Safety risks associated with the use of unpreloaded anchor bolts to fix High Mast light Poles

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Revision history

Revision	Date	Description
1	12 Apr 21	Draft for discussion, Unchecked, Work-In-Progress
2	13 Apr 21	2nd draft for discussion, Unchecked, Work-In-Progress Additional sections, references and general modifications.
3	21 Apr 21	3rd draft for discussion, Unchecked, Work-In-Progress Additional sections, references and general modifications.
4	30 May 21	Place names removed for confidentiality
5	5 July 21	Section 6 Bolt preload added. Section 8.2 rewritten. General modifications.

Note!

This report has been written for the author's personal interest and to address the author's concern for public safety.

This report has not been motivated, financed or otherwise encouraged by any other party, either private or commercial.

All drawings and illustrations have been produced by the author using his own resources unless attributed otherwise.

Contents

1 Summary

This report discusses the author's concerns about the design of High Mast Light Poles (HMLPs) using stand-off base plates with unpreloaded anchor bolts.

The same concerns apply equally to other applications using similar base plate designs that are also subject to repeated loads which induce stresses in the anchor bolts in excess of the anchor bolt endurance limit. e.g. Small wind turbines.

Fatigue failure of unpreloaded anchor bolts would cause the structure to collapse. Collapse has occurred in several small wind turbines (< 50 kW).

The author believes that HMLPs are a particular concern because of their location. e.g. Alongside motorways, at road junctions, airports, car parks etc. Collapse of HMLPs with heights up to 35 m would create a very high risk of death and injury.

The author's concerns include:

a) The difficulty of accurately predicting the safe design life of stand-off base plates with unpreloaded anchor bolts subject to repeated and dynamic loads.

b) The difficulty of accurately confirming that the basis of design for unpreloaded anchor bolts has been achieved or is maintained throughout the life of the structure.

c) Higher fatigue stresses in the lower part of the HMLP caused by flexure of the base plate when compared with that of fully grouted base plates restrained using preloaded anchor bolts. (Fatigue cracking of welds in the lower part of HMLPs has occurred at several installations [Ref 7]).

d) The absence of any requirement in the HMLP design standard ILE TR07 to consider fatigue failure of the structure and anchor bolts. (ILE TR07 was replaced by PLG07 in 2013. To be confirmed - does PLG07 now include requirements to consider fatigue?).

2 Objective

To explain why the author does not believe that it is possible to undertake realistic fatigue design calculations for unpreloaded anchor bolts necessary to justify the safe long term design of High Mast Light Poles (HMLPs) using stand-off base plates.

3 Terms

The terms and their definitions below apply to this report only.

Terms listed together are intended to be used interchangeably.

Term	Abbreviation	Description
Anchor bolt Anchor rod Hold down bolt		Partially or fully threaded rod cast into a concrete foundation for purpose of locating and restraining HMLPs.
Bolt preload Bolt post-tension		Imposing a tensile load in a bolt during installation for purpose of ensuring joint tightness and resistance to fatigue failure when subject to dynamic load. The term "preload" is often used in mechanical engineering applications. The term "post-tension" is often used in civil engineering applications.
Grout		Load bearing non-shrink material applied as a liquid after installation and levelling to ensure a full and uniform structural and specified load bearing contact between the base plate and the foundation. Anchor bolt preload is applied after the grout has reached its full structural strength. (Grout DOES NOT refer to "cosmetic grout" applied or "trowelled" AFTER bolt installation and tightening and thus not subject to preloading).
High Mast Light Pole High Mast Lighting Tower	HMLP HMLT	Vertical cantilever structures used to support CCTV or large light fittings for illumination of motorways, carparks, airports where large numbers of smaller lamp posts are not practicable. Supported using a base plate flange with pre-cast anchor bolts. Anchor bolts normally arranged as a "stand-off" base plate.
Lamp post		A vertical cantilever structure used to support CCTV cameras or single light fittings for illumination of streets or pedestrian areas. Lamp posts are smaller than HMLPs. Most lamp posts are supported by "planting" the lower part of its structure directly into the ground or foundation. i.e. Supported in the same way as a fence post.
Stand-off base plate (See Figure 7)		A base plate supported above the top of concrete level. All vertical downwards and upwards loads are carried entirely by the anchor bolts using levelling and top nuts. The gap between top of concrete and underside of base plate might or might not be filled with "cosmetic grout" after anchor bolt tightening. The "cosmetic grout" does not carry any significant structural load. (See xx)

4 Introduction

High Mast Light Poles (HMLPs) are a type of "lamp post" used to support larger, multiple light fittings at higher elevations than lamp posts typically used for street lighting. (See Figure 1, Figure 2). HMLPs are typically between 20 m and 35 m high.

HMLPs are typically located in areas of high occupancy or high road traffic. Typical locations include:

- Adjacent to some motorways.
- Motorway and road junctions.
- Motorway service areas.
- Air Ports, Car parks

Figure 1 Typical High Mast Light Pole

Figure 2 High Mast Light Pole located in retail park

5 Risk of death and injury

The location of most HMLPs, combined with the high occupancy of adjacent areas within a radial distance equal to their height, means that collapse would cause a very high risk of death and injury.

6 Bolt preload

6.1 Introduction

An understanding of bolt preload is key to understanding this report. The explanation in this section is applicable to any bolted joint.

Threaded bolts used in structures subject to varying loads are a paradox.

It is common knowledge that cutting a sharp notch in an object and then bending it "backwards-and-forwards" will cause many objects to break.

Breakage or failure occurs for two reasons.

1. The notch creates a stress concentration. Stresses in the material at the notch caused by external loads are disproportionately higher than they would have been if the object had remained smooth.
2. Crack growth caused by repeated applications of load or displacement.

If a high enough load is applied, the high local stresses will initiate a crack. Repeated application of the load or bending the object "backwards-and-forwards" causes the crack to grow. When the crack reaches a critical length, the object is weakened enough such that it fails catastrophically. The progressive crack growth caused by repeated loads is known as Fatigue.

A threaded bolt is covered in one long spiral notch.

Why do threaded bolts not fail when subject to repeated "backwards-and-forwards" displacement or reversing loads?

Road vehicle wheels are retained using threaded bolts (studs) AND subject to reversing loads.

Why do vehicle wheels not fall off after being driven for a short distance?

Vehicles wheel bolts **DO** fail catastrophically **WHEN** subject to significantly changing loads.

Bolted joints are **normally** designed to ensure that bolt loads **DO NOT** change significantly - even when the joint **is subject to significant load changes.**

(In real life, bolt loads vary by small amounts when bolted joints are subjected to load change. Small changes of bolt load in even tight joints are caused because no material is theoretically rigid. Reference to changing loads above mean loads that change significantly for purpose of calculating stress range and fatigue life).

6.2 Preventing bolt load variation by preload

Preloading every bolt in a joint is key to preventing the repeated load changes that would cause bolt fatigue failure. Preloading is achieved by tightening the nut or by hydraulic pre-tensioning.

The preload induced in a bolt is normally much higher than the external loads applied to the joint.

> For comparison, my car weighs about 19 kN. Each wheel is retained using 5 stud bolts. The preload induced in each stud bolt is over 50 kN at a torque of 140 Nm. Therefore, each wheel is retained with a preload that is more than 15 times the weight of the car.

A compressive clamping force is induced in the joint by the bolt preload.

Changes to the external load applied to the joint causes the clamping force to change. However, with the correct preload, the tension induced in the bolt would change very little. (See Figure 3 and Figure 4).

The bolt will not fail by fatigue if the changes of bolt tension are very small - even if the loads applied to the joint are both large and cyclic.

Bolt tension will only change and risk bolt fatigue failure if the external load is high enough to overcome the bolt preload.

Fatigue failure would only occur if the bolt preload was lost. For example, if the nuts were not tightened properly during installation or loosened due to vibration.

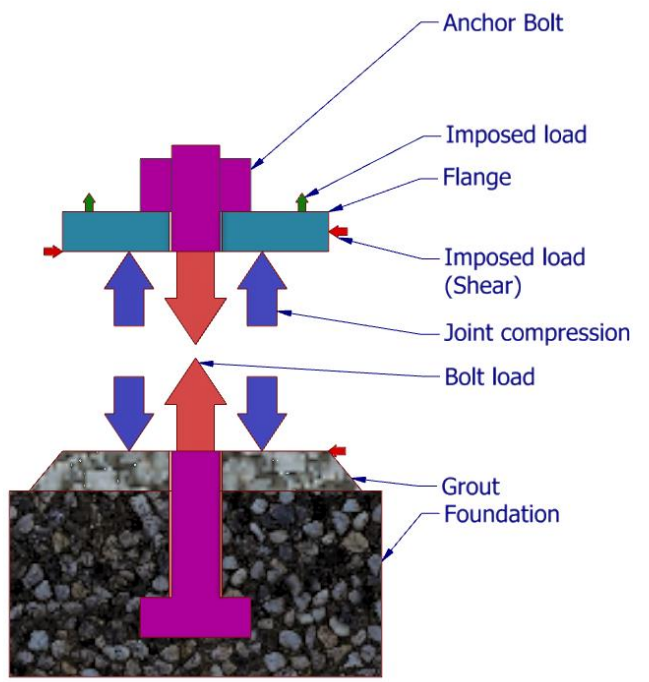

Figure 3 Free body diagram of a structural anchor bolt WITH grout when subject to an external loads that are much smaller than the bolt preload and grout friction

(Imposed shear load resisted by friction on grout)

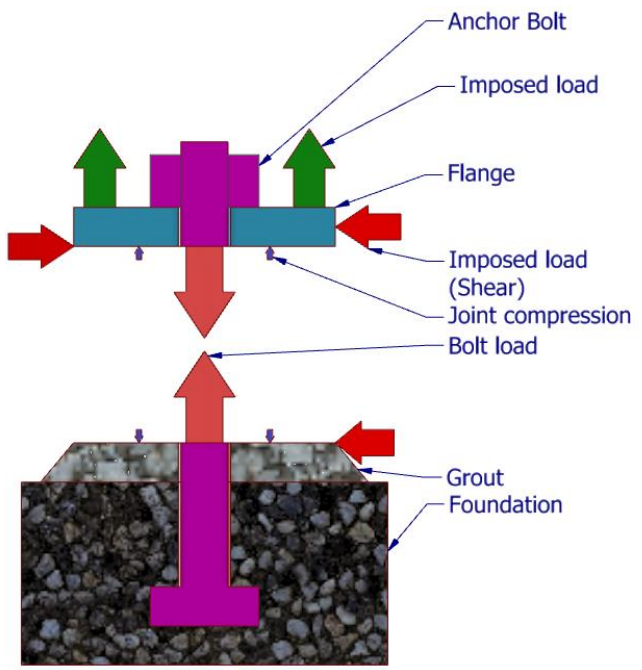

Figure 4 Free body diagram of a structural anchor bolt WITH grout when subject to an external loads that are nearly as large as bolt preload and grout friction

(Imposed shear load resisted by friction on grout)

6.3 Unpreloaded bolts

Unpreloaded bolts would normally only be used when external loads applied to the joints are constant or external load changes are negligibly small.

An example of an unpreloaded bolt is when used in a "Stand-off" base plate. (See Figure 7). A "Stand-off" base plate is only connected to its foundation through the anchor bolts. The section of bolt below the levelling nut is not preloaded. Every change to the loads applied to the joint is applied equally to the unpreloaded anchor bolts. (See Figure 5 and Figure 6).

NOTE! Lateral loads applied to the joint are also transmitted through the anchor bolts as shear and bending. In contrast, bolt loads in preloaded joints consist only of simple tension. i.e. Lateral loads in preloaded joints are carried by friction induced in the grout by the bolt preload.

Compressive loads applied to a joint with unpreloaded bolts would also cause an equal compression in the anchor bolts. Compressive loads in threaded bolts are not normally a limiting factor. However, compressive loads applied to cast-in anchor bolts could also increase the tendency of the bolt-to-concrete bond to shear and loosen.

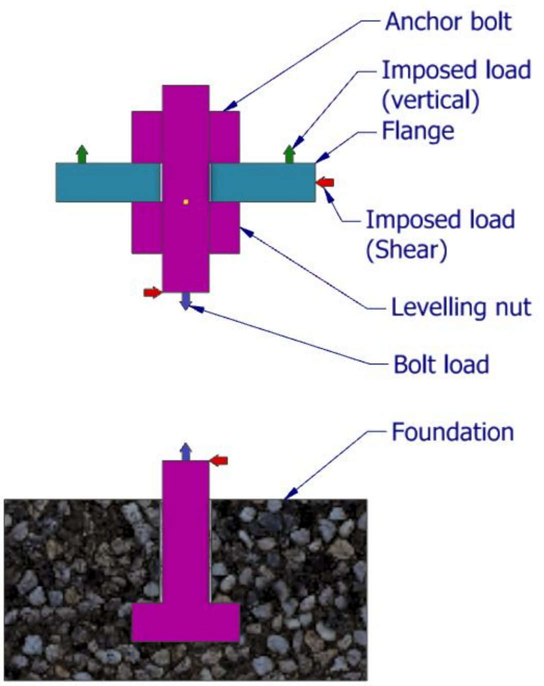

Figure 5 Free body diagram of structural anchor bolt WITHOUT grout subject to a small external axial and lateral loads
Imposed shear load resisted by anchor bolt shear/bending.

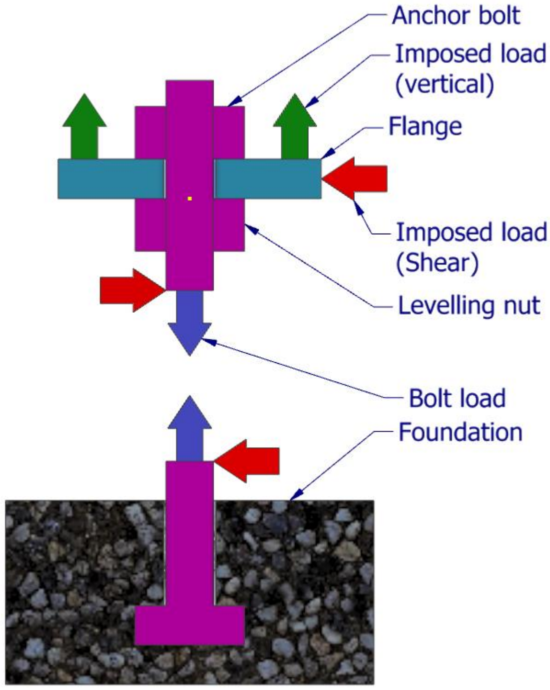

Figure 6 Free body diagram of structural anchor bolt WITHOUT grout subject to a large external axial and lateral loads
Imposed shear load resisted by anchor bolt shear/bending.

7 Author's concerns about HMLP design

Note The author accepts that unpreloaded anchor bolts used for HMLPs are normally fairly reliable despite the author's concerns about their design.

For example, the author was informed that anchor bolts have been reused when some HMLPs have been replaced during upgrades.

One of the author's concern is that absence of frequent failure is not a reasonable design justification for future safe operation - especially when not supported by realistic calculations, logical reasoning and the absence of effective inspection.

The author's concerns that make the risk of collapse impossible to predict are as follows.

a) Unknown installation loads in individual anchor bolts. i.e. A need to make design assumptions that cannot be measured, substantiated or monitored during the HMLP operating life.

b) Unknown bond strength between concrete and individual anchor bolts.
i.e. A more effective axial bonding between anchor bolt and concrete would create a stiffer anchor bolt that would attract a higher load compared with a less effectively bonded neighbour.

c) High stress range in each anchor bolts even if installation loads could be accurately measured and monitored.

d) Cyclic bending loads applied to threaded anchor bolts for which design acceptance data is not available. i.e. Fatigue design acceptance data is only available for cyclic axial loading. (See 14.2 below [Ref 4 Table 8.1]).

NOTE! Ref 4 Table 8.1 states "*Anchor bolts and rods with rolled or cut threads in tension*." i.e. **Not** '..*tension* ***and/or bending****..'*.

The diagram shown Ref 4 Table 8.1 under construction details only shows axial tensile forces. The diagram **does not show** bending moments or compressive forces.

Ref 4 Table 8.1 states *"Bending and tension resulting from prying effects and bending stresses from other sources must be taken into account"*. **No advice or methods are given in the BS Standard [Ref 4] about how bending stresses in threaded bolts are to be "*taken into account" for purpose of fatigue life design*.** For example, should bending stresses be treated as though they were pure axial stresses? Should the same "Detail Category" be applied to bending stresses as used for uniform tensile stresses. i.e. Detail Category 50.

e) The HMLP anchor bolt design below the levelling nut is impossible to inspect or prove by load testing or by checking of nut tightness.

f) Most HMLP installations are subject to high levels of corrosion. e.g. Moisture, road salt, dirt or even completely buried in soil and vegetation.

g) Higher stresses imposed in the base plate and in welds connecting the base plate to the mast caused by increased flexing when compared to a fully grouted and preloaded anchor bolts. i.e. HMLPs using stand-off base plates are fully supported by anchor bolts acting as point restraints rather than by directly and uniformly supporting on the grout.

h) Reduced stiffness of HMLP caused by use of stand-off base plate and unpreloaded anchor bolts. i.e. The axial and lateral stiffness of the anchor bolts is integral to the overall stiffness of the HMLP structure. A reduced overall stiffness is expected to reduce natural frequency and increase the dynamic load factor. An increased dynamic load factor will increase loads and hence stresses created during the application of variable wind loads.

8 Inspection

8.1 Inservice inspection of anchor bolts to confirm design intent

The author does not believe that there are any effective methods to predict the actual loads in individual anchor bolts below the levelling nuts. Such data is essential to confirm that each HMLP installation is in accordance with all assumptions and calculations used to justify its design.

8.2 Inservice inspection of anchor bolts to predict failure

8.2.1 Ultrasonic inspection

The author believes that ultrasonic inspection is the only practicable method of anchor bolt inspection that might give advance warning of imminent collapse due to fatigue failure of anchor bolts below the levelling nut.

Ultrasonic inspection might also be used to assess the degree of anchor corrosion.

Note! The author is not an expert in ultrasonic inspection. Further advice should be obtained from experienced NDT inspectors to assess the reliability and accuracy of ultra sonic inspection when used to detect fatigue cracks and corrosion of HMLP anchor bolts.

8.2.2 Nut tightness

Checking anchor nut tightness has no effect on bolt forces below the levelling nut. Anchor nut tightness could not be used to confirm that the anchor bolt had not completely fractured below the levelling nut.

8.2.3 Anchor bolt load testing

The only other option to test anchor bolts would be to sequentially loosen both top nuts and levelling nuts on individual anchor bolts. *** A load "pull" test could then be applied to the anchor bolt. This method would be time consuming. The author would not recommend load testing for regular use. However, a load test might give a valuable insight if used for sample testing when used in conjunction with ultrasonic inspection.

*** A more practicable option might be to temporarily demount the HMLP.
Another option would be to load test the anchor bolts when an HMLP had to be removed for other purposes.

A unique benefit of load testing would be the ability to confirm the axial stiffness of each anchor bolt. i.e. A variation in stiffness between anchor bolts caused by a difference in anchor bolt-to-concrete bonding would indicate that anchor bolt fatigue would be a higher risk).

8.2.4 Visual inspection of anchor bolts

Access to visually check for corrosion varies between installation even when all soil and coverings have been removed. Visual inspection might not be very accurate or reliable in every situation.

8.3 Inservice inspection of welds

In contrast to anchor bolts, welds at the base of most but not all HMLPs are easily accessible for normal visual and NDT / examination. However, the base of some HMLPs is buried in grass, soil, compost, tarmac and other materials which would prevent monitoring and inspection of welds and anchor bolts. The age of some covering materials would indicate that many HMLP installations have not been inspected for many years. (See Figure 12).

9 HMLP Design

HMLPs are normally much taller and larger than lamp posts.

The primary design difference between lamp posts and HMLPs is their method of anchorage.

Lamp posts are normally "planted" or cast into the ground or foundation in the same way as a fence post.

HMLPs are connected to anchor bolts cast into a concrete foundation. The HMLP is fitted with a circular or polygonal base plate flange. Matching anchor bolt holes are drilled in the flange in a uniformly spaced circular pattern. (See Figure 7, Figure 9 below).

Anchor bolts are typically cast directly into concrete without any type of sleeving **. i.e. The anchor bolts are in direct contact with the concrete.

** Based on the author's observations of HMLPs mostly situated in xxxx. HMLP installations in other parts of xxxx and yyyy appear similar.

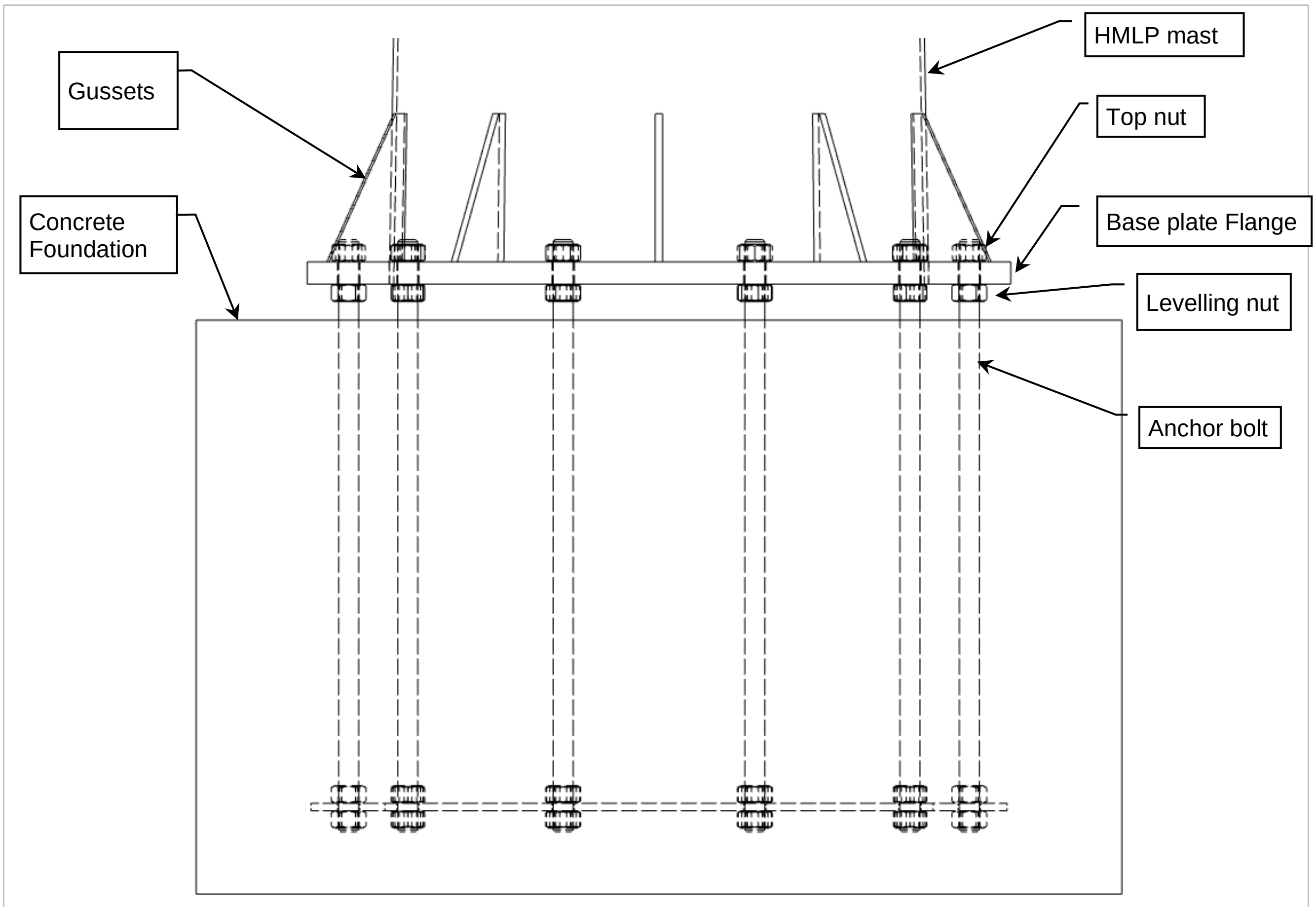

Figure 7 Typical design of HMLP base plate showing anchor bolts

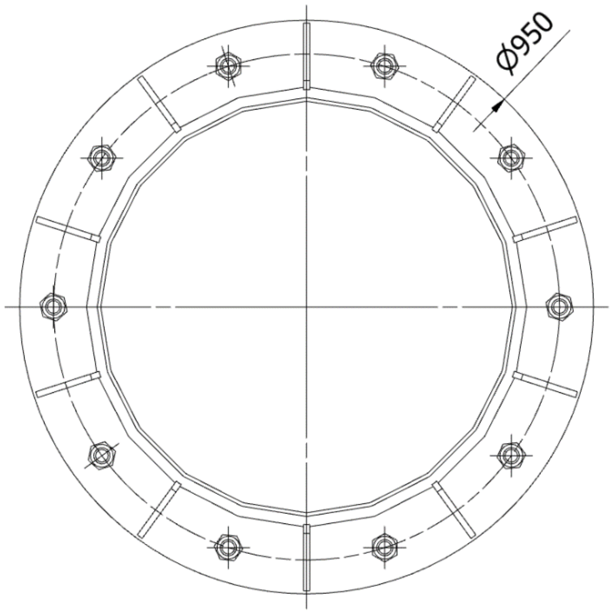

Figure 8 Typical HMLP base plate

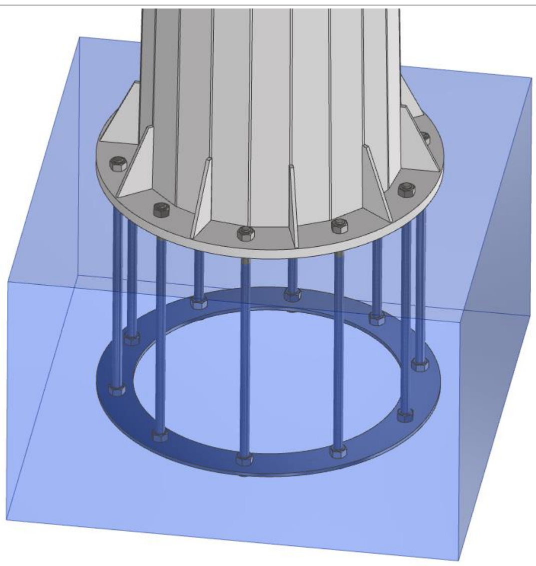

Figure 9 Typical HMLP base showing anchor bolts

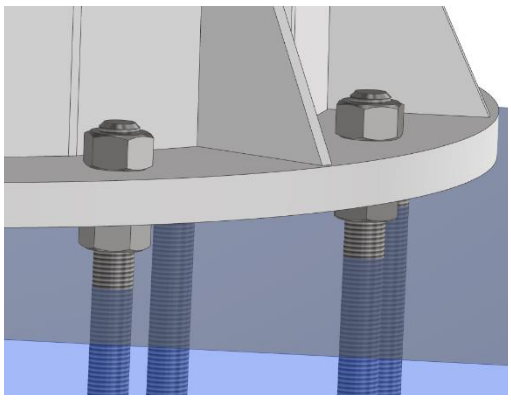

Figure 10 Close-up view
(with approx. 25 mm clearance between concrete and underside of nut).

10 Observation of publicly accessible HMLP installations by the author

10.1 Levelling nuts

When inspection has been possible, all HMLPs have been supported using "levelling nuts" ***. See Figure 7 above. Most levelling nuts appear to be typically located at a distance above the concrete equal to the anchor bolt diameter. Some installations have been observed with little or no gap between the top of concrete and the underside of the levelling nuts.

*** Some installations are impossible to inspect because they are either completely covered with grass, compost, soil, tarmac etc. (See Figure 12, Figure 15, Figure 16). In other cases, the levelling nuts are too close to the top of concrete to allow inspection of the threads below the levelling nut. It would also be impossible to confirm that the anchor bolt has not snapped below the levelling nut. (See Figure 11).

10.2 Anchor bolt corrosion

Some anchor bolts inspected by the author were corroded such that all evidence of thread form was lost. Corroded anchor bolt diameters in some cases as measured by the author were less than that corresponding to the effective stress area. (Measurement using vernier callipers). (See Figure 16, Figure 17, Figure 18).

Note The author was able to observe five HMLPs from the same retail park installation before and after removal following prolonged opposed action by the author ****. Two of the five HMLPs are shown in Figure 15, Figure 16, Figure 17. The anchor bolts of one of the five were buried under tarmac. Three of the five were buried and fully covered in at least 150 mm soil and compost with standing water during wet weather. One of the five HMLPs was installed above ground. (None of the buried installations showed evidence of corrosion following cursory visual inspection. Burial appeared to have been very effective in preventing corrosion.
However, the author would not recommend burial as a method of preventing corrosion. i.e. Soil conditions at other sites could be very different and very much more corrosive).

**** Removal of the HMLPs was because one HMLP had a defect that had existed for the full life of the installation. i.e. Several top nuts were engaged by less than half of their depth. Some nuts were engaged by less than two thread pitches! Understanding or acknowledgement of the author's concern about anchor bolt fatigue was not acknowledged or commented on by the XXX, the local authority or the owner (xxxxx). Further details of this installation are shown in CROSS Safety Report 610 (Ref 9).

NOTE! CROSS is a confidential safety reporting system operated by the Institution of Structural Engineers. It was NOT the author's intent to keep the report confidential. The author had previously limited success after contacting several universities, organisations including the xxx, and other experts, with the objective of obtaining "moral" and technical support. CROSS and Highways England took a positive and constructive interest.

Figure 11 xxxx services xxx westbound located close to the main service area buildings.

Figure 12 xxx Service xxx eastbound

<table>
<tr>
<td>
Figure 13 xxxx
(Installation not unusual)</td>
<td>
Figure 14 xxx
(Installation not unusual)</td>
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<td>
Figure 15 Car park, xxxx
(Installation not unusual)</td>
<td>
Figure 16 Car park, xxxx
(Now removed following prolonged action by author - installation not unusual)</td>
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Figure 17 Car park, xxxx

(HMLP now removed). The installation is not unusual).

Figure 18 Car park, xxxx

(HMLP now removed) The installation is not unusual.
(Same HMLP as Figure 17)

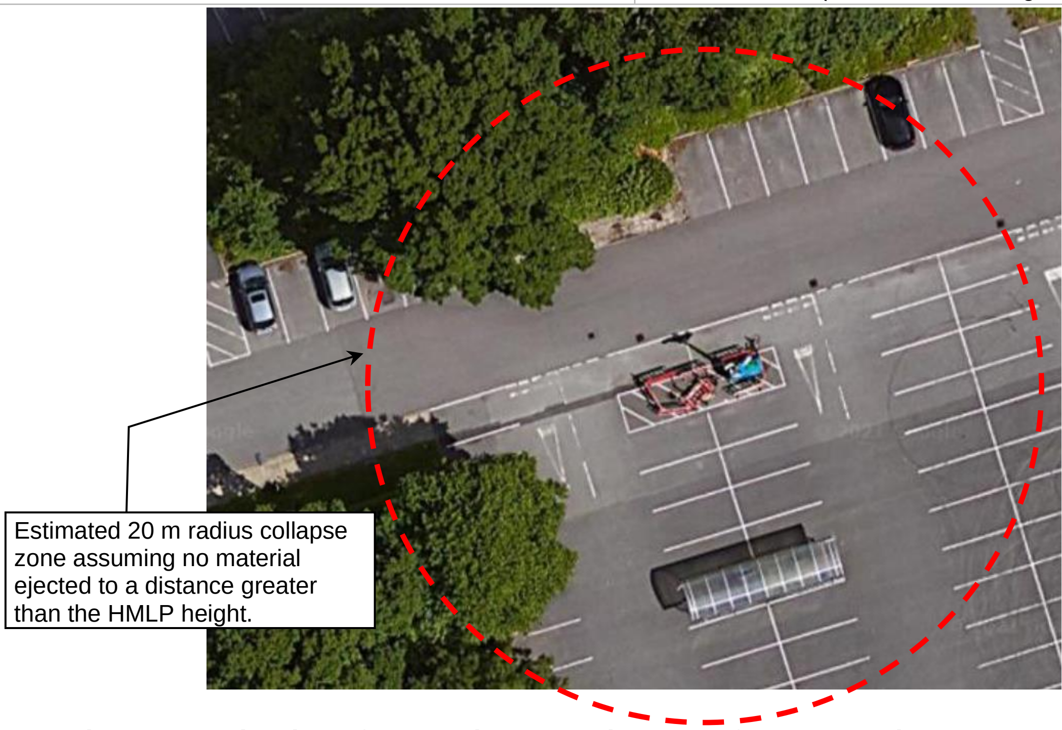


Figure 19 Aerial view of HMLP Figure 17, Figure 18 after removal (Courtesy Google Maps)

The HMLP was located in centre of the hatched area to the left side of the blue temporary lighting generator shown above.

11 Installation

11.1 Assumed installation procedure

The author is not familiar with the precise methods used for installation of HMLPs. However, some assumptions can be made about installation sequence following completion of the foundations. For example.

Step 1 All levelling nuts are placed on the anchor bolts.

Step 2 Alignment of the levelling nuts before installation of the HMLP.

Option 1 (Fully determinate). This could involve 3 levelling nuts located at 120 deg apart with all remaining nuts at lower levels thus not taking part in the HMLP levelling process.

Option 2 (Indeterminate). This could involve levelling all nuts together.

Step 3 Placement of the HMLP

Step 4 Fitting of the top nuts.

Step 5 Adjusting the levelling nuts to plumb the HMLP.

Option 1 Adjusting one of the 3 active levelling nuts, then adjusting the second active levelling nut.

Option 2 Adjusting more than one levelling nut at each adjustment step.

Step 6 Bringing all remaining levelling nuts into contact with the HMLP base plate and progressively tightening nuts not used in the levelling process.

Option 1 would appear to be the simplest and most likely method of installation and plumbing of the HMLP.

12 Nut installation accuracy

12.1 Anchor bolt load distribution

Is the design intent and calculation based on an assumption that in the absence of wind, each anchor bolt carries an equal axial load below the levelling nut?

Is so, to achieve this objective, both levelling and anchor nuts for each anchor bolt need to be adjusted relative to every other anchor bolt.

How accurately do the nuts need to be aligned relative to each other to ensure that each anchor bolt carries the same axial load?

There are no simple methods that can be used to ensure or confirm that each anchor bolt carries an equal compressive load when installed.

Load distribution in each anchor bolt is therefore entirely reliant on the judgement of the installer.

HSE Report RR1081 (Ref 8) includes the following under the heading "Key Messages" ***"….. Slight variations in the use of levelling nuts, underbase grouting, and sequence and level of torquing can intentionally but radically alter the load transfer mechanisms and, in particular, fatigue resistance. …"***

Note HSE Report RR1081 (Ref 8) also compares the method of support using stand-off base plates used for small wind turbines with "***Other structures - Lighting / Gantry bases***…"

12.2 Relationship between anchor bolt load distribution and nut turn

The following is a grossly simplistic calculation is for purpose of comparison and illustration. (See Table 1 below). This calculation assumes that each anchor bolt is fitted with a frictionless sleeve over its full embedded length. This assumption is for purpose of establishing the most optimistic limiting condition. (The author is unaware that anchor bolt sleeves have been used on any HMLP installation).

The purpose of the calculation below is to estimate the relationship between nut-turn and force generated in the anchor bolt below the levelling nut.

For purpose of discussion, let it be assumed that the installer is able to install nuts to an accuracy of 0.25 turns when attempting to create equal force in each anchor bolt below the levelling nut.

Table 1 Change of bolt force caused by nut turn

Description	Data	Units	Comments
Anchor bolt size	M27		
Thread pitch	3	mm	
Elastic modulus	205	GPa	
Effective anchor bolt length	680	mm	Maximum possible length if frictionless in concrete
Anchor bolt tensile diameter	24.17	mm	ISO metric thread form
Anchor bolt tensile cross section	459	mm^2	Under estimate of cross section for axial stiffness calculation
Anchor bolt unthreaded cross section	573	mm^2	Over estimate of cross section for axial stiffness calculation
Anchor bolt axial stiffness - Minimum	138	kN/mm	Using anchor bolt tensile area
Anchor bolt axial stiffness - Maximum	173	kN/mm	Using unthreaded area
Nut turn	0.25		Assumed accuracy achieved by judgement and "feel".
Min anchor bolt force for nut turn	104	kN	Underestimate because axial stiffness used is too low.
Max anchor bolt force for nut turn	129	kN	Overestimate because axial stiffness used is too high.
Mean anchor bolt force for nut turn	117	kN	Based on anchor bolt cross section equal to mid way between tensile and unthreaded cross section.

Discussion of calculation

a) The effective anchor bolt length used to calculate the anchor bolt axial stiffness is likely to be too long. i.e. A fully cast-in threaded anchor bolt without sleeving will not be effectively frictionless unless the bond between concrete is fully broken or non-existent. Reducing the effective anchor bolt length will increase its calculated axial stiffness and thus the calculated anchor bolt force for the nut turn assumption.

The author has not seen any evidence in at least twenty HMLP installations to justify that any anchor bolts are axially loose in their foundations **. All anchor bolts observed have appeared to be fully bonded to the concrete foundation at their top of concrete level. It is difficult to believe that the concrete-to-anchor bolt bond has not at least partially sheared. i.e. FEA calculations with fully bonded anchor bolts would indicate that the concrete bond at the top of the anchor bolt will shear.
However, such dis-bonding has not been observed by visual inspections - at least visually obvious cracking around the anchor bolts has not been observed.
(** When visual inspection has been possible).

Conclusion - The variations of axial load in adjacent anchor bolts created by judgement based installation is likely to be very much higher than implied by the above calculation.

b) The accuracy of nut-turn used in the calculation is no more than a crude estimate based on practical experience. The actual values might be much higher or much lower? In the author's experience, a judgment based accuracy of less than 0.125 nut turns does not seem credible. At the other extreme, an accuracy of 0.5 nut turns would seem easily achievable given the relatively high thread pitch.

Conclusion - The calculated anchor bolt force based on nut turn could be very inaccurate.

13 Wind loads

Wind is the most significant load imposed on HMLPs. The HMLP foundations must be designed to resist maximum wind loads as pseudo-static loads with the appropriate load factor.

Typical unfactored design wind loads are shown in xxxx vendor data [See Ref xx].

Typical unfactored wind loads for HMLPs of similar dimensions used in Figure 7 - Figure 10 above are as follows.

Table 2 Typical vendor wind data

Description	Value	Units	Comments
HMLP model number	xxxx		35 m high
Head area	3.2	m2	
Wind speed	45	m/2	
Horizontal shear	11	kN	
Over turning moment	259	kNm	

The following is an approximate simplistic "ball-park" calculation to compare with **Table 2** above.

Table 3 Approximate calculation to confirm range vendor wind data

Description	Value	Units	Comments
Head area	3.2	m^2	[Ref xx]
cd Head	0.5		Author's estimate
Wind velocity	45	m/s	[Ref xx]. Assumed constant over height for simplicity.
Air density	1.24	kg/m^3	Typical value for air
Wind pressure	1256	Pa	Calculated using data above
Column Height	35	m	[Ref xx]
Wind force on head	2.0	kN	Calculated using data above
Over turning moment - Head	70	kNm	Calculated using data above
Mean diameter of column	400	mm	Author's estimate
Cd column	0.6		Author's estimate
Column area	14	m^2	Calculated using data above
Wind force on column	11	kN	Calculated using data above
Over turning moment - column	185	kNm	Calculated using data above
Total over turning on base	255	kNm	Calculated using data above
Total shear force on base	13	kNm	Calculated using data above

Table 4 Approximate calculation of anchor bolt axial load and axial stress

Description	Value	Units	Comments
Overturning moment at HMLP base due to wind	259	kNm	Typical value. [See **Table 2**] (Note 1).
Number of anchor bolts	10		For example HMLP data. i.e. xxxx [Ref xx]
Pitch Circle Diameter (PCD)	838	mm	For example HMLP data. i.e. xxxx [Ref xx]
Anchor bolt Size	M27		For example HMLP data. i.e. xxxx [Ref xx]
Anchor bolt stress diameter	24.17	mm	ISO thread form
Anchor bolts stress area	459	mm^2	ISO thread form
Maximum anchor bolt force	123628	N	Linear anchor bolt load distribution. (Note 4).
Maximum anchor bolt axial stress	269	MPa	Corresponding to maximum anchor bolt load. (Note 2, 3).

Notes
1. Excludes dynamic load factor (DLF). A DLF will increase anchor bolt forces and stresses because of the rapid application and changes of magnitude and direction of wind forces. e.g. Gusting.
2. Excludes stress concentration factor (SCF) due to threads. An SCF is likely to be at least 2.8. i.e. The maximum axial stress could be 3 times higher than calculated without consideration of an SCF
3. Excludes anchor bolt bending stresses induced by a) Lateral wind loads and b) Deflection of the anchor bolts due to flexing of the HMLP base plate.
4. Calculated assuming a perfectly rigid based plate with a linear anchor bolt load increase from a central neutral axis.

Table 5 Approximate calculation of anchor bolt bending stress

Description	Value	Units	Comments
Lateral force due to wind	11	kN	Typical value. [See **Table 2**] (Note 1).
Number of anchor bolts	10		For example HMLP data. i.e. xxxx [Ref xx]]
Pitch Circle Diameter (PCD)	838	mm	For example HMLP data. i.e. xxxx [Ref xx]
Anchor bolt Size	M27		For example HMLP data. i.e. xxxx [Ref xx]
Anchor bolt stress diameter	24.17	mm	ISO thread form
Anchor bolts section modulus	1386	mm^3	ISO thread form
Bending length	30	mm	
Maximum shear load per anchor bolt	1100	N	Assume uniform load distribution.
Bending moment (assuming encastre restraint)	16.5	Nm	
Bending stress	12		(Note 3)

Notes
1. Excludes dynamic load factor (DLF). A DLF will increase anchor bolt forces and stresses because of the rapid application and changes of magnitude and direction of wind forces. e.g. Gusting.
2. Excludes stress concentration factor (SCF) due to threads. An SCF is likely to be at least 2.8. i.e. The maximum axial stress could be 3 times higher than calculated without consideration of an SCF
3. Excludes anchor bolt bending stresses induced by deflection of the anchor bolts due to flexing of the HMLP base plate.

14 HMLP Fatigue calculations

14.1 Fatigue loading

Wind loads are variable and cyclic. Wind loads are applied at different times and in different directions. Fatigue calculations must be undertaken to ensure that no part of the HMLP is at risk of failure by repeated applications of loads.

The only exception would be if the stress range at every location was always below the endurance limit of the material for every load application.

The following data is required to complete a fatigue calculation.

a) Input data

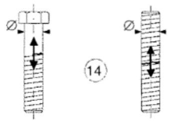
Description	Comments
Stress range	Calculated or measured based on imposed load. Note 1
Number of cycles at each stress range	Calculated or measured based on imposed load. Note 1
Minimum stress	Unknown for unpreloaded parts of the anchor bolts. The minimum stress depends on the combination of imposed load with installed stresses. As discussed above, installed stresses in the anchor bolts are unknown and cannot be measured.
Maximum stress	Unknown for unpreloaded parts of the anchor bolts. The maximum stress depends on combination of imposed load with installed stresses. As discussed above, installed stresses in the anchor bolts are unknown and cannot be measured.
Mean stress	The minimum and maximum stresses are unknown. Hence, the mean stress is unknown.
Thread fatigue stress history - axial	Calculated or measured. Normal design practice is to preload anchor bolt threads to reduce anchor bolt stress range to an absolute minimum.
Thread fatigue stress history - bending	Calculated or measured. Anchor bolt bending is caused by a) Lateral loads applied to the exposed section of anchor bolt below the levelling nut. b) Flexure of the HMLP base plate. Such a calculation would be very complex and would need confirmation by testing i.e. The load distribution is difficult if not impossible to predict. Normal design practice is to ensure that bolt threads are not subject to dynamic bending loads and thus avoid the need to do such a calculation. The author is unaware of any other object or system that relies on the need to know the design life of bolt threads in bending.

b) Allowable design data for design confirmation

Thread fatigue life - axial	Obtained from design codes. For example Ref 4 BS EN 1993-1-9 Eurocode 3: Design of steel structures - Part 1-9: Fatigue.
Thread fatigue life - bending	Not available. i.e. Ref 4 Table 8.1 states "Anchor bolts and rods with rolled or cut threads in tension". The only available design option using Ref 4 would be to consider all stresses as tensile for purpose of fatigue life calculation.

14.2 BS EN 1993-1-9 BS EN 1993-1-9 Eurocode 3: Design of steel structures - Part 1-9: Fatigue. [Ref 4]

a) Extract from BS EN 1993-1-9 Table 8.1

Detail Category	Construction Detail		Description	Requirements
50	size effect for t > 30 mm: $k_s=(30/t)^{0.25}$		14) Anchor bolts and rods with rolled or cut threads in tension. For large diameters (anchor bolts) the size effect has to be taken into account with k_s	14) $\Delta\sigma$ to be calculated using the tensile stress area of the anchor bolt. Bending and tension resulting from prying effects and bending stresses from other sources must be taken into account. For preloaded anchor bolts, the reduction of stress range many be taken into account.

b) Extract from BS EN 1993-1-9 Annex A

Annex A [normative] - Determination of fatigue load parameters and verification formats

A.1 Determination of loading events

(1) Typical loading sequences that represent a credible estimated upper bound of all service load events expected during the fatigue design life should be determined using prior knowledge from similar structures, see Figure A.1 a).

A.2 Stress history at detail

(1) A stress history should be determined from the loading events at the structural detail under consideration taking account of the type and shape of the relevant influence lines to be considered and the effects of dynamic magnification of the structural response, see Figure A.1 b).

(2) Stress histories may also be determined from measurements on similar structures or from dynamic calculations of the structural response.

15 Finite element Analysis

15.1 Introduction

A simple linear static finite element analysis (FEA) has been completed using the unfactored wind load used in section 13 above.

The wind load has been applied as though it was slowly applied from zero to maximum and then slowly removed. i.e. The effect of a DLF and oscillations (swing back) have not been included. The calculations will therefore under estimate the stresses induced in the HMLP structure and anchor bolts.

Note! The following calculation is based on a design maximum wind speed of 45 m/s.

Results for other wind speeds could be estimated in proportion to the wind velocity squared.

Wind velocity	Factor
45	1
40	0.79
35	0.6
30	0.44
25	0.31
20	0.2
15	0.11
10	0.05

15.2 Anchor bolt stress below the levelling nut (Tensile load side of HMLP)

Calculated stresses are shown in Figure 22 below.

Description	Value	Comments
Max tensile stress	~300 MPa	FEA (approximate nodal selection)
Min tensile stress	~ 180 MPa	FEA (approximate nodal selection)
Mean tensile stress	~ (300 - 180)/2 = 240 MPa	Based on 25 mm effective diameter
Bending stress	~ (300 - 180)/2 = 60 MPa	Based on 25 mm effective diameter
Corrected mean tensile stress	~ 260 MPa	Manual calculation 269 MPa **Table 4** (Note 1)
Corrected mean bending stress	~ 67 MPa	Manual calculation 12 MPa **Table 5** (Notes 1, 2)

Notes

1. Calculated stresses above are based on an anchor bolt diameter of 25 mm. A larger diameter than that of the equivalent stress area was used to better represent the stiffness of an M27 anchor bolt. The effective stress diameter of an M27 thread is 24.17 mm. i.e. The calculated axial stress above will be 7% too low. The calculated bending stress above will be 11% too low.
2. The manual calculation is expected to be very inaccurate because of the following:

a) The manual bending stress calculation is based on a beam that has a length nearly equal to its depth. Classical bending theory is a poor representation of this geometry. Accurate results are not expected.

b) The manual calculation is based on an idealised ~ 30 mm long encastre beam to represent the exposed thread length below the levelling nuts. This idealisation is not expected to accurately represent the interaction between the concrete and the anchor bolts. i.e. The anchor bolts are being held in place with concrete that has an elastic modulus equal to 11% of the steel anchor bolts.

c) The manual calculation does not consider the effects of flexing of the base plate. The anchor bolts are connected to the base plate and thus will be deflected by the same amount.

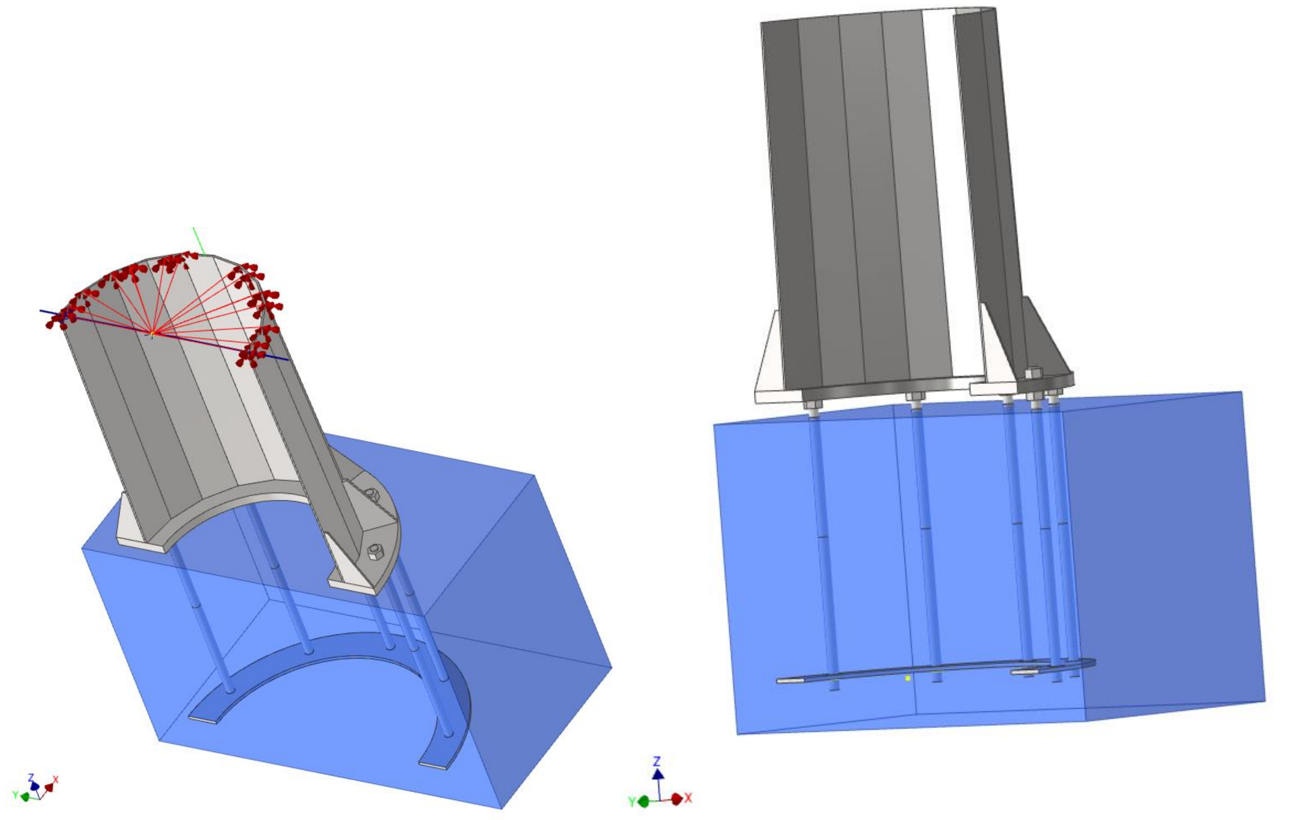


Figure 20 Half symmetry FEA model of example 35 m high HMLP

Lateral force and bending moment at base created by imposing the equivalent force FY = 5.5 kN at a elevation of 23.515 m above the underside of the HMLP base plate. (Equivalent total load for full model of 11 kN, 259 kNm)

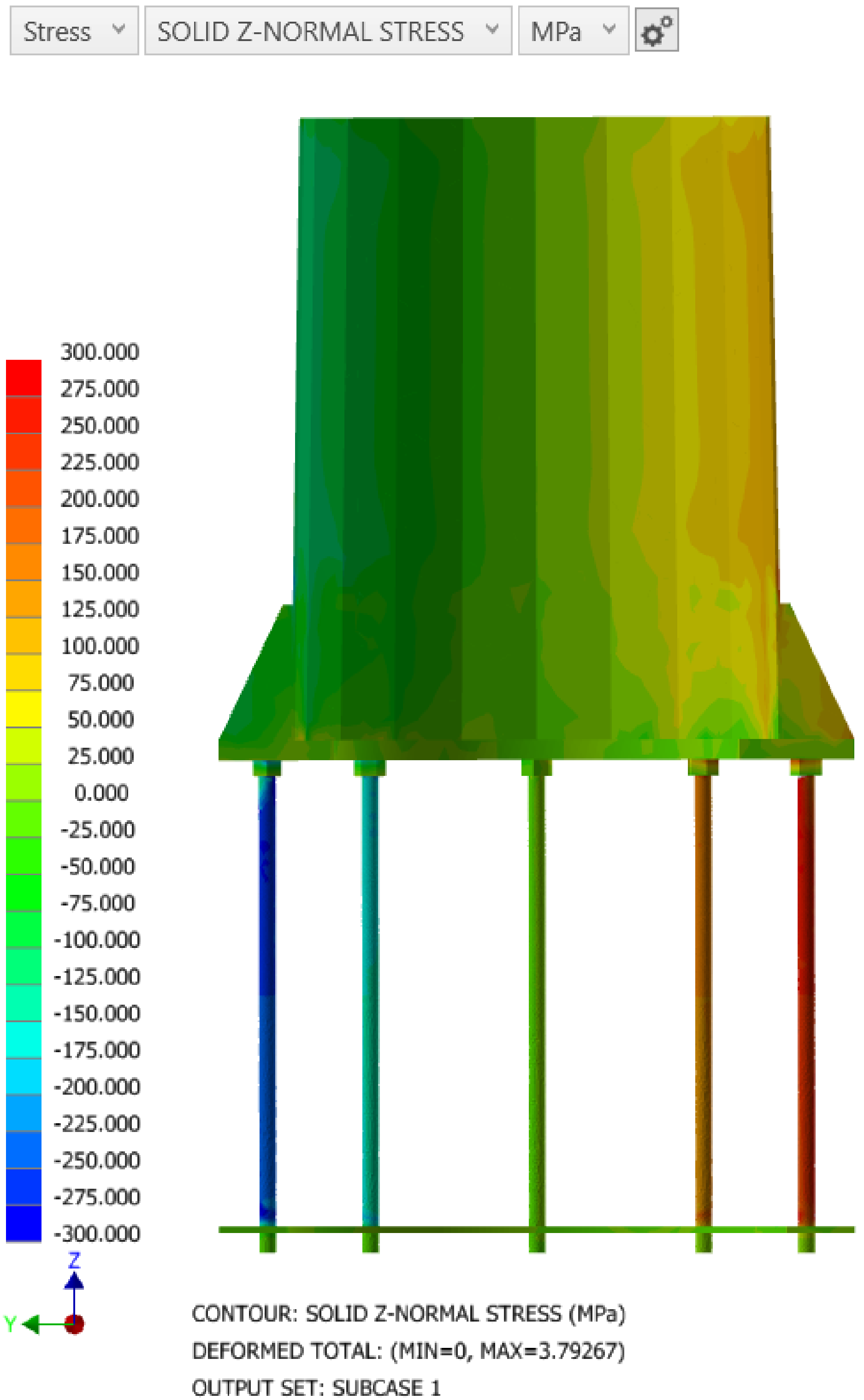


Figure 21 Axial stress (Z) results for load applied right to left

(Visibility of concrete foundation turned off for clarity)

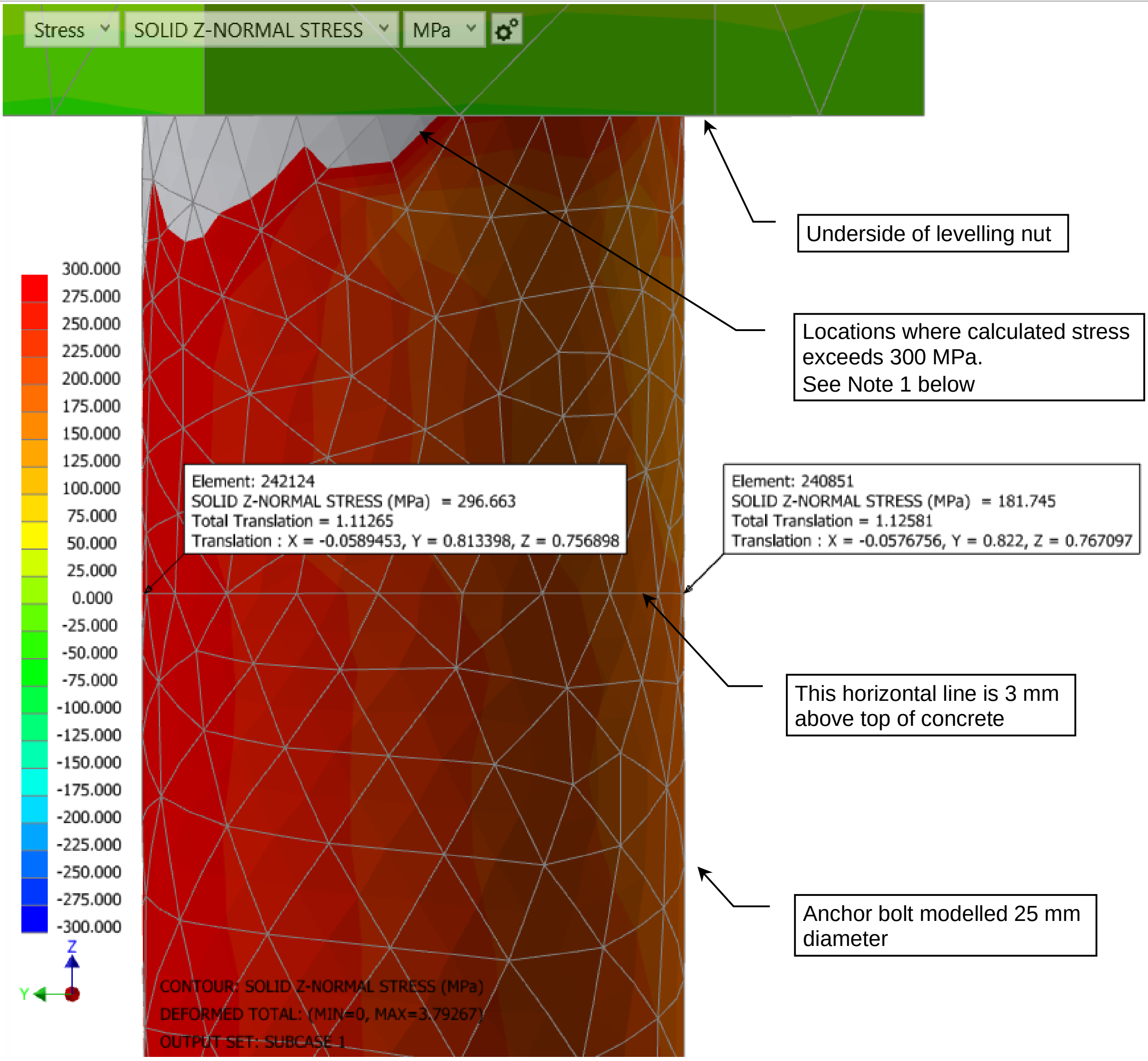


Figure 22 Axial stress at 3 mm above top of concrete (tension side anchor bolt)

The concrete foundation is present in calculation. Visibility of the concrete has been turned off in this view for clarity.

Note 1 Areas of the anchor bolt shown in grey have calculated axial stresses greater than 300 MPa. Values greater than 300 MPa are not expected to be realistic because they are close to a perfectly sharp internal corner thus creating a singularity.

16 HMLP with fully grouted based plate and preloaded anchor bolts

16.1 FEA model

The same FEA model as used for unpreloaded anchor bolts, with same imposed loads was used to calculate comparable stresses and deflections.

The only difference is that the anchor bolts are modelled as preloaded beam elements. Solid elements were used for the unpreloaded FEA model because contact between the anchor bolts and concrete foundation are required to maintain lateral position of the model. In contrast, lateral position of the preloaded model is maintained by friction between the HMLP base plate and the foundation.

In reality, the anchor bolts would maintain lateral position if the preload friction was lost. This model uses a preload of 125 kN per anchor bolt. i.e. 1250 kN of preload for 10 anchor bolts. A nominal coefficient of friction / stiction of, say 0.3 would provide a resistance of 500 kN. The maximum lateral load due to wind for this model is 11 kN. i.e. A safety factor against lateral slippage of about 40.

16.2 Discussion of results

Results with preloaded anchor bolts indicate that this HMLP design is probably not at risk from fatigue failure of the anchor bolts. i.e. In the absence of dynamic load factors, the maximum anchor bolt stress range is less than the endurance limit even at the design wind load of 45 m/s. Dynamic load factors should be confirmed before the above statement can be treated as fact.

Stress levels in the HMLP structure are slightly lower than for the unpreloaded FEA model. Lower stresses in the HMLP structure might be expected because the HMLP base plate is prevented from flexing by full contact with the concrete foundation.

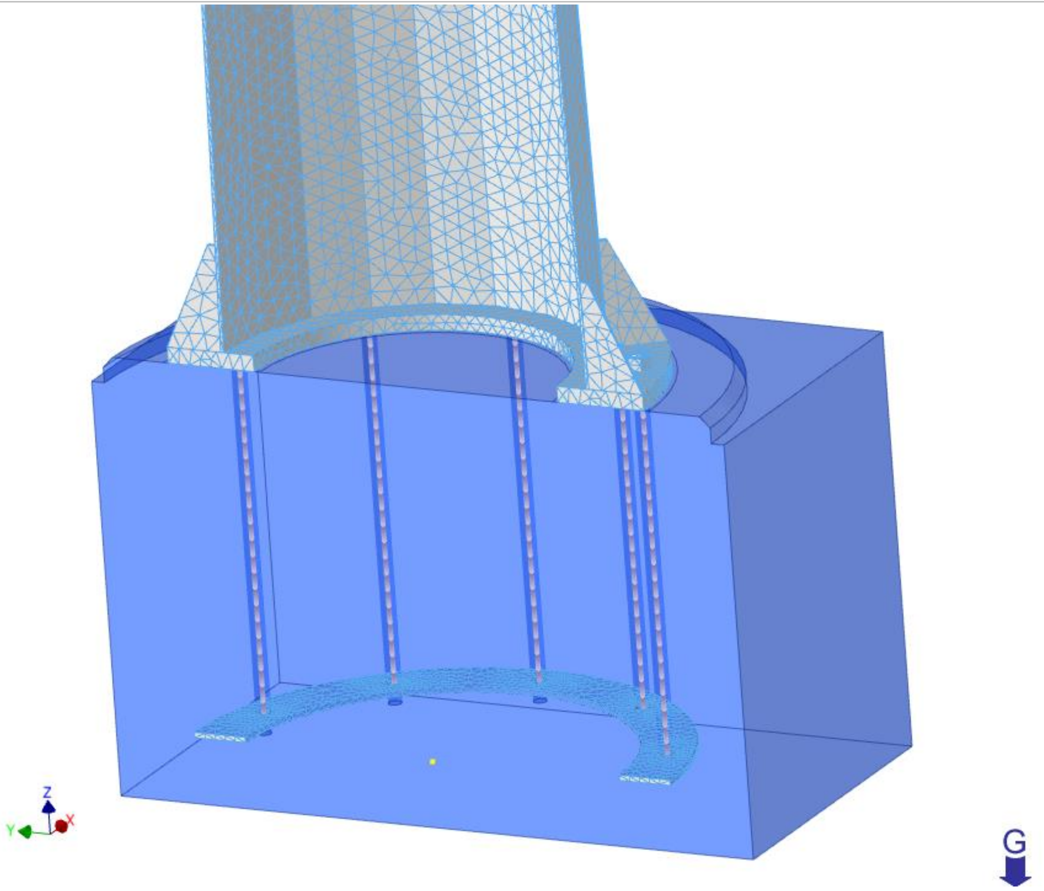


Figure 23 FEA model with grout

Anchor bolts are modelled as preloaded rods of 24.2 mm diameter. i.e. Diameter corresponding to the anchor bolt thread axial stress area. Preload = 125 kN per anchor bolt. Axial preload stress = 272 MPa.

Foundation mesh excluded for clarity. Separation contact with friction modelled between HMLP and foundation.

Half symmetry model with load corresponding to a wind speed of 45 m/s. i.e. Loads equivalent to a total force of 11 kN at an elevation of 23515 mm above bottom of HMLP base plate. (5.5 kN for half symmetry model).

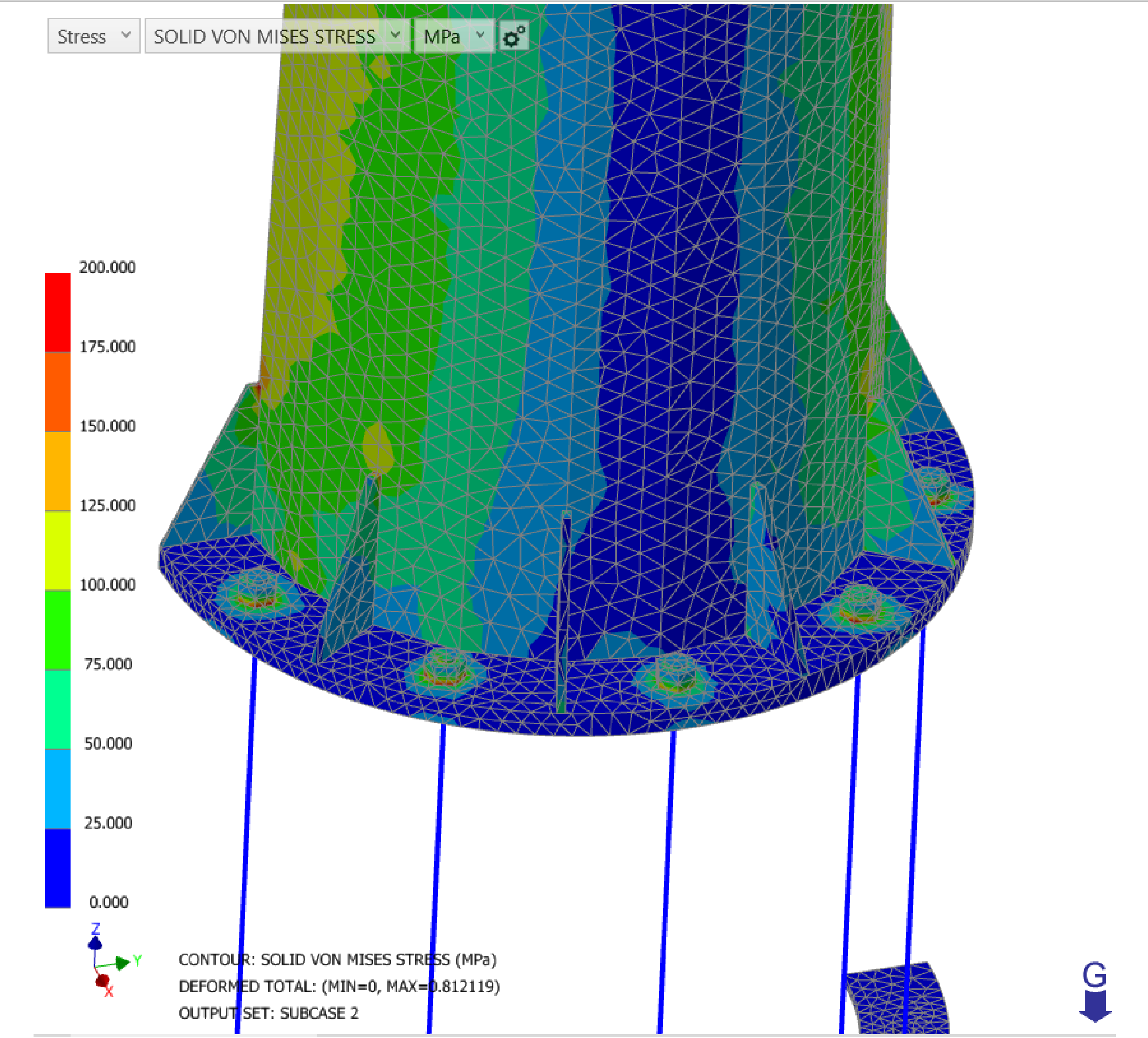


Figure 24 Von Mises stress with preloaded anchor bolts and full wind load

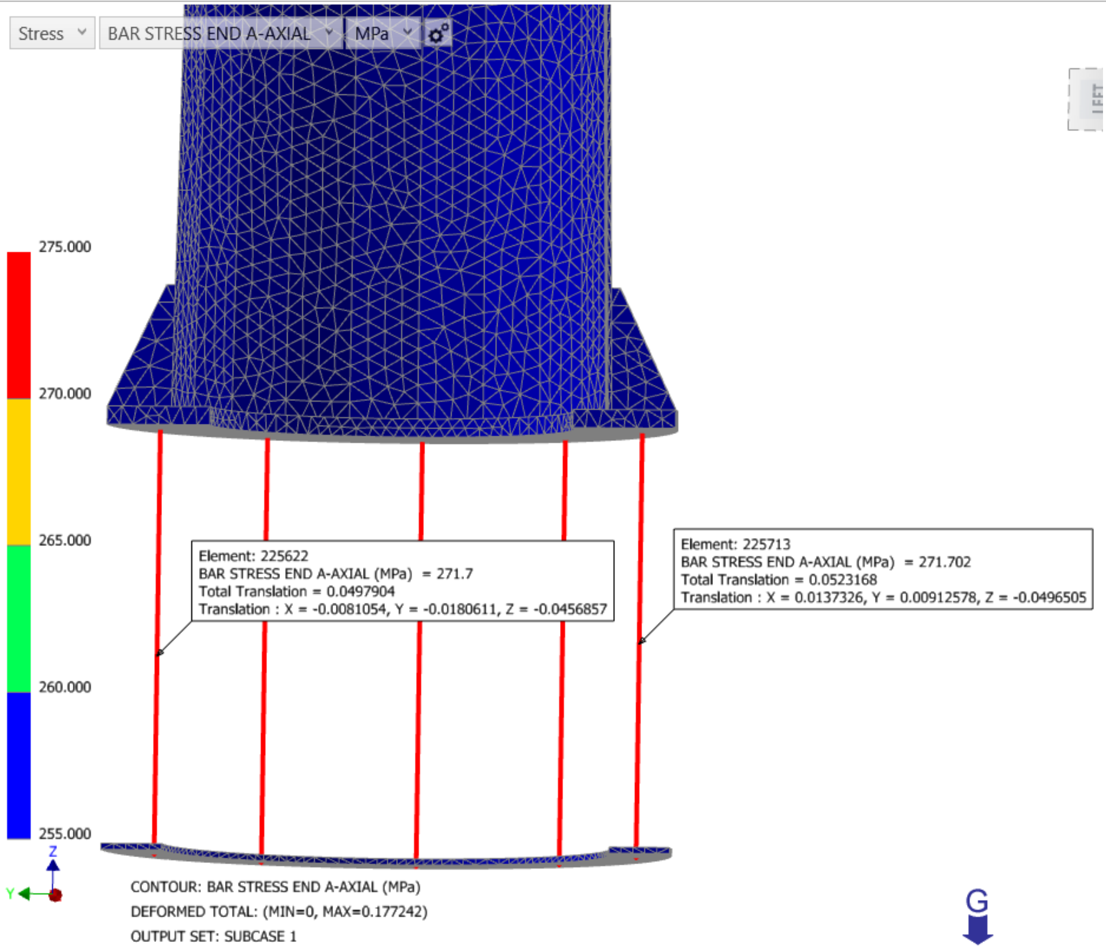


Figure 25 Anchor bolt axial stress results for Subcase 1 - Application of anchor bolt preload and gravity.

i.e. As installed. No wind.

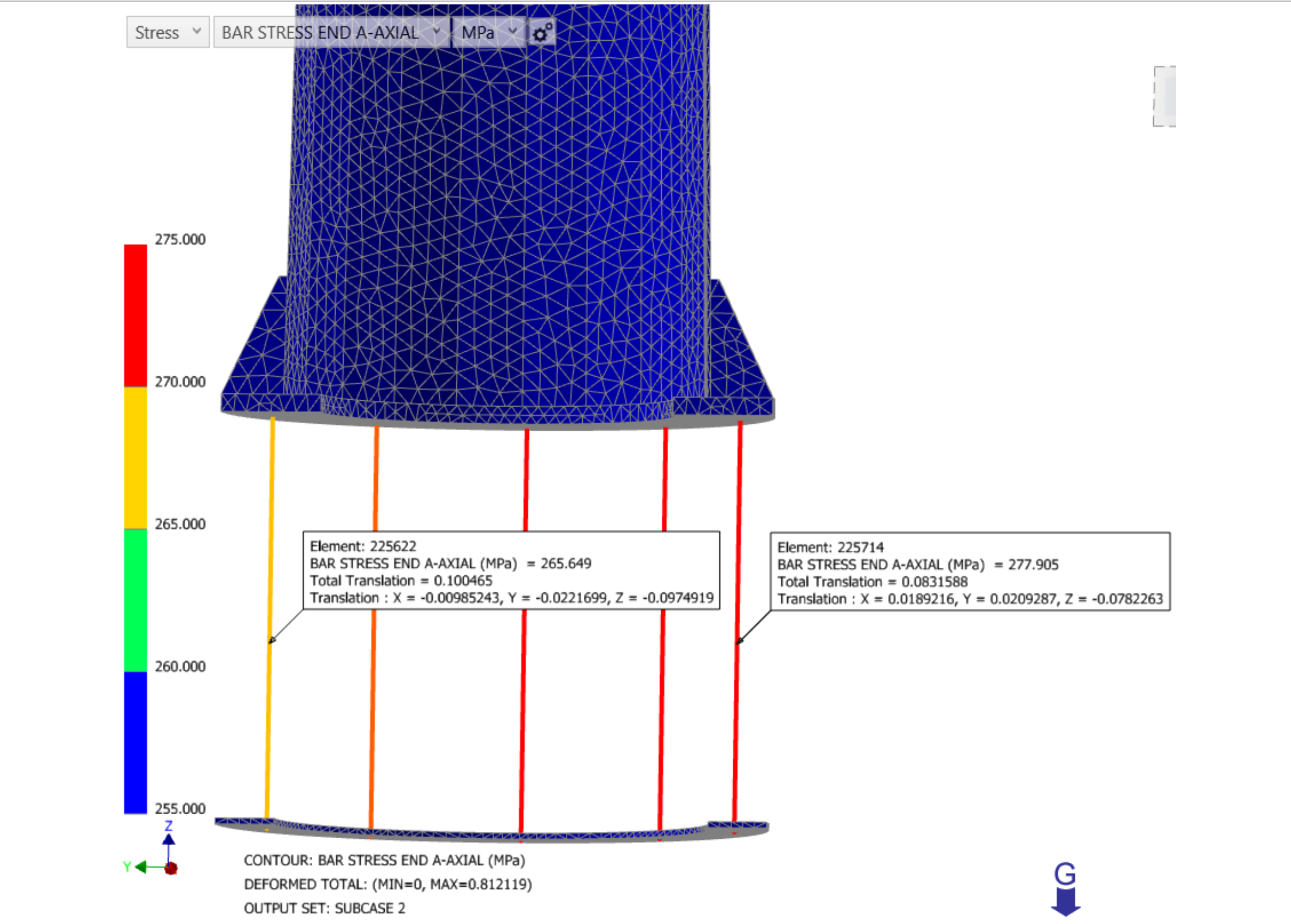


Figure 26 Anchor bolt axial stress results for Subcase 2 - Application wind load.

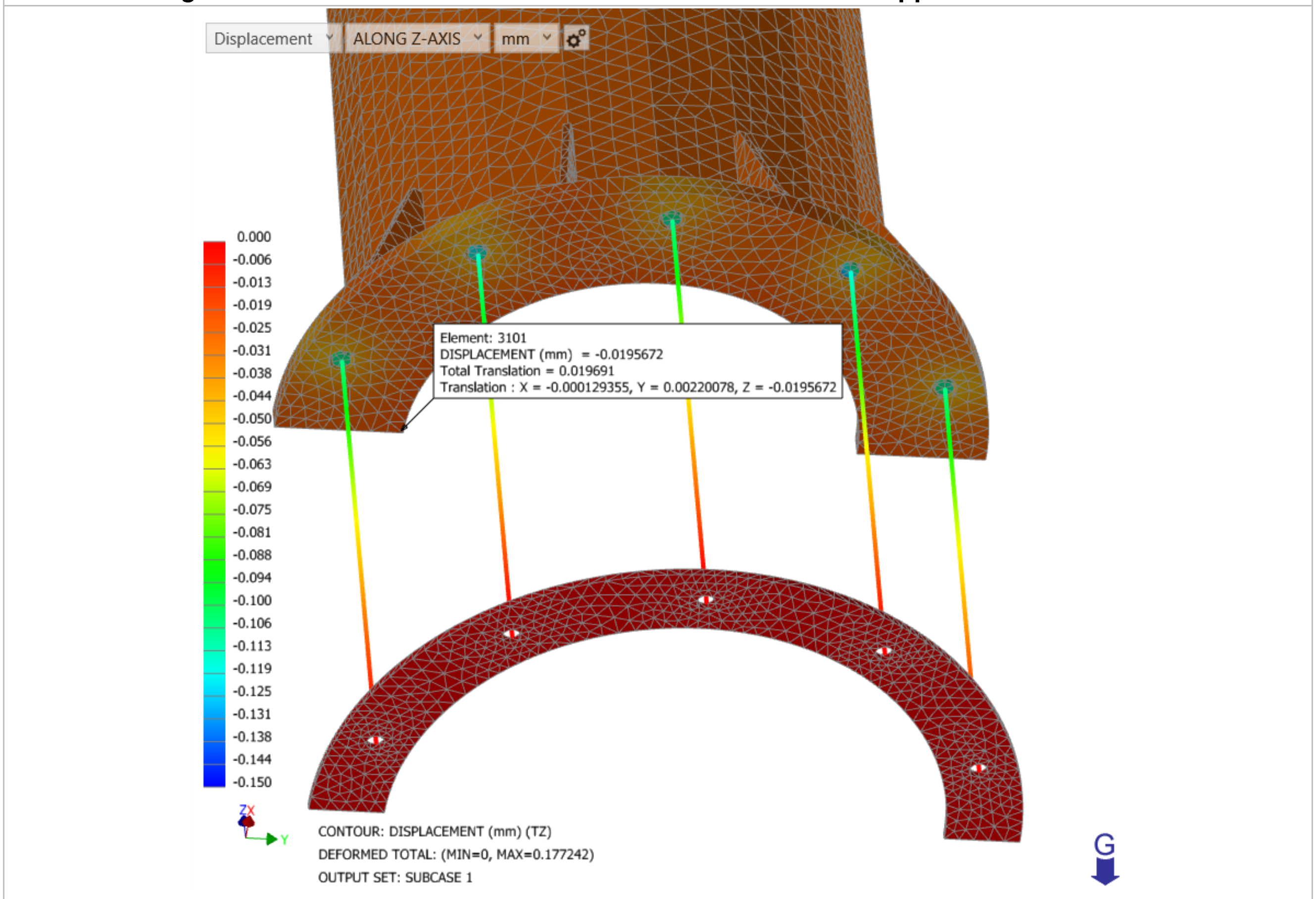


Figure 27 Vertical (Z) displacement caused by application of anchor bolt preload

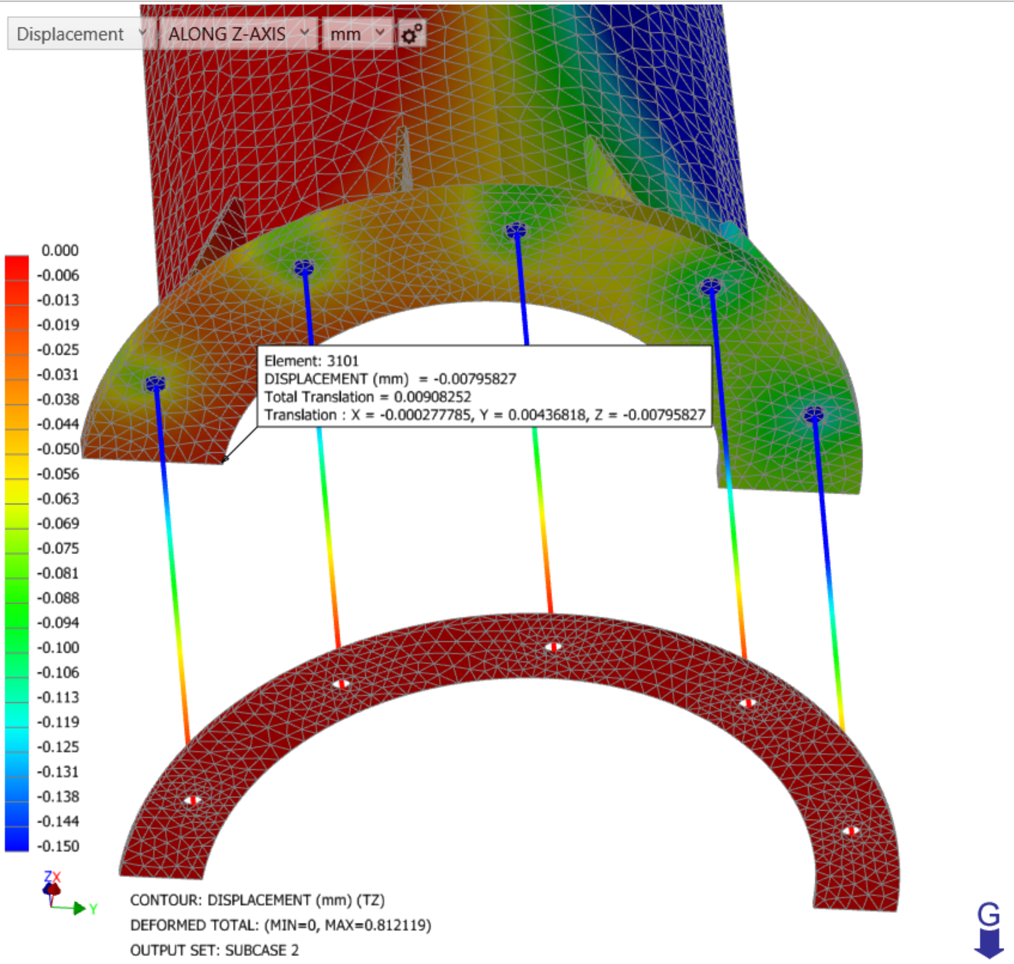


Figure 28 Vertical (Z) displacement with preload and wind wind load

Underside of base plate does not move upwards thus indicating that the HMLP remained in contact with the foundation.

17 Stresses in HMLP base plate and mast

17.1 FEA models

The purpose of this section is to compare two FEA calculations rather than to obtain accurate stress values. i.e. The mesh used for this model is too large for accurate stress calculations. Particularly, the mesh size needed to accurately calculate peak stresses which are required for fatigue analysis.

The mesh size and loading in each model is identical. The only difference is that one model uses a stand-off base plate with unpreloaded anchor bolts. (See Figure 29). The second model uses a fully grouted base plate with preloaded anchor bolts. (See Figure 30).

Note A mast wall thickness of 6 mm has been used in the absence of vendor data. It is possible that the mast wall thickness might be less than 6 mm. e.g. 4 mm or 5 mm.

17.2 Discussion of results

When compared with the fully grouted base plate with preloaded anchor bolts, the stand-off base plate model shows:

a) More areas of high stress than the fully grouted model.

b) Higher stresses in the base plate.

The differences in calculated stress are assumed to be caused by the difference in method of support. i.e. The stand-off base plate is effectively point supported. The only external resistance to base plate deflection is the bending strength of the anchor bolts.

In contrast, a fully grouted base plate with preloaded anchor bolts is continuously supported on the grout. The grout will also resist torsional deflection of the base plate due to eccentric load transfer from the mast wall.

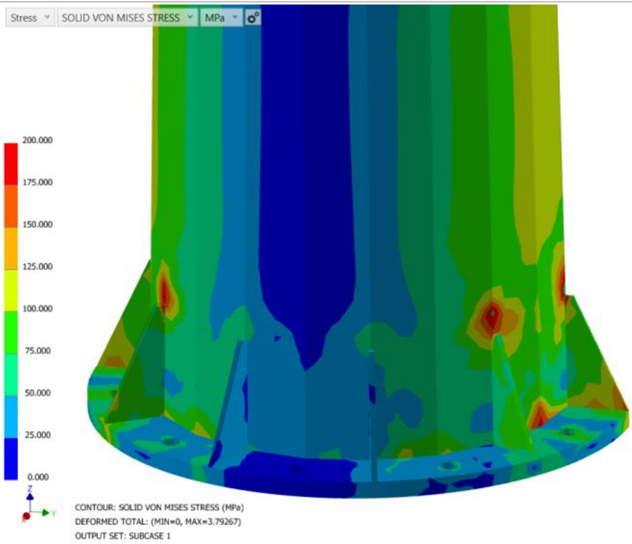


Figure 29 Von Mises stress with stand-off base plate (See Figure 20)

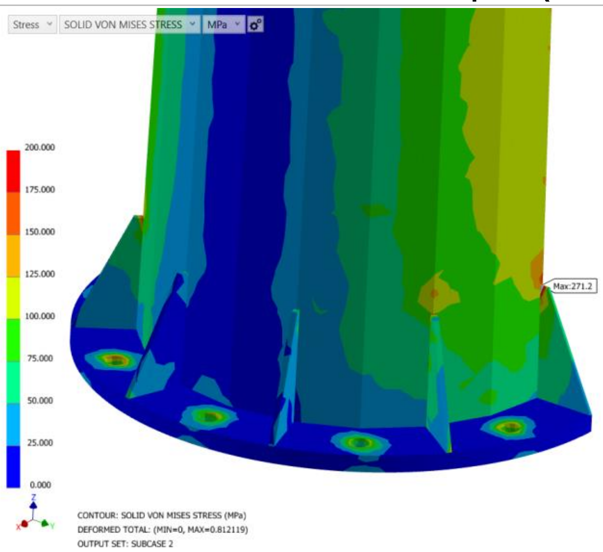


Figure 30 Von Mises stress for grouted base plate and preloaded anchor bolts (See Figure 23)

18 References

Title	Location / File	Comments
1. High Mast Light Pole Foundation Design	https://civilweb-spreadsheets.com/other-design-spreadsheets/high-mast-light-pole-design-spreadsheet/high-mast-light-pole-foundation-design/	
2.	xxxx	
3. xxxx	xxxx	Contains dimensional and load data.
4. BS EN 1993-1-9 Eurocode 3: Design of steel structures - Part 1-9: Fatigue		UK structural steelwork design standard for fatigue assessment.
5. xxx		xxxx
6. xxx		xxxx
7. NCHRP Report 718 Fatigue loading and design methodology for high mast lighting towers. RJ Connor et al	nchrp_rpt_718.pdf	Transportation Research Board Washington DC 2012 (Applicable to US installations only?)
8. HSE report RR1081 Review of small wind turbine construction instructions and specifically for structural supports and foundations	https://www.hse.gov.uk/research/rrpdf/rr1081.pdf	
9. CROSS Safety Report 610 High mast light poles removed from UK site	https://www.cross-safety.org/uk/safety-information/cross-safety-report/high-mast-light-poles-removed-uk-site-610	Original report written by A Weighell (Personal cost and public interest motivation).

Appendix A Anchor bolt axial stress range BS EN 1993 Part 1-9: Fatigue

References

1. Eurocode 3: Design of steel structures - Part 1-9: Fatigue

$\Delta\sigma_C := 50 \cdot MPa$

Reference value of the fatigue strength for anchor bolts in tension at N_C cycles. [1, Table 8-1]

$N_C := 2 \cdot 10^6$

Reference number of cycles for $\Delta\sigma_C$

$N_D := 5 \cdot 10^6$

Reference number of cycles for $\Delta\sigma_D$

$N_L := 10^8$

Reference number of cycles at cut-off limit $\Delta\sigma_L$

$$\Delta\sigma_{R1}(N_R) := \Delta\sigma_C \cdot \left(\frac{N_C}{N_R}\right)^{\frac{1}{3}}$$

Allowable stress range (N_R < N_D) with [1, 7, (3)]

$\Delta\sigma_D := \Delta\sigma_{R1}(N_D)$ $\Delta\sigma_D = 36.8\ MPa$

Fatigue limit for constant amplitude stress ranges at the number of cycles N_D

$$\Delta\sigma_{R2}(N_R) := \Delta\sigma_D \cdot \left(\frac{N_D}{N_R}\right)^{\frac{1}{5}}$$

Allowable stress range ($N_D < N_R$ < N_L) [1, 7, (3)]

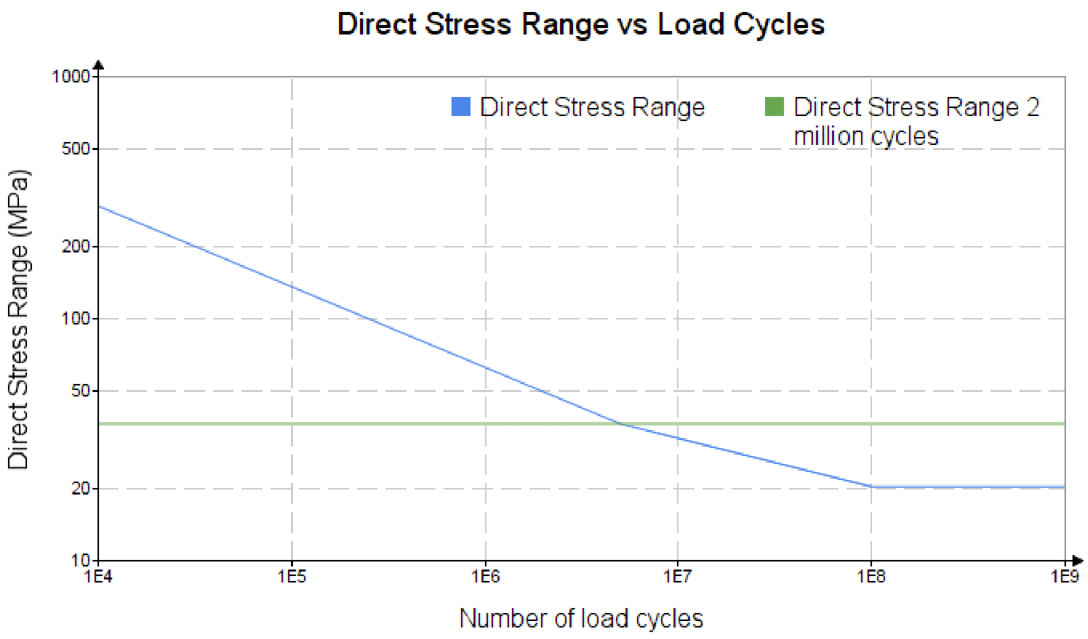
$\Delta\sigma_L := \Delta\sigma_{R2}(N_L)$ $\Delta\sigma_L = 20.2\ MPa$

$$\Delta\sigma_R(N_R) := \begin{array}{l} \text{if } N_R \le N_D \\ \quad \Delta\sigma_C \cdot \left(\frac{N_C}{N_R}\right)^{\frac{1}{3}} \\ \text{else if } N_D < N_R \le N_L \\ \quad \Delta\sigma_D \cdot \left(\frac{N_D}{N_R}\right)^{\frac{1}{5}} \\ \text{else} \\ \quad \Delta\sigma_L \end{array}$$

Direct stress range for nominal stress spectra with stress ranges above and below the constant amplitude fatigue limit $\Delta\sigma_D$

$$N\_typ := \begin{bmatrix} 1 \cdot 10^4 \\ 1 \cdot 10^5 \\ 5 \cdot 10^5 \\ 1 \cdot 10^6 \\ 2 \cdot 10^6 \\ 5 \cdot 10^6 \\ 1 \cdot 10^8 \end{bmatrix} \qquad \Delta\sigma_R\left(N\_typ_n\right) = \begin{bmatrix} 292 \\ 136 \\ 79 \\ 63 \\ 50 \\ 37 \\ 20 \end{bmatrix} MPa$$

Values of Direct Stress Range at various cycle counts


Direct Stress Range vs Load Cycles
Direct Stress Range
Direct Stress Range 2 million cycles
Direct Stress Range (MPa)
1000
500
200
100
50
20
10
1E4
1E5
1E6
1E7
1E8
1E9
Number of load cycles

Appendix B Wind turbine failure

B.1 Introduction

Several small wind turbines (~50 kW) supported using the same design as that used for HMLPs (unpreloaded anchor bolts with levelling nuts) have collapsed.
e.g. Bradworthy (Ref xx). (Reported by the BBC on 29 Jan 2013).

Appendix C Proposed solution

C.1 Fatigue design

It is impossible to design threaded anchor bolts to resist fatigue failure caused by dynamic and repetitive loads unless either

a) The threaded section is preloaded or

b) The stress range is below the anchor bolt material endurance limit. i.e. ~20 MPa. (See Appendix A above).

Option b) is impractical because the permitted allowable stress range would be negligible thus requiring very large anchor bolts. (The use of large anchor bolts (> 50 mm) and thick base plates (50 mm - 75 mm) to minimise stress range appears to have been a design approach used in the USA for some installation).

A practical and commercially viable solution is required to create a low and predictable risk to public safety.

One solution would be to convert existing stand-off base plates to structurally grouted base plates with preloaded anchor bolts.

The existing anchor bolts and HMLP base plates in most if not all cases should be large suitable for such conversion without modification. i.e. Modification or replacement would not be required to HMLP, the anchor bolts or the foundation.

C.2 Proposed corrective design solution

1. Fit segmented a support ring to the circumference of HMLP base plate. (See Figure 31 below). Ensure the underside of the support ring is coated with release agent to prevent bonding when the grout is cast.
2. Use the jacking anchor bolts in the support ring to carry the weight of the HMLP while maintaining alignment and plumbness of the HMLP. Sacrificial load spreader feet will be fitted to each jacking anchor bolt. i.e. The jacking anchor bolt feet will be left in place when the jacking anchor bolts are withdrawn following grouting).
3. Lower the levelling nuts to create a gap between the top of the levelling nut and the underside of the HMLP base plate.
4. Insert a sponge "washer" above the levelling nut. The sponge would create a compressible gap in the grout to prevent the levelling nut resisting anchor bolt tension during later preloading. Coat the levelling nuts with a release agent or plastic tape to prevent bonding with the grout.
5. Fit plastic drain tubes to allow drainage of any water that might collects inside the HMLP.
6. Fill the gap between foundation and underside of HMLP base plate with load bearing non-shrink grout of suitable compressive strength. Monitor grout from the inside of the HMLP to ensure that no air bubbles are left under the HMLP base plate.
7. Leave the temporary support collar in place until the grout has set and achieved its design compressive strength.

8 Remove the support ring.

9 Add "Bellville" spring washers to each anchor bolt in turn. Clean bolt threads and add anti-corrosive lubrication.

Note The existing anchor bolts are not sleeved. It is likely that anchor bolts might still be bonded to the concrete. The concrete bond might break in the future and thus release the preload. Bellville washers will reduce the loss of preload if the concrete-to-anchor bolt bond breaks during operation. Bellville washer might also provide some visual indication that preload has been lost.

10 Preload the anchor bolts by tightening the nuts to the specified torque.

C.3 Features of support ring (See C.1)

1 High torsional rigidity. The support ring will be formed as a hollow section to ensure maximum torsional rigidity to resist eccentric loading.

2 Rotation resistant "Spigot and Sockets" on each end of the support ring segments to maintain alignment and torsional continuity.

3 2, 3 or 4 support ring segments to allow fitting and removal. (3 segments will probably the the cheapest and most practicable option).

4 Narrow lip at bottom of ring to engage with HMLP base plate. The lip should be as narrow as practicable to maximise the volume of grout. (See C.2).

5 Support ring design strength and jacking anchor bolt spacing to allow operation with a single jacking anchor bolt removed to gain access to loosen tight levelling nuts.

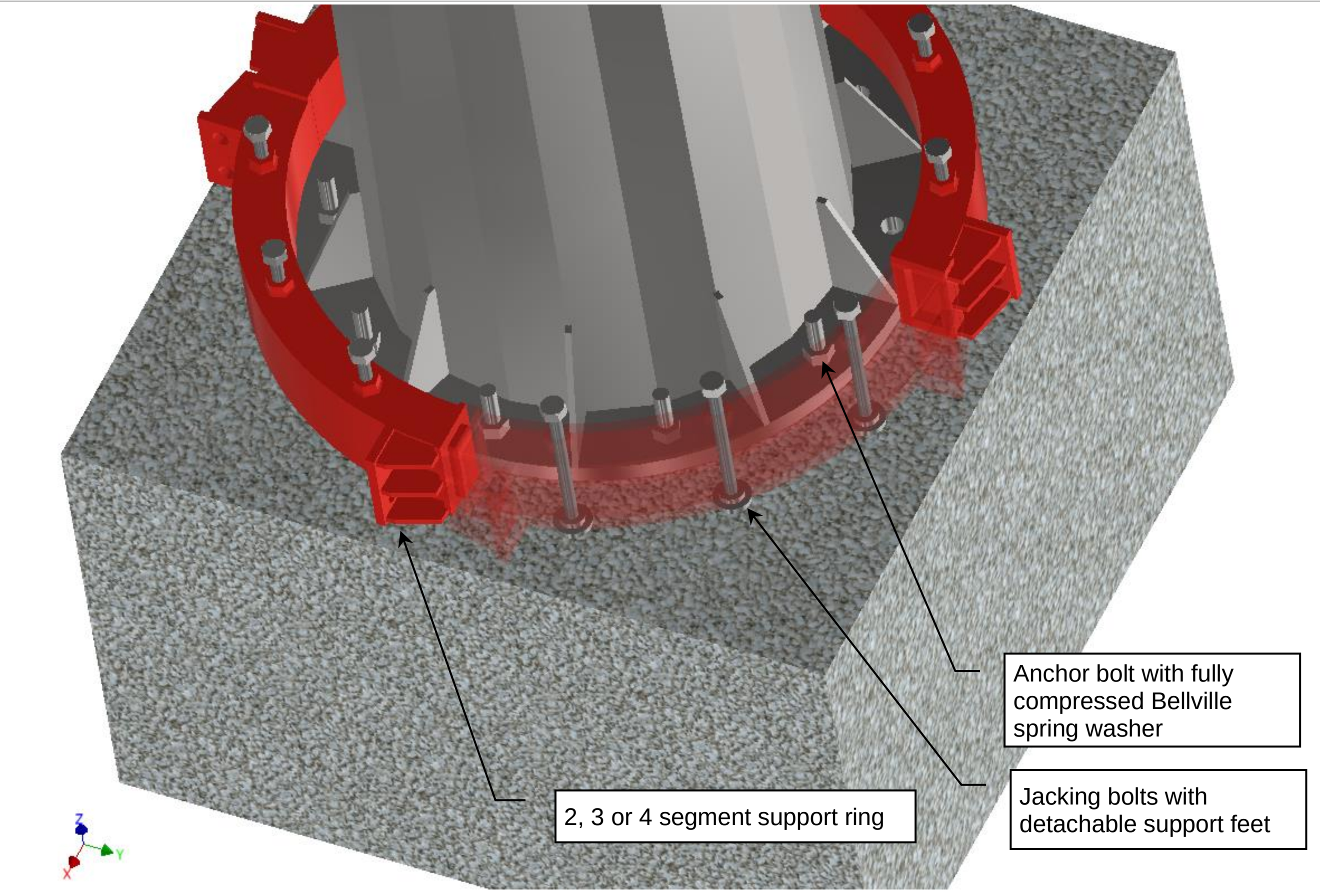


Figure 31 HMLP base plate supported on a torsionally stiff multi-segment support ring (Support ring design copyright A Weighell)

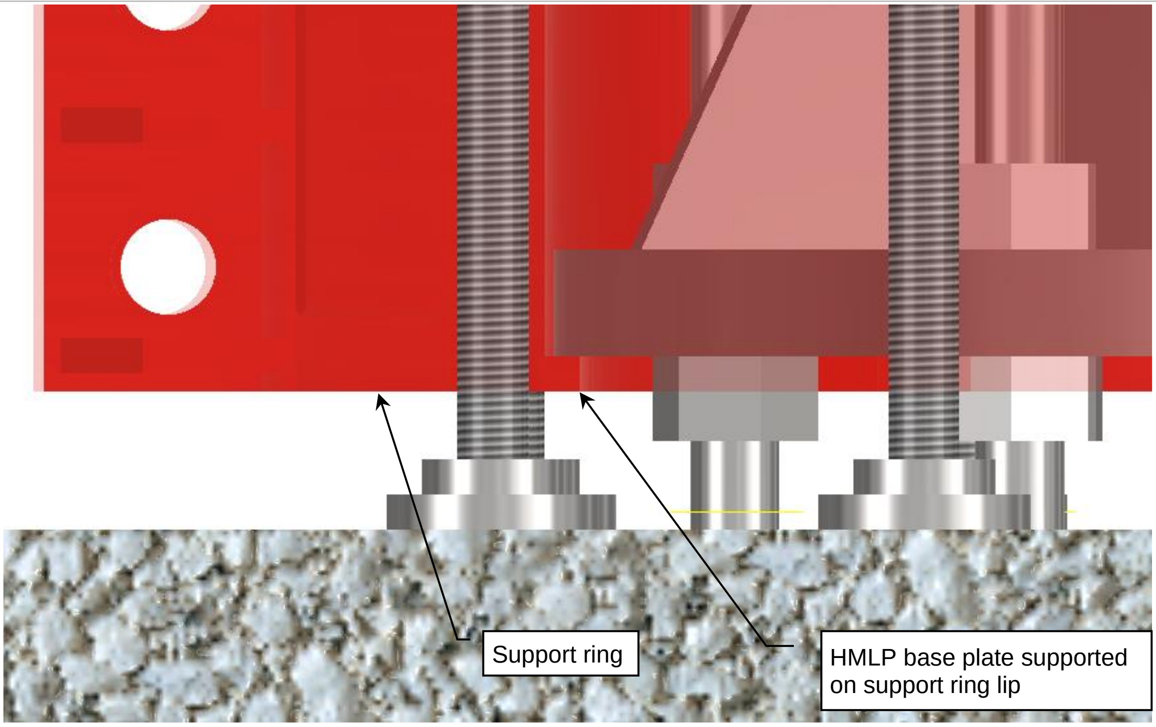


Figure 32 Close up view of HMLP flange supported on lip projecting from bottom of support ring (Design copyright A Weighell)

Appendix D An automotive equivalent of unpreloaded anchor bolts?

During the late 1960s and early 1970s, it was common practice among some teenagers in the UK to "widen" the wheels of their "Mini" cars. i.e. Increasing the track width in the hope of cornering at higher speeds. (This practice became less common by the mid-1970s because insurers enforced rules on what was classed as an unauthorised non-factory modification).

Which method would require the same fatigue calculation methods and unjustifiable design assumptions as HMLPs with unpreloaded anchor bolts?

Which method would YOU trust with you life?

(The author is unaware of wheel widening that did not use spacers and wheel hub stud bolts that were not preloaded over their full length).

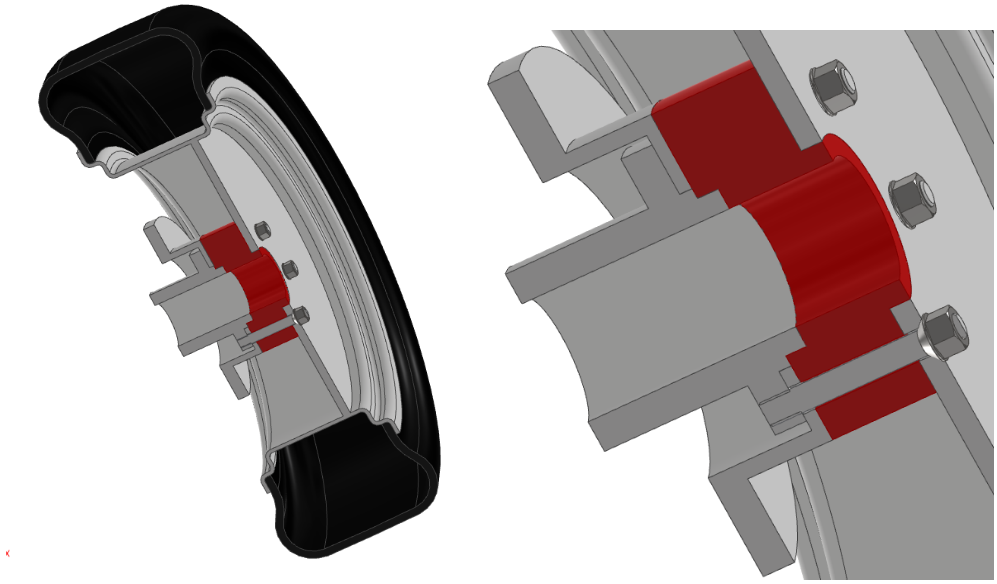


Figure 33 Widened car wheel using a spacer

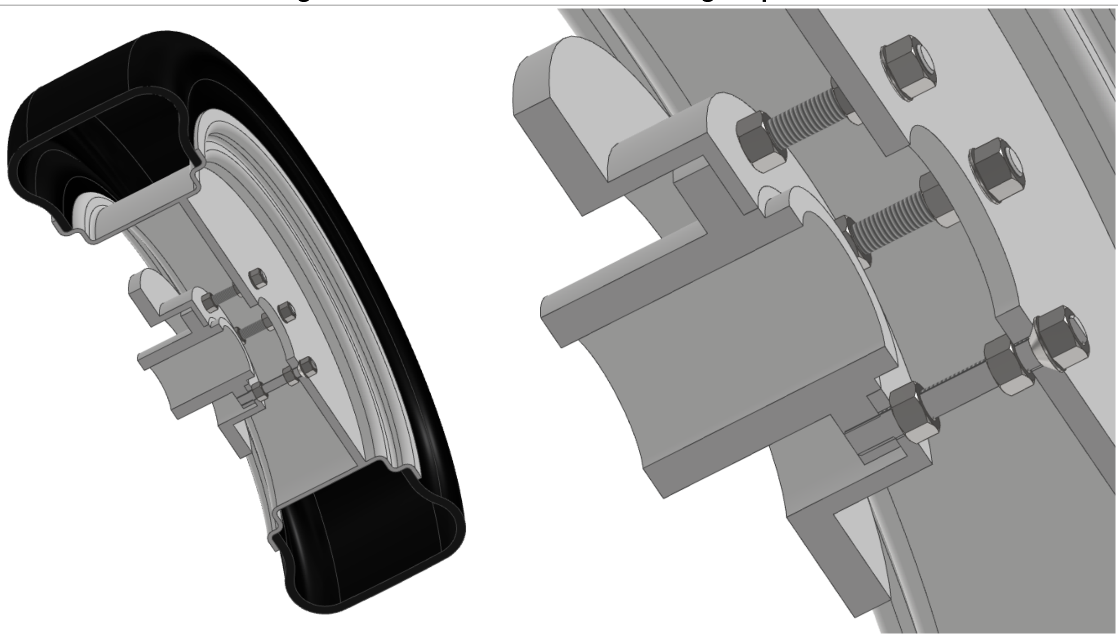


Figure 34 Widened car wheel using nuts rather than spacers?

Appendix E Anchor bolt stress range with structural grout and anchor bolt preload

Title Stress range calculation for preloaded High Mast Light Pole anchor bolt with structurally grouted base plate.

By A Weighell Date 17 April 2021

Objective To estimate bolt loads and bolt stresses in a typical High Mast Light Pole (HMLP) base plate flange due to wind loads - if structurally grouted and using preloaded anchor bolts.

Introduction The general principles and discussion in this calculation apply to all HMLPs from all manufacturers. This is a hypothetical calculation assuming that the existing stand-off base plates were installed with full structural grouting and preloaded anchor bolts.

The purpose of this calculation is to compare anchor bolt stress ranges for
a) Structurally grouted base plates and anchor bolt preload with
b) Stand-off based plates and un-preloaded anchor bolts normally used for HMLPs.

The calculation has been completed because concerns by the author with regard to the fatigue life of the un-preloaded holding down bolts. The un-preloaded anchor bolts are subject to repeated bending and axial loads caused by wind loading.

It is normal structural and mechanical engineering design practice to use preloaded bolts to mitigate against bolt fatigue failure when the joint is subject to repeated loading.

The purpose of bolt preloading is to minimise the load variation in the bolt threads that would risk fatigue failure. i.e. Bolt preload would normally be high enough to ensure that bolt preload prevented the repeated bolt stress variations exceeding the endurance limit of the bolt. i.e. The bolt and bolted joint would be designed to have an infinite life despite the joint being subject to high numbers of load cycles.

Without preload, the life of a bolt subject to variable loads is relatively short. For example, properly preloaded (torqued) vehicle wheel stud would be safe to operate for many hundreds of thousand of miles. A wheel nut that is not adequately preloaded (torqued) might fail after few tens of miles.

Smaller bolts subject to repeated loads would be preloaded by tightening to a minimum torque. e.g. Vehicle wheel nuts. Larger structures would be preloaded either hydraulically. e.g. Large wind turbines. Specially designed nuts could also be used to apply preload for large bolts if hydraulic tensioning was not practicable. e.g. Nord-lock "Superbolt".

HMLPs are one of the few common structures subject to repetitive loading that do not use preloaded anchor bolts. Small wind turbines (up to 50 kW) have previously used to have used un-preloaded holding down bolts. Several small wind turbines collapsed when their un-preloaded anchor failed by fatigue.

The dimensional details used for this calculation are based on [redacted] was selected because it appears one of the most similar to HMLP installations that the author has observed in the UK. The use of this model number does not imply any specific criticism of either the manufacturer or the model number selected. The discussion and intent of this calculation applies equally to every manufacturer and model of high mast light pole using un-preloaded anchor bolts in conjunction with stand-off baseplates.

Details not publicly available in the manufactures data have either been estimated or measured at publicly accessible UK installations.

The motivation for this calculation is both a personal technical interest and a concern for public safety. i.e. Most HMLPs are located in areas of relative high vehicle and population density. e.g.

Alongside motorways, car parks, motorway service areas. Collapse of an 20-35 m tall HMLP would create a high risk of death and injury at most installations.

References

1. Eurocode 3: Design of steel structures - Part 1-9: Fatigue
2. [redacted]
3. https://civiltoday.com/civil-engineering-materials/concrete/84-modulus-of-elasticity-of-concrete

Assumptions

This calculation is intended to be for illustration of typical values of data.
The data used is intended to be representative and realistic but is not verified.
This calculation is intended to be simplistic and conservative to assist explanation.

Foundation concrete strength	C30
Anchor bolt material	Grade 8.8

$fc := 30 \cdot MPa$ — Strength of concrete 28 day C30

$Ec := 4700 \cdot \sqrt{\frac{fc}{MPa}} \cdot MPa$ $\quad Ec = 25743\ MPa$ — Elastic modulus of normal weight concrete (approx) [Ref 3]

$dc\_max := 1100 \cdot mm$ — Approximate mean outside diameter of concrete resisting compression

$dc\_min := 500 \cdot mm$ — Approximate mean inside diameter of concrete resisting compression

$Nb := 10$ — Number of bolts

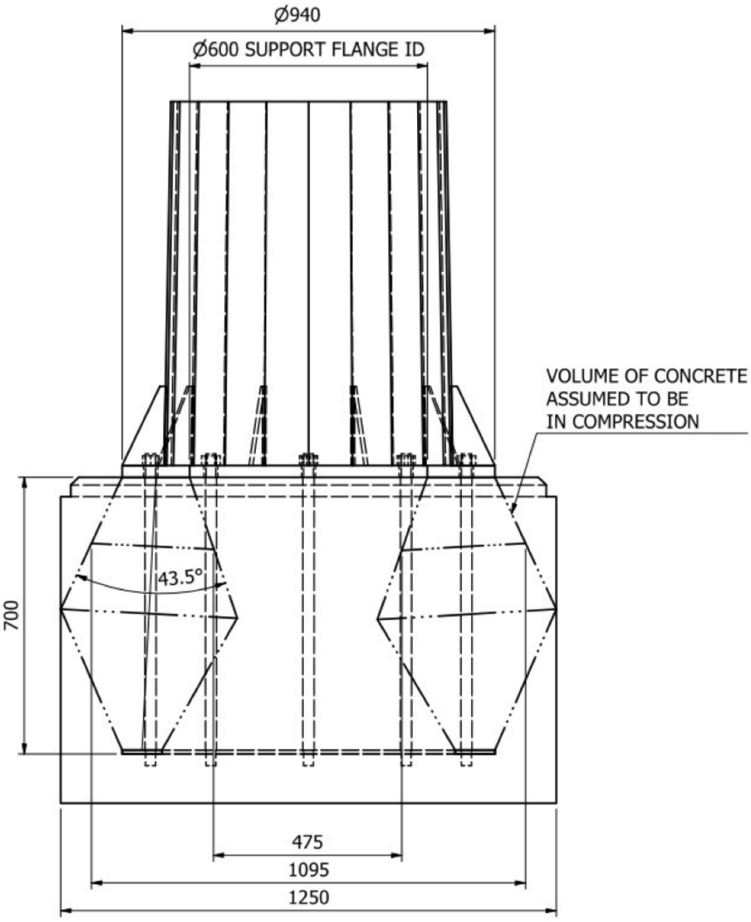
$$Acb := \frac{\left(\frac{\pi}{4} \cdot \left(dc\_max^2 - dc\_min^2\right)\right)}{Nb} \qquad Acb = 754\ cm^2$$

Approximate mean cross sectional area of concrete per anchor bolt resisting compression

$Lc := 690 \cdot mm$ — Depth of concrete subject to compression

$$kcb := \frac{Acb \cdot Ec}{Lc} \qquad kcb = 2.8\ \frac{MN}{mm}$$

Approximate stiffness of concrete per anchor bolt



$Es := 205 \cdot GPa$ — Elastic modulus of steel anchor bolts

$db := 24.8 \cdot mm$ — Approximate effective diameter of M27 anchor bolt assumed for purpose of calculating axial stiffness

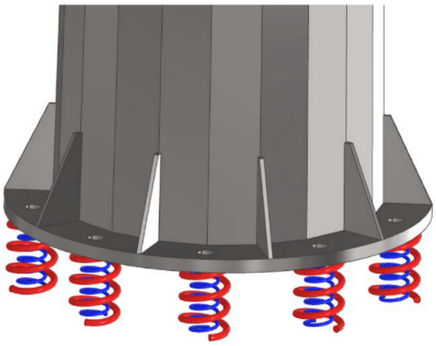
$Lb := 700 \cdot mm$ — Length of anchor bolt in tension for purpose of calculating axial stiffness

$Ab := \frac{\pi}{4} \cdot db^2$ $\quad Ab = 483\ mm^2$ — Cross section of bolt for purpose of anchor bolt axial stiffness calculation

$kb := \frac{Ab \cdot Es}{Lb}$ $\quad kb = 141\ \frac{kN}{mm}$ — Axial stiffness of anchor bolt

$\frac{kcb}{kb} = 19.88$ — Ratio of concrete stiffness to anchor bolt stiffness

This calculation conservatively assumes that the high mast light pole baseplate flange is supported on "springs". Each "spring" corresponds to one anchor bolt location. Until the imposed anchor bolt load exceeds the anchor bolt preload load, the stiffness of each "spring" is equal to the combined stiffness of the anchor bolt and associated concrete. Each "spring" is centred on each holding anchor location.



Conceptual support of high mast light pole for purpose of calculation
The outer (red) "spring" represents the vertical stiffness of the concrete at each anchor bolt location.
The inner (blue) "spring" represents the axial stiffness of each anchor bolt.
Note: This is a conservative simplification for purpose of explanation. This simplification is also for direct comparison with anchor bolts using stand-off base plates.

$k := kcb + kb$	$k = 2.95\ \frac{MN}{mm}$	Combined stiffness of each "spring"
$M\_total := 259 \cdot kNm$		Over turning moment at 45 m/s wind velocity
$pcd := 838 \cdot mm$		Pitch circle diameter of circular anchor bolt group
$F\_max := \frac{4 \cdot M\_total}{pcd \cdot Nb}$	$F\_max = 124\ kN$	Maximum force in support "spring" furthest from axis of bending. Assumes a) Linear force distribution / rigid flange. b) Uniformly spaced support points. c) Equally stiff support points. d) Circular anchor bolt pattern
$\Delta c := \frac{F\_max}{k}$	$\Delta c = 0.042\ mm$	Deflection of support "spring" furthest from axis of bending
$\Delta b := \Delta c$	$\Delta b = 0.042\ mm$	Deflection of anchor bolt is equal to the deflection of concrete because both are connected together at their end points.
$\Delta Fb := kb \cdot \Delta b$	$\Delta Fb = 5.92\ kN$	Maximum anchor bolt load change caused induced by bending load M_total
$Fb\_preload := 150 \cdot kN$		Preload applied during installation to each anchor bolt. Representative value for purpose of this calculation.
$fgrout := 30 \cdot MPa$		Assumed compressive strength of grout
$fgrout\_allow := 0.6 \cdot fgrout$	$fgrout\_allow = 18\ MPa$	Allowable grout compressive strength
$dflange\_OD := 940 \cdot mm$		HMLP baseplate flange outside diameter

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$dflange\_ID := 600 \cdot mm$ — HMLP baseplate flange inside diameter

$d\_bolt\_hole := 35 \cdot mm$ — Assumed for this calculation

$$Agrout := \frac{\pi}{4} \cdot \left(\left(dflange\_OD^2 - dflange\_ID^2 \right) - Nb \cdot d\_bolt\_hole^2 \right)$$

$Agrout = 0.402\ m^2$ — Total bearing area of grout

$$\sigma\_grout\_preload := \frac{Nb \cdot Fb\_preload}{Agrout}$$

$\sigma\_grout\_preload = 4\ MPa$ — Mean compressive stress in grout due to anchor bolt preload

$Fb\_max := Fb\_preload + \Delta Fb$ $\quad Fb\_max = 156\ kN$ — Maximum anchor bolt load induced by bending load M_total

$D := 27 \cdot mm$ — Anchor bolt diameter M27

$p := 3 \cdot mm$ — Thread pitch for M27 metric

$d_t := D - 0.938194 \cdot p$ $\quad d_t = 24.19\ mm$ — Anchor bolt stress diameter

$A_t := \frac{\pi}{4} \cdot (D - 0.938194 \cdot p)^2$ $\quad A_t = 459.41\ mm^2$ — Anchor bolt stress area

$\sigma\_preload := \frac{Fb\_preload}{A_t}$ $\quad \sigma\_preload = 327\ MPa$ — Anchor bolt axial stress induced by preload

$\sigma\_proof := 585 \cdot MPa$ — Proof stress for grade 8.8 anchor bolt.

$Ratio\_preload := \frac{\sigma\_preload}{\sigma\_proof}$ $\quad Ratio\_preload = 0.56$ — Anchor bolt preload compared with bolt capacity

$\sigma\_range := \frac{\Delta Fb}{A_t}$ $\quad \sigma\_range = 12.9\ MPa$ — Change of anchor bolt axial stress caused by bending load M_total

$\sigma\_endurance := 20 \cdot MPa$ — Endurance limit for bolt tensile stress [Ref 1]

$\Delta c\_preload := \frac{Fb\_preload}{kcb}$ $\quad \Delta c\_preload = 0.053\ mm$ — Deflection due to preload is greater than the deflection due imposed loads. The base plate will not separate from the concrete when bending load M_total is applied.

$\mu := 0.3$ — Coefficient of friction between base plate and concrete (assumed)

$F\_total\_preload := Fb\_preload \cdot Nb$ — $F\_total\_preload = 1500\ kN$

Total clamping force between base plate and concrete / grout foundation

$F\_friction := F\_total\_preload \cdot \mu$ — $F\_friction = 450\ kN$

$F\_lateral := 11 \cdot kN$ — Typical maximum lateral load [Ref 2]

$SF\_slip := \dfrac{F\_friction}{F\_lateral}$ — $SF\_slip = 41$ — Safety factor against slip caused by imposed lateral load.

Conclusion
The stress induced in the anchor bolt by the imposed load is less than the endurance limit for bolting specified by BS EN 1993 1:9 [Ref 1]. Therefore, the anchor bolts would have an infinite life even if subjected to the peak design wind load at every cycle.

Dynamic bending stresses in the anchor bolts will not be very small because
a) Friction prevent lateral load transfer to bolts.
b) Structural grout prevents baseplate deflection and hence bolt flexing.
i.e. The base plate friction created by anchor bolt preload is at least 40 times greater than the highest lateral load that will be experienced by the high mast light pole.

Note! The calculation above is very simplistic and intended to be conservative. Actual results should be more favourable.