1
**
*NSET,NSET=TOWERLEG
20001 , 20101 , 30001 , 30101
**
***** Boundary conditions
**
*BOUNDARY
ROCK , 1 , 3 , 0
**
TOWERLEG , 1 , 5 , 0
**1,1,1,0
**
**

```

\*\*  
\*\*  
\*\*

\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*

COUPLINGS

\*\*  
\*\*  
\*\*  
\*\*  
\*\*

\*\*\*\*\* The nodes in the top of the towers are made  
hinged such that the cables not are rigid  
\*\*\*\*\* RELEASE a rotational DOF or comb. of rotational  
DOF at one or both ends. S2 endpoint, ALLM-comb. of all the rot. DOF  
(M1,M2 and T).

\*RELEASE  
20030 , S2 , ALLM  
20130 , S2 , ALLM  
30030 , S2 , ALLM  
30130 , S2 , ALLM

\*\*

\*\*\*\*\* Coupling between cross beams and towerlegs  
\*\*\*\*\* Mulit-Point constraints (MPC), "TIE" makes  
all active DOF equal at two nodes

\*MPC

\*\*

\*\*Tower north

TIE , 20201 , 20014  
TIE , 20207 , 20114  
TIE , 20301 , 20029  
TIE , 20307 , 20129

\*\*

\*\*Tower south

TIE , 30201 , 30012  
TIE , 30207 , 30112  
TIE , 30301 , 30029  
TIE , 30307 , 30129

\*\*

\*\* The cross beam under the bridge girder, and the bridge girder:

\*\* Fixed in the length direction in one side of the bridge

\*\*

\*RELEASE

29501 , S2 , M2-T  
39501 , S2 , M2-T  
39501 , S1 , M2

\*\*

\*\*

\*\*

\*\*

\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*

ELEMENT-SETS

\*\*

\*\*  
\*\*  
\*\*  
\*ELSET, ELSET=TOWER, GENERATE

20001 , 20030 , 1  
20101 , 20130 , 1  
20201 , 20206 , 1  
20301 , 20306 , 1  
30001 , 30030 , 1  
30101 , 30130 , 1  
30201 , 30206 , 1  
30301 , 30306 , 1

\*\*  
\*ELSET, ELSET=TOWERLEG1, GENERATE

20001 , 20030 , 1  
30001 , 30030 , 1

\*\*  
\*ELSET, ELSET=TOWERLEG2, GENERATE

20101 , 20130 , 1  
30101 , 30130 , 1

\*\*  
\*ELSET, ELSET=FRAME

20025 , 20028  
20125 , 20128  
20301 , 20306

\*\*  
\*\*  
\*\*  
\*\*  
\*\*

\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*

NODES TO PUT ON

LOAD

\*\*\*\*\*  
\*\*\*\*\*

\*\*  
\*\*  
\*\*  
\*\*  
\*\*

\*NSET , NSET=GIRDER\_NA , GENERATE  
1 , 37 , 1

\*\*  
\*NSET , NSET=CABLE , GENERATE  
1001 , 1037 , 1  
2001 , 2037 , 1

\*\*  
\*NSET , NSET=DummyNodes , GENERATE  
3002 , 3036 , 1  
4002 , 4036 , 1

\*\*  
\*\*\*\*\* H09 - H18 - H24 - H30  
\*NSET , NSET=SensorWest  
3009 , 3018 , 3024 , 3030

\*\*  
\*NSET , NSET=SensorEast  
4009 , 4018 , 4024 , 4030

\*\*  
\*\*  
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\*\*\*\*\*  
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\*\*\*\*\*  
\*\*\*\*\*  
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\*\*  
\*\*

STEPS AND LOADS

\*\*

\*\* Deadload is given as static load, with non-linear geometry  
\*\* Need to do it in a two step fashion, because of the backstay cables  
\*\*

\*\*\*\*\* STEP 1 - DEADLOAD GIRDER, MAINCABLE, TOWER  
\*\*

\*STEP ,        AMPLITUDE=RAMP        ,        NAME=EGENVEKT        ,        NLGEOM, INC=5000  
\*STATIC, STABILIZE=1E-10  
1E-6 ,        1E-6 ,        1E-9 ,        1E-6  
\*\*

\*\*\*\*\* Deadload cable  
\*\*\*\* Cable with half the hanger and hangerlink:  
\*\*\*\* RHO-MED H.STENGER = 8524.2  
\*\*\*\* RHO-WITH HANGERS = 408kg/m / 0.0443 m2 = 9210  
\*\*\*\* RHO-WITHOUT HANGERS = 356kg/m / 0.0443m2 = 8036  
\*\*\*\*  $g = 9,81 \times (8036/9210) = 8,5595$   
\*\*

\*DLOAD  
MAINCABLE, GRAV, 8.5595, 0 , 0 , -1  
\*\*

\*\*\*\*\* Deadload bridge girder  
\*\*\*\* Deadload bridge girder included hangerlink and half the hanger =  
5350 kg/m  
\*\*\*\*  $5350 \times 9,81 = 52483.5$   
\*\*

\*DLOAD  
GIRDER, PZ, -52484  
\*\*

\*\*\*\*\* Deadload tower  
\*\*

\*DLOAD  
TOWER, GRAV, 9.81, 0 , 0 , -1  
\*\*

\*NODE PRINT, TOTALS=YES, FREQUENCY=100  
RF

\*NODE PRINT, TOTALS=YES, FREQUENCY=100

U  
\*EL PRINT, ELSET=MAINCABLE, FREQUENCY=100  
SF

\*EL PRINT, ELSET=GIRDER, FREQUENCY=100  
SF

\*EL PRINT, ELSET=BACKSTAYCABLE, FREQUENCY=100  
SF

\*\*  
\*\*



```
*END STEP
**
**
***** STEP 2 - DEADLOAD BACKSTAYCABLE
**
*STEP ,      AMPLITUDE=RAMP      ,      NAME=EGENVEKT_BACKSTAY ,
      NLGEOM,INC=5000
*STATIC,STABILIZE=1E-10
1E-6 ,      1E-6 ,      1E-9 ,      1E-6
**
**** Backstayable
**** Need to do this in 2 steps, because the backstays need tension
before loading
**
*DLOAD
BACKSTAYCABLE, GRAV, 9.810, 0 , 0 , -1
**
*NODE PRINT, TOTALS=YES, FREQUENCY=100
RF
*NODE PRINT, TOTALS=YES, FREQUENCY=100
U
*EL PRINT, ELSET=MAINCABLE, FREQUENCY=100
SF
*EL PRINT, ELSET=GIRDER, FREQUENCY=100
SF
*EL PRINT, ELSET=BACKSTAYCABLE, FREQUENCY=100
SF
**
*Output, history, frequency=1
*Element output, elset=MAINCABLE
SF1, SF2, SF3, SM1, SM2, SM3
**
*END STEP
```

## File Eigenfreq.inp

```
*HEADING
Analysis of Lysefjord bridge
**
** Include the model
*INCLUDE, INPUT=LYSEFJORD_B31.inp
**
***** STEP 3 - EIGENVALUES
**
*STEP, , NAME=EIGENVALUES, NLGEOM
*FREQUENCY
150
**
*NODE PRINT, TOTALS=YES,NSET=GIRDER_NA
U
*END STEP
**
**
**
***FREQUENCY
**150, ,
***NODE PRINT
**U,
***EL PRINT,FREQUENCY=0
***EL FILE
** SF,
***OUTPUT, FIELD
***ELEMENT OUTPUT
**SF,
***NODE FILE
** U,
***OUTPUT, FIELD
***NODE OUTPUT
**U,
***MODAL FILE
***OUTPUT,HISTORY,FREQUENCY=1
***MODAL OUTPUT
***END STEP
**
```

## File Dynamic.inp

```
*HEADING
Analysis of Lysefjord bridge
**
** Include the model
*INCLUDE, INPUT=LYSEFJORD_B31.inp
**
**
***** AMPLITUDES
**
**Amplitude,      name=Amp_NTW      ,      definition=EQUALLY SPACED  ,
      smooth=0    ,      fixed interval=0.04
*****
*Amplitude ,      name=Amp_H09W     ,      definition=EQUALLY SPACED  ,
      smooth=0    ,      fixed interval=0.04
      0., -0.000674886484742226, -0.00103774359221402, -
0.00102482082863311, -0.000960120521671107, -0.000870446347395252, -
0.000656871723794076, -0.00068728483600856
-0.000675009118796054, -0.000734692865392808, -0.000573924661776872, -
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0.00042100490412203, -0.000404968954339183
-0.000103156999816281, -5.17561809723378e-05, 4.22413815001648e-05, -
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0.000198325574553499, 0.000300137380749613
0.000326294284359923, 0.000112522413889554, 8.94208164934484e-05,
0.000328407930752287, 0.000533784638529718, 0.000371855945685189,
0.000377138416385709, 0.000352236233579339
0.000365918680372866, 0.000384916737762923, 0.000448388414596288,
0.000529039285165293, 0.000445211411079717, 0.000506528469867366,
0.000422246480722836, 0.000527896674321166
0.000165138901740469, 0.000240959810387513, 0.000188667164009058,
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0.000203023731843924, 0.000156645955235528, -0.000131400893581819, -
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0.00014563107324597, 0.00015133849743694
0.000100048274806923, 0.00064507512177426, 0.000233371808366196,
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-0.000125975040460421, 0.000174739061054242, 0.000128477440351884,
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0.000312985459486023, 8.50359747334347e-05
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```

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\*\*  
\*\*  
\*\*

\*\*\*\*\* STEP 4 - DYNAMIC LOADING

\*\*

\*\*\*\*\* DIRECT DYNAMIC, CONTROLL INCREMENT

\*Step, name=Amplitudes, nlgeom=YES, inc=2500

\*Dynamic , direct=NO STOP , initial=NO

0.04, 100

\*\*

\*\*BOUNDARY, TYPE=ACCELERATION, AMPLITUDE=Amp\_NTW

\*\*##NODENUMBER , 3 , 3 , 1

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\*BOUNDARY, TYPE=ACCELERATION, AMPLITUDE=Amp\_H09W

3009 , 3 , 3 , 1

\*\*

\*BOUNDARY, TYPE=ACCELERATION, AMPLITUDE=Amp\_H18W

3018 , 3 , 3 , 1

\*\*

\*\*BOUNDARY, TYPE=ACCELERATION, AMPLITUDE=Amp\_H24W

\*\*3024 , 3 , 3 , 1

\*\*

\*BOUNDARY, TYPE=ACCELERATION, AMPLITUDE=Amp\_H30W

3030 , 3 , 3 , 1

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**BOUNDARY, TYPE=ACCELERATION, AMPLITUDE=Amp_H18E
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**BOUNDARY, TYPE=ACCELERATION, AMPLITUDE=Amp_H24E
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**BOUNDARY, TYPE=ACCELERATION, AMPLITUDE=Amp_H30E
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**
***** OUTPUT REQUESTS
**
*Restart, write, frequency=0
**
**          FIELD OUTPUT:
**
*Output, field, frequency=1
*Node Output
A, RF, U, V
**
**
**          HISTORY OUTPUT:
**
*Output, history, frequency=1
*Node Output, nset=DummyNodes
U1, U2, U3, UR1, UR2, UR3, A3
*Node Output, nset=GIRDER_NA
U3, UR1
**
**Element output, elset=MAINCABLE
SF1, SF2, SF3, SM1, SM2, SM3
*Element output, elset=GIRDER
SF1, SF2, SF3, SM1, SM2, SM3
*End Step

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