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EurolinK

PONTE SULLO STRETTO DI MESSINA

PROGETTO DEFINITIVO

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1 Introduction

COWI has been asked to review the tower construction methods proposed by Eurolink/Cimolai with the purpose of verifying the permanent structures.

The proposed construction methods are reviewed to verify the feasibility, to evaluate the global and local impacts on the permanent structure and to assess the need for additional reinforcement of the structure.

Cimolai's proposed tower erection method comprises the following main steps:

- Erection of the first three tower leg segments from temporary erection towers constructed at the base of each tower leg;
- Installation of a climbing erection platform on each tower leg. The platform is guided along the previously erected tower leg segments with four rollers located at each of the top and bottom of the platform. The platform climbs the tower leg using strand jacks connected to the underside of the platform and to the top of the previously erected tower leg segment;
- Installation of tower leg segments four to 21 and the main cable saddle with the climbing erection platform; and
- Installation of the three cross beams by strand jacks supported at the tower leg segment just above the cross beam elevation.

1.1 Scope of review

The tower construction method review is based on information provided by Cimolai/Eurolink. The Cimolai reports and drawings reviewed as part of this task are listed in Table 1-1 and Table 1-2, respectively:

Table 1-1 Cimolai reports reviewed.

Report Title	Document No.
Platform Elevator for Pylon Assembly	Document No. 2002159RCDO033 0
Tower Bolted and Welded Joint Procedure	2002159RCDO036 0
Answers to "Comments on tower Construction Method by Cimolai" (20 April 2010)	2002159RCDO032 A

Metodo do Montaggio Torri	2002159-CIM-EMS-0100 0

Table 1-2 Cimolai drawings reviewed

Drawing Title	Document No.
Torri - Montaggio - Fasi di attracco della chiatta e scarico dei conci di pilone	2002159DO00616C
Montaggio - Torri - Sistemazione aree di cantiere - Messina	2002159DO00618A
Montaggio - Torri - Sistemazione aree di cantiere - Reggia Calabria	2002159DO00619A
Torri - Fasi di montaggio - Montaggio del concio di base	2002159DO00104B
Torri - Fasi di montaggio - Montaggio del concio 2	2002159DO00105B
Torri - Fasi di montaggio - Montaggio del concio 3	2002159DO00106B
Torri - Fasi di montaggio - Manovra di rotazione dei conci da posizione orizzontale a posizione vertical	2002159DO00121A
Torri - Fasi di montaggio - Montaggio del concio 4	2002159DO00109B
Torri - Attrezzatura di montaggio dei conci prefabbricati insieme di carpenteria	2002159DO00103B
Torri - Fasi di montaggio - Montaggio dellèattrezzatura di sollevamento	2002159DO00107B
Torri - Fasi di montaggio - Montaggio dei concio 4 (a/b)	2002159DO00108B
Torri - Fasi di montaggio - Montaggio traverse inferior	2002159DO001300
Torri - Fasi di montaggio - Montaggio traverse intermedio	2002159DO001310
Torri - Fasi di montaggio - Montaggio traverse superior	2002159DO001320
Torri - Montaggio - Fasi del sistema di appensione dell'attrezzatura posa conci per lo spostamento del concio n al successivo	2002159DO001420
Torri - Montaggio - Dettaglio ancoraggio strandjack su parete esterna	2002159DO00140A
Montaggio impalcato - Montaggio concio 28C - Sottofase a, b, c	2002159DO002370
Montaggio impalcato - Montaggio concio 27C - Sottofase d, e	2002159DO002380

In cases where information was not available, reasonable assumptions were made, as described in the relevant sections of this report. The construction method review is primarily focused on verification of the permanent tower structure for the construction loads and includes the following scope:

- General comments and observations regarding the proposed construction method;
- Global tower verification for the construction stage prior to the installation of cross beam 1. This stage considers the lifting/platform loading during the erection of tower leg segment 8 and the erection of cross beam 1;
- Global tower verification for the construction stage prior to the installation of cross beam 2. This stage considers the lifting/platform loading during the erection of tower leg segment 14 and the erection of cross beam 2;
- Global tower verification for the construction stage prior to installation of cross beam 3. This stage considers the lifting/platform loading during the erection of the cable saddle and the erection of cross beam 3;
- Global tower verification for the erection of the deck segments adjacent to the towers using strand jacks attached to cross beam 3;
- Global tower verification for the construction stage prior to the erection of the deck segment at tower. This stage considers wind loading on deck, cables, hangers and tower prior to the deck being restrained transversally by the towers;
- Local verification of the erection platform strand jack anchors' connection to the tower leg;
- Combined global and local verification of the longitudinal stiffeners and transverse diaphragms for the erection platform reactions applied during segment erection; and
- Feasibility review of the proposed procedure for making the horizontal splices between tower leg segments.

The freestanding tower with and without tie-back were considered in the tower design, as the tower demands in these conditions are not dependent on the details of the overall tower erection method. Loads on the freestanding tower with and without tie-back were found only to govern the design of the tower base anchorage.

The erection equipment and its connections to the permanent structure, other than those indicated above, have not been reviewed or verified.

The extent of the analysis and design calculations are intended to provided a general indication of the feasibility of the construction methods and expected modifications to the permanent tower structure. The results presented are based on the expected loadings at the critical cross sections. A more comprehensive detailed analysis of all construction stages and loadings may result in some optimizations, particularly for portions of the tower legs away from the critical cross sections.

2 General comments to proposed construction method

As part of the review and verification, a general review of the various methods proposed by Cimolai has been made based on experience with similar processes. The comments are not specifically related to the overall strength of the permanent structure, but should be considered as part of the overall review process.

2.1 General handling and transport of elements

Fabrication and erection of major bridges involves mass production and repetitive handling of similar elements. The handling starts with plates and profiles, then panels and subassemblies, and finally erection of the elements.

The following are comments regarding the general handling and transport of the segments:

1 The handling, including sea fastening, should preferably be made without welded attachments. It is our experience that the repetitive use of non-welded handling appliances is cost effective and requires fewer repairs of the permanent structure. At the same time, it is also quicker to fasten and unfasten the elements during operations that are often on the critical path.

2 During sea transport, slamming by the waves on the bridge elements should be avoided. Possible slamming would increase the loads on the elements considerably. Also, sea water should be prevented from entering the internal parts of the tower elements because salt deposits are not acceptable and cleaning is difficult in practice.

3 Tower leg segment support points during transport appear not to be limited to diaphragm and stiffener locations. The tower leg segment capacity at support points should be verified.

4 In further stage of the project, the shipment in upright position of highest tower segments can be verified. Doing so would eliminate the need to turn them on their side and back upright prior to erection.

2.2 Tower leg segment erection

The erection of the tower leg segments is based on the lifting of an erection platform with the tower leg element on top. The erection platform is supported with strand jacks from the tower top and the strand jack hoists are located under the platform. The concept is shown on Cimolai drawing No. 2002159D000103B.

During lifting, the erection platform is guided by the tower leg and the eccentric load from the tower leg segment and the platform is transferred to the tower through the guide rollers. Tower cranes and access platforms are all attached to the erection platform and thus travel along the tower leg during lifting.

Access to the erection platform during lifting is not permitted for safety reasons and therefore the platform is remote controlled.

The following are comments to the proposed erection concept:

1 The lifting machinery and the tower cranes are located on the lifting platform during the lift and the machinery is operated by remote control. Safe access must be provided to the erection platform during lifting in case of malfunction and for completing maintenance or repairs during the lifting operation.

2 The maximum tower leg segment weight is ~ 1525 t (Sicilia tower leg segment 14 including TMDs) and the weight of the erection platform is ~ 600 t.

3 The erection platform is also the working platform for all work to be performed from the outside of the tower legs. That means that all work from the platform is on the critical path. For instance welding, NDT weld testing and weld repairs.

4 During the lifting of a tower leg, segment heavy loads are transferred to the tower legs through the guide rollers. It is therefore expected that all of the contact areas for the rollers will need to be painted after completion of the tower erection.

2.3 Cross beam erection

Cross beams are erected with strand jacks from the top of the previously erected tower leg segments and with a pull-back system that enables passing of cross beams at lower levels during hoisting.

The following are comments to the proposed erection concept:

1 The tower legs are sloped towards each other and so will deflect towards each other as they are built. It should be possible to counteract the deflection before fitting of the cross beams so as to achieve the target geometry and not generate unaccounted for stresses in the tower legs.

2 It should also be possible to counteract possible deviations of plumbness outside the tolerances in the longitudinal direction.

3 The cross beams appear to be lifted with two strand jack at each end and thus there is redundancy.

4 During cross beam erection a pull back cable is used to hold the cross beams away from the tower centerline. This is required as the cross beams cannot be lifted directly in line with tower legs due to the foundations and/or the previously erected cross beams. The strand jacks' orientation while lifting the cross beams will vary from vertical to an angle of approximately 6°-8° from vertical.

3 Analysis model

As part of the tower verification and assessment during construction, an analysis model was created. The tower construction model was created using the COWI/Buckland and Taylor Ltd. in-house structural analysis software CAMIL. The model was built stage-by-stage following the proposed erection procedure. The model comprises beam elements with the appropriate section properties and masses to represent the tower legs and cross beams. The foundations are modelled using the soil springs from the global IBDAS model. The Calabria tower was modelled as it generally comprises thinner tower leg plates than the Sicilia tower and therefore is slightly less tolerant of the erection loads. To confirm the appropriateness of the model, mode shapes and forces for the completed free standing tower were verified using those determined by the global IBDAS analysis model.

The following loads are considered in the construction stage analysis:

- Dead.
- Wind.
- Seismic (based on Response Spectrum Analysis).
- Erection of tower segment using erection platform (including impact and an allowance for geometric fabrication tolerances).
- Erection of the cross beam (including impact).

The wind and seismic loads were determined for an appropriate return period considering the approximate tower construction duration. The tower components are analysed and verified considering two sets of load combinations: i) loads acting on the tower during general construction; and ii) reduced loads acting on the tower during lifting operations.

Lower return periods are considered for wind loads during lifting operations than during general tower construction, as lifting operations can be suspended during periods of high winds. The characteristic wind speeds considered during construction are listed in Table 3-1.

Table 3-1 Characteristic wind speeds during construction

Elevation (m)	Characteristic Wind Speed During General Construction (m/s)	Characteristic Wind Speed During Lifting Operations (m/s)
10	36.5	10.5
70	47	14
400	60	20

The design wind pressures are proportional to wind speed squared and so the wind pressures during lifting operations will be reduced to only 10% ($100 \times 10.5^2/36.5^2$) of the values during general construction.

The total design wind pressures are determined based on the wind speeds in Table 3-1, multiplied by a gust factor of 2.0 and an appropriate drag coefficient for the wind angle considered. Drag factors are as specified in the General Design Principles (CG.10.00-P-RG-D-P-SV-T4-00-00-00-00-01). For wind acting along the bridge axis, the drag coefficient is 0.5 for each leg. For wind acting transverse to the bridge axis, a total drag factor of 1.8 is specified for the upwind and downwind legs. For the purpose of this assessment a drag factor of 1.8 is applied to both the upwind and downwind legs.

Seismic forces during general construction are based on the design spectrum given in the design basis (GCG.F.04.01) and a peak ground acceleration of 1.4 m/s^2, as specified in "Design Criterion: Verification of construction stages for permanent structures" (A9055-NOT-3-028). During short duration lifting operations, the probability of a significant seismic event is below that typically associated with other ULS loads. Therefore, seismic loading during lifting operations is not considered.

Tower components are verified using ULS combinations 1 and 3:

ULS1: (0.95 / 1.15) PP + (0 / 1.5) PN + 1.0 VV

ULS3: (0.95 / 1.15) PP + (0 / 1.5) PN + 1.0 VS

where PP is the structural component self weight, PN is the non-structural component self weight, VV is the wind load and VS is the seismic load.

4 Global verification of construction stages

Using the analysis model described in the previous section, global force effects are determined for the tower during the critical construction stages. The following three construction stages result in the maximum tower leg force effects:

- Just prior to the installation of cross beam 1. This stage considers the lifting/platform loading during the erection of tower leg segment 8 and the erection of cross beam 1;
- Just prior to the installation of cross beam 2. This stage considers the lifting/platform loading during the erection of tower leg segment 14 and the erection of cross beam 2; and
- Just prior to the installation of cross beam 3. This stage considers the lifting/platform loading during the erection of cross beam 3.

Figure 4-1 shows the model in the critical stages described above and after tower construction is complete.

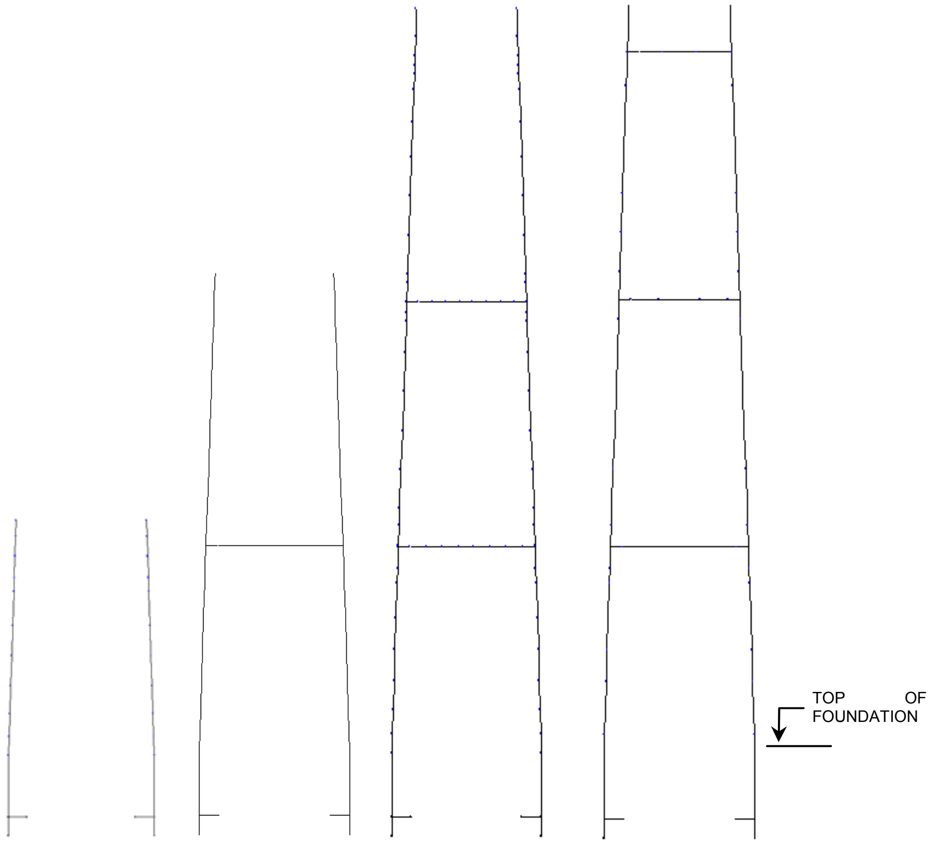
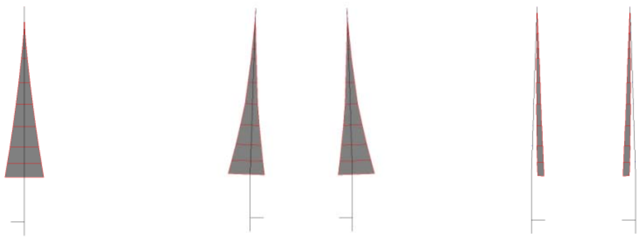
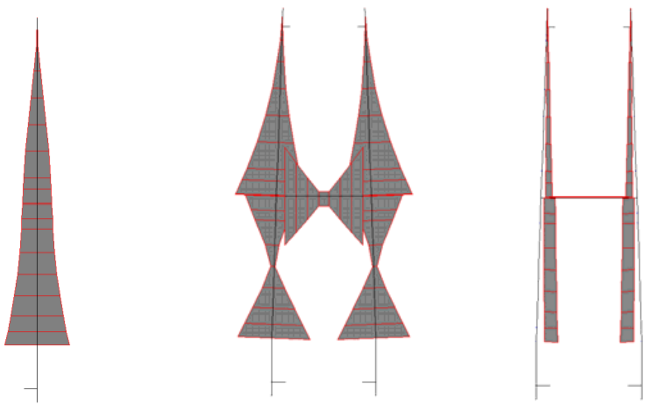


Figure 4-1 Analysis models for critical construction stages and completed tower

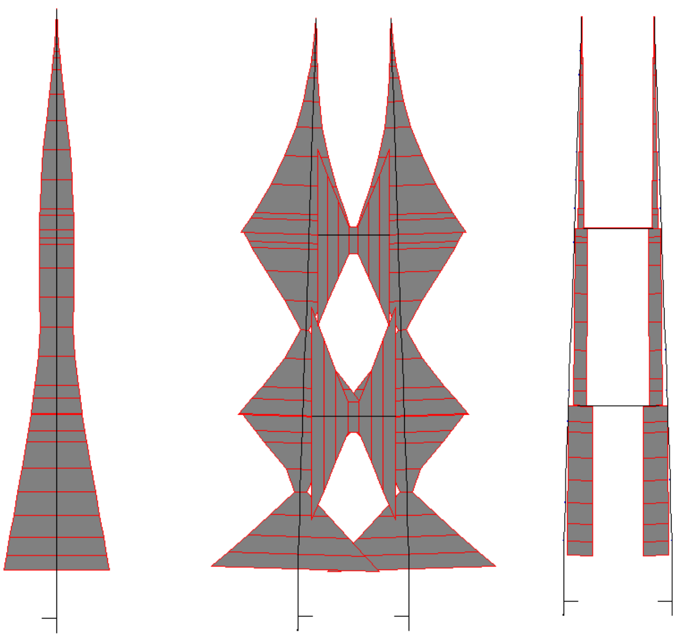
Figure 4-2 shows envelopes of longitudinal moment (M_y), transverse moment (M_z) and axial force (P) at the critical construction stages, just prior to the installation of each cross beam. The force effect envelopes are for general construction (i.e. no lifting operation) and include dead, wind and seismic effects, as described in the previous section. Governing tower leg moments in the critical construction stages are typically caused by the wind load combination.



$M_{y,max}$ = 1142MN-m $M_{z,max}$ = 1070MN-m P_{max} = 125MN



$M_{y,max}$ = 1247MN-m $M_{z,max}$ = 1450MN-m P_{max} = 265MN



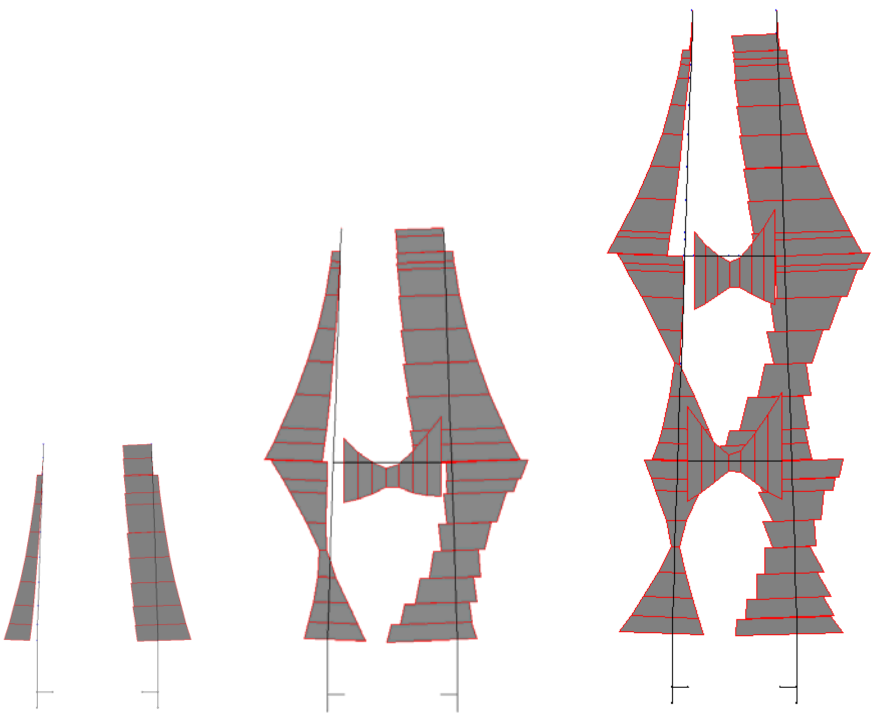
$M_{y,max}$ = 1390MN-m $M_{z,max}$ = 2275MN-m P_{max} = 506MN

Figure 4-2 Force effect envelopes at critical construction stages during general construction

The force effects shown in Figure 4-2 are well below those present in the completed bridge, for which the tower components have been designed. Therefore, the current tower design is adequate for global forces during the temporary construction stages.

During lifting operations, the tower is assumed to be subjected to lower wind speeds and seismic loads are not considered. However, the tower leg segment erection platform applies significant transverse moments to the tower legs.

Axial load and transverse tower leg bending dominates stresses during lifting operations. Figure 4-3 shows the tower leg transverse moment envelopes for the three critical construction stages. The envelopes consider the effects of dead and wind loads. The envelopes also include the effects of erecting the cross beams and the tower leg segments. The loads from the erection of a tower leg segment were moved along the entire height of the tower leg in order to capture the critical position. The erection platform carrying a tower leg segment was placed on the right tower leg only, so as to more clearly show the effect in the moment envelopes.



$M_{z,max}$ = 380MN-m $M_{z,max}$ = 530MN-m $M_{z,max}$ = 610MN

Figure 4-3 Transverse moment envelopes during lifting operations

The global force effects during lifting of the cross beams and tower leg segments do not govern the tower leg design. However, the stresses caused by the global effects must be combined with the local stresses caused by the erection platform reactions on the tower legs. Therefore, an estimate of the maximum global stresses in the tower legs is needed.

The maximum compressive stress in the loaded plate B longitudinal stiffener is approximately 50 MPa and occurs in tower leg segment 6 when the lifting platform is located in approximately the same location. The coexisting axial load, longitudinal moment and transverse moment causing the maximum stress are:

P_{max} = 291 MN

$M_{y,max}$ = 80 MN-m

$M_{z,max}$ = 97 MN-m

The maximum compressive stress in the loaded plate C longitudinal stiffener is approximately 70 MPa and occurs in tower leg segment 7 just below cross beam 1 when the lifting platform is located in approximately the same location. The coexisting axial load, longitudinal moment and transverse moment causing the maximum stress are:

P_{max} = 259 MN

$M_{y,max}$ = 67 MN-m

$M_{z,max}$ = 563 MN-m

The maximum expected tensile stresses in the tower are determined using the same approach as described above. The maximum tensile stresses in the tower legs are less than the maximum compressive stresses because of the counteracting compressive dead load stress. A maximum tensile stress of approximately 30 MPa was calculated at the locations of the loaded longitudinal stiffeners.

For simplicity, a maximum longitudinal stiffener global axial stress of 70 MPa in compression and 30 MPa in tension are assumed for the loaded plate B and C longitudinal stiffeners for combination with the local stresses caused by the erection platform reactions on the tower leg.

As indicated in Section 1.1, the tower was also verified for the effects of erecting deck segment 28C near the tower using stand jacks attached to cross beam 3, as shown in Figure 4-4.

The approximate weight of the 58 m long deck segment is 1,064 tonnes. The maximum strand jack force is estimated based on the cable angle for the critical deck segment position during lifting. Using conservative assumptions regarding the cross beam connection location and details at the cross beam it was found that the maximum stand jack forces will cause low cross beam stresses. Therefore, this construction stage will not govern the design of the tower legs or cross beam 3.

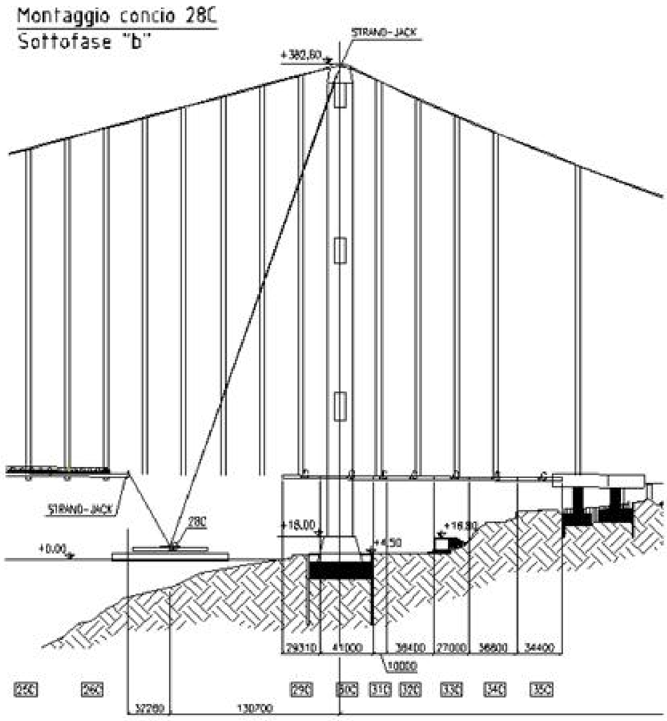


Figure 4-4 Erection of deck segment 28C using cross beam 3

As indicated in Section 1.1, the tower was also verified for the wind loads present when the suspended deck is almost completely erected, but not yet restrained transversely at the towers. In this condition all wind loading applied to the deck, hangers and cables must be transferred to the tower through the main cables at the cable saddle. Tower demands in this condition are based on the SLS2 wind speed specified in the design basis. The force effects for this construction stage were compared with the force effects from ULS/SILS wind on the completed structure. It was found that the ULS/SILS wind on the completed bridge will govern the design of the permanent tower structure.

5 Erection platform

Tower leg segments one to three are erected directly by the temporary erection towers at each tower leg base. The temporary erection towers have no effect on the permanent structure and are not considered in this review. Tower leg segments four to 21 and the main cable saddles are erected using the climbing erection platform. The erection platform is lifted by strand jacks attached to the top of the previously erected tower leg segment. The tower leg segment being erected is supported at the top of the erection platform on the outside of the tower leg. The large eccentricity of the erection platform and tower leg segment weights is balanced by supports that roll along the tower leg faces. The proposed method of erecting the tower leg segments with the erection platform is shown in the Figure 5-1 with an isometric view of the platform. The erection platform applies very large reactions to the tower leg and the effects are assessed in this section.

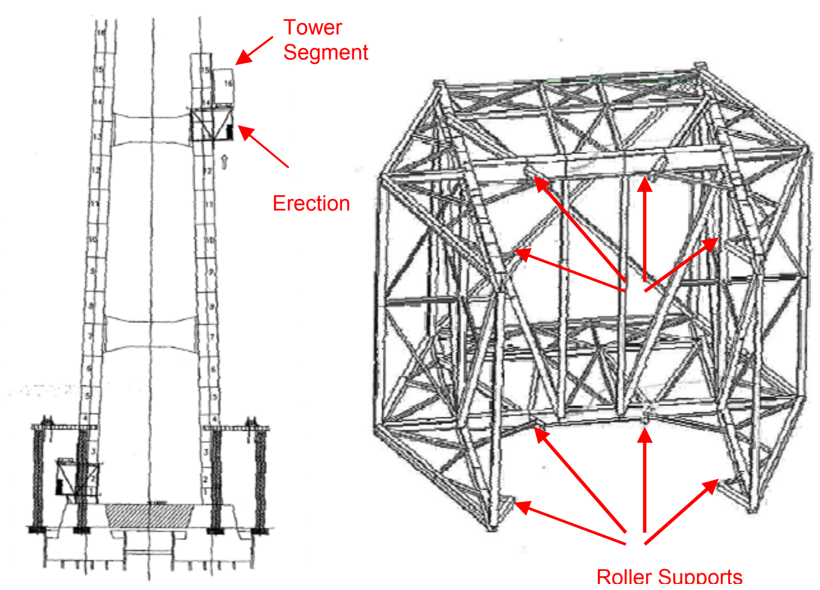


Figure 5-1 Erection platform on tower leg and isometric view of platform

5.1 Erection platform reactions

Using the erection platform configuration and the platform and segment weights, the roller support reactions are estimated. The following weights are used in determining the reactions:

Maximum segment weight (including 6-25.5 t tuned mass dampers): 1525 t

Platform weight provided by Cimolai (Doc. no 2002159RCD0032 A): 600 t

Figure 5-2 illustrates the free body diagram of the erection platform carrying the tower segment during lifting and placement at the tower top.

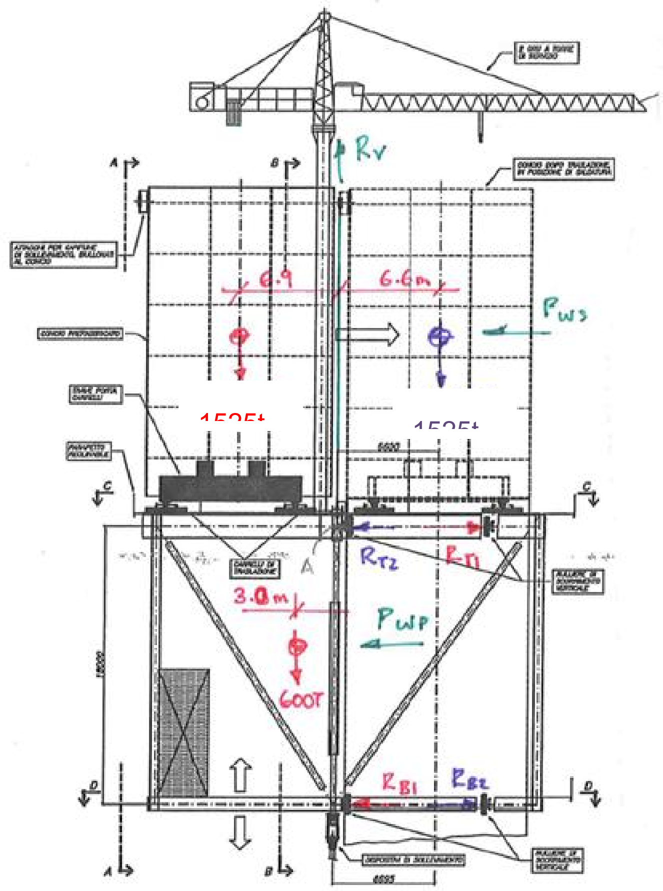


Figure 5-2 Free body diagram of erection platform

Using the free body diagram in Figure 5-2 and neglecting the effects of the small tower leg slope, the maximum reactions, R_{t1} and R_{b1}, at the top and bottom of the erection platform, respectively, are 684 tonnes.

Assuming, in a very conservative way and only for feasibility verifications, to increase those reactions by the dead load factor of 1.15 and a dynamic impact factor of 1.15, the resulting reaction is 684 x 1.15 x 1.15 = 905 tonnes. In addition to the gravity loads, wind loading will cause additional reactions. For the maximum 20 m/s wind speed considered during lifting operations, it is estimated that the resulting reaction would be 75 tonnes. The calculated reaction is divided between the two roller supports engaged at each of the top and bottom of the platform. As the supports are orientated on an approximately 30° angle to the applied force, the total reaction placed on the longitudinal stiffener is (905+75)/2/cos(30°) = 565 tonnes.

The magnitude of the reaction imbalance is a function of the erection platform stiffness, transversal friction on rollers and the permitted geometric tolerances. An analysis of these parameters is beyond the scope of this review, however, for this assessment the computed reactions are increased by 50% resulting in a maximum reaction of:

R_{1max} = 565 x 1.5 = 848 tonnes.

As shown in Figure 5-2, the eccentricity created by the platform and tower segment is reversed when the new tower segment is positioned in line with the existing tower leg. In this configuration the erection platform and tower leg segment weights have opposing eccentricities, reducing the maximum reaction. Using the same approach as above, but ignoring the impact factor as the platform is not being lifted when in this position, the maximum reaction is:

R_{2max} = 522 tonnes.

The maximum reaction is caused while the erection platform is being lifted with a tower leg segment.

For the large diameter rollers proposed by Cimolai, additional vertical loading due to roller friction is likely minimal. Given the uncertainty in the magnitude of the reaction imbalance, the small effects of roller friction are neglected.

5.2 Longitudinal stiffeners - local stresses

The platform reactions described in the previous section are applied to the corner tower leg longitudinal stiffeners on plates B and C, as shown in Figure 5-3. The longitudinal stiffeners carry the platform reactions in bending, spanning between the transverse diaphragms. The platform reactions are assumed to be applied to the longitudinal stiffeners through the roller configuration provided by Cimolai.

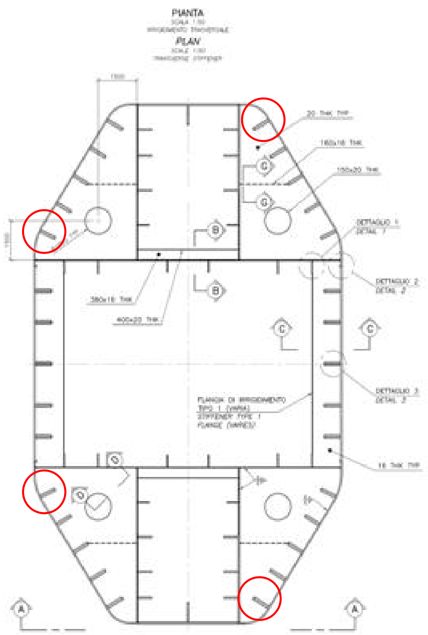
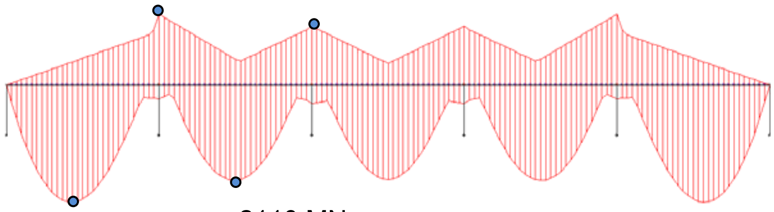
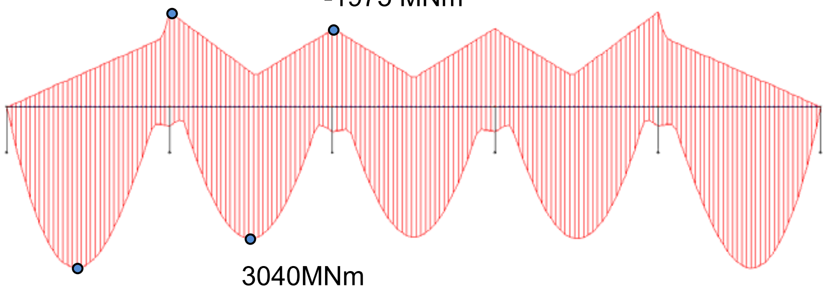


Figure 5-3 Longitudinal stiffeners supporting erection platform rollers

Figure 5-4 shows the longitudinal stiffener moment envelopes for 3.0 m and 3.5 m transverse diaphragm spacings, which cover the typical range of stiffener spacings present in the tower legs. At the top of the partially erected tower, the erection platform reaction is applied to the last longitudinal stiffener span below the horizontal segment splice. At the transverse diaphragm immediately below the segment splice there is no moment continuity and so the largest longitudinal stiffener bending moments occur when the platform reactions are applied in this longitudinal stiffener span. The maximum moments for the 3.0 m and 3.5 m transverse diaphragm spacings are 2,720 kNm and 3,520 kNm, respectively. Away from the segment splice at the tower top, the interior span moments between transverse diaphragms are smaller, having maximum values of 2,110 kNm and 3,040 kNm for the 3.0 m and 3.5 m diaphragm spacing.



(a) 3.0 m transverse diaphragm spacing.



(b) 3.5 m transverse diaphragm spacing.

Figure 5-4 Longitudinal stiffener moment envelopes for erection platform loading

The maximum longitudinal stiffener stresses are calculated using the maximum moments shown in Figure 5-4. Table 5-1 identifies the critical longitudinal stiffener and the resulting local bending stresses for the two transverse diaphragms spacings considered. In the longitudinal stiffener span below the transverse splice, the maximum local bending stresses in the loaded plate B and C longitudinal stiffeners vary between 353 MPa and 673 MPa for the two transverse diaphragm spacings considered. In the interior longitudinal stiffener spans, the maximum local bending stresses in the loaded plate B and C longitudinal stiffeners vary between 275 MPa and 522 MPa.

The largest stiffener moments are caused by the erection platform reactions between diaphragms, and so the largest local stiffener bending stresses are the induced tensile stresses on the stiffener outstand. Therefore, the maximum total stress results from the combination with the maximum global tensile stress of 30 MPa. The resulting total stresses are in many cases larger than the design stress resistance, f_y/γ_{m0} = 438 MPa, for both diaphragm spacings, indicating that the longitudinal stiffeners loaded by the erection platform will have to be modified.

The maximum local compressive stresses are caused by the hogging (negative) moments at the diaphragms. When combined with the maximum global compressive stress of 75 MPa, the local stresses are always less than the maximum tensile stresses.

The combination of the 30 MPa global tensile stress with the higher end span bending stresses is conservative and somewhat unrealistic, as there is very limited global axial stress near the top of the partially erected tower. However, the global axial stress is generally a small portion of the total stress and for simplicity in this assessment, it is assumed to apply in all cases.

Table 5-1 Longitudinal stiffener stresses due to erection platform loading

Stiffener Location	Critical Segment	Stiffener Size (mm)	Plate t (mm)	Min. Section Modulus (mm^3)	Stiff. Span	Span Length (m)	M_{max} (KNm)	σ (MPa)
Plate B	Calabria Segment 10	625 x63	45	7.67 x 10^6	Int.	3.0	2110	275
						3.5	2730	358
					End	3.0	2720	353
						3.5	3520	458
Plate C	Sicilia Segment 16	550 x 55	40	5.22 x 10^6	Int.	3.0	2110	404
						3.5	2730	522
					End	3.0	2720	520
						3.5	3520	673

In order to limit the local stresses to the estimated maximum permissible stress of 408 MPa (438 MPa – 30 MPa), it is estimated that the four loaded stiffeners must be increased in size to approximately 700 mm x 70 mm in the end span below the horizontal splice and to approximately 625 mm x 63 mm in the interior spans. For a main plate thickness of 40 mm, such stiffeners would have elastic section moduli of 10 x 10^6 mm^4 and 7.45 x 10^6 mm^4, respectively. For the maximum moments of 3,520 kNm and 2,720 kNm, this results in maximum local bending stresses of 352 MPa and 366 MPa.

As the shear forces are roughly the same at all supports, the maximum shear stresses will occur in the 625 x 63 stiffeners. The maximum average shear stress on a 625 x 63 stiffener is approximately 141 MPa, resulting in a shear utilization ratio of 0.55. To consider the interaction of shear and moment, von Mises stresses were calculated based on the maximum shear forces and moments at the support. At this location the utilization ratio for von Mises stresses is approximately 0.9.

It may be possible to reduce the amount of additional steel necessary to provide the required capacity by considering alternative modifications to the tower leg cross section and possibly the fabrication/assembly sequence. Such alternatives are described in Section 7.

5.3 Transverse plate diaphragms – local stresses

The transverse diaphragm plates supporting the longitudinal stiffeners must also be designed for the erection platform reactions. The current design contains no direct connection between the longitudinal stiffener and transverse diaphragm, as shown in Figure 5-5.

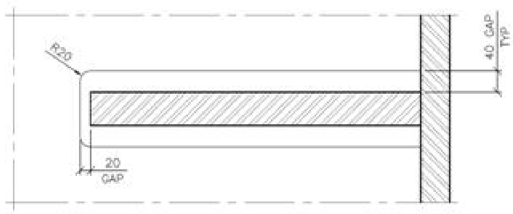


Figure 5-5 Current transverse diaphragm detail at longitudinal stiffener

This detail is inadequate for the large erection platform reactions on the longitudinal stiffeners and a direct connection must be provided by means of welding the longitudinal stiffener directly to the transverse diaphragm as shown in Figure 5-6

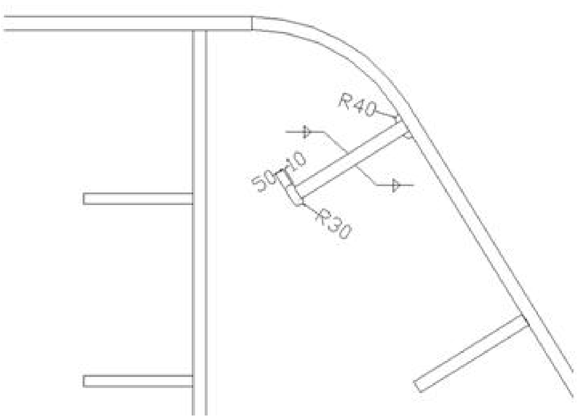


Figure 5-6 Modified longitudinal stiffener to transverse diaphragm connection detail

From the continuous beam model used to analyze the longitudinal stiffener under the erection platform loads, the maximum reaction at a transverse diaphragm is approximately 7,780 kN, and varies little with the diaphragm location. For the given roller configuration, this reaction occurs when the roller is centered over the diaphragm. In this position the center rollers would be located 187.5 mm from the diaphragm plate centerline. The rollers are narrow and so it is appropriate to assume that the entire reaction is carried by the longitudinal stiffener (negligible transverse distribution) and is transferred in shear between longitudinal stiffener and transverse diaphragm.

For the connection detail shown in Figure 5-6, the available weld length accounting for the cope holes is typically (625-50) x 4 sides = 2,300 mm. For the 7,780 kN reaction, the required weld capacity is 3.38 kN/mm, requiring a 14 mm throat. However, the transverse diaphragm plate thickness of 20 mm does not have sufficient shear capacity to transfer the forces from a double sided 14 mm fillet weld. Therefore, for the calculated loading, one of the following strengthening options will likely be required:

1 Locally increase the thickness of the transverse diaphragm plate to 28 mm and provide double sided fillet welds with 14 mm throats, as shown in Figure Figure 5-6 (slightly smaller diaphragm plate thickening and weld sizes would be feasible at the diaphragms below the segment splice, where a longitudinal stiffener depth of 700 mm is required); or

2 Increase the longitudinal stiffener depths to provide additional weld length for the transfer of shear forces. The required stiffener depth would be approximately 800 mm and the fillet welds would have 10 mm throats.

The overall plate diaphragm must also be checked for the direct stresses resulting from the load delivered through the connection to the longitudinal stiffener. Due to the irregular shape and the large cut-out in the diaphragm plate, a finite element shell model was used to assess the diaphragm plate stresses and buckling resistance. Details of the model used are provided in Design Report - Tower Legs incl. Joints and Splices (CG1000-P-CL-D-P-SV-T4-00-00-00-00-01). Erection platform reactions were applied to the affected longitudinal stiffeners concurrently with the deviation (kick) forces from the supported tower leg plates B, C, E, F and H. The finite element model is used to assess the minimum load amplifier, α_{cr}, for the design loads to reach the elastic critical load and to determine the maximum von Mises stress on the plate from a linear analysis of the applied loads. The finite element model used in the analysis is shown in Figure 5-7.

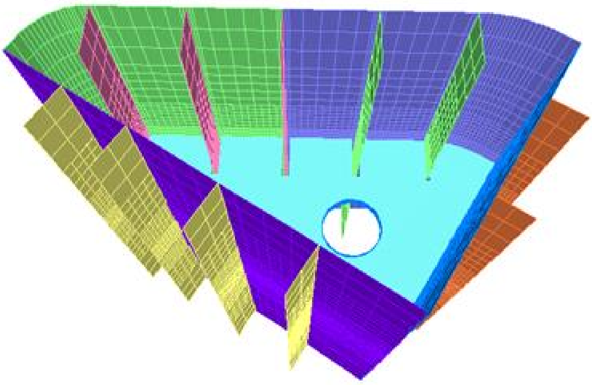


Figure 5-7 Finite element model of transverse diaphragm

The finite element model was run with all relevant load combinations and a buckling analysis was performed using the complete stress field. The critical buckling mode was caused by loading the longitudinal stiffener on plate C, near the intersection of plates C, D and H, and was found to have a minimum load amplifier, α_{cr} of 1.02. The associated deformed shape of the transverse diaphragm is shown in Figure 5-8. The von Mises stresses in the diaphragm plate are shown in Figure 5-9 for the same loading used in the buckling analysis.

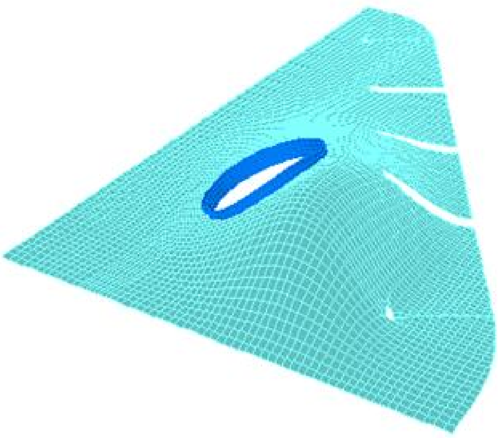


Figure 5-8 Deformed shape of transverse diaphragm for critical buckling mode

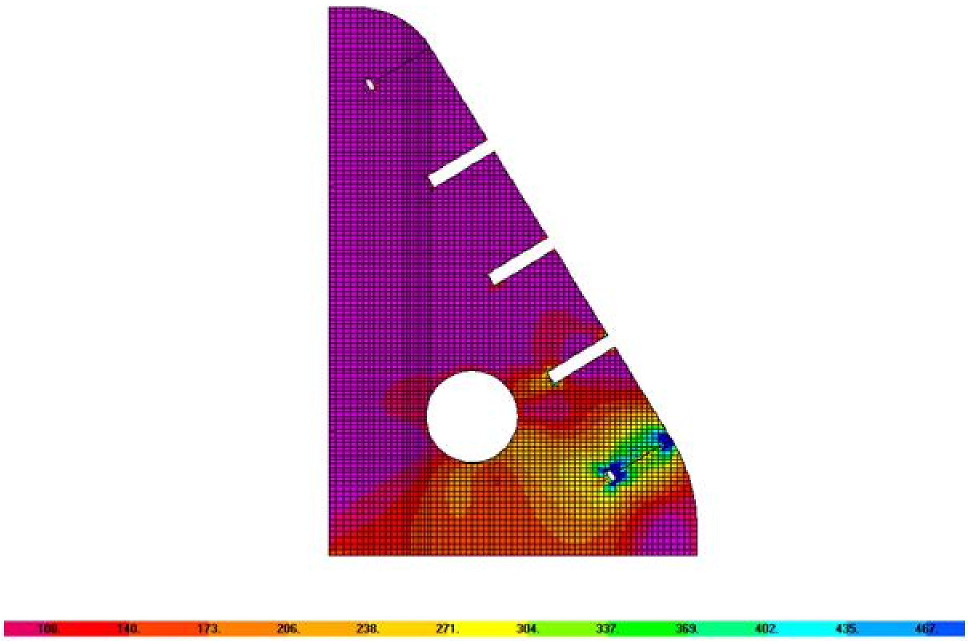


Figure 5-9 Maximum von Mises stresses in transverse diaphragms

The largest von Mises stresses occur around the perimeter of the cope hole at the end of the loaded longitudinal stiffener. Stresses larger than yield are predicted within approximately 100 mm of the cope hole. The high stresses are very localized and dissipate to much less than the yield stress away from the opening. The high stress concentrations are indicative of the linear elastic analysis that was performed. In a more representative elastic-perfectly plastic analysis, as the diaphragm plate started to yield around the cope hole and strains increased, the load would spread to a much larger area, generally decreasing the stresses from those shown. The cope hole detail is consistent with that specified by the Eurocode for use in orthotropic steel decks, and it is expected that for the relatively few loading cycles experienced during the tower erection, that the diaphragm's performance in the completed bridge will be unaffected.

For the areas of the diaphragm plate that are not located directly adjacent to the loaded longitudinal stiffener, the maximum von Mises stress is approximately 250 MPa, occurring just beyond the tip of the stiffener. Using the criteria of EN 1993-1-5 Section 10 and Annex B, the resulting reduction factor, ρ, is 0.56 and the utilization ratio is 1.06 indicating that the diaphragm is overstressed. As is indicated by the small reduction factor, the diaphragm plate is quite slender. The buckling resistance can be improved markedly by moving the currently specified 160 mm x

16 mm diaphragm plate stiffener, as shown in Figure 5-10. The repositioned stiffener more effectively resists the buckling deformations shown in Figure 5-8.

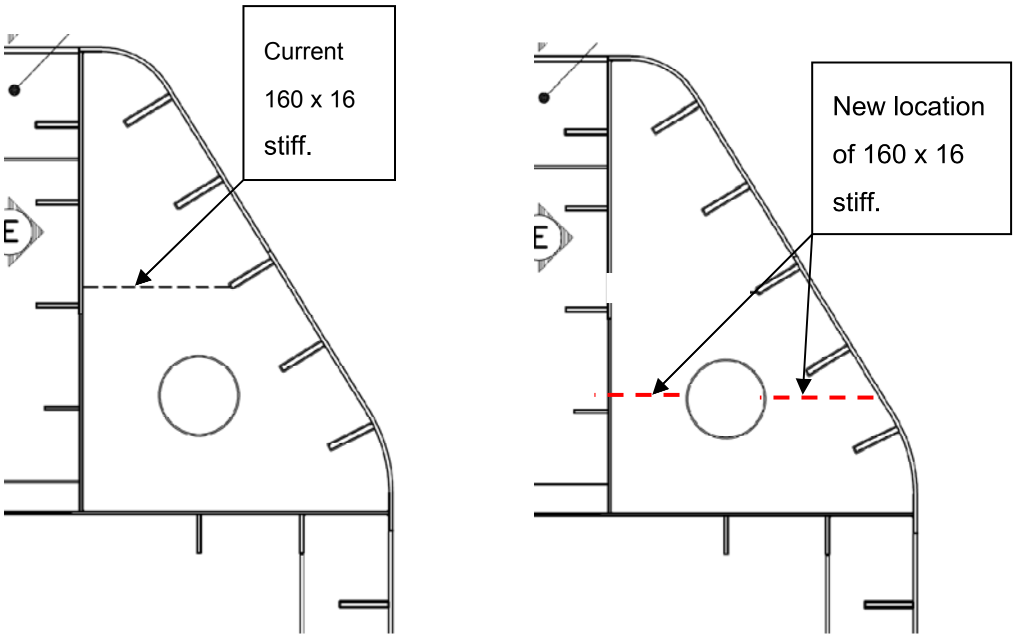


Figure 5-10 Proposed modification to location of diaphragm stiffener

The FE model was updated with the revised diaphragm plate stiffener location and the elastic buckling analysis was re-run. With the repositioned stiffener the minimum load amplifier for the critical buckling mode is increased to 1.53. The associated deformed shape of the diaphragm for the critical buckling mode is shown in Figure 5-11. The corresponding von Mises stresses in the diaphragm plate are shown in Figure 5-12. The repositioning of the stiffener has a negligible effect on the von Mises stresses. Repositioning the diaphragm plate stiffener reduces the diaphragm utilization ratio to 0.90, with a negligible effect on quantities.

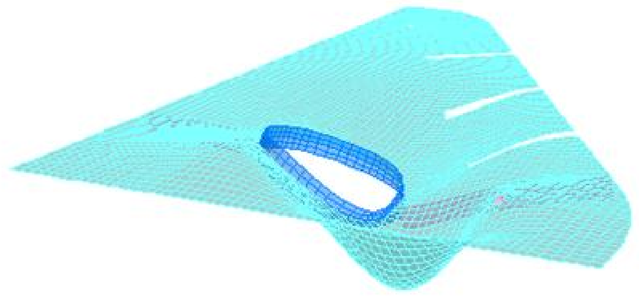


Figure 5-11 *Deformed shape of transverse diaphragm for critical buckling mode after proposed modification to diaphragm stiffener*

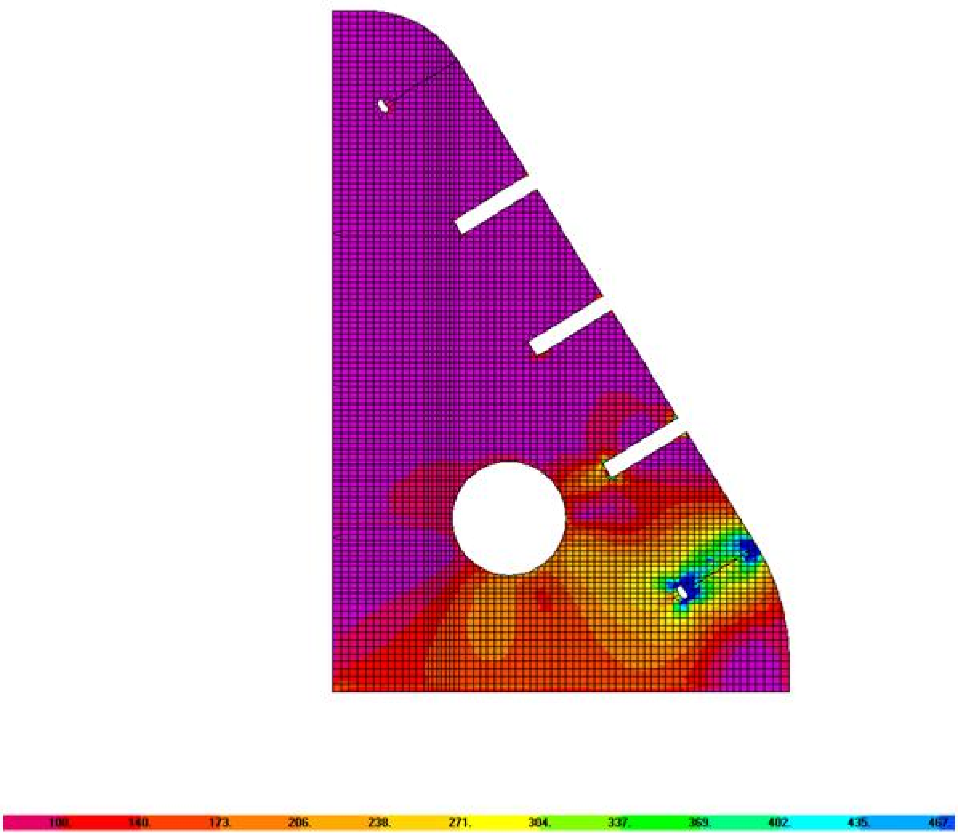


Figure 5-12 *Maximum von Mises stresses in transverse diaphragms after proposed modification to diaphragm stiffener*

5.4 Erection platform strand jack connection to tower legs

The erection platform is hoisted up the tower leg by four strand jacks connected to the underside of the platform. The lifting strands are anchored to the top of the previously erected tower leg segment at the intersection of plates C, D and H. The top anchorage is eccentric to the tower leg face, and so its connection to the tower leg must resist the vertical strand force and the moment induced by the eccentricity. The vertical strand force is transferred to the tower leg plate by a 90 mm thick lug plate welded to the tower leg face and on to which the strand anchor plates bear. The moment is transferred to the tower leg through a force couple comprising a tensile component provided by four 50 mm diameter Macalloy bars passing through the tower leg skin plate and a compressive component provided by bearing of the strand anchor plates on the tower leg face.

The strand anchorage was verified for the maximum factored lifting force of approximately 28 MN computed using the free body diagram in Figure 5-2. The proposed strand anchorage was generally found to provide sufficient capacity, however, the some of the proposed details of the connection to the tower leg will have a detrimental effect on the permanent tower structure. The following aspects of the proposed connection detail require further consideration and possibly modification:

- It is not clear whether the 90 mm thick lug plate welded to the tower leg face is intended to be removed after segment erection. Removing the lug plate with out damaging the permanent steel work is unlikely practical and leaving the lug plate in place will likely result in ongoing maintenance issues;
- It will likely be difficult to curve the relatively narrow lug plate to the same profile as tower leg plate. Any misalignment of the lug plate and tower leg plate will make it difficult to place the required connecting welds;
- The 50 mm diameter Macalloy bars require holes in the tower leg skin plate. After erection is complete the holes in the skin plate must be filled in manner that will not affect the long term durability of the tower;
- The 50 mm diameter Macalloy bars require two holes in one of the plate H longitudinal stiffeners. The holes remove approximately 25% of the stiffener area and because of the position of one of the holes at the tip of the stiffener, remove an even greater fraction of the

inertia. The affected longitudinal stiffener must be made deeper for a sufficient length above and below the required holes so as to not locally reduce the stiffener area or inertia; and

- The 80 mm thick plate to which two of the Macalloy bars are anchored is inadequate for transferring the bar force to the reinforced plate D transverse stiffener, without the assistance of the plate H longitudinal stiffener. Deformations of this connection may damage the permanent longitudinal stiffener. It is recommended that the plate thickness be increased to 120 mm, similar to that of the plate provided for the other two Macalloy bars.

6 Joint splicing procedure

The horizontal splices between the tower leg segments are hybrid connections, with all interior plates and vertical stiffeners being connected with slip-resistant bolts and the exterior plates being connected with full penetration butt welds. In accordance with EN 1993-1-8, the use of such connections is permitted provided the welds are made prior to the final tightening of the bolts.

The proposed procedure for the making the horizontal splice involves connecting several tower leg segments initially using temporary construction bolts only. After a certain number of tower leg segments are erected and connected by the temporary bolts, the welded connections in the exterior plates are made, after which the temporary construction bolts are removed, transferring all of the load to the welds. The final slip-critical bolts are then installed and fully tensioned, completing the splice. The proposed procedure is illustrated in Figure 6-1.

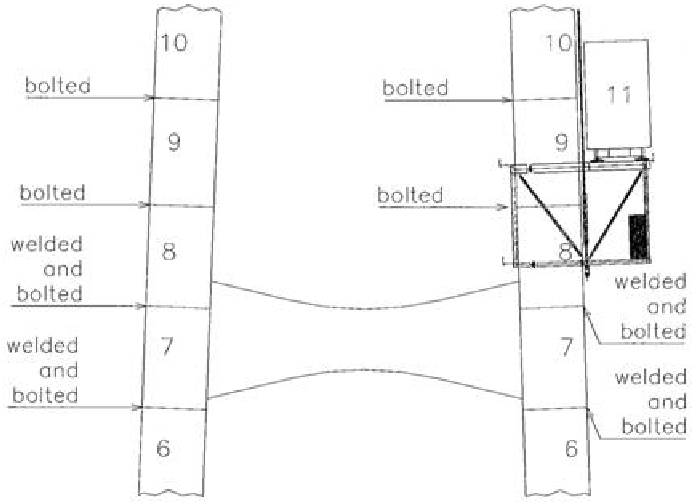


Figure 6-1 Proposed procedure for making horizontal tower leg splices

This procedure for making the horizontal splices satisfies Eurocode criteria for hybrid connections, as the welds will be completed prior to the final tightening of the bolts. However, the proposed procedure specifies that after the welds are made the next step is "de-installing the temporary HR-10.0 [bolts] and let the load on the welds alone (or the welds that have no mutual influences on the respective bolted part)." The meaning of the statement in parentheses is not entirely understood, however, based on the requirements of EN 1993-1-8, all the temporary bolts used in the cross section should be removed prior to final tightening of any of the permanent bolts.

An additional concern regarding the proposed procedure is that it is not clear if appropriate consideration has been given to the potential consequences of the weld shrinkage. Due to the size of the welds and number of weld runs required, weld shrinkage of at least 2 mm - 4 mm should be considered. For the standard hole diameters used in the splice plates, it is likely that such shrinkage will cause the bolts to bear against the bolt holes of the connected plates causing shear forces on both the temporary bolts and the final bolts that have been installed but not tensioned. These shear forces may make removing the temporary bolts difficult, and more critically, may cause additional shear demands on the permanent bolts that were not considered for the permanent design. The restraint from the bolts may also result in additional stresses in the tower plates and welds. An in-depth investigation of these issues is beyond the scope of this assessment and is not possible without more detailed information regarding the welding and bolting procedures. However, prior to completion of the final design phase, it should be demonstrated that these issues have been investigated and addressed in the joint splicing procedure.

The proposed procedure involves erecting several tower leg segments using bolted only connections before making the welds. As such, the partial splice must be verified to have adequate strength for the construction loads in two conditions:

1 Capacity of the internal plates and stiffeners only when the temporary bolts are used; and

2 Capacity of the skin plates only after the welds are completed and the temporary bolts are removed.

As the exact assembly procedures and number of temporary bolts used is not known at this time, an approximate procedure was used for assessing the strength of the tower splice for these two conditions. The capacities are based on the full capacity of either the internal plates and stiffeners only or the skin plates only. This approach is valid for verifying if the proposed procedure is feasible, with the extreme condition being that where the number of temporary bolts required provides the full capacity of the internal plates and stiffeners.

The results of this assessment indicate that it is feasible to erect a certain number of segments using bolted only connections before returning to make the final welds. The maximum construction stage moments occur near the cross beam locations. Therefore, at these locations the final welded/bolted splices should be completed before erecting additional segments above the cross beam. The cantilever portion of the tower leg extending above the cross beams can then be constructed with several bolted only tower segments as the resistance of the internal plates and stiffeners or skin plates is adequate to resist the applied forces.

The general procedure proposed for the horizontal splices is considered feasible. The exact number temporary bolts required and the number of segments that can be erected using bolted only connections will need to be verified after detailed assembly procedures are developed.

The previously described assessment of the local stresses in the longitudinal stiffeners on plates B and C due to the erection platform loading assumed that the effective stiffener section comprised the stiffener outstand and a tributary width of the tower skin plate (see Section 5). However, this assumption is not valid for the tower splices that have not been welded, and the resistance of the section should be based on that of the longitudinal stiffener only (i.e. no skin plate). A review of the bending moment diagrams from Section 5 indicates that the maximum moment in the stiffeners in the area of the tower splice is approximately 2,084 kNm. For the 625 mm x 63 mm stiffener proposed in Section 5 for the typical spans, the resulting stress would be approximately 508 MPa. Therefore, the longitudinal stiffeners in these areas must also be increased to 700 mm x 70 mm, as was already proposed for the diaphragm span just below the horizontal tower splice.

7 Summary of design modifications required to accommodate proposed construction method

The study carried out shows that:

- The described methods are considered to be feasible with limited reinforcement of the permanent structures
- Verification of more detailed methods descriptions may result in requirement for further reinforcement of the permanent structures

The conclusions below are based on the verification of the feasibility base on the described methods of construction. Further detailing of the methods may result in changes on the conclusions below.

As described in Section 5.2, the current longitudinal stiffeners on plates B and C are inadequate to carry the erection platform reactions. It estimated that to resist the applied local moments concurrently with the global axial load in the stiffener, the four loaded stiffeners must be at least 700 mm x 70 mm in the end spans below the segment splice and in the span of the segment splice. The stiffeners in typical interior spans must be at least 625 mm x 63 mm in the.

As described in Section 5.3, the loaded longitudinal stiffeners must be connected directly to the diaphragm plates. Modifications to the diaphragm plate thickness may also be required, depending on the longitudinal stiffener depths provided and the available load transfer length.

Modifications to the longitudinal stiffeners and diaphragm plates must be considered together as one influences the other. The following three combinations of stiffener (those loaded by the erection platform only) and diaphragm plate modifications may be used to provide the required capacities:

1 Use a 700 mm x 70 mm longitudinal stiffener over the entire tower height and thicken all diaphragm plates to 25 mm around the longitudinal stiffener connections (assumed thickened

area is approximately 4 m^2 per diaphragm). Longitudinal stiffeners must be connected directly to the diaphragm plates with double 12 mm throat fillet welds. The estimated quantity increases are:

Sicilia Tower: 175 tonnes for stiffeners
35 tonnes for diaphragms

Calabria Tower: 200 tonnes for stiffeners
35 tonnes for diaphragms

<u>445 tonnes total</u>

2 Use a 700 mm x 70 mm stiffener only in end spans and over the length of the segment splice (assumed over 5.5 m length) and a minimum 625 mm x 63 mm stiffener in all other interior spans. This option requires local thickening of the diaphragm plate to between 25 mm and 28 mm, depending on the stiffener depth at each diaphragm. Longitudinal stiffeners must be connected directly to the diaphragm plates with double 12 mm to 14 mm throat fillet welds, depending on the stiffener depth at each diaphragm. The estimated quantity increases are:

Sicilia Tower: 75 tonnes for stiffeners
50 tonnes for diaphragms

Calabria Tower: 85 tonnes for stiffeners
50 tonnes for diaphragms

<u>260 tonnes total</u>

3 Use a constant longitudinal stiffener depth of 800 mm and a thickness equal to the maximum of the current stiffener thickness or 65 mm (i.e., a 700 mm x 70 mm stiffener becomes a 800 mm x 70 mm stiffener and a 575 mm x 58 mm stiffener becomes a 800 mm x 65 mm stiffener). Longitudinal stiffeners must be connected directly to the diaphragm plates with double 10 mm throat fillet welds. This option does not require local diaphragm plate thickening. The estimated quantity increases are:

Sicilia Tower: 270 tonnes for stiffeners

0 tonnes for diaphragms

Calabria Tower: 290 tonnes for stiffeners
0 tonnes for diaphragms

560 tonnes total

- It may be possible to reduce the amount of additional steel required by considering the following modifications:
- Changing the loaded flat longitudinal stiffeners to T-shapes, as shown in Figure 7-1, to be more effective in bending. This option would likely result in a reduced steel area for these stiffeners. Therefore, to maintain the same cross section capacity, additional steel may need to be provided on nearby plates and stiffeners. The resulting net change in total steel tonnage is expected to minimal.

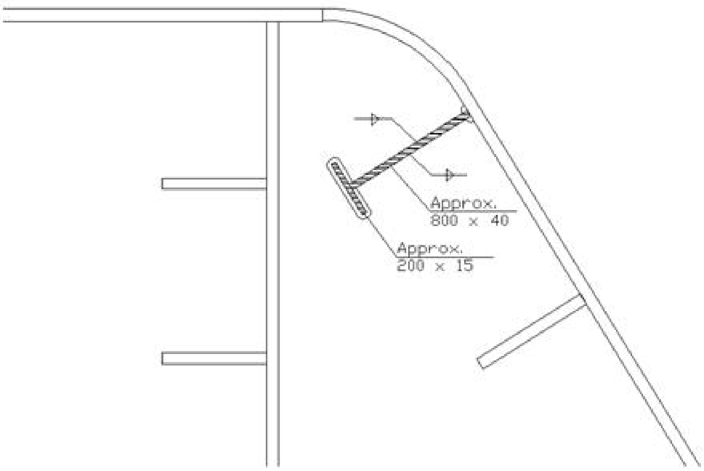


Figure 7-1 Modified longitudinal stiffener using T-shape

- Making the loaded stiffeners so deep that they meet (and are welded to) the opposite tower leg plates (E or H), and making a proper access hole through the new plates. The new plate could also function as a longitudinal stiffener for the opposite tower leg plates, allowing the removal the current stiffener. Figure 7-2 shows two variations of the option described.

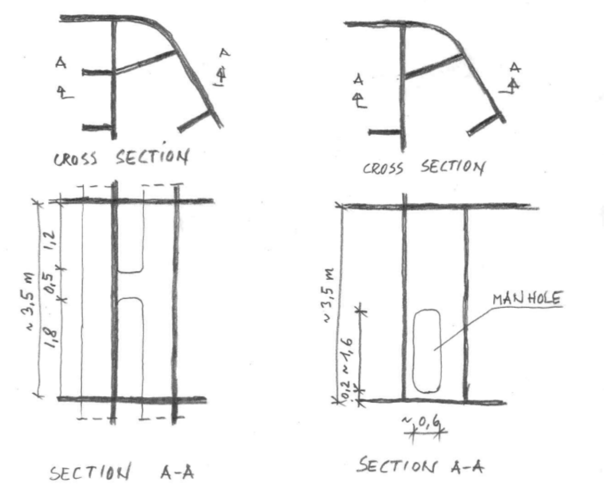


Figure 7-2 Modified longitudinal stiffener using new plate

The alternatives listed above may reduce the additional steel required to accommodate the proposed construction method, however, the tower leg cross section and possibly the fabrication/assembly sequence may have to be modified.

Annex 1

Tower Aerodynamic during Construction

Table of Contents

1. Summary

This memo summarizes the tower vibration under wind during the construction and the need of TMD (tuned mass damper) to mitigate them where necessary, resulting from the investigation carried out based on the 80 days design, the erection method proposed in the design definitive and the wind tunnel test performed on the full aero elastic tower model in BMT (Doc. No. 431163rep1v4), as follows.

- The tower vibration of only 1st bending mode along the bridge axis shall be considered to be investigated and controlled.
- The tower vibration shall be mitigated at the construction step Nos. 11 and 12 (at erection of the tower leg segment No.13 and 14) for the structural safety and at the construction steps No.15 through No.22 (from erection of the tower leg segment No.17 to the early stage of PPWS installation) for the worker's comfort.
- The tower vibration can be mitigated for the construction steps Nos. 11 and 12 by TMD for in-service condition provided in the tower leg segment No.13 and 14 and for the construction steps Nos. 15 through No.22 by another TMD temporary provided.

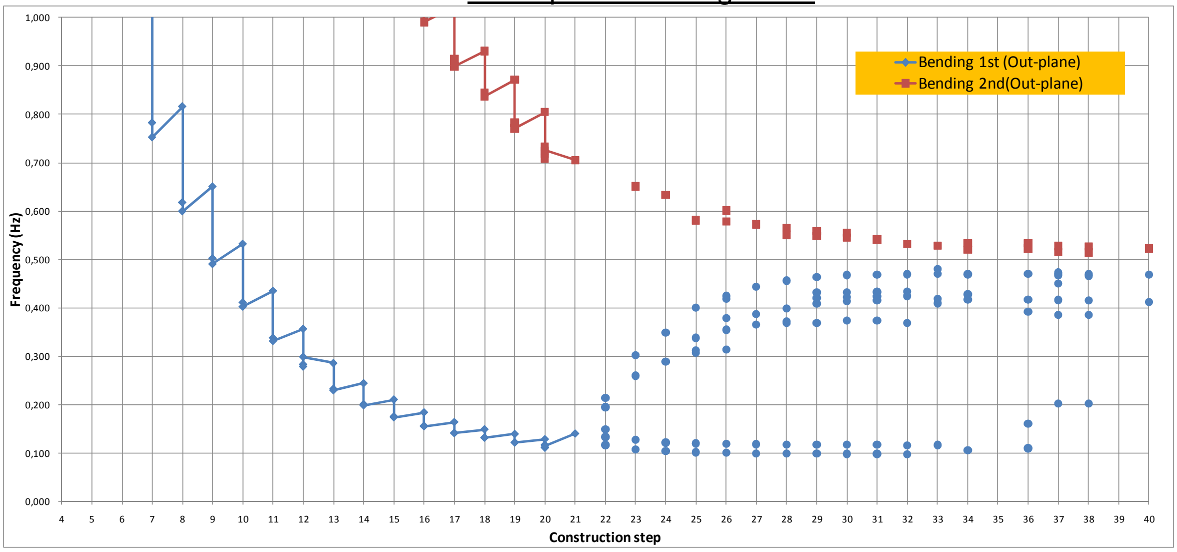
The construction steps of superstructure used for the investigation are shown in Appendix A.

2. Vibration Modes and Frequencies

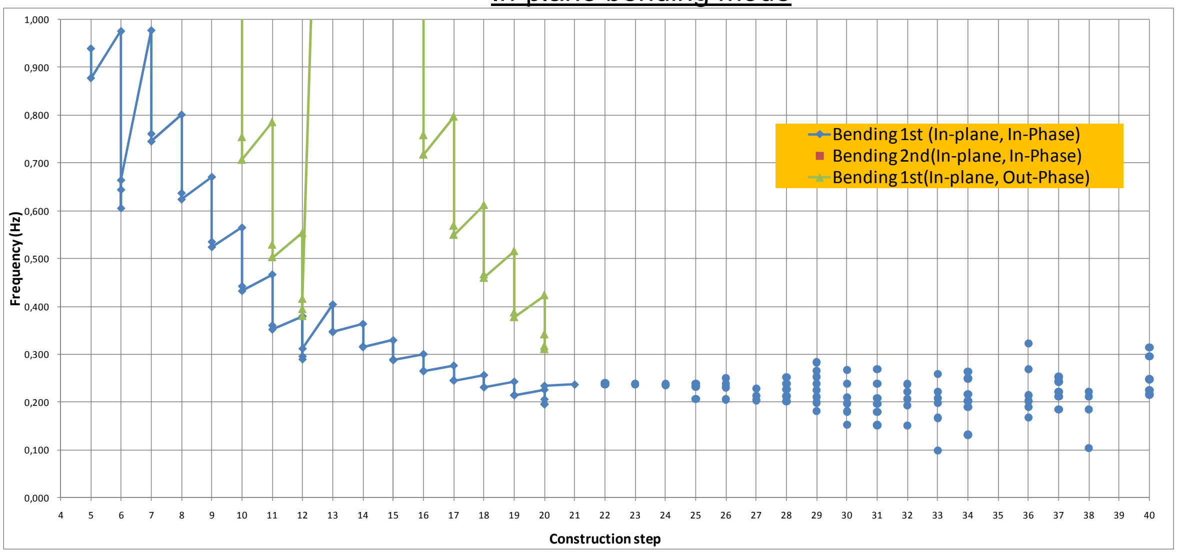
The construction of superstructure was represented by 40 construction steps, No.1 through No.20 for the tower construction, Nos.21 and 22 for the catwalk installation, No. 23 through No.32 for the cable installation, No.33 through No.39 for the deck erection and No. 40 for a reference condition, as shown in Appendix A.

3D global bridge model was established based on the 80 days submission design including the tower erection platform but just by the mass of 8,000 kN, the catwalk system and the temporary tie-back system. An eigen value analysis of the bridge under construction at all the construction steps was performed and the eigen frequencies and the equivalent mass as refer to the tower vibration modes were obtained, as shown in Appendix B and illustrated in Figure 2-1.

out of plane bending mode



In-plane bending mode



torsional mode

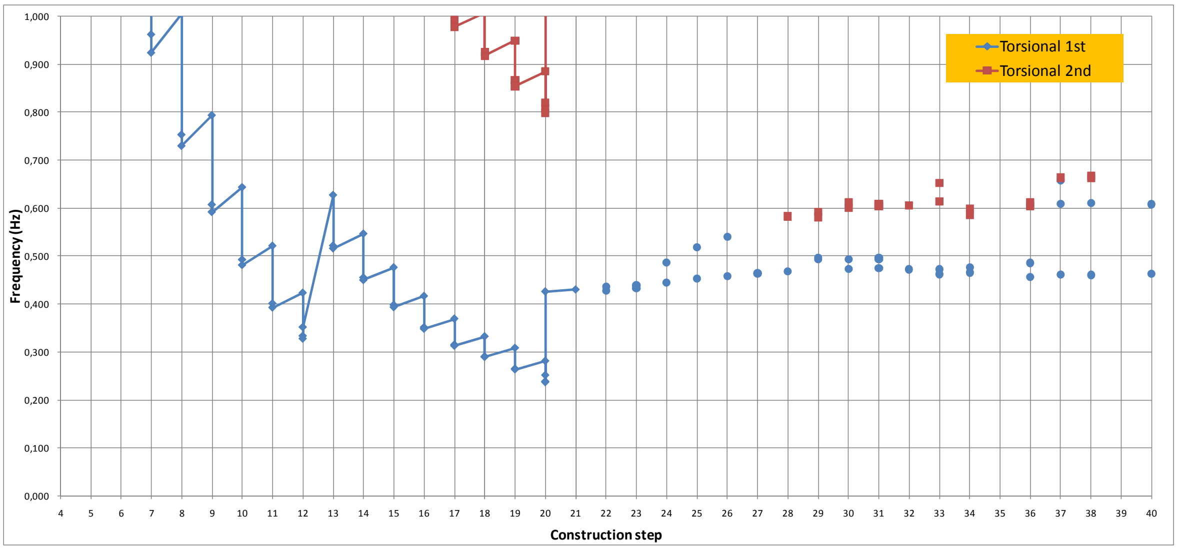


Figure 2-1 Eigen frequencies during construction as refer to tower vibration modes

3. Evaluation of Tower Vibration

The evaluation of the tower vibration was made for the safety and worker's comfort criteria and was performed by assessing the resonant wind speed of the tower at all construction steps and thereafter by assessing the response amplitude of the tower at the construction steps considered to occur below the design wind speed.

3.1 Resonant Wind Speed

The resonant wind speeds were calculated using the formula below, where the Strouhul number is 14 for a freestanding tower from the wind tunnel test.

Resonant wind speed $V = \frac{f \cdot D}{St}$

f : eigen frequency (Hz)

D: representative length (20m)

The design wind speeds were calculated using the formula below for the safety and worker's comfort criteria respectively for the tower erection top, where the 11.875 m/s is expedient wind speed to obtain the design wind speed at the tower erection top using the formula below for the working wind speed of 15 m/s at 10mm height.

Design wind speed $u(z) = 1.07 \cdot (u_{ref} \cdot 0.17 \cdot \ln\left(\frac{z}{0.01}\right) + 0.01 \cdot z)$

z : elevation of erection top of tower

u_{ref} : 29 m/s for the structural safety (corresponds to SLS2 in the design basis)
11.875 m/s for the worker's comfort

The resonant wind speeds and the design wind speeds of the tower of 1st bending mode along the bridge axis at all the construction steps are shown in Figure 3-1, since only the 1st bending mode was found in the wind tunnel test to be assessed for the structural safety and worker's comfort criteria.

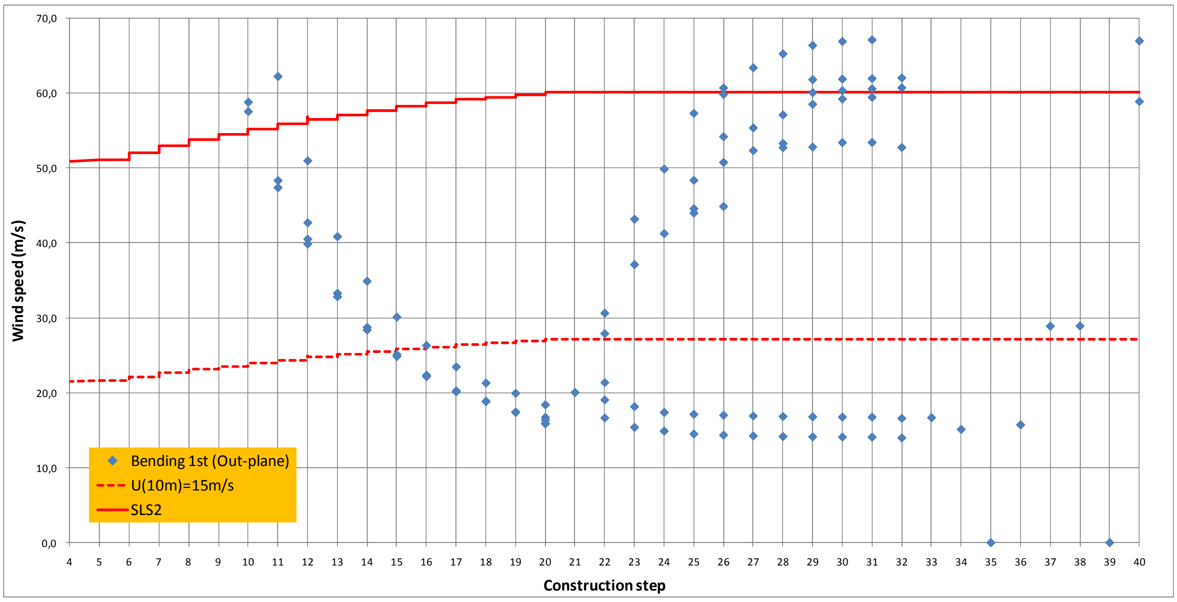


Figure 3-1 Resonant wind speed and design wind speed of tower during construction

In Figure 3-1, the blue dots show the resonant wind speed of the 1st bending mode and the red lines show the design wind speed for the safety and comfort criteria. The construction steps whose resonant wind speed appear below the design wind speed were assessed for the response amplitude.

3.2 Response Amplitude

The response amplitudes were obtained using the amplitude-Scruton number relation obtained in the wind tunnel test as shown below, where the equivalent masses are those calculated in an eigen value analysis, the inherent damping was assumed to be 0.16% rel-to-crt considered realistic from the measurements on several previous.

Scrouton number $S_c = \frac{2 \cdot M_{eq} \cdot \delta}{\rho \cdot D^2}$

Air density ρ=1.23 kg· s/m,

Representative length D = 20m

Structural damping δ=0.01 in logdec

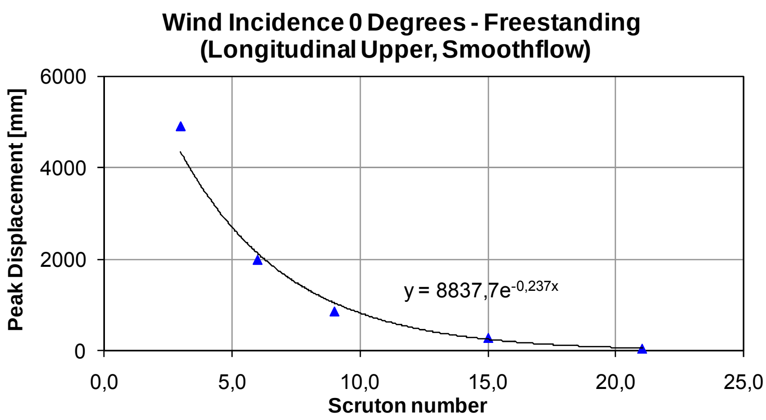


Figure 3-1 A_{peak}-Sc diagram (Freestanding tower under smooth flow)

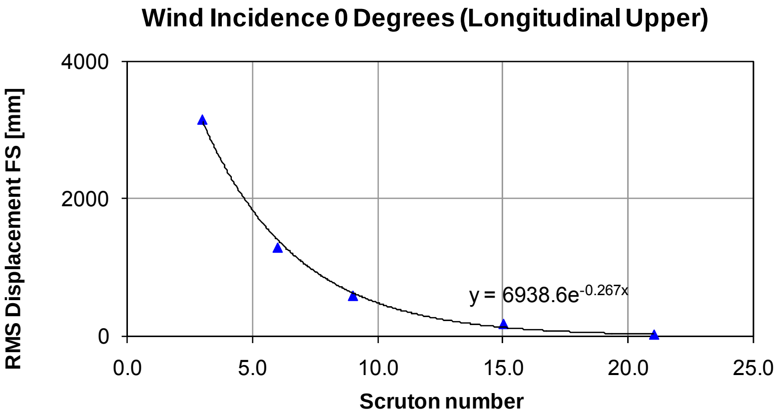


Figure 3-3 $A_{R.M.S.}$-Sc diagram (Freestanding tower under smooth flow)

The expected peak amplitudes and the permissible amplitudes of the tower during construction at the tower erection top are shown below for the construction steps where the mitigation measures is required.

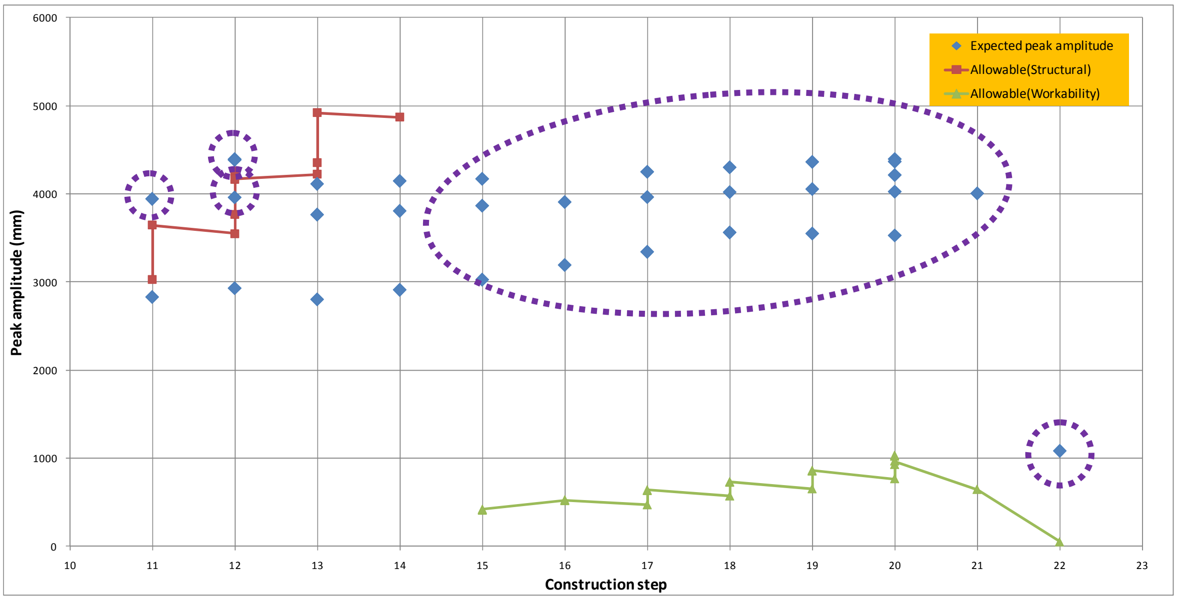


Figure 3-4 Expected peak amplitude and permissible amplitude

The permissible amplitudes were calculated in such a way that the wind force in the tower does not exceed the capacity for the load combination of 1.0DD+1.0VV for the safety evaluation, where VV is caused by the vortex shedding oscillation and the gamma factor for material of 1.05 is taken into consideration, and a peak acceleration which workers on the structure feel does not exceed 0.5 m/s^2 (50gal) for the worker's comfort evaluation.

In Figure 3-4, it was found that the tower vibration shall be mitigated at the construction steps Nos. 11 and 12 (at erection of the tower leg segment No.13 and 14) for the structural safety and at the construction steps No.15 through No.22 (from erection of the tower leg segment No.17 to the early stage of PPWS installation) for the worker's comfort.

4. Tuned Mass Damper (TMD)

The tower vibration at the construction steps Nos. 11 and 12 can be mitigated by TMD for in-service condition provided as installed in the tower leg segments No.13 and 14, since TMD for in-service condition is provided as the frequency is far from the tower frequency but the mass is larger and the required additional damping is relatively small.

The tower vibration at the construction steps Nos. 15 through No.22 shall be mitigated by another TMD temporary provided for the tower during construction. Allowing for an error margin of frequency tuning of 5% and damping tuning of 10%, the temporary TMD mass was calculated as M_{TMD}= 45 tones per tower leg and the internal damping of 9% for the construction step No.22 in order to mitigate the tower vibration with the conditions below. The detailed achievements by the temporary TMD at all the construction steps are shown in Appendix B.

- TMD is put to the tower at 290m in elevation first after the tower leg segment No.16 is erected, moved to at 350m in elevation after the tower leg segment No.19 is erected (the construction step No.9), and then moved close to / on the top of the crossbeam No.3 after the tower completion (the construction step No.19).
- TMD is put to the tower leg segment No.16 as the frequency is set to fit to the construction step 15 (for erection of the tower leg segment No.17), and re-adjusted 5 times to better fit to the following construction steps, since the tower frequency varies from 0.176 to 0.115 Hz. The final re-adjustment is made after the temporary tie-back system is installed.

The temporary TMD has been designed, as one of possible solutions, as inverted pendulum arrangement each having the mass of 22.5 tones with spring and hydraulic damper, as shown in Figure 4-1, assuming two TMDs being attached to each tower leg.

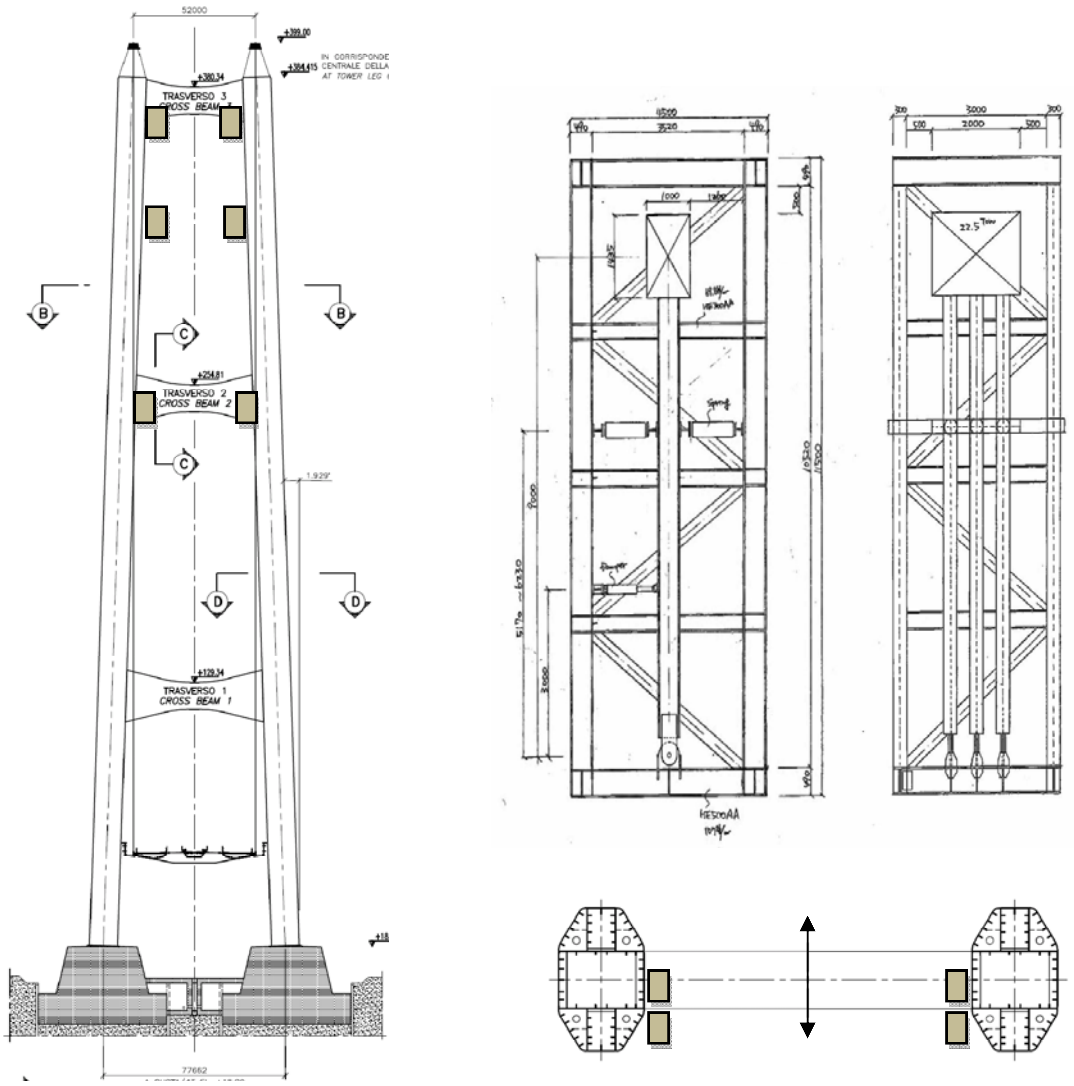
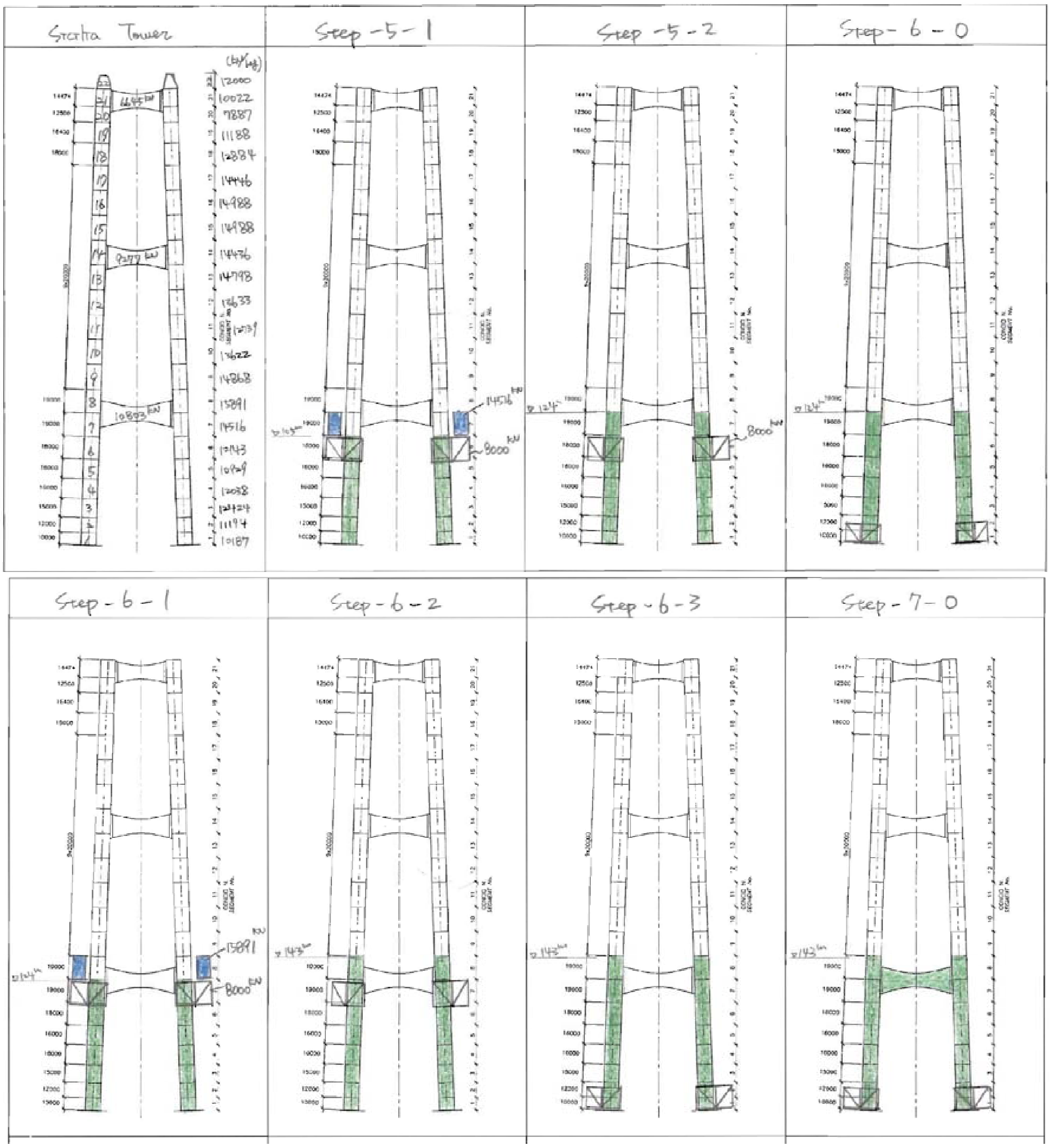
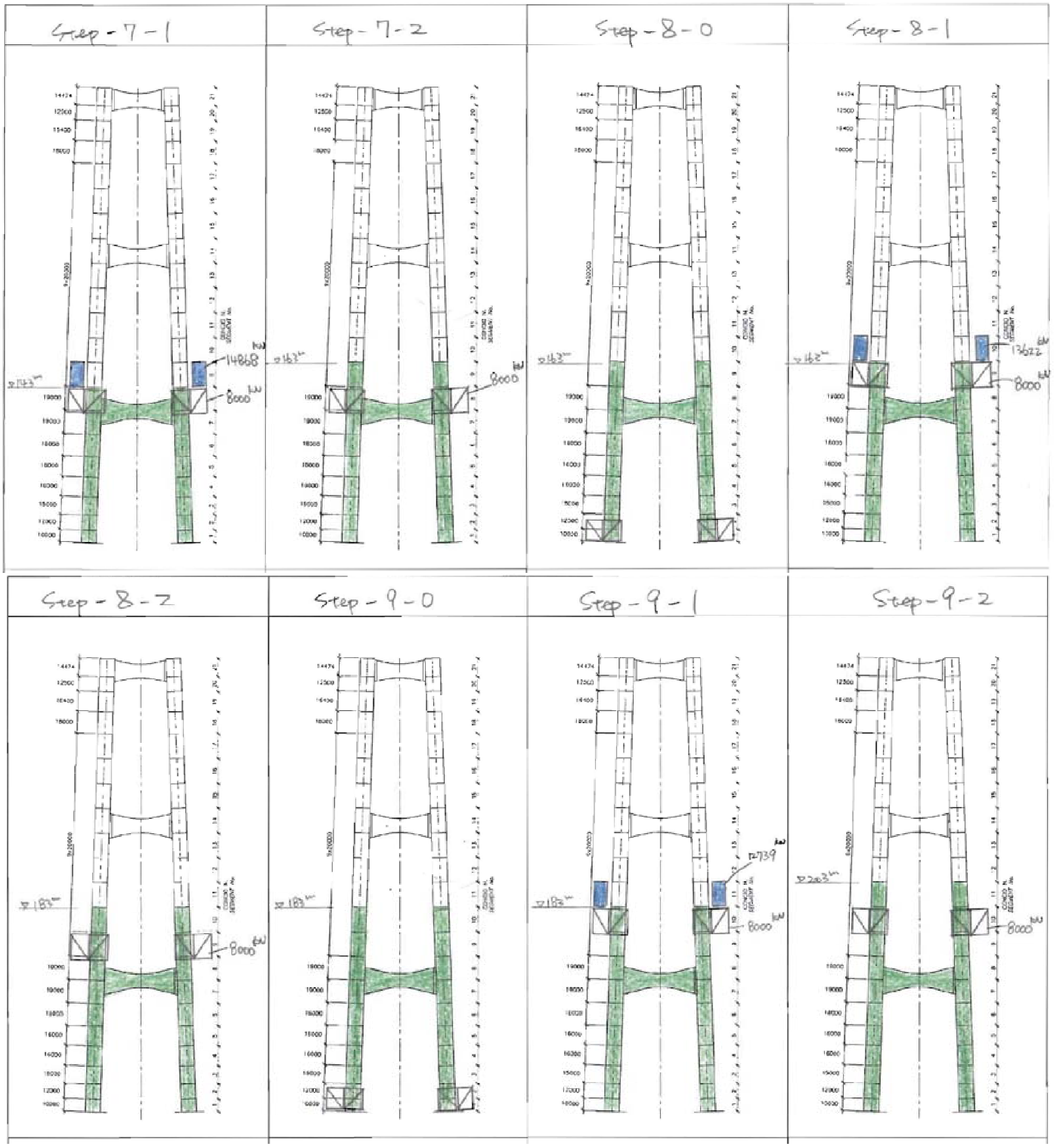


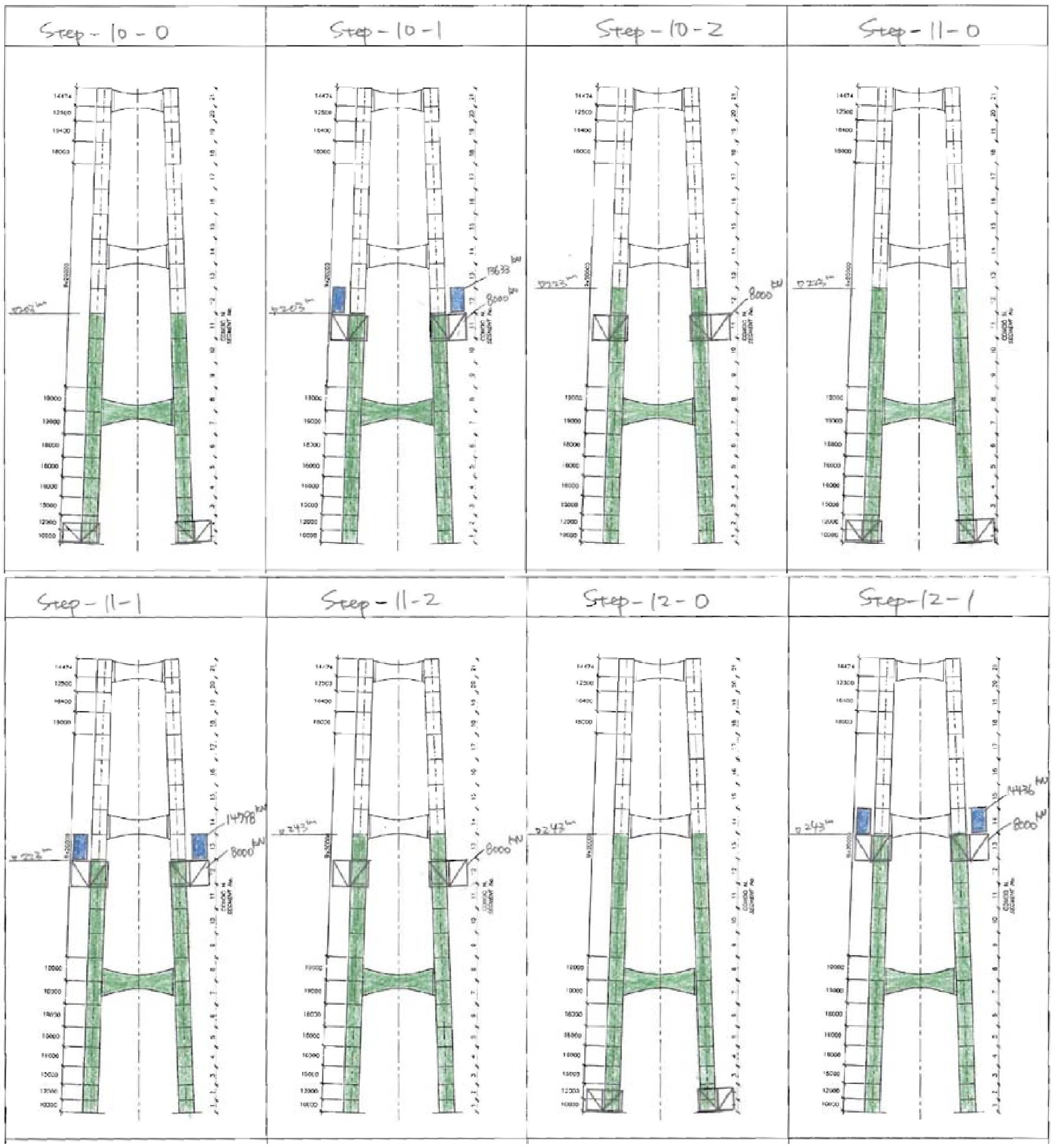
Figure 4-1 Temporary TMD and its expected position

Appendix A : Construction Step

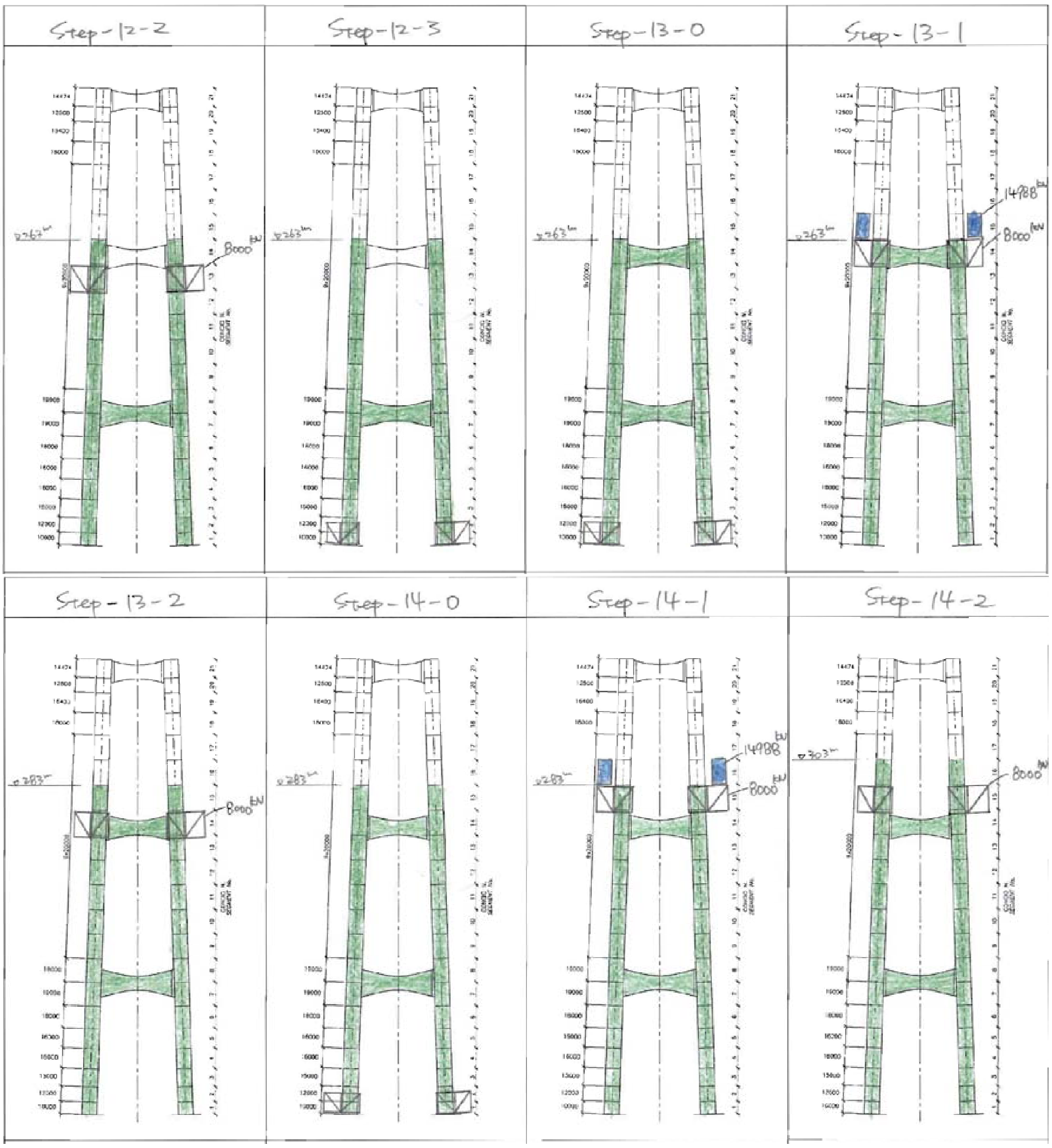


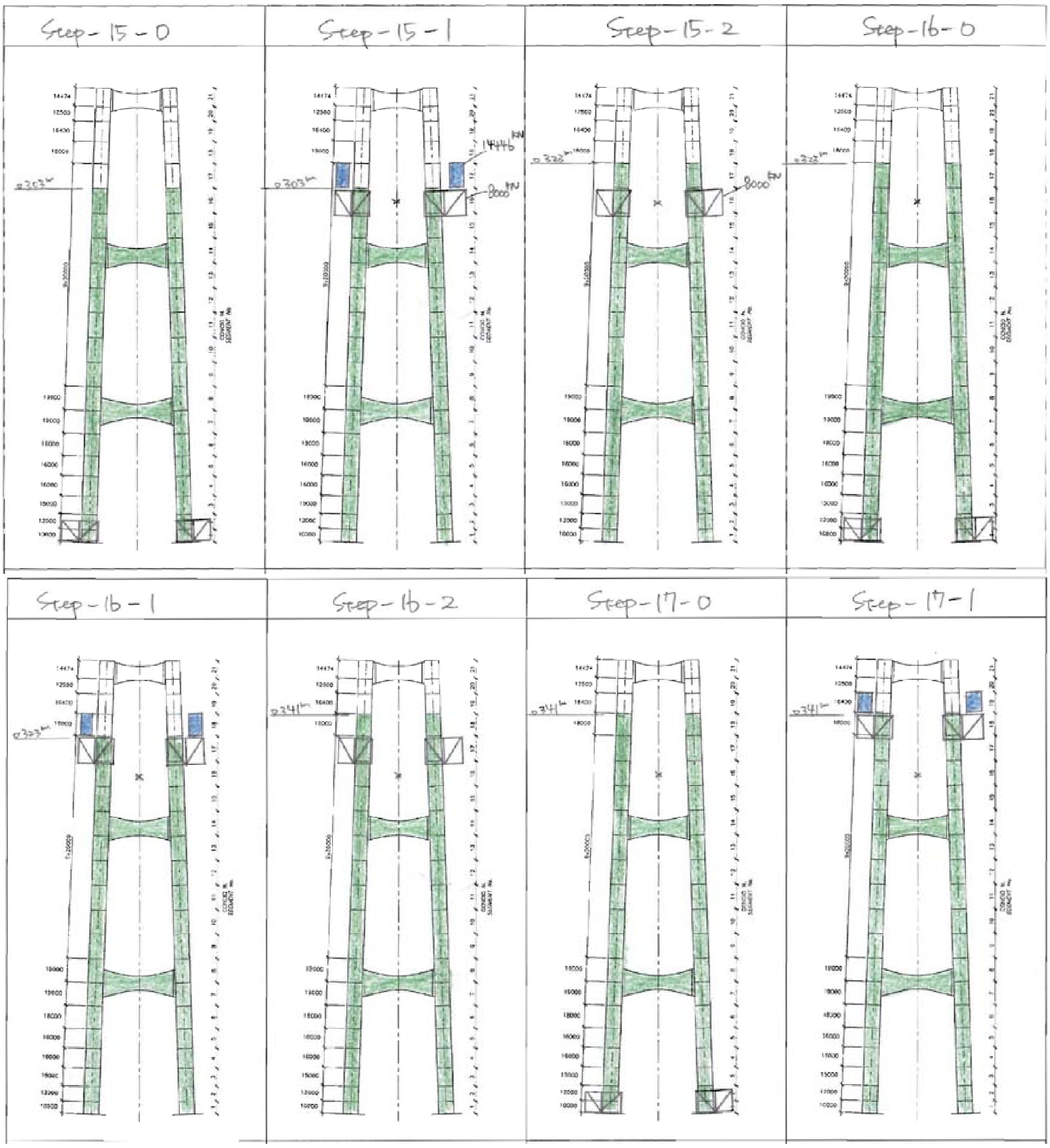


Step - 7 - 1
Step - 7 - 2
Step - 8 - 0
Step - 8 - 1
Step - 8 - 2
Step - 9 - 0
Step - 9 - 1
Step - 9 - 2
14868 kN
8000 kN
13622 kN
12739 kN

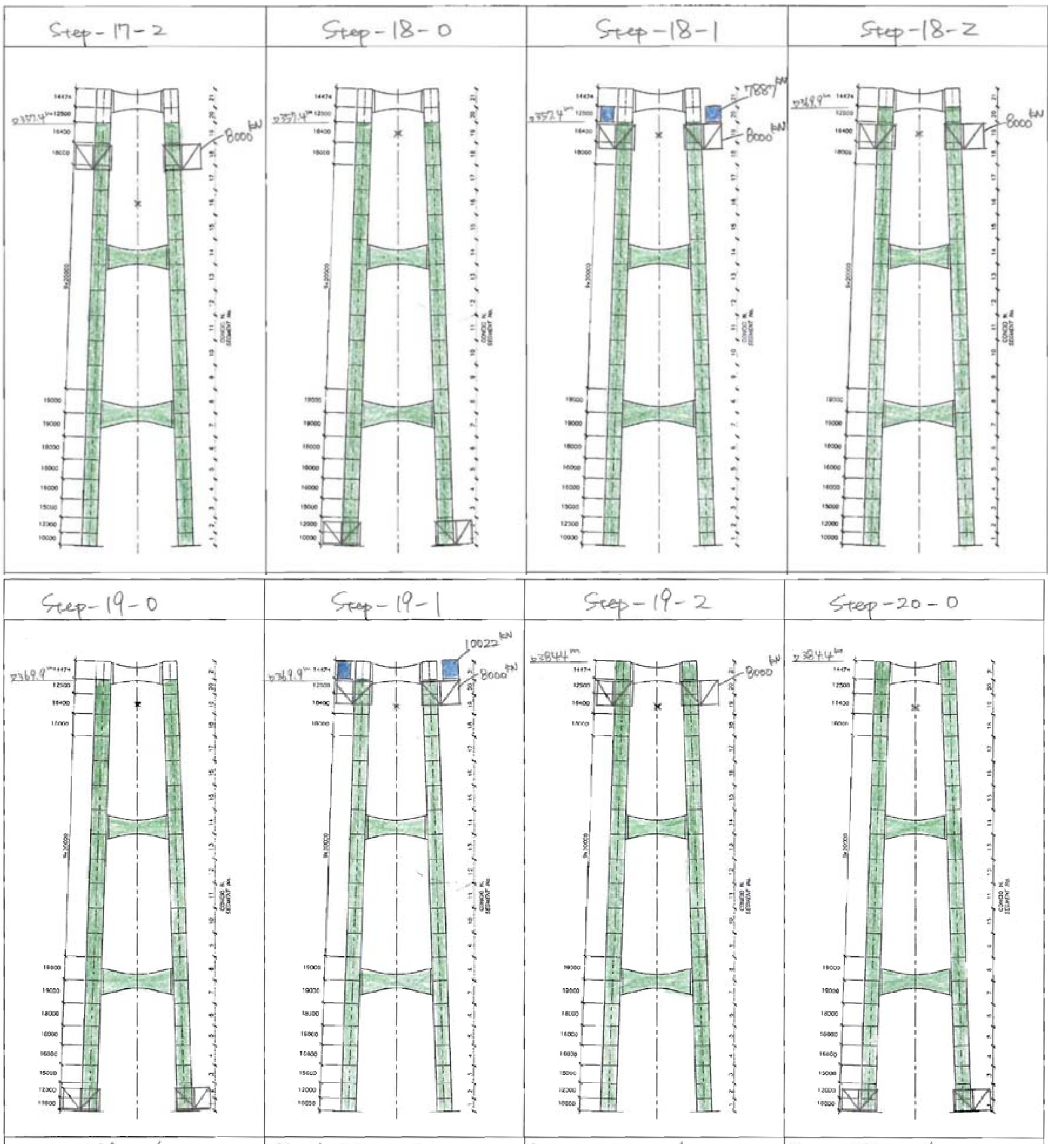


Step - 10 - 0
Step - 10 - 1
Step - 10 - 2
Step - 11 - 0
Step - 11 - 1
Step - 11 - 2
Step - 12 - 0
Step - 12 - 1

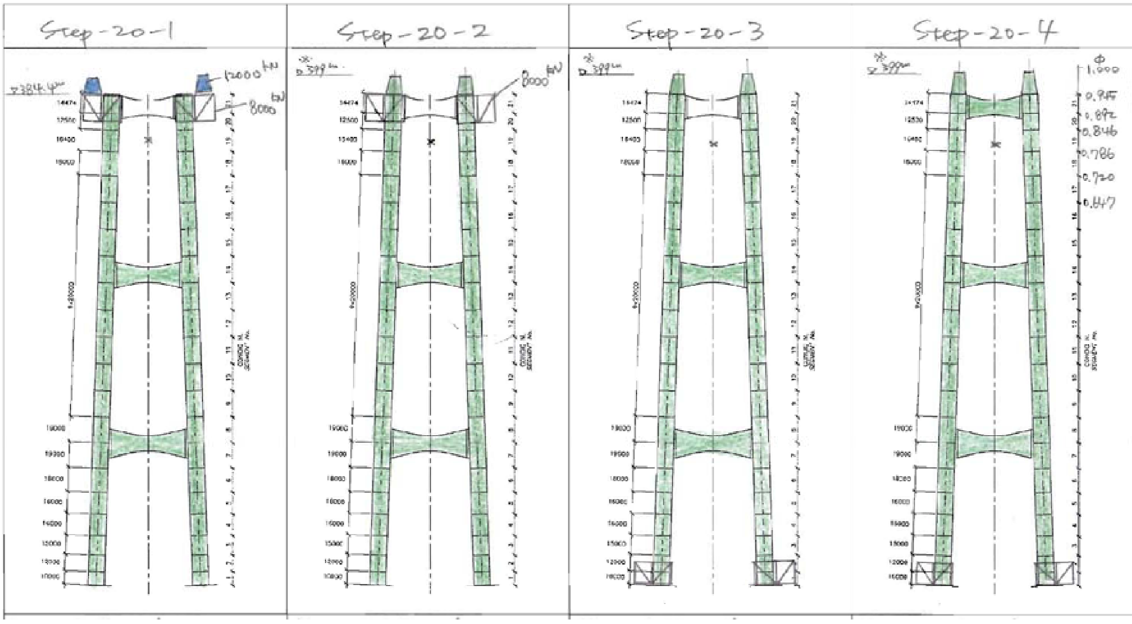

Step-12-2
Step-12-3
Step-13-0
Step-13-1
Step-13-2
Step-14-0
Step-14-1
Step-14-2
8000 kN
14988 kN



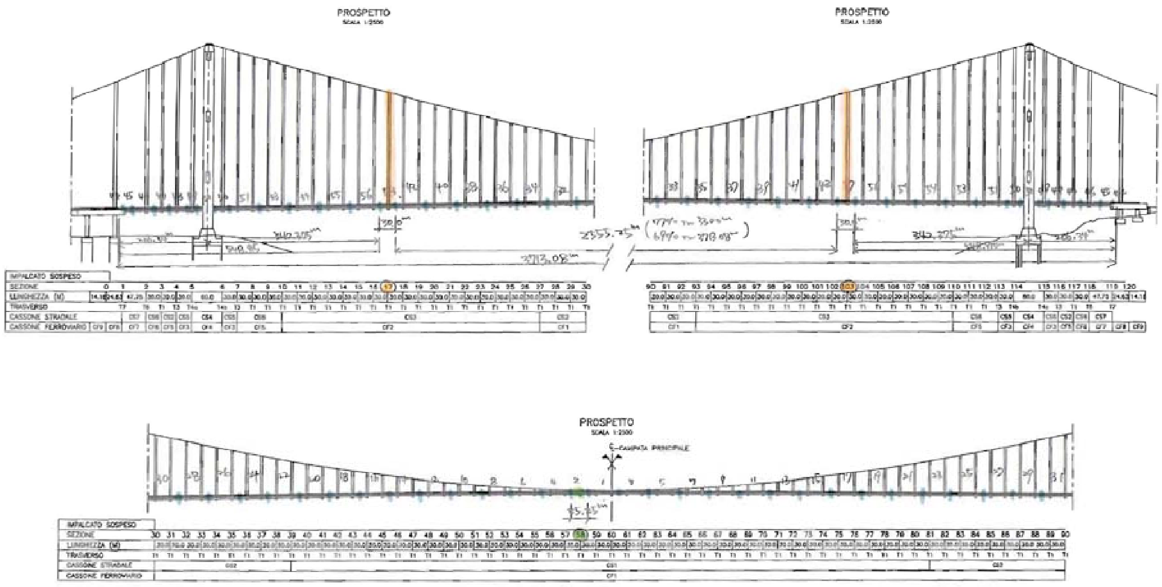
Step-15-0
Step-15-1
Step-15-2
Step-16-0
Step-16-1
Step-16-2
Step-17-0
Step-17-1



Step - 17 - 2
Step - 18 - 0
Step - 18 - 1
Step - 18 - 2
Step - 19 - 0
Step - 19 - 1
Step - 19 - 2
Step - 20 - 0
8000 kN


Step-20-1
Step-20-2
Step-20-3
Step-20-4

Step-21	Catwalk installation	Catwalk installation
Step-22		Temporary Set-back
Step-23	Cable erection	10 / 324 strands erected
Step-24		20 / 324 strands erected
Step-25		40 / 324 strands erected
Step-26		60 / 324 strands erected
Step-27		90 / 324 strands erected
Step-28		130 / 324 strands erected
Step-29		180 / 324 strands erected
Step-30		230 / 324 strands erected
Step-31		270 / 324 strands erected
Step-32		Cable completion
Step-33	Deck erection	13% deck erected
Step-34		26% deck erected
Step-35		39% deck erected Tie-down hangers are installed
Step-36		52% deck erected
Step-37		69% deck erected
Step-38		79% deck erected
Step-39		Deck closure
Step-40	Reference condition	



Deck installation sequence

Appendix B - Frequencies, Equivalent Mass and TMD Design

Re	Tower f Hz	Tower Meq ton/m/leg	Tower Mmode ton/leg	Tower δ (logdec)	Tower φ	Tower Sc /leg	TMD f Hz	TMD m ton/leg	TMD c kN･s/m	TMD h	TMD δ (logdec)	TMD φ	M(TMD)/M(Towe	f(TMD)/f(Tower)	Sc-req /leg	δeq-req (logdec)
Step-15-1	0,176	111,6	7654	0,01	1,00	4,5	0,178	45	6,0	0,060	0,375	0,96	0,54%	1,011	14,1	0,031
Step-15-2	0,174	86,1	6417	0,01	1,00	3,5	0,178	45	6,0	0,060	0,375	0,92	0,59%	1,023	14,1	0,040
Step-16-0	0,184	77,1	5772	0,01	1,00	3,1	0,178	45	6,0	0,060	0,375	0,92	0,66%	0,967	14,1	0,045
									Frequency tunning							
Step-16-1	0,156	106,1	7804	0,01	1,00	4,3	0,160	45	6,0	0,066	0,418	0,92	0,49%	1,026	14,1	0,033
Step-16-2	0,155	85	6710	0,01	1,00	3,5	0,160	45	6,0	0,066	0,418	0,82	0,45%	1,032	14,1	0,041
Step-17-0	0,164	76,3	6049	0,01	1,00	3,1	0,160	45	6,0	0,066	0,418	0,82	0,50%	0,976	14,1	0,046
									Frequency tunning							
Step-17-1	0,142	101,3	7910	0,01	1,00	4,1	0,142	45	6,0	0,075	0,471	0,82	0,38%	1,000	14,1	0,034
Step-17-2	0,141	83,5	6935	0,01	1,00	3,4	0,142	45	6,0	0,075	0,471	0,82	0,44%	1,007	14,1	0,042
		TMD is moved to Sec.19														
Step-18-0	0,149	75	6260	0,01	1,00	3,0	0,142	45	6,0	0,075	0,471	0,94	0,64%	0,953	14,1	0,046
									Frequency tunning							
Step-18-1	0,132	94,7	7795	0,01	1,00	3,8	0,131	45	6,0	0,081	0,511	0,94	0,51%	0,992	14,1	0,037
Step-18-2	0,132	82,1	7067	0,01	1,00	3,3	0,131	45	6,0	0,081	0,511	0,91	0,53%	0,992	14,1	0,042
Step-19-0	0,139	73,5	6369	0,01	1,00	3,0	0,131	45	6,0	0,081	0,511	0,91	0,59%	0,942	14,1	0,047
Step-19-1	0,122	95	8111	0,01	1,00	3,9	0,131	45	6,0	0,081	0,511	0,91	0,46%	1,074	14,1	0,037
Step-19-2	0,121	81,2	7280	0,01	1,00	3,3	0,131	45	6,0	0,081	0,511	0,87	0,47%	1,083	14,1	0,043
Step-20-0	0,129	72,8	6565	0,01	1,00	3,0	0,131	45	6,0	0,081	0,511	0,87	0,52%	1,016	14,1	0,048
									Frequency tunning							
Step-20-1	0,111	95,7	8499	0,01	1,00	3,9	0,115	45	6,0	0,092	0,582	0,87	0,40%	1,036	14,1	0,036
Step-20-2	0,111	81,9	7035	0,01	1,00	3,3	0,115	45	6,0	0,092	0,582	0,82	0,43%	1,036	14,1	0,042
Step-20-3	0,117	73,5	6343	0,01	1,00	3,0	0,115	45	6,0	0,092	0,582	0,82	0,48%	0,983	14,1	0,047
Step-20-4 (Freestanding)	0,115	77,2	7223	0,01	1,00	3,1	0,115	45	6,0	0,092	0,582	0,82	0,42%	1,000	14,1	0,045
		TMD is moved to Top							Frequency tunning							
Step-21 (Temporary Set-back)	0,140	82,4	9301	0,010	1,00	3,3	0,140	45	6,0	0,076	0,478	0,97	0,46%	1,000	14,1	0,042
Step-22 (Catwalk completion)	0,133	218,5	19508	0,010	1,00	8,9	0,140	45	6,0	0,076	0,478	0,97	0,22%	1,053	14,1	0,025

Re	Robustness α	h	δeq	A(tower)	ζ	A(TMD)
Step-15-1	100%	100%	0,128	0,000	7,3	0,000
	95%	90%	0,094	0,000	6,7	0,002
	95%	110%	0,095	0,000	6,2	0,002
	105%	90%	0,066	0,008	5,4	0,040
	105%	110%	0,074	0,003	5,2	0,016
Step-15-2	100%	100%	0,116	0,001	6,7	0,004
	95%	90%	0,120	0,000	7,2	0,003
	95%	110%	0,116	0,001	6,5	0,004
	105%	90%	0,061	0,058	4,9	0,261
	105%	110%	0,069	0,029	4,7	0,126
Step-16-0	100%	100%	0,121	0,001	6,7	0,007
	95%	90%	0,058	0,117	4,7	0,510
	95%	110%	0,066	0,066	4,5	0,274
	105%	90%	0,133	0,000	7,3	0,003
	105%	110%	0,135	0,000	6,7	0,003
Step-16-1	100%	100%	0,100	0,000	6,5	0,002
	95%	90%	0,106	0,000	7,1	0,001
	95%	110%	0,100	0,000	6,2	0,002
	105%	90%	0,058	0,023	4,8	0,102
	105%	110%	0,069	0,008	4,6	0,035
Step-16-2	100%	100%	0,090	0,006	6,4	0,030
	95%	90%	0,108	0,001	7,4	0,008
	95%	110%	0,098	0,003	6,4	0,016
	105%	90%	0,054	0,110	4,7	0,421
	105%	110%	0,064	0,046	4,5	0,169
Step-17-0	100%	100%	0,108	0,003	6,9	0,019
	95%	90%	0,059	0,117	5,0	0,483
	95%	110%	0,069	0,058	4,7	0,222
	105%	90%	0,103	0,005	7,1	0,027
	105%	110%	0,102	0,005	6,4	0,027
Step-17-1	100%	100%	0,090	0,001	6,7	0,007
	95%	90%	0,072	0,008	5,8	0,037
	95%	110%	0,081	0,003	5,2	0,015
	105%	90%	0,072	0,008	5,9	0,038
	105%	110%	0,083	0,003	5,4	0,012
Step-17-2	100%	100%	0,100	0,003	6,6	0,015
	95%	90%	0,081	0,013	6,0	0,067
	95%	110%	0,086	0,009	5,3	0,040
	105%	90%	0,071	0,029	5,5	0,131
	105%	110%	0,083	0,012	5,1	0,049
Step-18-0	100%	100%	0,098	0,008	5,6	0,041
	95%	90%	0,058	0,140	4,1	0,539
	95%	110%	0,069	0,061	3,9	0,222
	105%	90%	0,160	0,000	7,1	0,001
	105%	110%	0,136	0,000	6,3	0,003
Step-18-1	100%	100%	0,108	0,000	6,2	0,003
	95%	90%	0,079	0,006	5,2	0,032
	95%	110%	0,089	0,003	4,7	0,012
	105%	90%	0,089	0,003	5,8	0,014
	105%	110%	0,097	0,001	5,2	0,007
Step-18-2	100%	100%	0,112	0,001	6,2	0,007
	95%	90%	0,081	0,015	5,2	0,073
	95%	110%	0,090	0,007	4,7	0,031
	105%	90%	0,091	0,007	5,7	0,036
	105%	110%	0,098	0,004	5,2	0,019
Step-19-0	100%	100%	0,087	0,020	5,0	0,091
	95%	90%	0,055	0,181	3,8	0,621
	95%	110%	0,070	0,066	3,6	0,214
	105%	90%	0,144	0,000	6,9	0,002
	105%	110%	0,117	0,002	5,9	0,012
Step-19-1	100%	100%	0,080	0,006	4,5	0,025
	95%	90%	0,099	0,001	6,2	0,006
	95%	110%	0,092	0,002	5,2	0,009
	105%	90%	0,052	0,079	3,5	0,250
	105%	110%	0,068	0,019	3,3	0,057
Step-19-2	100%	100%	0,077	0,022	4,3	0,084
	95%	90%	0,096	0,005	5,9	0,026
	95%	110%	0,095	0,005	5,1	0,024
	105%	90%	0,050	0,182	3,3	0,522
	105%	110%	0,065	0,055	3,2	0,151
Step-20-0	100%	100%	0,107	0,005	6,0	0,026
	95%	90%	0,097	0,010	5,9	0,054
	95%	110%	0,095	0,012	5,1	0,053
	105%	90%	0,075	0,047	4,9	0,201
	105%	110%	0,088	0,019	4,6	0,075
Step-20-1	100%	100%	0,090	0,002	5,1	0,010
	95%	90%	0,082	0,005	5,7	0,024
	95%	110%	0,070	0,014	4,7	0,058
	105%	90%	0,072	0,012	4,2	0,045
	105%	110%	0,093	0,002	3,9	0,006
Step-20-2	100%	100%	0,093	0,006	5,1	0,025
	95%	90%	0,087	0,010	5,7	0,046
	95%	110%	0,074	0,027	4,7	0,102
	105%	90%	0,072	0,031	4,2	0,106
	105%	110%	0,093	0,006	3,9	0,020
Step-20-3	100%	100%	0,090	0,015	5,4	0,067
	95%	90%	0,080	0,031	4,7	0,120
	95%	110%	0,093	0,013	4,1	0,044
	105%	90%	0,096	0,010	5,7	0,048
	105%	110%	0,097	0,010	4,9	0,039
Step-20-4 (Freestanding)	100%	100%	0,081	0,021	5,4	0,094
	95%	90%	0,083	0,018	5,1	0,077
	95%	110%	0,087	0,014	4,4	0,050
	105%	90%	0,085	0,016	5,3	0,069
	105%	110%	0,096	0,007	4,6	0,028
Step-21 (Temporary Set-back)	100%	100%	0,104	0,002	6,6	0,015
	95%	90%	0,078	0,018	5,7	0,102
	95%	110%	0,086	0,010	5,1	0,050
	105%	90%	0,078	0,019	5,7	0,106
	105%	110%	0,087	0,009	5,2	0,045
Step-22 (Catwalk completion)	100%	100%	0,074	0,000	5,6	0,000
	95%	90%	0,058	0,000	7,0	0,000
	95%	110%	0,049	0,000	5,7	0,002
	105%	90%	0,049	0,000	4,2	0,001
	105%	110%	0,067	0,000	4,0	0,000