Transformation of a Large Diameter Monopole to a Pinned Support Guyed Tower

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SUMMARY

This paper presents the engineering challenges encountered during the design of a large steel pin foundation as part of the detailed engineering of a new 138 kV transmission line near Stewart, British Columbia. The purpose of this project was to modify the foundation type of a monopole steel tower (LL-32) located in an area of poor rock condition from a fully rigid connection to a pin connection. This new 138 kV transmission line project is located in a remote area presenting site access challenges.

At the beginning of the project, a site investigation was performed to evaluate the different rock classes and to determine the line routing and structure locations. The evaluation of the rock was based on visual observations of the area and eight boreholes performed nearby.

Originally designed as a rigid foundation, the LL-32 monopole strain tower is a 40 m high, 2.6 m in diameter at its base, which supports significant loads. This tower was moved from its initial position to decrease the loads acting on the existing LL-33 adjacent structure. Blasting and excavation of the new LL-32 site uncovered a rock surface with a rock capacity inferior to what was expected (poor rock mass rating). Fracturing and shear zones were encountered and the LL-32 site's rock class was therefore assessed as an inferior category. The location of the LL-32 structure foundation was weakened due to shear, fracture zones and weathering. There was no better rock condition in the vicinity of the structure. A concrete pad reinforcement solution was first proposed, but too costly since all work was done using helicopters. This solution had too great of an impact on the schedule and was ultimately rejected. The foundation of LL-32 had to be adapted to reduce the amount of rock anchors, thus decreasing its structural capacity. Since the original exterior steel foundation was already shipped to site at the time of the site blasting and the tower was already fabricated, a guyed approach was requested.

The decision was made to move forward and limit the impact on the schedule. The first step was to locally modify the tower structure to install guys. The second step was to modify the tower base connection to the foundation to limit the loads transferred. This entailed modifying a fully rigid base connection to a pinned connection of almost three metres in diameter, limiting the loads transferred to the main foundation of the tower and meeting the requirements of the decreased foundation and rock capacity.

BBA has developed a large pinned connection at the base of the tower. Since all the guys intersected at the same point in the tower, this assembly resisted the torsion loads of the tower while supporting vertical and lateral loads. The modification had to match the existing stub and foundation flanges. To facilitate construction, the pinned connection had to rigidly support the tower during construction. In order to limit reactions under the various line loads, the pinned connection was released after the guys were installed. It will remain in this condition throughout the service life of the line.

KEYWORDS

Transmission line, monopole, steel foundation, guying, tower modification, heavy steel fabrication, torsion load, galvanization

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INTRODUCTION

This paper presents the engineering challenges tackled during the design of a large steel pin foundation as part of the detailed engineering of a new 138 kV transmission line near Stewart, British Columbia. The purpose of this project is to modify the existing foundation type of a large diameter monopole steel tower located in an area of poor rock quality from a fully rigid connection to a pin connection.

The new 10 km long 138 kV transmission line represents significant engineering and construction challenges given its remoteness, the local mountainous terrain and associated climatic loads. Since the beginning of its operation, the existing transmission line was subjected to climatic conditions such as rime ice and snow creep. Unfortunately, the existing line has proven inadequate to withstand the severe climatic conditions prevailing in this area. A total of 32 new structures were designed to replace the existing line. The new transmission line is mostly supported by rigid and guyed circular hollow steel structures.

Figure 1: LL-32 Tower

GEOTECHNICAL INVESTIGATION AND GEOLOGY [1]

At the start of the new project, geotechnical works including site investigations, borehole drilling, core sampling and laboratory tests, rock mass classification, and geomechanical parameters of rock mass were performed. All the original data and geotechnical information prepared for the original project was also used.

The natural rock outcrop and stripped rock surface were used for assessing the rock mass and, at new borehole sites, the rock surface was classified as fair to very good. The final evaluation of the rock was based on visual observations and eight boreholes. The rock observed in the borehole cores and in the stripped areas or rock outcrops are mostly fine to medium grain andesite. In some areas, basalt, agglomerate, and tuff were observed.

Geological Strength Index (GSI) values were determined for every single foundation using two different methods. First, GSI values were estimated during the site visits using the GSI chart. Second, the GSI values were calculated with using Rock Mass Rating (RMR_{89}) method. The rock mass at the foundation of structures were classified into four groups. After classification of the rock mass in every single structure, the geomechanical parameters of rock such as subgrade reaction modulus and bonding strength of rock anchors were determined for each group.

The design of the tower structures and foundation was carried out based on the results of the geotechnical investigation. During the construction, the specific depth of rock excavation required controlled blasting.

LL-32 TOWER

The LL-32 tower structure is a 40 m high strain tower, 2.6 m diameter steel monopole (at its base), which supports significant loads (See Figure 1 and 2). This tower supports spans of approximately 500 m and features an angle of approximately 30°. The tower is supported by a steel foundation anchored to the rock. The initial foundation design overturning moment was ±20,000 kN-m.

This structure is placed on the top of a rocky cliff, on a fairly flat and extended area. The average slope of the rock surface is 10°. The rock is a fine grain dark grey /green andesite. GSI of the rock mass is around 70 based on the data collected from stripped rock surface.

Since LL-32 tower is the last tower before tying to existing line it was moved from its initial position to reduce the loads applied on the existing LL-33 adjacent structure. This was done after the structural evaluation of its existing foundation capacity. Blasting and excavation at the LL-32 new site uncovered a rock surface with an inferior than expected rock capacity and a poor rock mass rating. Fractured rock and shear zones were encountered and the rock class at this new location dropped to a lower category. This reduced the rock design bearing capacity from 6.8 to 1.1 MPa. Unfortunately, there was no better rock condition in the vicinity of the structure.

A concrete pad reinforcement solution was first proposed to support the steel foundation and to redistribute the loads on a larger rock surface. This required a large surface area and was judged too costly since all work was done using helicopters and had too much impact on the project schedule. This option was ultimately rejected.

LL-32 TOWER MODIFICATIONS – GUYED STRUCTURE

Since the original steel foundation was already shipped on site at the time of blasting operations and the tower structure was already fabricated, a guyed tower approach using the original steel foundation was studied. The steel foundation was adapted to reduce the amount of bearing pressure on rock. Next, the LL-32 tower structure had to be modified to reduce the loads transferred to its steel foundation.

Since the construction timeframe was limited to the summer months and to limit the impact on the schedule, a decision was quickly taken after some preliminary structural and geotechnical analysis. Our preliminary approach featured four guys attached to the two vertical top arms splices of the common section. With this configuration, the guys would have restrained the tower in torsion.
For various reasons, the tower fabricator had already modified and installed nine guy attachments at the upper connection of the centre section of the tower. Analysis showed that the simple addition of guys to the structure did not reduce the overturning moments at the base of the tower and also that this guy configuration was inefficient in restraining the tower in torsion. In fact, the tower rigidity in cantilever was far greater than the lateral stiffness offered from the additional guys. Using the original tower base connection, the added guys did not provide enough rigidity to decrease the overturning moments at the base of the tower. The solution was to modify the tower base connection at the foundation to limit the overturning moments transferred to the foundation and to restrain the structure in torsion. This entailed going from a fully rigid base connection to a special pin connection of almost three metres in diameter.

The main challenges of the base connection modification were:

- To offer torsion resistance while supporting vertical and lateral loads and freeing rotation around both horizontal axes, since all the guys intersected at the same working point in the tower;
- To match the existing stub (tower) and foundation flanges bolt patterns with no modifications of the already fabricated parts;
- To respect the helicopter weight-limit constraints;
- To facilitate the construction and tower erection, the pinned base connection had to rigidly support the tower during construction prior to the guy installation;
- After construction and the final guy installation, the temporary rigid connection had to be released for the normal operation of the transmission line, limiting the loads transferred on the main foundation of the tower.

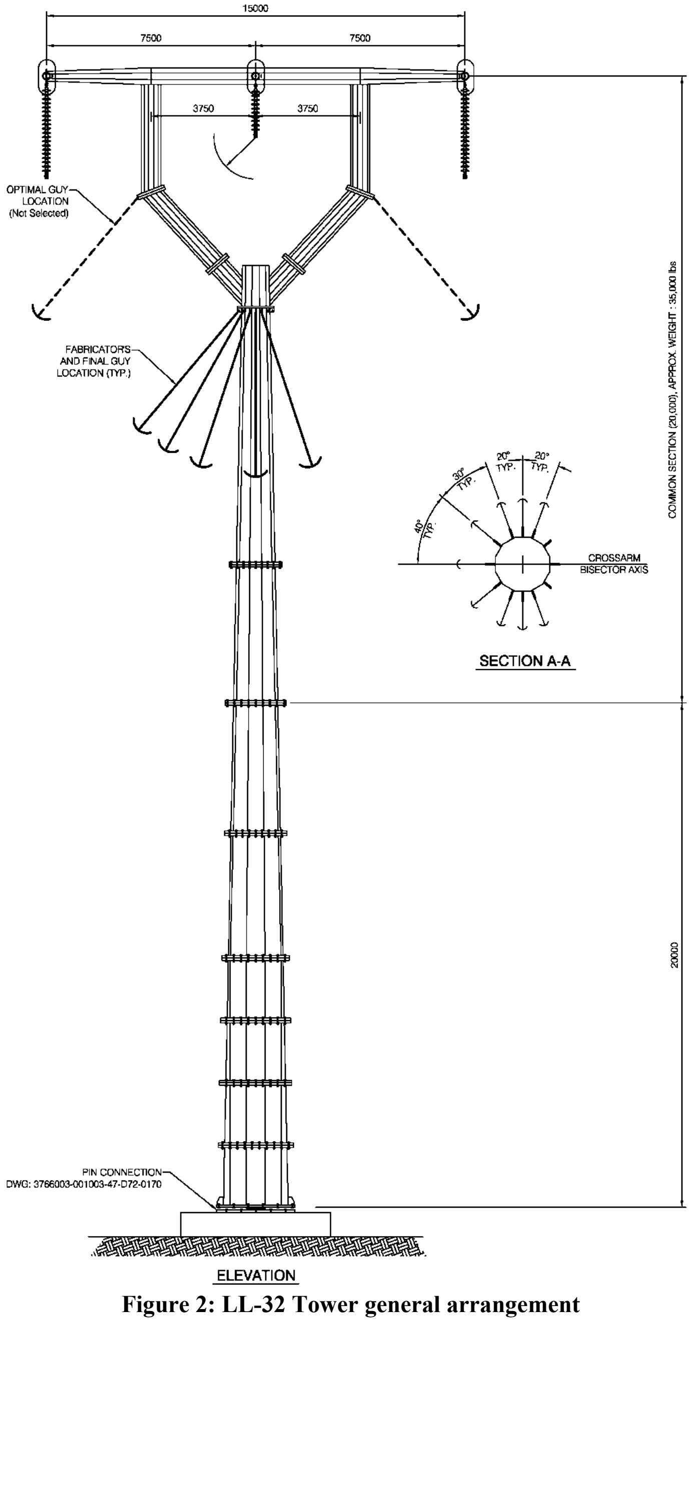

Figure 2: LL-32 Tower general arrangement

A combination of hand calculations and finite element analysis was used to verify the resistance of the base connection modifications. A reaction envelope was used for the design and was obtained from three different PLS-POLE analyses. These analyses were:

1. A completely pin support model with all guy wires;
2. A semi-rigid support model with all guy wires and;
3. A fully rigid model with no guy wires (construction loads during erection).

Table 1: Tower base reactions – Envelope

ENVELOPE LOADS			
P =	1500	kN	Max vertical load
V =	200	kN	Shear
Torsion =	850	kN.m	Torsion around tower vertical axis
CONSTRUCTION LOADS			
M_{max} =	1500	kN.m	Max. bending moment during tower construction passing thru the spacers during erection

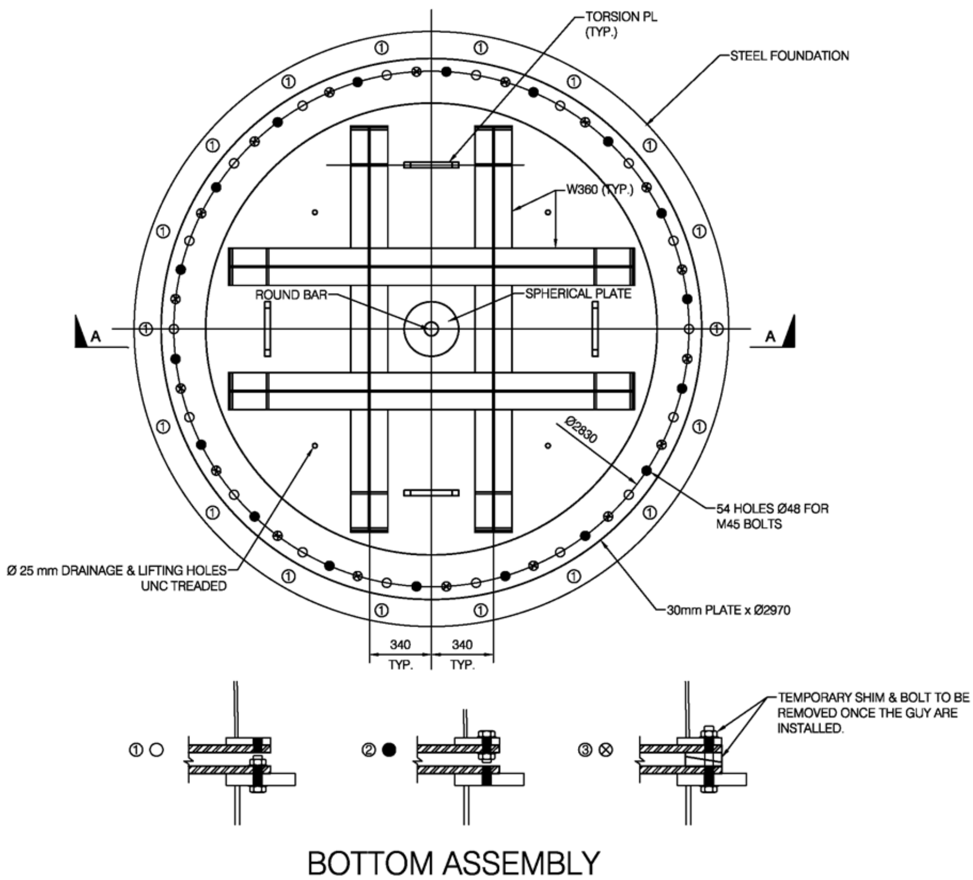

Figure 3: Base Connection - Bottom assembly

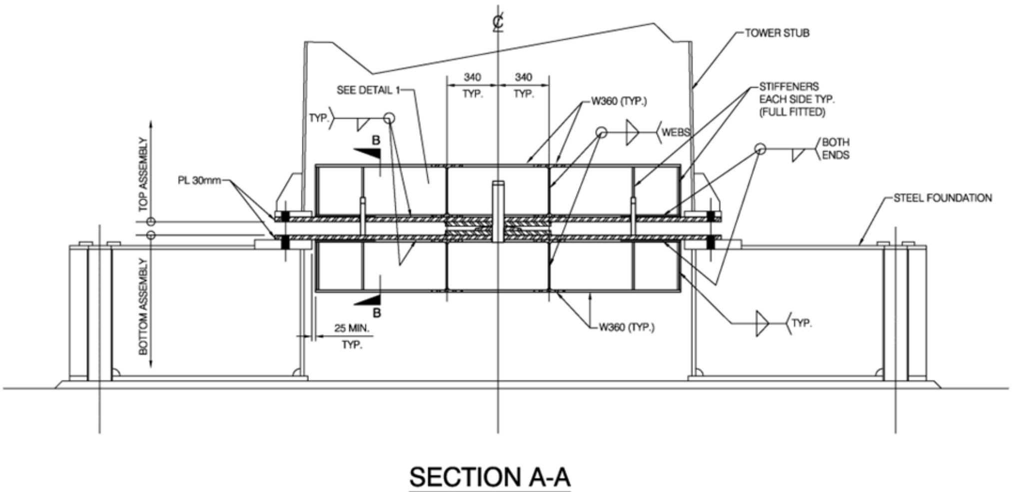

Figure 4: Base Connection – Section A

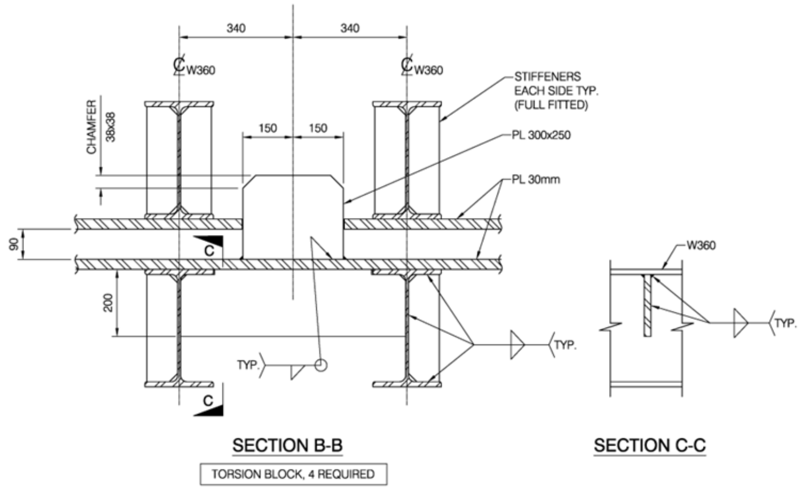

Figure 5: Base Connection – Section B and C

GENERAL DESCRIPTION OF THE BASE CONNECTION MODIFICATION - TOP AND BOTTOM ASSEMBLIES:

The base connection modifications consisted of installing two symmetrical assemblies placed one on top of each other. A top assembly is bolted on the tower stub and a bottom assembly is bolted to the original steel foundation (See Figure 3, 4, 5 and 6).

Both assemblies feature:

1. a 2,970 mm diameter main plate, drilled using the existing tower-to-foundation bolts patterns and hole diameter;
2. a 700x700 square reinforcing plate in the centre;
3. W360x72 beams welded in an H-shaped arrangement.

The top assembly main plate is slotted at four locations and features an 81 mm diameter hole in the centre. The bottom assembly features four rectangular torsion blocks, a circular spherical plate and a 76 mm diameter round bar in the centre all welded on the main plate. The four rectangular slots of the top assembly allow the insertion of the four torsion blocks of the bottom assembly. These four blocks resist and transfer the tower torsion loads. On the bottom assembly, installed below the four torsion blocks are four vertical reinforcing plates ensuring the proper torsion load transfer to the main beams and to the steel foundation. The centre hole in the top assembly allows the insertion of the centre pin of the bottom assembly. The centre pin resists all the lateral loads transferred at the bottom of the tower. A gap of 90 mm between the two assemblies was selected to offer enough rotational clearance to allow movement and full involvement of the new guys. The vertical loads are applied in the centre via a circular spherical plate. The total weight of one assembly was limited to a maximum of 6,000 lbs (2,725 kg) due to the helicopter lift capacity.

LOAD PATH

The regular stub-to-foundation load path is modified with the addition of the top and bottom assemblies. The axial and shear loads found in the tower stub walls are transferred to the top assembly top plate. The vertical load is resisted by the W360 section, main and centre plate and is transferred to the bottom assembly at the centre via the spherical plate. The shear and torsion loads, coming from the tower flange-to-assembly bolts, are transferred in the main plate of the top assembly. The base shear is transferred on the centre pin causing shear and flexure of the pin and the torsion load is transferred to the shear blocks in bearing. The load takes a mirror pathway in the bottom assembly (See Figure 5).

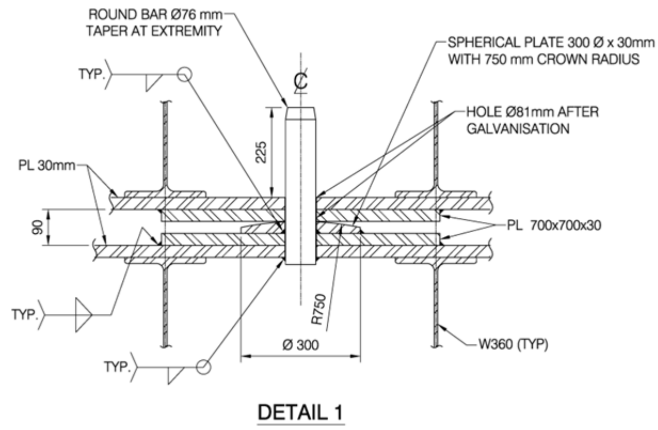

Figure 6: Base Connection – Detail 1

MAIN PLATE DIMENSIONING:

For the 2,970 mm diameter main plate thickness calculation, we assumed two conservative load distributions. For both cases, shear of the main plate was not a governing failure mode:

1. Main plate – Beam ends (Figure 7):
   The main load path occurs from the stub/foundation flanges to the beams end via the main plate. For the design of the main plate section at the W360 beam location nearest to the stub walls, we assumed that 100% of the load was applied on ⅛ of the stub surface area. Eccentricity is defined as the distance between the beam ends and the stub/foundation flanges and the effective plate design width defined as 300 mm. W360 to main plate welds and the effect of local stresses were also verified with the same load assumptions;

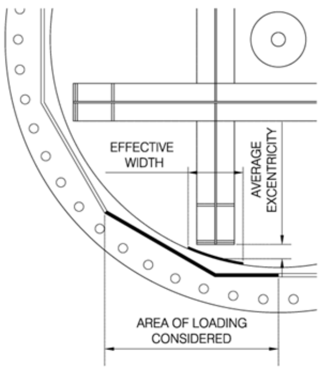

Figure 7: Main Plate Thickness - 1

2. Main plate – Between beams (Figure 8):
   For the design of the main plate section in between the W360 beams (more flexible plate areas), we assumed that 50% of the maximum load will pass through ⅛ of the stub surface area between the W360 beams. To be conservative, an eccentricity of 215 mm was considered acting on half the effective plate length. Plate stresses in different directions were considered.

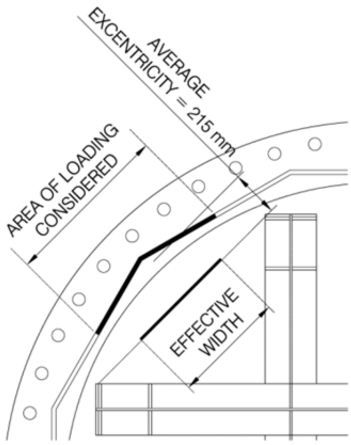

Figure 8: Main Plate Thickness - 2

CENTRE PLATE DIMENSIONING:

The additional 700x700 square plate in the centre of both assemblies was added to transfer the vertical load from the spherical plate directly to the four W360 sections (See Figure 6). Extra bending resistance was required locally in the centre square area between the W360 sections. This plate was added only where required because of the weight transportation limits.

W360 SECTIONS DIMENSIONING:

W360x72 sections were selected and designed using the maximum loads from the tower stub. The vertical load was considered to be acting on the top and bottom assemblies using a conservative level of eccentricity thus increasing the moments in an individual beam. The flange stresses at beam-to-beam connections were calculated for the member selection and welding design.

BOLTS DIMENSIONING:

Bolts in the top and bottom assemblies were designed based on the shear and torsion loads. Torsion loads were conservatively applied on the bolts closest to the torsion blocks only. For installation simplicity, a minimum of eighteen M45 Grade 8.8 bolts were used for each assembly, equivalent to exactly one third of the original bolt quantity (54 original bolts). Also, bolts in the top and bottom assemblies were not aligned with each other to allow maximum clearance and free rotation of the tower (See Figure 9).

The remaining eighteen bolt holes were temporarily used during the tower construction phase. Full length 1-½" ASTM F3125 Grade A325 bolts with temporary adjustable shims were installed to provide rotational restraint at the bottom of the tower. These bolts attached the stub flange, the top assembly, the bottom assembly and the foundation plate rigidly. Special temporary shims were also designed according to the maximum construction loads (See Figure 9).

Figure 9: Pin during tower erection

CLEARANCES VERIFICATIONS:

The centre hole final dimension and the rectangular slots dimensions in the top assembly were verified and adjusted to meet a 2° rotational requirement. The final and complete assembly was fit-tested after galvanization to verify both fit and rotational requirements.

GALVANIZING CHALLENGES

Large steel parts require large galvanizing baths and this handling constraint can cause significant delays and shipping costs. The local steel fabricator proposed metallizing, but this coating method did not provide the required design life [2]. Since construction tower erection was ongoing and due to the construction window nearing the end, it was accepted to use a smaller galvanizing bath and to perform a "double dip".

The W360 flange to the main-plate area were drilled in certain areas to provide venting (See Figure 10). Holes in the centre of the W360 webs were also added to help the flow during the galvanizing process.

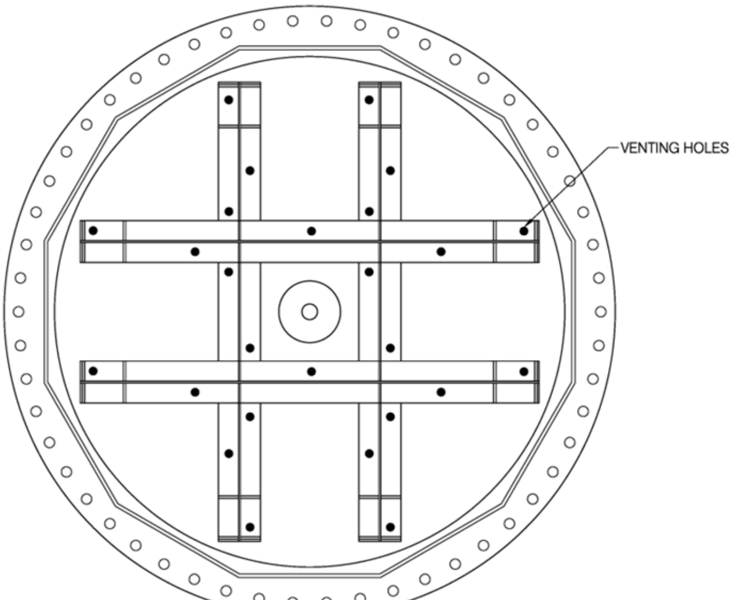

Figure 10: Venting holes

CONCLUSION

During the detailed engineering phase, the weakness of the existing foundation at LL-33 tower was discovered. LL-32 tower was moved to a new location in order to reduce as much as possible the load on the span leading to LL-33. The site blasting and excavation at the new LL-32 location showed a rock surface with a capacity much inferior to what was expected.

A reinforced pad solution was proposed to address this situation but the concrete pad solution was rejected by the Client and a guyed tower solution was requested. Since the guy attachment modifications were not optimized and, as a result, all the guys intersected at the same working point in the tower, BBA had to develop a more complex pin connection at the base of the tower that had to transfer torsion loads. The original steel foundation was adapted to reduce the bearing stress on rock.

Symmetric top and bottom assemblies were designed to offer torsion resistance while supporting vertical and lateral loads and freeing rotation around both horizontal axes. To facilitate the construction and tower erection, the pinned base connection rigidly supported the tower. During fabrication of the assemblies, the galvanization requirement was adapted to allow fabrication, shipping and installation before the construction season ended. The impact of these changes was deemed minimal for the performance of the assemblies.

With precise analysis of the new behaviour of the structure and control over the base reactions during construction and final installation stages, the tower remained free-standing, without guys, during construction and was then converted to a guyed and pinned tower during the final.

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