Design Report

Reduction of support structures

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Summary

Huisman is a privately owned company operating globally with extensive experience in the design and manufacturing of heavy construction equipment for world's leading on- and offshore companies. The facility in Fujian Province (Xiamen area) in China is one of the production facilities of Huisman. The production facility has been fully operational since April 2007 and delivers a significant contribution to the overall Huisman engineering and production capacity. Huisman China (HCN) is a production facility with a total area of 284,000 m^2.

Due to the opening of the first quayside owned by an international company in China in 2012 a large storage area is obtained. In combination with requests of clients to store their products passed the delivery date, the amount of products currently stored have rapidly increased. During storage additional support structures are used for support of the products. Amount of supports that are used for one product is, on average, equal to 19 percent of the own weight of the product.

The big stock of support structures, equal to 2600 metric tons, is divided into two types; workshop supports and spreader boards. Each support has its own functions and requirements. Some of the support structures are used for different functions which has as result that they are not used in the most efficient way. By evaluating the different functions of the supports the efficiency of the support with respect to weight is increased.

Spreading out the product weight over the floor and supporting the product to make sure transportation, by use of self propelled modular transport vehicles, is possible are the main functions of the supports.
Investigating the current max allowable local floor pressure resulted in some remarkable findings: 1) local floor pressure is equal to the global floor pressure and 2) there are no calculations available within Huisman China about the current floor pressures. A local design institute has determined the current floor pressure according to a soil survey without providing HCN calculations.

With use of different models (according to the American Concrete Institute) the validity of the current floor pressure is examined and a max allowable local floor pressure according these models is determined. These models (based on Westergaard's equations and footing calculations) calculate the stresses in the concrete foundation. This is done for different local loadings with keeping the properties of the soil layer, laying beneath the concrete foundation into account. According these models, the max allowable local floor pressure can be increased with a factor of four up to 20mT/m^2 for the big storage yard and with a factor of 15 up to 75mT/m^2 for the quayside.

With keeping different floor pressure scenarios in mind, new support structures are designed. During design different loading scenarios for the support structures are also taken into account. The new designs for the support structures result in a significant reduction of stock. The reduction is varying for the different combinations of max allowable floor pressure and loading conditions between 486mT up to 1186mT. This is equal to a reduction of 19 – 46% with respect to the total amount of stock at HCN and a reduction of 29 – 71% with respect to the stock that can be replaced by the new design.

Further investigation about the floor pressure at HCN and evaluating more design possibilities should give final outcome about the design that will lead to support structure that is most efficient with respect to weight.

For remaining questions please contact me at: arnoreimert90@gmail.com

Terminology & abbreviations

Term	*Explanation*
CoG	Centre of Gravity
HCN	Huisman China
HLMC	Heavy Lift Mast Crane
LTB	Lateral Torsional Buckling
mT	Metric Ton
OMC	Offshore Mast Crane
PMC	Pedestal Mounted Crane
PMOC	Pedestal Mounted Offshore Crane
QS	Quayside
SB	Spreader board
SPMT	Self Propelled Modular Transport
SS	Support Structure
SWL	Safety Working Load
SYI	Storage Yard Phase I
SYIII	Storage Yard Phase III
WSS	Workshop Support

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1 Introduction

Huisman is a privately owned company operating globally with extensive experience in the design and manufacturing of heavy construction equipment for world's leading on- and offshore companies. With engineering and production capacity in The Netherlands, Czech Republic and China, Huisman is generating close to 100,000 m^2 of total production surface. The new to build production facility, which should be operational in 2014, will increase this production surface even more. Together with the local sales, engineering and service supports in Australia, Norway, Brazil, Singapore, Slovakia and the USA a dedicated service team of skilled professionals is always able to provide advice and service support before, during and after installations and delivery.

The facility in Fujian Province (Xiamen area) in China is founded to facilitate customers in this region and to increase the overall production capacity. The production facility has been fully operational since April 2007 and delivers a significant contribution to the overall Huisman engineering and production capacity. Huisman China (HCN) is a production facility with a total area of 284,000 m^2. Part of the terrain is the first quayside owned by an international company in China that is opened up in 2012. This quayside is specially constructed by Huisman to increase production area and be self-reliant for loading and unloading.

Owing to the opening of the new quayside in 2012 a large storage area is obtained. In combination with requests of clients to store their products passed the delivery date, the amount of products currently stored have rapidly increased. During production and storage of the products, additional support structures are needed which results in a big stock. This stock exists out of two types of support structure; workshop supports and spreader boards. The usage of these two types of support structures can be split up into mainly two categories; distribution of the floor pressure to ensure that the max allowable floor pressure is not exceeded and supporting of the product. The distribution of the floor pressure is in its turn divided into the floor pressure of the different areas on the site of HCN and the deck pressure on the transport barges that are sometimes used to transport the product to the customer.

The total stock of support structures results in a big amount of steel that is used for storage of products. By having a look at the validity of the current floor pressure values and by assessing the current design of the support structures it should be possible to come up with possibilities to reduce the total amount of steel needed for support structures.

The current floor pressure is based on a soil survey performed by a local design bureau. In theory it is not allowed to exceed the max allowable floor pressure determined by the design institute because of warranty reasons. The stacking units are in the current situation constructed out of separated stacking units; the spreader boards and workshop supports. For the support structures that are used to support a product it should always be possible to transport the product together with all the used support structures over the site of HCN by use of self-propelled modular transport vehicles. The dimensions and safety working load of the SPMTs results is some constraints for the support structures.

In Chapter 2 the current situation of the floor pressure distribution and the support structure use is discussed. In Chapter 3 the floor pressure at HCN is discussed into more detail. Chapter 4, 5 and 6 contain information about the design of the new support structures.

The end of this report contains a conclusion accompanied by recommendations for further investigation.

2 Current situation and problem definition

The final purpose of this report is to come up with a potential reduction of the total stock of support structures (SSs). To realize this, it is important to be aware of the current situation. Investigating the current situation shall reveal potential points of improvement, which can lead to reduction of stock. This is described in this chapter.

2.1 Current situation

HCN uses SSs for multiple purposes. The SSs are used to support the products during the production phase in the workshops and during transport to, and storage at the quay side (QS) or storage yard (SY) until delivery date. Except for support of the products are the SSs also used to distribute floor- and deck pressure during storage on the terrain of HCN and during transport on transport barges.

2.1.1 Support structures

The SSs can be split into two types of supports; spreader boards (SBs) and workshop supports (WSSs). On March 17th of 2014 the stock at HCN was counted and inventoried. Distinction is made between the different type of SSs, between the storage of SSs on different areas at the site of HCN and between the usages of the SSs. This way it is possible to divide the total stock over different categories to indicate efficiency opportunities in a later phase.

In Appendix A, a list is shown containing the entire stock at HCN and the location of the SSs on the site. From this list it can be noticed that the total stock has a weight of 2845 metric tons. [1] 240mT of this stock is due to 25 meter long SBs. From this point on these SBs are not taken into account because they are only used for ‘special cases’, which is out of the scope of this project. Furthermore it can be noticed that the 830mT stored at the (QS) together with the 972mT stored at the storage yard phase III (SYIII) represent the largest part of the stock. This part of the stock is only used to store out the products at the site of HCN. For the remaining 800mT (322mT is in use in the big workshop (WS B1A) and 170mT is in use in the small workshop (WS 01&02)) it is assumed that the SSs are used in the same way as the SSs at the QS and the SYIII. From the same list it can also be noticed that WSSs represent 23 percent of the total stock while the SBs represent the remaining 77 percent.

WSSs are mainly used to bring the product to a certain height. Sometimes this height is needed to perform work on the product but it is almost always needed for transportation with self-propelled modular transport vehicles (SMPTs). The SPMTs (Drawing *A08-00001-00-00A* [5]) have a height clearance of 1450mm and have to drive below the product. Since the main function of the WSSs is to bring the product to a certain height and thus to support the structure, it is assumed that 100 percent of the WSSs are used for supporting purpose.

The SBs, are in contrast to the WSSs, used to fulfil two functions. First function is spreading out the floor pressure; the max allowable floor pressure has to be kept in check always. This means that when this floor pressure seems to be exceeded, SBs are placed beneath the WSSs to make sure this will not happen. For the distribution of the product load to the deck of the transport barges, the SBs are used in the same manner. Second function of the SBs is supporting the structure. The SPMTs that are used for transportation of the product result besides height clearance also in restriction of the passage width for the complete SS. Due to the width of the SPMTs the WSSs must be placed at least 6 meters away from each in order to drive in between. In order to overcome this distance, SBs are used; the SBs have supporting function.

To determine the influence of the two use categories (supporting of the product and load distribution to the floor) of the SSs, the load out configuration for multiple products is examined; this results in the product range for this project. The following cranes (or a part of that crane) are evaluated: 300 and 900mT PMOCs, 400, 850, 2200 and 3000mT OMCs, 1500 and 1800mT HLMCs and 400 and 900mT PMCs. This product range covers a large area of HCNs products and is therefore assumed to be representative for the entire product range of HCN. For the (parts of the) cranes, information about the crane and information about the SSs used are listed in Appendix B. From this Appendix it can be seen that 44% of the stock is used for distribution of the floor pressure, while 56% is used to support the product. With the full support function of the workshop supports this means that almost 60 percent of the

[1] Price for one mT of stock is equal to 2 euro’s.

SBs are used for distribution of the floor pressure and the remaining 40 percent is used for support of the structure. Check:

$$0.23 \cdot 100 + 0.77 \cdot 60 + 0.77 \cdot 40 = 100\% \qquad 2\text{-}1$$

Appendix B also gives information about the amount of SSs that is on average used to support a product. This amount is equal to 19 percent of the own weight of the product. Besides this 19 percent it is found that:

- In 65% of the cases, the SS support the product on a second level (Figure 2). The amount of SSs used for that second level is equal to 13% of the total stock.
- Two products can't be transported by only making use of SPMTs; they are not supported by a complete SS, see Figure 3. These products have to be lifted on the SPMTs by use of a crane (for example in the workshop). The SPMTs drive to the QS and with use of the Skyhook the product is stored at the QS.
- The transportation of the remaining 24 products is done by making use of only SPMTs. In total there are 52 complete SSs used for the storage of the 24 products. Sometimes one (set of) SPMT(s) is used to transport one complete SS and sometimes it is possible that one (set of) SPMT(s) can transport two complete SS (Figure 4).
- From the 52 complete SSs, 37 structures are transported by use of a single SPMT, 13 structures are transported by use of a set of two SPMTs and 2 structures are transported by use of a set of three SPMTs.

A short overview of the results obtained by evaluating the product range is given in the figure below.

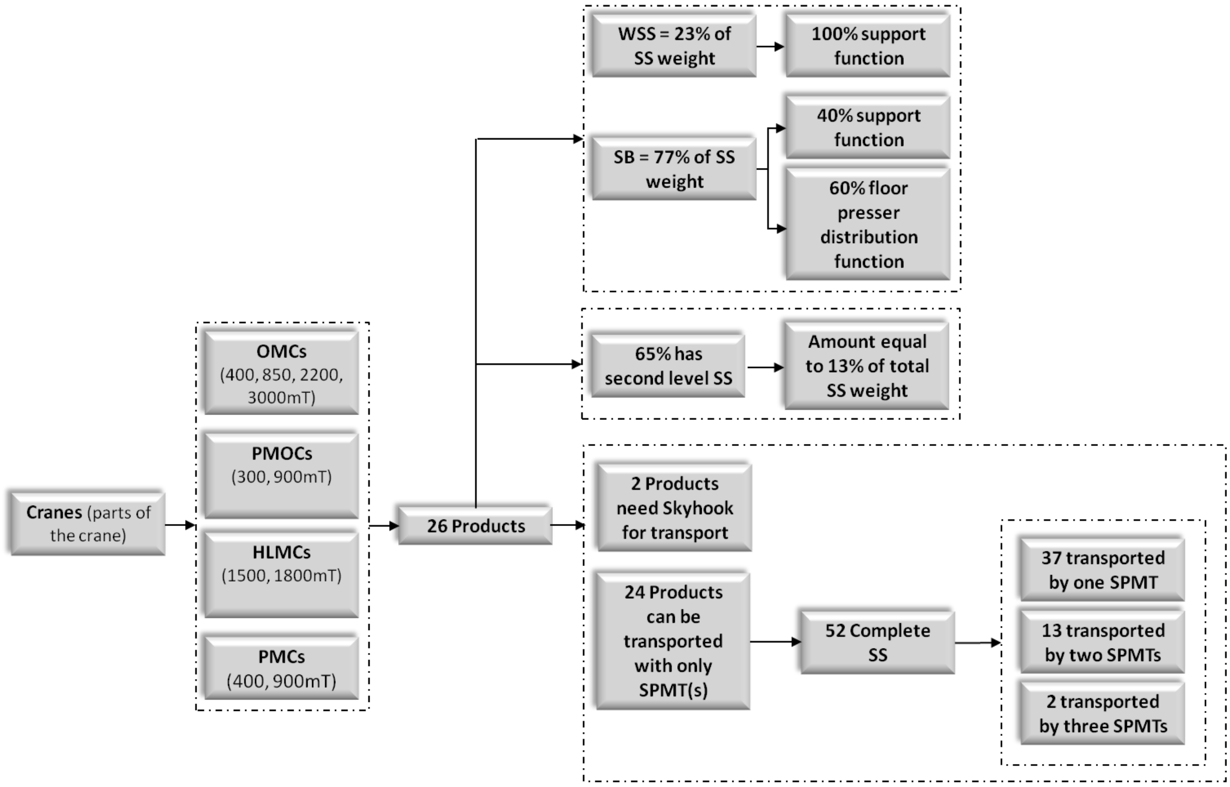

Figure 1: Evaluation of the product range

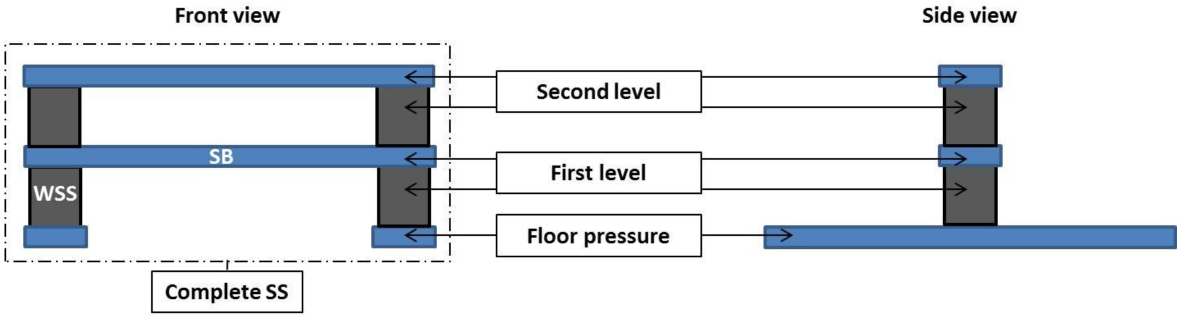

Figure 2: One complete SS with a second level support on top

Figure 3: 1800mT HLMC stored at the QS with use of the Skyhook.

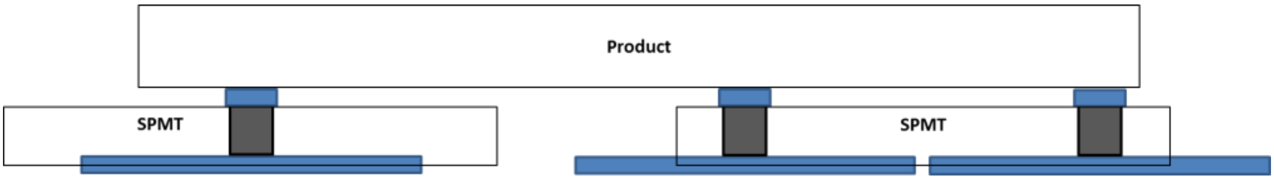

Figure 4: Three complete SSs are transported by use of two singe SPMTs

2.1.2 Floor pressure

For the evaluation of the floor pressure distinction is made between the different areas at HCN. The total area of HCN is split into different areas, with all their own max allowable floor pressures. There are the two main workshops, WS B1A and WS 01&02 (10mT/m^2), the storage yard phase I (SYI) (3mT/m^2), the storage yard phase III (SYIII) (5mT/m^2) and the quay side (QS) (again divided into several floor pressures, storage yard at quay side is however 5mTm^2). Drawing of the complete site of HCN with all max allowable floor pressures for each area can be found in drawing "*A07-20280-00-10F"* [6]. The site of HCN is shown in Figure 5.

	Max allowable floor pressure [mT/m^2]
WS B1A and WS 01&02	10
Storage yard Quayside	5
Storage yard Phase III	5
Storage yard Phase I	3

Table 1: Max allowable floor pressure at different sites of HCN

Figure 5: Site of HCN with the EMAS AMC in front of the Quayside

The different max allowable floor pressures are determined by a local design institute; “Fujian Architectural & Light Textile Design Institute”. The max allowable floor pressures are based on a soil survey. During this survey samples of the different ground layers are taken. For all the layers, different tests are performed to determine the properties of the layer. When the results are known, the max allowable floor pressure is determined.[2] The tests and calculations that are necessary to determine the max allowable floor pressure are according the Chinese code “Load code for the design of building structures” [7].

In the current situation no distinction is made between local floor pressures and global floor pressure. For the QS and SYIII it is at the moment not possible to make this distinction since it is unclear how the exact value for the floor pressure is determined. For the workshops the differences between the local and global floor pressure is however already known but not implemented yet. There are drawings available which contain information about the maximum SWL for different store out configurations in the workshops. The foundation beneath the workshop is a pillar foundation; the SWL depends on the position of the local loads with respect to the piles. Drawings of the pillar foundation and plans containing the max allowable local floor pressure in the workshops are shown in Appendix C. It can be seen that local floor pressure in the big workshop starts from 40mT/m^2 varying up to 200mT/m^2.

The floor pressure is based on the soil survey, ‘Report for geology survey of dream hall’ [8] performed at the QS. The soil survey of the SYIII is not available. Therefore it is assumed that the soil beneath the QS and SYIII is the same. The foundation on top of the upper soil layer is not the same for the SYIII and QS. The foundation at the SYIII exists out of a detritus layer with concrete cast-in-situ structures on top. For the QS there is a detritus layer, with a lean concrete layer, a sand layer and precast concrete blocks on top. More information about the two foundations can be found in Appendix D.

At the moment it is not allowed to exceed the max allowable floor pressure at one of the sites of HCN. When this max allowable floor pressure is exceeded the design institute is no longer responsible when damage occurs and therefore the design institute will not repair the damage in the form of warranty.

2.1.3 Transport barges

At HCN there is almost no information available about the transport barges. Only thing that is known is the max allowable deck pressure of the barges. This is a global deck pressure and is different for most of the transport barges. Since the deck construction is almost the same for most barges some simplified beam calculations can be performed to see if difference can be made between local and global deck

[2] The design institute wasn’t willing to provide information about the calculations report of the foundation because that is classified information.

pressure. A small model, based on the Ballast Pontoon of HCN [9], is constructed. For this model a few assumptions are made:

- The total deck can be divided into sections with the same size as the tank cross sections.
- The web frames absorbing all the forces.
- Longitudinal stiffeners are only used to reinforce the deck plate.

In this model two situations are taken into account. Situation one where it is assumed that the walls of the tanks have rigid walls and option two where it is assumed that the walls have certain stiffness. Filling in the dimensions of the deck of the pontoon it can be seen that the local force, if executed on the web frame, can be much higher than the 10mT/m^2. This depends however a lot on the contact area of the local force. More information about the model and values for the pontoon can be found in Appendix E.

2.2 Problem definition

From the current situation it becomes clear that it is unknown how the value for the floor pressure is assigned to the QS and SYIII. Another remarkable thing is that the same kinds of SSs are used for multiple functions; the SSs can be used more efficiently.

It is likely that a stock reduction is possible when these two points are evaluated in more detail. Chapter 3 is about the floor pressure; distinction is made between local and global floor pressure. Chapter 4 will start with discussing the design of a new type SSs.

3 Max allowable local floor pressure

This chapter describes the current max allowable floor pressure at HCN. Since there is no detailed information available about the methodology that is used by the design institute to determine/calculate the current used allowable floor pressure it is chosen to calculate the floor pressure with use of constructed models. Failure of the concrete foundation can occur due to shear (punch and beam) and bending stresses in the concrete. The models will calculate these stresses. Since the QS and SYIII have both a different foundation, distinction is made between these two different areas at the site of HCN.

3.1 Models

Two models are constructed; one model will calculate the punch and beam shear in the concrete and one model will calculate the stresses in the concrete due to bending. The model used to calculate the bending stress is useful for three different loading conditions; loading on the interior of the slab, loading at the corner of the slab and loading on the edge of the slab (Figure 6). For the QS there is only one loading condition since the lean concrete layer is one big layer. For the SYIII the different loading conditions are however very useful since the foundation is build up out of multiple ground slabs. More information about the foundation can be found in Appendix D.

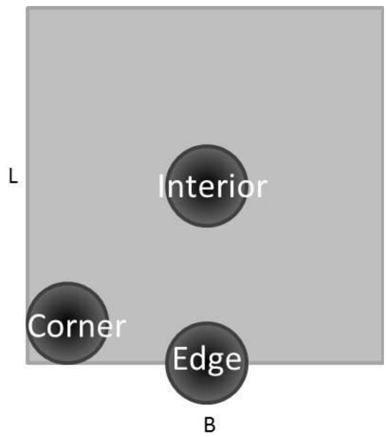

Figure 6: Loading conditions of concrete slabs.

The model that will calculate the shear stresses in the concrete is based on footing calculations. The concrete foundation represents the footing which is concentrically loaded by a SS under loading. This SS can be seen as the column. [10] The shear stresses due to a local load depend on:

- Max allowable global floor pressure: 5mT/m^2.
- The foundation on top of the soil. For the calculation of the shear stresses in the concrete at the QS it is assumed that the lean concrete layer is split into slabs with the same width and length as the slabs on SYIII.
- Floor surface of the support structure that is in contact with the concrete slab.
- Properties of the used concrete.

Figure 7 shows the model used for shear and beam stresses.

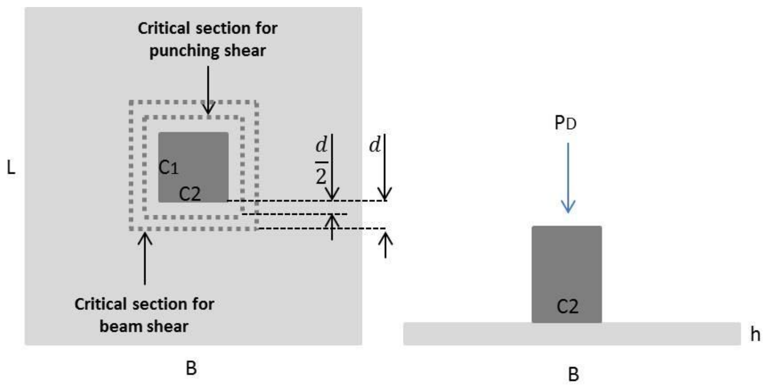

Figure 7: Critical shear sections

The model that will calculate the bending stresses in the concrete is based on Westergaard's stress equations. [11] Westergaard's equations are used in slab-on-grade pavement design. It calculates the max allowable wheel load, which is a local load. This report is however about high floor pressures due to loaded supports and not due to wheel loads.

Westergaard analysed three particular loading conditions (interior, edge and corner). For loading on the edge and corner of slabs, Westergaard only derived equations based on wheel loads, where it is assumed that the load is uniformly distributed over the area of a small circle with radius a. For interior loading of the slab, Westergaard also derived an equation that can be used in case of a uniformly distributed square area, which is the case at HCN.

A complete report which contains more information about the models and that will discuss the results into more detail than is done in the next section can be found in Appendix F.

3.2 Limitations

The two described models that will be used to calculate the allowable local floor pressure have some limitations. The model used to calculate the shear stresses in the concrete is based on footing calculations.

- In case of footings, the column is always attached to the footing. In case of HCN, the supports are not attached to the concrete.
- Footings are always beneath the surface and therefore there is always some type of soil on top of the footing. In case of HCN, the concrete forms the floor surface.
- At the quay side, the foundation exists out of one big lean concrete layer. To perform shear calculations the concrete layer is divided in squares of 5 by 5 meter.

The Westergaard's equations that are used to calculate the bending stresses in the concrete also have some limitations:

- The vague definition of '*Removed considerable distance*'. For this study it is assumed that a considerable distance is equal to a distance of 1000mm between the center of the support to the edge.
- Westergaard's equations calculate the max allowable wheel load, which is a local load. This report is however about floor pressure due to loaded supports and not due to wheel loads, so it is an approximation.
- The modulus of subgrade reaction (k-value) which is needed to calculate the bending stress in the concrete is always a constant. From the soil survey at the quay side it is found that there are a lot of different layers in the soil. For the calculations at HCN it is assumed that the upper two layers (backfilling stone and backfilling sand) have the biggest impact on this k-value. These two layers have a depth of at least 9 meters for the whole QS. An average of these two layers is used.

3.3 Results

With use of the two constructed models is it possible to calculate the max allowable local floor pressure for the QS and for the SYIII. The limitations of the models however have to be taken into account; therefore the complete Report about the local floor pressure should be checked by a civil bureau.

3.3.1 Storage Yard Phase III

Since the max allowable global floor pressure is $5mT/m^2$, the shear stress in the concrete will not cause any trouble. The shear stress will start causing trouble in case the total max allowable global floor pressure for one slab (125mT) is exerted to a surface area of $250 \cdot 250$mm $(C_1 \cdot C_2)$; the critical punching shear will become larger than the punching shear force resisted by the concrete. This will however never be the case for Huisman.

The bending stress in the concrete will therefore determine the max allowable local floor pressure. For SYIII there are the three loading conditions. The results are based on a loaded WSS on top of a concrete slab, see Table 2. Local floor pressures are given for multiple values for the modulus of subgrade reaction.

	Modulus of subgrade reaction (k-values) x 10000 [kN/m^3])						
	k = 1	*k* = 1.5	*k = 2*	*k = 2.5*	*k = 3*	*k = 3.5*	*k = 4*
Interior	19	20	21	22	23	24	24
Edge	14	15	17	18	19	20	21
Corner	18	21	26	30	34	39	46

Table 2: Max allowable local floor pressures at the storage yard for the three loading conditions. [mT/m^2]

3.3.2 Quayside

For the QS punch will also not cause any problems; the bending stresses in the concrete will determine the max allowable local floor pressure. Since the lean concrete layer is just one big layer, only one loading condition is taken into account; the interior loading condition. For the calculation of the bending stress only the concrete layer is taken into account. The detritus layer, the sand layer and the precast concrete blocks are ignored. Results can be found in the table below.

	Modulus of subgrade reaction (k-values) x 10000 [kN/m^3])						
	k = 1	*k* = 1.5	*k = 2*	*k = 2.5*	*k = 3*	*k = 3.5*	*k = 4*
Interior	74	78	81	83	86	88	90

Table 3: Max allowable local floor pressures at the quay side for the interior loading condition. [mT/m^2]

At the moment the max allowable local floor pressure at the QS is 5mT/m^2. According to models where punch shear, beam shear and bending in the lean concrete layer (with ignoring the other layers) are calculated, the local floor pressure can be increased significantly. The bending stress is for the lowest k-value (equal to 10.000kN/m^3) not exceeding the critical stress in the concrete as long the local floor pressure is not exceeding 74mT/m^2. Of course the max allowable global floor pressure of 5mT/m^2 may not be exceeded.

The precast concrete blocks have a compressive strength of 50MPa, so this doesn't give any problems. Point of attention is however the 50mm sand layer between the concrete blocks and the lean concrete layer. This layer may result in sagging of the precast concrete blocks. The sand layer is probably also the reason for the current waving of the QS floor surface.

4 Redesign Specifications

In chapter 3 it is found that the max allowable local floor pressure at SYIII can be increased by a factor off four up to 20mT/m^2, as long as the max allowable global floor pressure is kept in check. During the evaluation of the product range in Chapter 2 it was found that the current average local floor pressure is already exceeding the prescribed floor pressure of 5mT/m^2 by a factor of two up to 10mT/m^2. Out of these values it can be concluded that the amount of SS used for the floor distribution can be halved. Remark is the assumed rigidly of the SBs used to distribute the floor pressure.

What currently is done is the following; “replace 8meter long SBs with a height of 400mm by 12meter long SBs with a height of 310mm in case the contact area of the 8meter SBs with the floor surface is not enough to keep the floor pressure beneath the max allowable floor pressure”. In Figure 8 gaps between the ends of the SBs and the floor surface are shown to indicate that the distribution of the floor pressure is not uniformly. When the length of all the SBs is halved so that a local floor pressure of 20mT/m^2 is reached this assumption will come closer to reality. Redesigning the SS used for floor pressure distribution can however lead to a better solution.

Figure 8: Gap between end of the SBs and the floor surface

The SSs used for floor pressure distribution are also used for the support of the product. In this chapter the requirements for the new SS that will be designed are discussed, but first information is given about some changes with respect to the current situation and about remarks/assumptions.

4.1 Changes

At the moment SBs are used for different functions. For the new support structures distinction will be made between the different functions to make sure the redesign will be designed most efficiently with respect to weight. The complete SS beneath a product exist, at the moment, out of multiple individual components (the simplest structure is build up out of three SBs and two WSSs). For the redesign it is considered if it is profitable to connect all the individual parts together so the complete SS exist out of one piece.

4.2 Remarks

The total amount of stock at HCN is equal to 2600mT. It is assumed that this total stock is used in the same way as the support structures in the product range.

The content of the report about the floor pressure first have to be verified before the results can be used. Therefore different local floor pressure scenarios will be evaluated during the design phase of the redesign. The current used local floor pressure is already found to be 10mT/m^2; this will be the first scenario. Second scenario is the local floor pressure of 20mT/m^2; according chapter 3 the actual allowable local floor pressure at the SYIII. With knowing that the allowable local floor pressure for WS B1A is varying between 40 and 200mT/m^2, the third and last scenario of a local floor pressure of 40mT/m^2 is embraced. For the third scenario the products will be transported directly from the workshops towards the QS.

During transport by SPMT it is assumed that the dimensions/shape of the redesign will not cause any troubles for the SPMT as long the loading conditions for the SPMT, as shown in '*Drawing D00000306-03-100-01 SPMT China*' [5] are met. This assumption can be made since the load bearing beams are over dimensioned with respect to the loading capacity of the SPMT.

4.3 Requirements

In this section all the requirements are set for the new design of the SSs. After discussing the requirements, all the information is available to start with the design phase of the SSs.

- The maximum allowable local and global floor pressure should always be kept in check for the different scenarios.
- The new SSs should be able to replace the SSs that are used for distribution of floor pressure and are used for support of the product at first level. The redesign is not intended to replace the SSs at higher level support.
- Since 65% of the products are also supported at second level it should be able to place a second level support on top of the new SSs.
- It should always be possible to transport the SSs together with the product by SPMT. With a SPMT height clearance of 1500mm and a width of 5500mm minimum dimensions for the SSs are defined.
- For the new SSs distinction has to be made between complete SSs that will be transported with use of one and two SMPTs. SSs that need three SPMTs for transport are out of the design scope.
- For the manufacturing of the redesign no machining operations or complex welding is allowed. This because of the price tag of the product.
- The redesign should be manufactured out of steel37 and/or steel52. It is preferable that material is used that is already on stock at HCN.
- Stability of the complete structure should be kept in check all the time.
- The weight of the product has to be distributed uniformly as possible to the floor.

5 Redesign Concepts

In this chapter different concepts for the new SS are discussed briefly. In Appendix G an elaborated version of the concept phase is given.

A complete SS consist out of three parts; a support beam, two columns and floor plates (Figure 9). The dashed lines show that there is a lot design space for each part. In the current situation is one complete SS constructed out of a number of stacking units (the WSSs and SBs). Advantage of these stacking units is the free design space for the complete SS. Another profit is the small storage area that is needed for the stacking units that are not in use. Big disadvantage however is that the same stacking units are used for different functions, which mean that they are not used most efficient with respect to weight.

The new SS will be a complete SS where the support beam, two columns and floor plates are welded together. This way it is possible to design most efficient with respect to weight. Since the freedom of adjusting the complete SS (with use of more SBs or WSSs) is no longer available, distinction has to be made between different loading scenarios. It is for example possible to design SSs for different SWLs based on the product range or to design SSs with a SWL equal to the SWL of the SPMT.

Figure 9: Simplified body of complete SS

5.1 Support beam

The support beam is subjected to two types of loading; point load in the middle of the beam and uniformly distributed load over the entire beam. Stresses due to bending and shear forces have to be kept in check together with the lateral torsional buckling (LTB). With neglecting the LTB the best section with respect to deflection is an I-beam. When the LTB is however taken into account, it is better to come up with a torsion box; the torsion box will give more stiffness to resist the LTB.

Because of the requirement that it should be possible to place a second level of support on top, it is preferable that the upper section of the beam is as wide as possible; a torsion box made out of I-beams becomes the best solution. The dimensions of the I-beams depend on the occurring stresses in the section, but it also depends on the design choices for the columns and at the plate material that is on stock at HCN.

Evaluation of the stock shows that St52 or Q345D will be used. Both have a yield stress starting from 345MPa. There are quite a number of plate thicknesses on stock, so that will not cause any bottlenecks.

In Microsoft Excel two models are created (‘Redesign single support’ and ‘Redesign double support’) to compare different kinds of support beams and column structures to come up with the most efficient support structure combination. Multiple SWLs (both point load and distributed load) are examined; this is based on the loading situations of the product range earlier discussed.

For both models (single SS and double SS) are two different box sections examined; box sections out of two I-beam profiles and box-sections out of three I-beam profiles, see Figure 10. The shear stresses and bending stresses in the support beams (Figure 11a and b) are calculated for the different box sections loaded by varying SWLs and having different lengths. With keeping the width of the SPMT in mind, the length of the support beam depends also on the column structure.

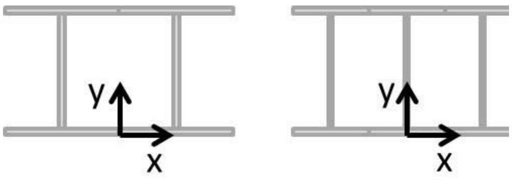

Figure 10: Two type of box sections

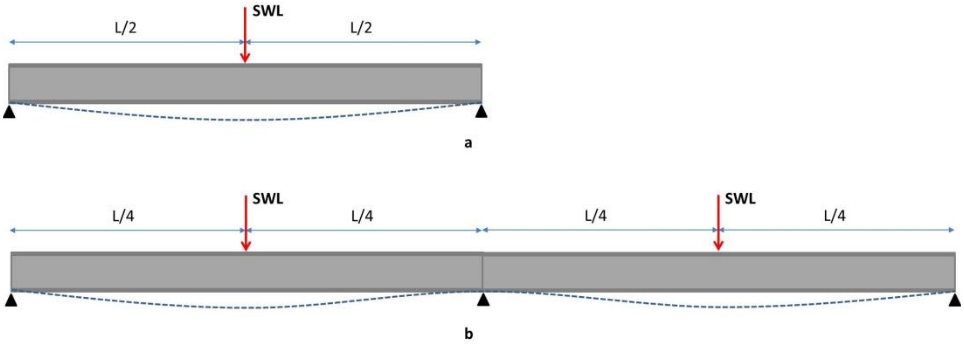

Figure 11: Deflection for two types of support beams

5.2 Column structure

For the column structure, watched in the front view, distinction can be made between placing the column under a certain angle with respect to the vertical axis or to place the columns structure perpendicular with respect to the support beam. The first option can result in a shorter support beam since the length of the beam depends on the drive through width of the SPMT; see Figure 12 (5500mm plus 200mm tolerance). However to absorb the high stresses that will occur in the weld (stress concentration is pointed by red circle), large stiffeners are needed; reducing the support beam length is not possible and use of columns positioned vertical (when the SS is seen from the front) is preferred.

Figure 12: Column structure possibilities

Furthermore there are different options for the orientation of the column structure in the side view, different profiles are possible and distinction has to be made between the column structures of the two different models (column structures that support the end of the support beam and column structures that support the support beam in the middle).

For both type of column structures I-beam columns and tube columns are examined as well as placing the columns vertical beneath the support beam or placing it under a certain angle (Figure 13). In case of placing the columns under a certain angle the two columns can be connected by use of a truss bar or by use of one large floor plate.

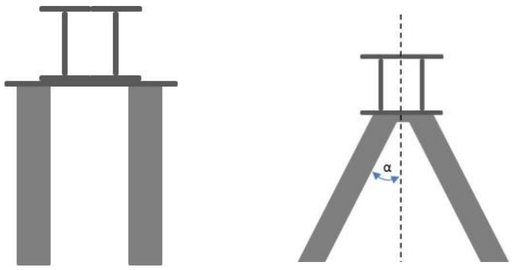

Figure 13: Orientation of support columns

The stresses in the columns have to be kept in check all the time as well as failure to buckling.

5.3 Floor plate

The design of the floor plate, that will be used to spread out the product weight over the floor, depends on the max allowable floor pressure, the loading on top of the support beam and the design of the column structure. For the design of the floor plate, stiffeners will be used between the floor plate and the column structure. Due to these stiffeners it is assumed that the floor plates will distribute the pressure equally to the floor. The use of stiffeners is however only valid as long as the dimension of the plate are not too large.

6 Redesign results

In this chapter all the results obtained after evaluation of the different concepts and a final design is composed. Result off the different SS loading conditions together with the three floor pressure conditions is a 3x3 matrix with the reduction for every scenario.

6.1 Final design

In the final design is the support beam a box section constructed out of two I-beam profiles. Each column structure consists out of two I-beam columns under an angle of 30 degrees with the vertical axis. One end of the columns is welded to the support beam while the other end is welded to the floor plate. The flange of the column will be positioned in line with the web of the I-beam profile of the box section; Figure 14 shows the good force introduction between the support beam and the column structure. This force introduction is less in case of tube columns. A box section existing out of two I-beam profiles is chosen because it has a better stiffness versus weight ratio in comparison with a three I-beam profile section.

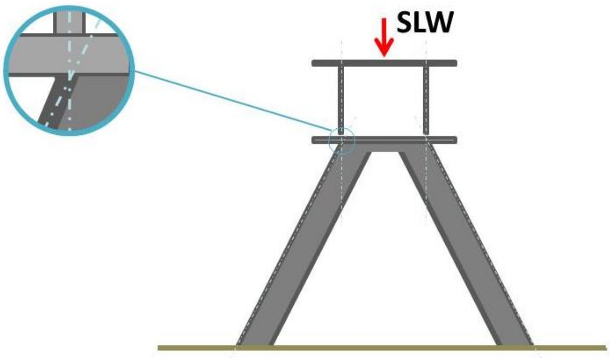

Figure 14: Positioning of columns with respect to support beam

The length of the support beam used for the first model, which will be transported by only one SPMT, is equal to 7 or 8 meters depending on the different scenarios. In case of the second model (transported by use of two SPMTs) the length is equal to 15 or 16 meters.

While all the column structures are constructed out of I-beam profiles, distinction is made between the SWL of the different column structures. The middle column structure in case of the second model has a higher SWL because of the variety of loading on top of that model. More information about choices made during the design phase and about the different loadings can be found in Appendix G.

As already discussed, are there different loading conditions on top of the SSs and different floor loading conditions. The floor plate should be able to uniformly distribute all the forces to the ground; this will result for some loading scenarios to a very large and heavy floor plate. A third SS model is needed to distribute the floor pressure uniformly to the floor in case the distance from the edge of the floor plate to the nearest point of the column is more than 2500mm. The different models are displayed in Figure 15, Figure 16 and Figure 17.

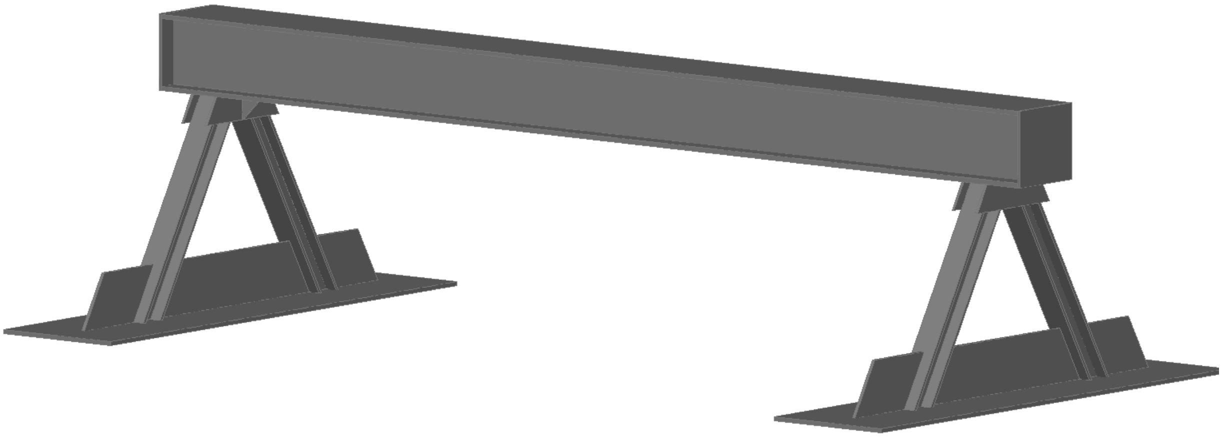

Figure 15: Single SPMT SS

Figure 16: Double SPMT SS

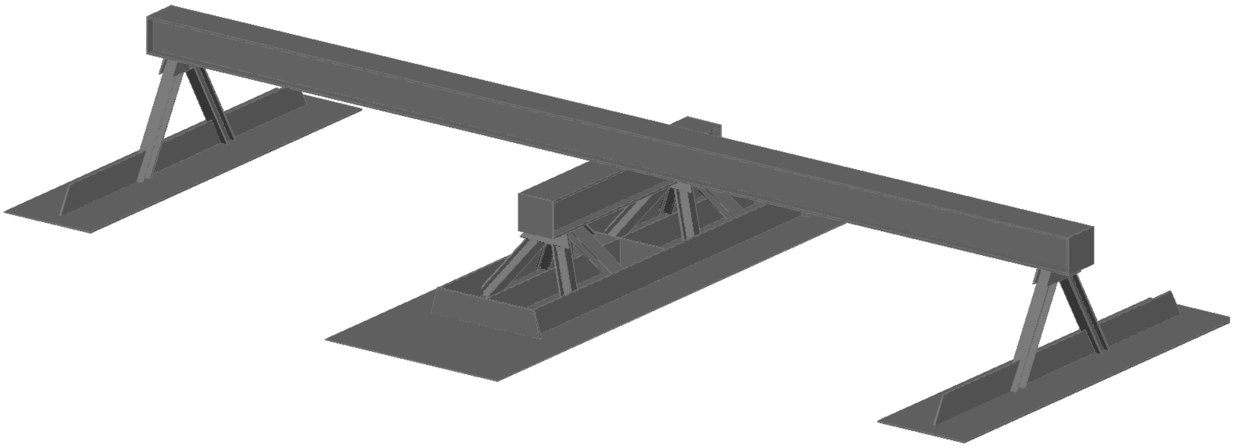

Figure 17: Double SPMT SS with additional SS in the middle

6.2 Loading scenarios

At the moment, HCN uses a lot of the same load out configurations for a variety of products; a product is placed on a 8 meter SB (this way it is possible to transport the product), the SB is supported by two WSSs and the WSSs are on their turn placed on top of two 8 meter SBs. The 8 meter SB has a SWL applied in the middle of the board of 120mT. When the described load out configuration is loaded by that 120mT, the configuration is used reasonably efficient. When the configuration is however loaded by a product of about 20, 30 or 40mT (this is done multiple times at the moment) the ratio between support weight (about 18mT) and product weight becomes relatively high. Therefore not only SSs with a SWL equal to that of the existing SBs are examined but also SSs with lower SWL.

The product range, listed in Appendix B, form the basis of this project. When the load out configuration of the products is examined into more detail some things can be noted. Of the 26 products it is found that five products are stored without a complete SS; three products are not supported correctly (no spreader boards are used to distribute the floor pressure) and two products are stored at the QS with use of the Skyhook (again no complete SS is used), see Figure 18. For the remaining 21 products, all the supports that are used for the SSs that are transported by use of three SPMTs are left out of the design scope; this is case for one product.

Figure 18: Load out of 300mT PMOC (left) and 1800mT HLMC (right) without using complete support structure

For the remaining complete SS, the loading of the SS due to the product is evaluated, for the twenty stored products that are taken into account it is found that 31 complete support structures need only one SPMT for transport and 13 complete support structures need two SPMTs. For the 31 single SPMT SSs the loading is divided into two groups; point load up to 50mT and point load between 50 and 100mT. 13 times the loading is less than 50mT and 18 complete support structures are loaded by a load between 50 and 100mT (one time an additional 8m SB is needed). For the 13 double complete support structures 9 times the structures is loaded by a point load less than 120mT and 4 times by a load between 120 and 160mT.

From the previous findings three different categories are composed. The first two scenarios are only based on information about the product range, while the third scenario is also based on the SWL of the SPMT.

1. - 13 SSs with a SWL of 50mT transported by one SPMT.
   - 18 SSs with a SWL of 100mT transported by one SPMT.
   - 9 SSs with a SWL of 2x120mT transported by two SPMTs.
   - 4 SSs with a SWL of 2x160mT transported by two SPMTs.

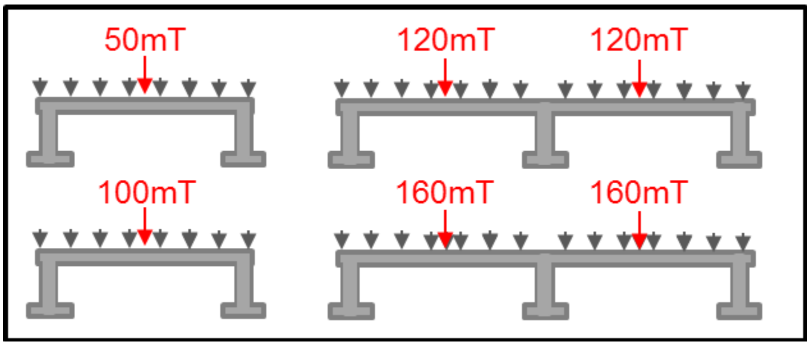

Figure 19: Loading scenario 1

2. - 31 SSs with a SWL of 100mT transported by one SPMT.
   - 13 SSs with a SWL of 2x160mT transported by two SPMTs.

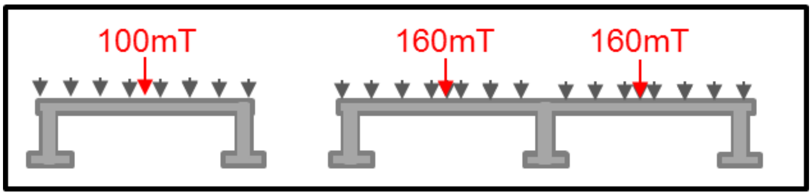

Figure 20: Loading scenario 2

3. - 31 SS with a SWL of 200mT transported by one SPMT.
   - 13 SSs with a SWL of 2x200mT transported by two SPMTs.

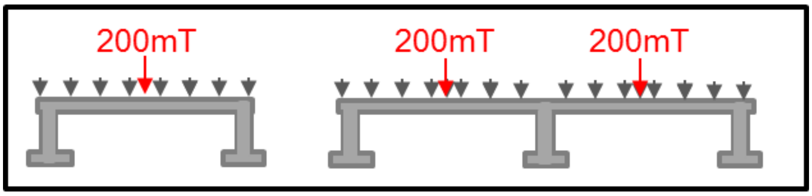

Figure 21: Loading scenario 3

All the safety loads are based on a point load in the middle of the support beam between two column supports.

6.3 Results

In order to determine the final reduction for all different scenarios an Excel sheet is constructed; “Reduction”. In this sheet multiple combinations of support beams and column structures are compared. Distinction is made between different support beams and column structures for the different floor pressure scenarios and loading scenarios. During the comparison of the beams and column structures the stresses in the material are taken into account and the mass of each component is kept as low as possible. This is done for all the above mentioned loading situations. Results are displayed in Table 4. Dimensions of the profile of support beam and column beams are also shown in this table, for information see Figure 22.

Complete SS	Length support beam	Profile support beam b*tf / h*tw	Profile outer columns b*tf / h*tw	Profile inner columns b*tf / h*tw	Total mass without floor plates
S50	7000	600*20 / 340*10	150*10 / 150*10		1967
S100	7000	600*30 / 490*12	200*12 / 150*12		3025
	8000	600*30 / 490*15	200*12 / 150*12		3588
S200	8000	700*30 / 720*25	200*12 / 150*12		4532
D120	15000	600*25 / 710*20	200*12 / 150*12	300*20 / 150*12	7706
	16000	600*25 / 720*20	200*12 / 150*12	300*20 / 150*12	8218
D160	15000	600*30 / 640*25	200*12 / 150*12	300*20 / 150*12	8860
	16000	600*35 / 650*25	200*12 / 150*12	300*20 / 150*12	9444
D200	15000	600*40 / 660*30	250*12 / 150*12	300*20 / 150*12	11487
	16000	600*40 / 670*30	250*12 / 150*12	300*20 / 150*12	12238

Table 4: Final concept results

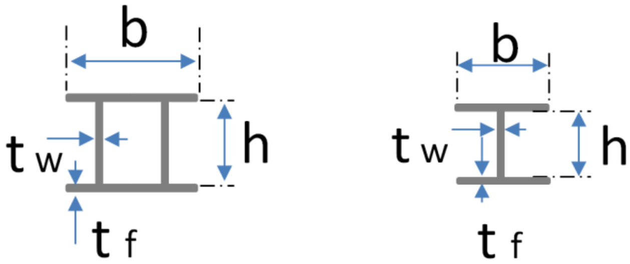

Figure 22: Profile of support beam (left) and columns (right)

There are besides the different loading scenarios also the three floor pressure scenarios. These three floor pressure scenarios in combination with the loading scenarios resulting in different floor plates. The thickness of the floor plate is equal for all the different scenarios; 30mm. The amount of stiffeners that is used in order to reinforce the plate depends on the length of the floor plate. The stiffeners are needed for uniformly pressure distribution over the floor. The total amount of stiffeners used to reinforce the plate is equal to a certain percentage of the mass of the plate self. This percentage depends on the length of the plate, see Table 5. For floor plates longer than 6850mm an additional support structure is needed since reinforcement with only stiffeners is not sufficient.

Length of floor plate [mm]	Total weight of stiffeners attached to one plate [% of mass of floor plate]
0 – 4000	15
4000-6000	20
6000-6850	30
6850 - →	Additional support structure

Table 5: Amount of stiffeners on floor plate

The combination of the loading scenarios together with the floor pressure scenarios results in a table containing the weight of the new design structure for every combination.

Loading scenario [mT]	Max allowable floor pressure [mT/m²]		
	10	20	40
S50	4810	3330	2650
S100	9750	5870	4390
S200	-	11000	8040
D120	26620*	15210	11110
D160	27850*	16440	12270
D200	-	24090	17040

Table 6: Weight of SS for all scenarios * Additional support is needed for the middle support column

The SSs used to support the 26 product, that forms the product range, represent a total weight of 1491mT. 1157mT is used to support the twenty products that are used to determine the previous mentioned loading scenarios. The new SS (only replaces the first level support) can replace 965mT of supports of the product range. The amount of the three before described cases can be extrapolated to the total amount of stock at HCN, see the table below.

	Supports used for total product range	Supports used for products with complete SSs	Supports used for first level of complete SSs
Product range	1491	1157	965
Extrapolated to total stock	**2600**	**2047**	**1683**

Table 7: Three comparison categories

In Table 8 the total reduction is given for the nine different scenarios. The reduction with respect to the three categories discussed in Table 7 is shown in three different colours. For scenario 3.1 it can be seen that it is not possible to replace the current SSs since the floor plates will become too big (SWL on every point on the beam is 200mT. With a floor pressure of 10mT/m^2 this result in floor plates with dimension: 2x10m).

Floor pressure scenario / **Loading scenario**	**1** 10 mT/m^2	**2** 20 mT/m^2	**3** 40 mT/m^2
1	Scenario 1.1 618mT 37% 31% 24%	Scenario 1.2 1031mT 61% 51% 40%	Scenario 1.3 1186mT 71% 59% 46%
2	Scenario 2.1 486mT 29% 24% 19%	Scenario 2.2 954mT 57% 47% 37%	Scenario 2.3 1116mT 66% 55% 43%
3	Scenario 3.1 -	Scenario 3.2 541mT 32% 27% 21%	Scenario 3.3 862mT 51% 43% 33%

Table 8: Reduction for different scenarios

Conclusion

In conclusion, it is possible to reduce the amount of stock that is needed for the storage of product at the site of Huisman China significant. This reduction is based on two different investigations; investigation of the current floor pressure and designing new support structures that can be used more efficient with respect to weight.

Besides the reduction of stock results the new support structure into several other advantages. The new support structure exists out of one piece, therefore it takes less time to build a complete support structure and to transport one complete support structure. One complete support structure is less heavy in the new situation which means that the SWL of the SPMTs can be used more efficient. Since the new support structures are less heavily it is also more easily to handle them within the workshops.

The floor plates which will be specifically designed to distribute the floor pressure to the floor will distribute the floor pressure more uniformly to the floor than the spreader boards that are used at the moment.

The results obtained for the floor pressure should be checked by a civil bureau before really using the outcomes of the investigation. For the new type support structure multiple other designs are possible and therefore it is recommended to make a following up study based in this report.

Recommendations for Implementation

The investigation about the floor pressure has to be checked by a civil bureau. This doesn't mean however that it is not possible to increase the floor pressure at any location at HCN. For the Workshop the distinction between the local and global max allowable floor pressure is already know.

Implementation of the max allowable local floor pressure according Appendix C will result in a direct reduction of stock. When the findings about the floor pressure are checked by a civil bureau it is still possible to adjust the floor pressure in the workshops according the conclusion of the civil bureau.

At the moment it is still difficult to decide which support structure type is the best. It is therefore recommended to perform a further investigation which will result in a final support structure which has a reduction of stock as result and maybe more advantages.

Next chapter will discuss information/recommendations that can be used in further investigation.

Recommendations for further investigation

The proposal for a new support structure presented in this report is not complete. This report provides a good basis but could use further expansion; further investigation should give more insights in the actual reduction of stock that is possible within HCN.

For further investigation it is recommended to design the support structures with keeping the SPMTs in mind. The costs associated with the SPMTs have, compared with the costs that are associated with the support structures, more impact on HCN. A support structure that will increase the efficiency of the SPMT can therefore maybe lead to a higher reduction of costs.

For the new internship assignment the following topics/requirements have to be taken into account:

1. All the different transportation scenarios (SPMTs located next to each other and in front of each other) result in different type of support structures. In consultation with the Management team of HCN, the Mechanical Engineering department and the workers in the workshop it should be possible to come up with a support structures that will increase the efficiency of the SPMTs and also lead to a significant reduction of stock. These two improvements will lead to significant savings.

2. During the new design phase of the support structures, the validity of the ‘floor pressure report at HCN’ has to be taken into account.

3. In further investigation the possibilities to build up a complete support structure out of separate components has to be evaluated. With use of separate components it is easier to modify the complete support structure with respect to the requirements for the products. Separate components also results in less storage space for the support structures in case they are not in use by HCN.

   Since the mass of the support structure should be kept as low as possible the connection between the different components can cause difficulties.

4. During the design of the support structures attention has to be paid to the floor plates. It must be ensured that the floor plates will distribute the pressure to the floor as uniformly as possible.

A complete internship assignment for further investigation can be found in Appendix H.

References Report

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Appendices

Appendix A	Inventory of stock
Appendix B	Usage of support structures
Appendix C	Local floor pressure workshop
Appendix D	Foundation storage yard and quay side
Appendix E	Loading of Ballast Pontoon
Appendix F	Report about Local floor pressure at HCN
Appendix G	Support Structure Design phase
Appendix H	Internship Assignment

Appendix A Inventory of stock

Support structures on 17-03-2014

Project number	Dimensions (mm) l*w*h	Weight (kg)	On ship	Behind fence	Storage Yard Other side	Hall Other side	Store	Storage Yard this side	High Hall	Low Hall	Total	Total Weight (T)
	Spreader Boards											
A08-00008-00-00B	8000*1200*400	5330	3	40	45			8	12		108	575.64
A08-00012-00-00B	12000*1440*310	12344		28	17			7	9	7	68	839.39
A07-00012-00-00A	12000*1224*524	13433		10	5			6	6		27	362.69
A07-00015-00-00A	15000*1224*524	16790		2	8				1	2	13	218.27
	25080*1836*700	60000						4			4	240.00
	5000*1224*524	5597						2			2	11.19
	Supports											
D00001180-01-100-01	1000*1000*400	425		8	22	6	14	4	1	5	60	25.50
"	1000*1000*500	453		27	45	5	4	7	10	3	101	45.75
"	1000*1000*750	523		25	36	3	14	3	25	9	115	60.15
"	1000*1000*1000	676		14	62	11	12		24	35	158	106.81
"	1000*1000*1250	771		20	35		4		4	4	67	51.66
"	1000*1000*1500	862		58	201	4	6	20	15	18	322	277.56
	New workshop Supports											
A14-26220-11-110A	1000*1000*2000	1634			8						8	13.07
"	1000*1000*2500	1741			10						10	17.41
											Total	2845.10

Costs for material + production + transport is according to Bart Brill, cost estimator: €2,-/kg

Costs Total € 5,690,192.00

Appendix B Usage of support structures

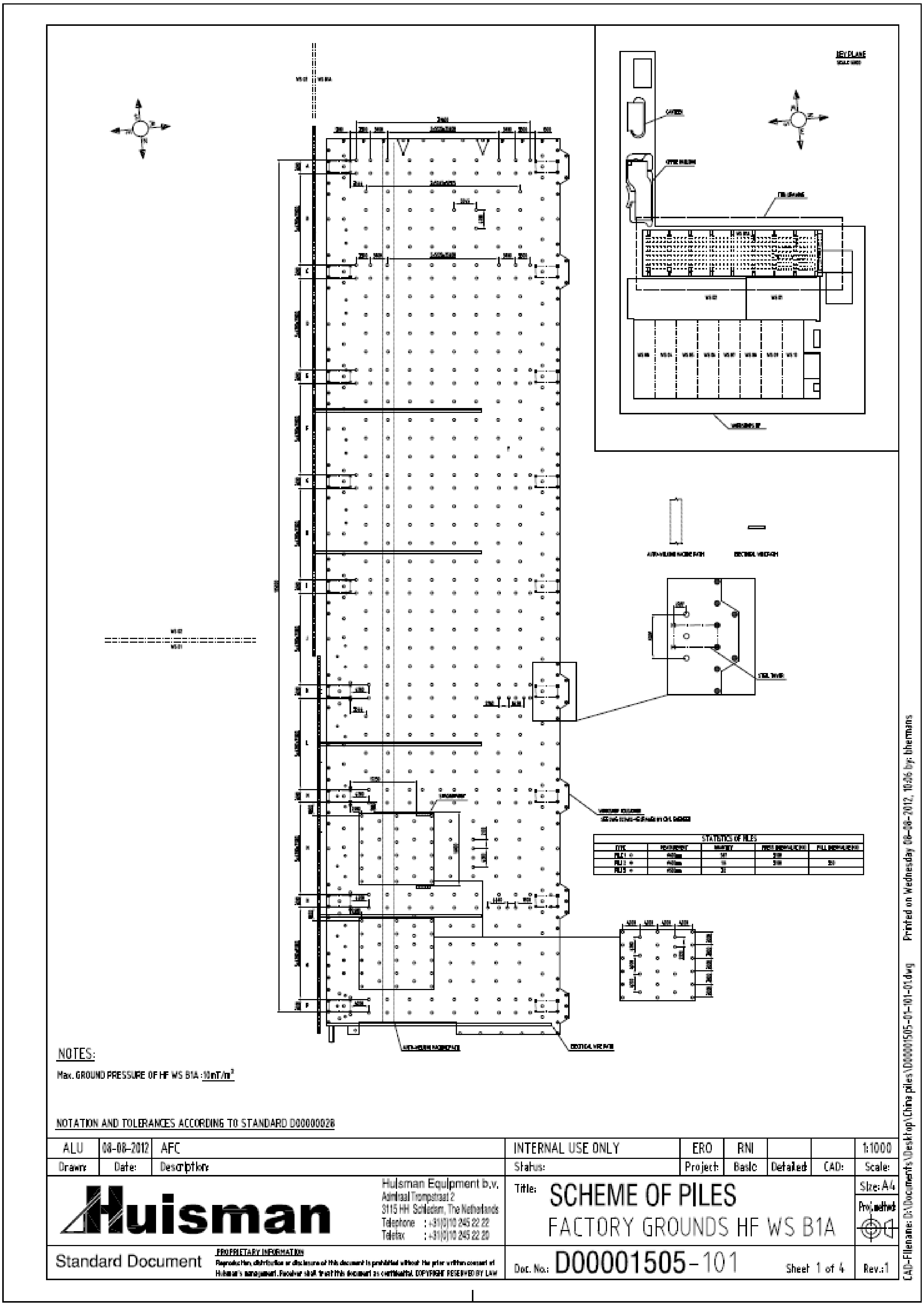
Projects are stored on the Storage Yard or Quay side

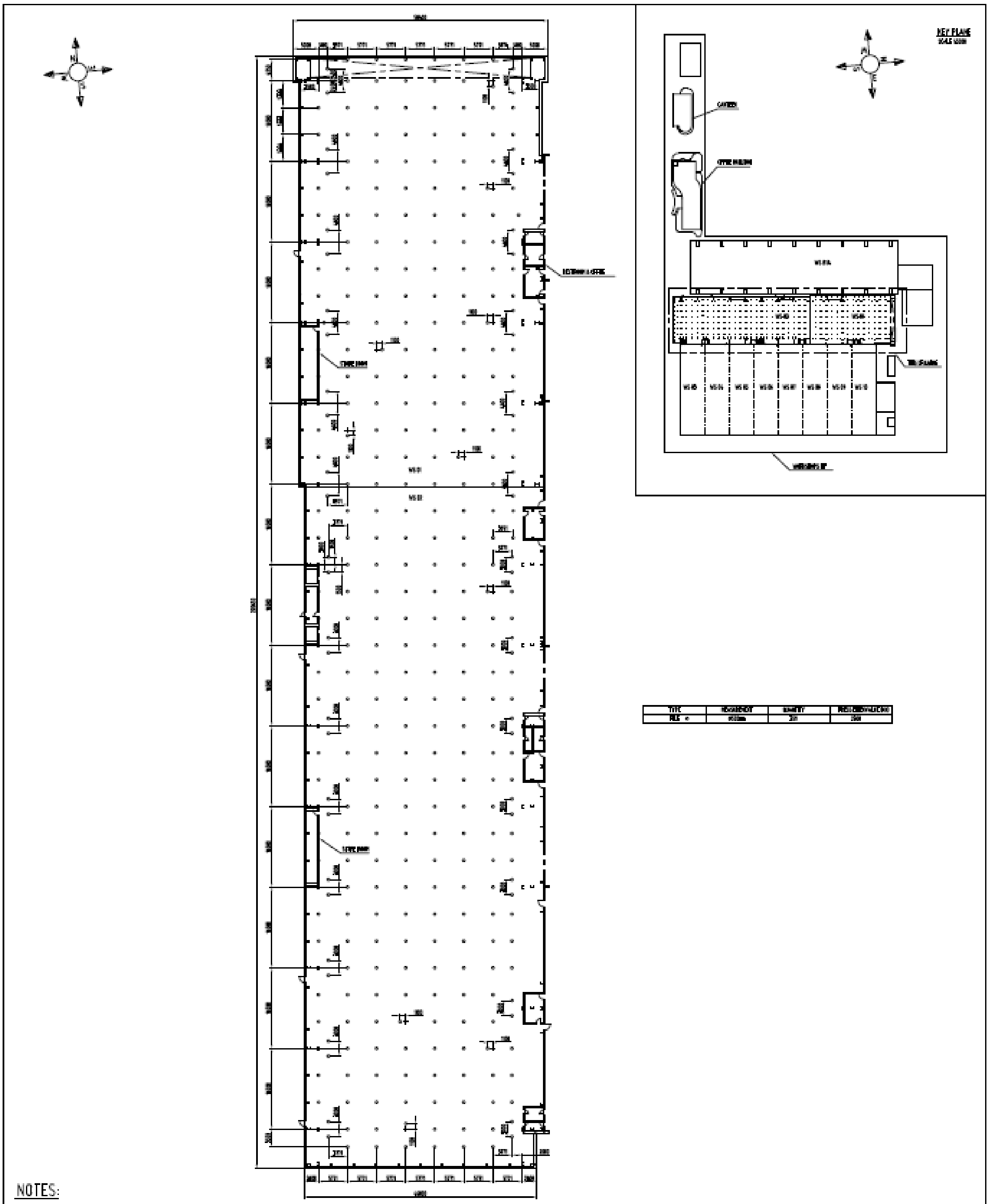
							Front side of stored product								Back side of stored product								
Project	Drawing	Kind of product	Remarks	Own weight of product	Weight of all the used support structures	Weight supports structures vs own weight product	Kind of support used to distribute floor pressure	Contact surface supports and floor	Weight support used to distribute floor pressure	Floor pressure	Weight support used to support product	# of SPMTs used to transport the product	Weight of support that can be replaced with redesigned supports	Redesign support that can substitute first level	Kind of support used to distribute floor pressure	Contact surface supports and floor	Weight support used to distribute floor pressure	Floor pressure	Weight support used to support product	# of SPMTs used to transport the product	Weight of support that can be replaced with redesigned supports	Redesign support that can substitute first level	Support not used in first layer support
				[mT]	[mT]	[%]	[-]	[m^2]	[mT]	[mT/m^2]	[mT]	[-]	[mT]	[-]	[-]	[m^2]	[mT]	[mT/m^2]	[mT]	[-]	[mT]	[-]	
A13-42000 400mt OMC (12m jib) - Ceona	38-030	Complete		359.0	81.0	22.6%	2x 8m	19.2	10.7	8.0	10.1	1	17.53	100T	3x 8m	28.8	16.0	10.0	43.5	2	35.3	D120T	28.2
A12-41000 400mt OMC - Helix																							0.0
																							0.0
A13-43000 400mt PMC - GMS	38-110A	Boom		80.0	6.0	7.5%	2x 100cm with	2	3.1	26.5		1		50T	2x 100cm with 2x	2	3.1	16.5				50T	
	38-121A	Boom part 1	Not supported in the correct way. Floor pressure not distributed and SPMT direct in contact with product	28.0	4.3	15.4%	2x 100cm on top	1	1.4	19.5		1		50T	2x 100cm with 2x 150cm on	2	3.1	7.0				50T	
	38-121A	Boom part 2		50.0	6.2	12.3%	2x 150cm with 2x 150cm on top	2	3.4	13.0		1		50T	2x 75cm with 2x 150cm on top	2	2.8	15.0				50T	
	38-130A	Crane		200.0	24.0	12.0%	2x 8m	19.2	10.7	12.0	13.744	1	24.4	2 x 100T				12.0					
																							0.0
A13-45000 300mt PMOC - Huisman	38-010A	Crana		131.0	40.0	30.5%	2x 12m HD	29.376	26.2	6.0	15.836	1	40.606	50T				6.0				100T	
A12-48000 300mt PMOC - Huisman	38-040A	Pedestal		27.0	16.0	59.3%	4x 150cm with 4x 40cm on	4	5.1	11.0	10.66	1	7.63	50T				11.0			7.63	50T	0.7
	38-060A	Boom		60.0	24.0	40.0%	2x 8m	19.2	10.7	5.0	15.808	1	24.72	50T				5.0				50T	
	38-061A	Boom part		28.0	36.0	128.6%	2x 8m	19.2	10.7		8.1	1	17.69	50T	2x 8m	19.2	10.7		7.904		17.69	50T	0.6
	38-061A	Boom part		32.0	36.0	112.5%	2x 8m	19.2	10.7		7.054	1	17.69	50T	2x 8m	19.2	10.7		7.904		17.69	50T	0.6
																							0.0
A13-48000 900mt Pedestal Mounted Crane - DOF	38-130A	Crane House		300.0	38.0	12.7%	2x 8m	19.2	10.7	9.0	8.224	1	17.69	100T	2x 8m	19.2	10.7	9.0	8.224	1	17.69	100T + 8m spreader or 200T	2.6
	38-210A	Boom		123.0	40.0	32.5%	2x 8m	19.2	10.7	6.0	9.576	1	17.69	50T	2x 8m	19.2	10.7	3.0	7.722	1	17.69	100T	4.6
	38-350A	Pedestal		180.0	57.0	31.7%	2x 12m HD	29.376	26.2	6.0	30.984	1	56.812	D120T				6.0				D120T	0.2
																							0.0
A12-40000 2x 2000mt Sheerleg Crane - STX Offshore	38-220A	Backmast		133.0	49.5	37.2%	2x 8m	19.2	10.7	4.0	14.108	1	17.69	50T	2x 8m	19.2	10.7	4.0	14.108	1	17.69	50T	14.1
	38-250A	A-Frame+Skit system+Preventor		583.5	108.0	18.5%	3x 8m	28.8	16.0	8.5	38.172	2	35.33	D120T	3x 8m	28.8	16.0	15.5	38.172	2	36	D120T + 15m spreader or D200T	36.7
																							0.0
A12-55000 2200mt OMC - EZRA	38-150C	Mast base	SPMT cannot drive below product. Product is stored at quayside with	780.0	53.0	6.8%	2x 12m	34.56	24.7	14.0					4x 8m	38.4	21.3	9.0					
	38-330A	Boom		334.0	113.0	33.8%	2x 8m	19.2	10.7	10.6	47.02	1	54.19	D120T	3x 8m	28.8	16.0	8.5	42.41	2	38.428	2 x 100T	20.4
	38-360A	Complete Mast	SPMT cannot drive below product. Product is stored at quayside with use of Skyhook.	1060.0	56.0	5.3%	2x 12m	34.56	24.7	11.6		2			2x 12m	34.56	24.7	24.3	21.32	3			
																							0.0
A11-42000 1500mt HLMC - Kahn	38-030B	Complete		546.3	85.0	15.6%	3x 8m	28.8	16.0	11.5	26.235	2	35	D160T	3x 8m	28.8	16.0	10.4	24.143	2	35	D160T	15.0
	38-031A	Mast		250.0	37.0	14.8%	2x 8m	19.2	10.7	6.0	8.9	1	17.69	100T	2x 8m	19.2	10.7	9.0	6.872	1	17.69	100T	1.6
	38-032A	Boom		120.0	38.0	31.7%	2x 8m	19.2	10.7	6.8	7.8	1	17.69	100T	2x 8m	19.2	10.7	1.5	7.778	1	17.69	50T	2.6
																							0.0
A11-44000 850mt OMC - Allseas	38-016A	Lower mast		540.0	74.0	13.7%	6x 8m	57.6	32.0	10.6	42.3	2	74.62	D160T				10.6		2		D160T	
	38-033A	Boom		148.0	41.0	27.7%	2x 8m	19.2	10.7	4.5	13.6	1	17.69	100T	2x 8m	19.2	10.7	5.4	9.074	1	17.69	100T	5.6
	38-036A	Complete Crane	Three SPMTs support structure is used.	910.0	208.0	22.9%	4x 12m	69.1	49.4	5.9	74.3	3		Tripple	4x 12m	69.12	49.4	7.3	31.372	3		Tripple	
																							0.0
A10-46000 3000mt OMC - EZRA	38-26C	Boom		420.0	107.0	25.5%	3x 8m	28.8	16.0	18.6	52.4	2	41.93	2x D120T	2x 8m	19.2	10.7	18.6	26.6	1	24.4	2x 100T	40.7
																							0.0
A07-66000 1800t HLMC - Sea Trucks	38-31A	Boom		334.0	113.0	33.8%	2x 8m	19.2	10.7	7.2	47.2	1	54.01	D120T	3x 8m	28.8	16.0	8.1	41.4	2	38.428	2x 100T	20.6
		Total		**7756.8**	**1491.0**	**19.2%**			**382.7**	**10.1**	**502.1**	**32.0**	**608.3**				**280.2**	**9.7**	**338.5**	**25.0**	**356.7**		**194.8**
		Percentage of total support weight [%]			100%				25.7%		33.7%		41%				18.8%		22.7%		24%		13%

Projects are stored on the Storage Yard or Quay side

								Front side of stored product								Back side of stored product								
	Project	Drawing	Kind of product	Remarks	Own weight of product	Weight of all the used support structures	Weight supports structures vs own weight product	Kind of support used to distribute floor pressure	Contact surface supports and floor	Weight support used to distribute floor pressure	Floor pressure	Weight support used to support product	# of SPMTs used to transport the product	Weight of support that can be replaced with redesigned supports	Redesign support that can substiture first level	Kind of support used to distribute floor pressure	Contact surface supports and floor	Weight support used to distribute floor pressure	Floor pressure	Weight support used to support product	# of SPMTs used to transport the product	Weight of support that can be replaced with redesigned supports	Redesign support that can substiture first level	Support not used in first layer support
					[mT]	[mT]	[%]	[-]	[m^2]	[mT]	[mT/m^2]	[mT]	[-]	[mT]	[-]	[-]	[m^2]	[mT]	[mT/m^2]	[mT]	[-]	[mT]	[-]	
1	A13-42000 400mt OMC (12m jib) - Ceona	38-030	Complete		359.0	81.0	22.6%	2x 8m	19.2	10.7	8.0	10.1	1	17.53	100T	3x 8m	28.8	16.0	10.0	43.5	2	35.3	D120T	28.2
	A12-41000 400mt OMC - Helix																							0.0
																								0.0
2	A13-43000 400mt PMC - GMS	38-110A	Boom		80.0	6.0	7.5%	2x 100cm with	2	3.1	26.5		1		50T	2x 100cm with 2x	2	3.1	16.5				50T	
3		38-121A	Boom part 1	Not supported in the correct way. Floor pressure not distributed and SPMT direct in contact with product	28.0	4.3	15.4%	2x 100cm on top	1	1.4	19.5		1		50T	2x 100cm with 2x 150cm on	2	3.1	7.0				50T	
4		38-121A	Boom part 2		50.0	6.2	12.3%	2x 150cm with 2x 150cm on top	2	3.4	13.0		1		50T	2x 75cm with 2x 150cm on top	2	2.8	15.0				50T	
5		38-130A	Crane		200.0	24.0	12.0%	2x 8m	19.2	10.7	12.0	13.744	1	24.4	2 x 100T				12.0					
																								0.0
6	A13-45000 300mt PMOC - Huisman	38-010A	Crana		131.0	40.0	30.5%	2x 12m HD	29.376	26.2	6.0	15.836	1	40.606	50T				6.0				100T	
7	A12-48000 300mt PMOC - Huisman	38-040A	Pedestal		27.0	16.0	59.3%	4x 150cm with 4x 40cm on	4	5.1	11.0	10.66	1	7.63	50T				11.0			7.63	50T	0.7
8		38-060A	Boom		60.0	24.0	40.0%	2x 8m	19.2	10.7	5.0	15.808	1	24.72	50T				5.0				50T	
9		38-061A	Boom part		28.0	36.0	128.6%	2x 8m	19.2	10.7		8.1	1	17.69	50T	2x 8m	19.2	10.7		7.904		17.69	50T	0.6
0		38-061A	Boom part		32.0	36.0	112.5%	2x 8m	19.2	10.7		7.054	1	17.69	50T	2x 8m	19.2	10.7		7.904		17.69	50T	0.6
																								0.0
11	A13-48000 900mt Pedestal Mounted Crane - DOF	38-130A	Crane House		300.0	38.0	12.7%	2x 8m	19.2	10.7	9.0	8.224	1	17.69	100T	2x 8m	19.2	10.7	9.0	8.224	1	17.69	100T + 8m spreader or 200T	2.6
2		38-210A	Boom		123.0	40.0	32.5%	2x 8m	19.2	10.7	6.0	9.576	1	17.69	50T	2x 8m	19.2	10.7	3.0	7.722	1	17.69	100T	4.6
3		38-350A	Pedestal		180.0	57.0	31.7%	2x 12m HD	29.376	26.2	6.0	30.984	1	56.812	D120T				6.0				D120T	0.2
																								0.0
4	A12-40000 2x 2000mt Sheerleg Crane - STX Offshore	38-220A	Backmast		133.0	49.5	37.2%	2x 8m	19.2	10.7	4.0	14.108	1	17.69	50T	2x 8m	19.2	10.7	4.0	14.108	1	17.69	50T	14.1
5		38-250A	A-Frame+Skit system+Preventor		583.5	108.0	18.5%	3x 8m	28.8	16.0	8.5	38.172	2	35.33	D120T	3x 8m	28.8	16.0	15.5	38.172	2	36	D120T + 15m spreader or D200T	36.7
																								0.0
6	A12-55000 2200mt OMC - EZRA	38-150C	Mast base	SPMT cannot drive below product. Product is stored at quayside with																				
7		38-330A	Boom		334.0	113.0	33.8%	2x 8m	19.2	10.7	10.6	47.02	1	54.19	D120T	3x 8m	28.8	16.0	8.5	42.41	2	38.428	2 x 100T	20.4
8		38-360A	Complete Mast	SPMT cannot drive below product. Product is stored at quayside with use of Skyhook.																				
																								0.0
9	A11-42000 1500mt HLMC - Kahn	38-030B	Complete		546.3	85.0	15.6%	3x 8m	28.8	16.0	11.5	26.235	2	35	D160T	3x 8m	28.8	16.0	10.4	24.143	2	35	D160T	15.0
0		38-031A	Mast		250.0	37.0	14.8%	2x 8m	19.2	10.7	6.0	8.9	1	17.69	100T	2x 8m	19.2	10.7	9.0	6.872	1	17.69	100T	1.6
:1		38-032A	Boom		120.0	38.0	31.7%	2x 8m	19.2	10.7	6.8	7.8	1	17.69	100T	2x 8m	19.2	10.7	1.5	7.778	1	17.69	50T	2.6
																								0.0
2	A11-44000 850mt OMC - Allseas	38-016A	Lower mast		540.0	74.0	13.7%	6x 8m	57.6	32.0	10.6	42.3	2	74.62	D160T				10.6		2		D160T	
3		38-033A	Boom		148.0	41.0	27.7%	2x 8m	19.2	10.7	4.5	13.6	1	17.69	100T	2x 8m	19.2	10.7	5.4	9.074	1	17.69	100T	5.6
4		38-036A	Complete Crane	Three SPMTs support structure is used.																				
																								0.0
5	A10-46000 3000mt OMC - EZRA	38-26C	Boom		420.0	107.0	25.5%	3x 8m	28.8	16.0	18.6	52.4	2	41.93	2x D120T	2x 8m	19.2	10.7	18.6	26.6	1	24.4	2x 100T	40.7
																								0.0
6	A07-66000 1800t HLMC - Sea Trucks	38-31A	Boom		334.0	113.0	33.8%	2x 8m	19.2	10.7	7.2	47.2	1	54.01	D120T	3x 8m	28.8	16.0	8.1	41.4	2	38.428	2x 100T	20.6
			Total		**5006.8**	**1174.0**	**23.4%**			**284.0**	**10.0**	**427.8**	**27.0**	**608.3**				**184.8**	**9.1**	**285.8**	**19.0**	**356.7**		**194.8**
			Percentage of total support weight [%]			100%				24.2%		36.4%		52%				15.7%		24.3%		30%		17%

Appendix C Local floor pressure workshop





NOTES:

Max. GROUND PRESSURE OF HF WS B1A : 10mT/m²

NOTATION AND TOLERANCES ACCORDING TO STANDARD D00000028

ALU	08-08-2012	AFC	INTERNAL USE ONLY	ERO	RNI			1:1000
Drawn:	Date:	Description:	Status:	Project:	Basic	Detailed	CAD:	Scale:

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Title: SCHEME OF PILES
FACTORY GROUNDS HF WS 01&02

Size: A4

Proj. method

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Doc. No.: D00001505-101 Sheet 2 of 4 Rev.:1

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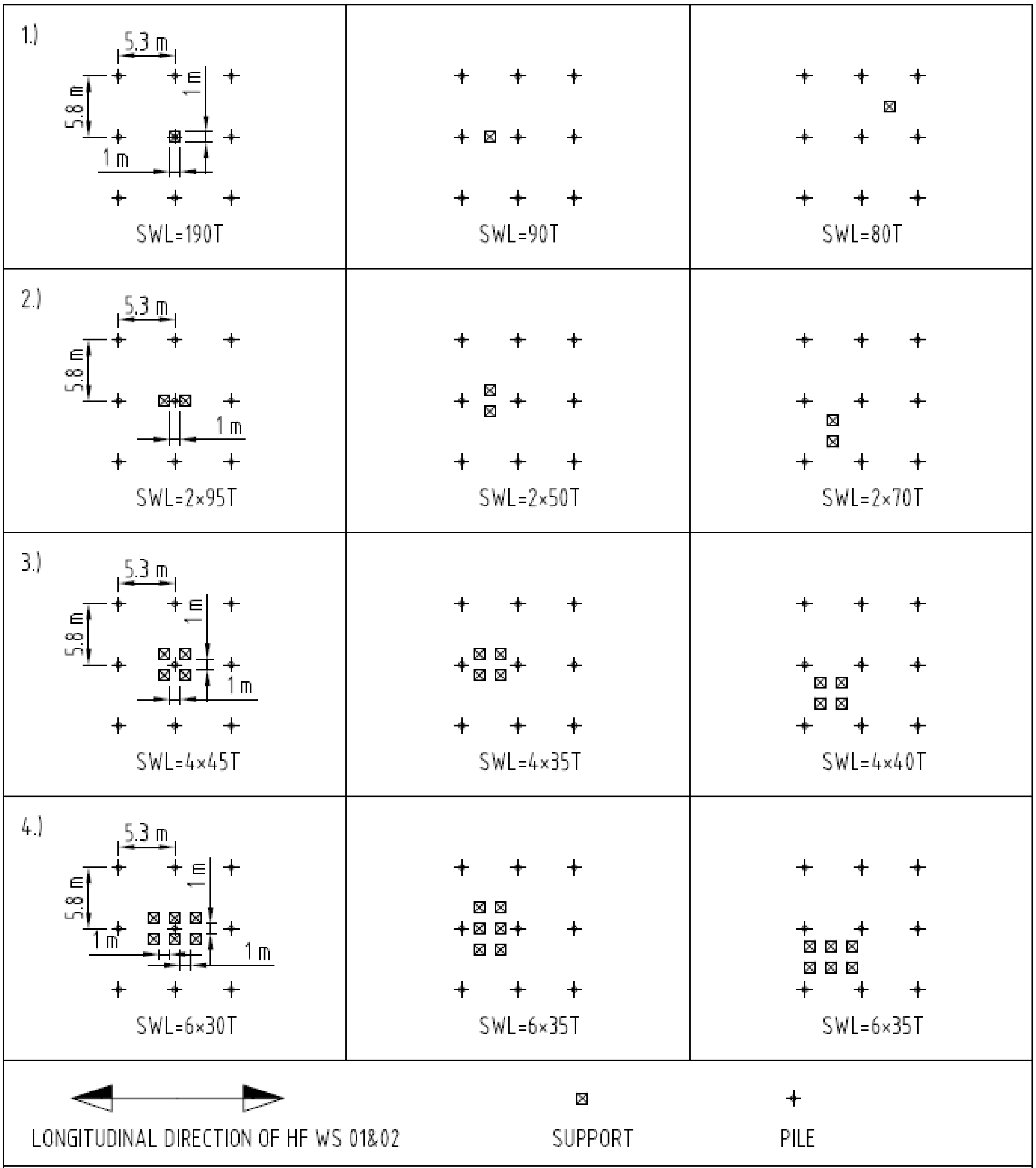
CAD-Filename: D:\Documents\Desktop\China piles\D00001505-01-101-02.dwg



NOTE:

1. MASS OF SUPPORTS IS NOT TAKEN IN CALCULATION
2. NO OTHER LOADS CAN BE PLACED WITHIN THE AREA 10m X 10m OF THE LOAD.

NOTATION AND TOLERANCES ACCORDING TO STANDARD D00000028

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Title: MAX FLOOR LOAD SUPPORTS
HF WS 01&02

Size: A4
Proj. method

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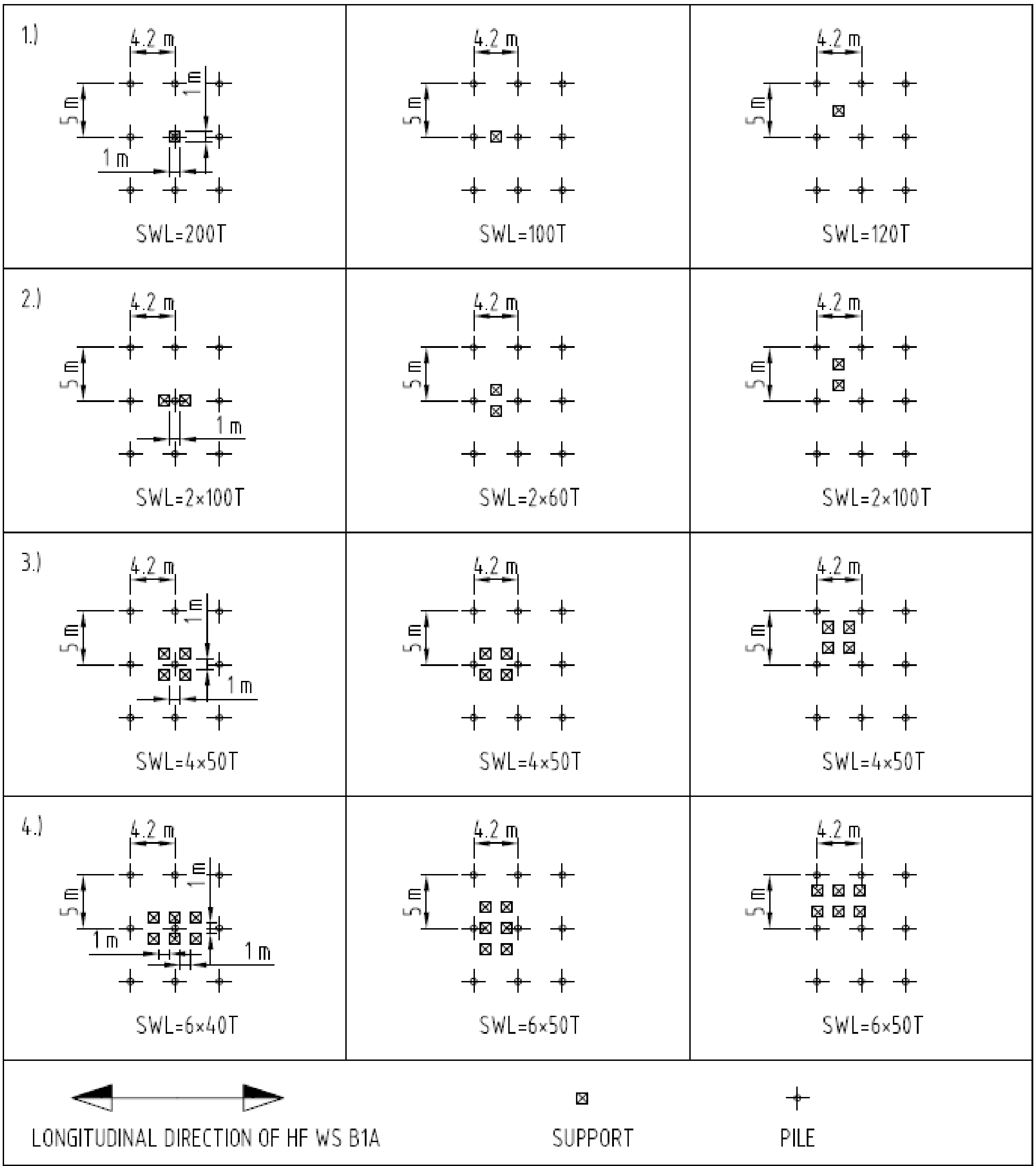
Doc. No.: D00001505-101 Sheet 3 of 4 Rev.:1

CAD-Filename: D:\Documents\Desktop\China piles\D00001505-01-101-03.dwg Printed on Wednesday 08-08-2012, 10:13 by: bhermans



NOTE:

1. MASS OF SUPPORTS IS NOT TAKEN IN CALCULATION
2. NO OTHER LOADS CAN BE PLACED WITHIN THE AREA 10m X 10m OF THE LOAD.

NOTATION AND TOLERANCES ACCORDING TO STANDARD D00000028

Drawn:	Date:	Description:	Status:	Project	Basic	Detailed	CAD:	Scale:
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Title: MAX FLOOR LOAD SUPPORTS
HF WS B1A

Size: A4

Projection:

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Appendix D Foundation storage yard and quay side

The foundation of the storage yard Phase III exists out of a detritus layer with cast-in-situ concrete structures on top. The cast-in-situ structures have dimensions 5000x5000x250mm. The compressive strength of the concrete has a value of 30MPa and a flexural strength of 3.0MPa. There are no reinforcements used. The cast-in-situ concrete structures on storage yard Phase I have dimensions 5000x5000x200mm and have a compressive strength of 25MPa.

The foundation of the quay side exists out of a layer of detritus followed by a lean concrete layer with a precast concrete block C50 on top. Main function of the lean concrete layer is to ensure a good distribution of the loads. The lean concrete has a compressive strength of 15MPa. The compressive strength of the detritus is for both the quay side and the storage yard equal to about 120KPa.

What is missing in the section view of the quayside is a sand layer of about 5cm between the precast concrete block and the lean concrete layer.

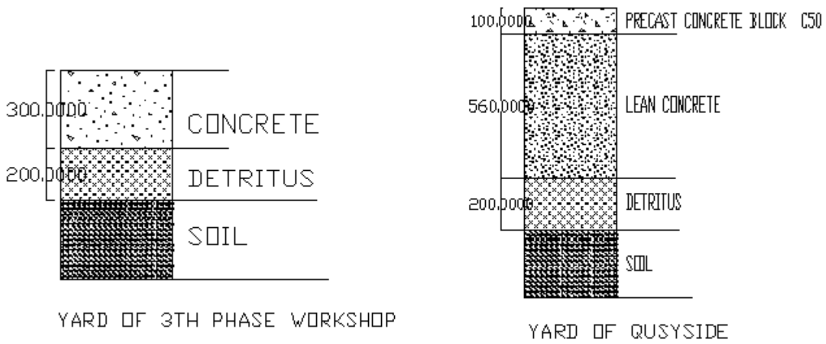

Figure 23: Foundation storage yard and quayside

Appendix ELoading of Ballast Pontoon

When using a number of assumptions it is possible to make a small model where the max local force deck pressure is calculated. For the ballast pontoon of HCN this model is build up in Excel. Since the deck construction of the transport barges are always more or less the same, the results of the model should also give an idea about the possibility of making distinction between local and global deck pressure for other transport barges.

For the Ballast Pontoon, the longitudinal frame spacing is equal to 600mm and the transverse web spacing is equal to 2400mm. The Pontoon is split up into nine different tanks. Calculations are based on a small section of the deck above the tank located in the middle. Simplified reproduction of the pontoon is given in Figure 24. Figure 25 shows the 3D section of the deck that is used for the model. In this section is L equal to the width of the tank and also the length of the section, which is 6000mm and W is the width of the section of the deck that is taken into account. Because the web spacing is 2400mm, this is also the maximum value for W.

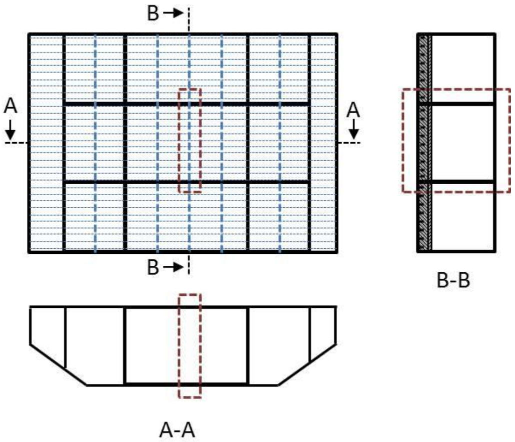

Figure 24: Simplified reproduction of the Ballast Pontoon. Cross section of the Side view is displayed without stiffeners and webs.

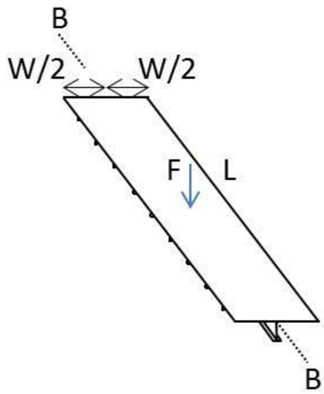

Figure 25: 3D section of the deck.

There are two loading situations (Figure 26); situation one where the walls of the tank are assumed to be rigid. In this situation it can be assumed that the web is clamped. For the second situation it is assumed that the walls have certain stiffness and therefore the web is assumed to be imposed. For both situations simple beam calculations are performed.

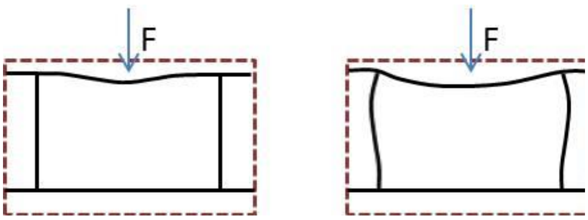

Figure 26: Loading situations of the deck. Rigid tank left, stiffness tank right.

With knowing the materials and dimensions of the web, the stiffeners and the plate it is possible to calculate the max point load F. Dimensions of the deck construction are given in the table below. The yield stress of the steel used is equal to 235Mpa.

Deck plate		Thickness [mm]			
		15			
Longitudinal frame:	210·15	Web height [mm]		Thickness [mm]	
		210		15	
Transverse web fr20:	700·10-250·20	Web height [mm]	Thickness [mm]	Flange width [mm]	Thickness [mm]
		700	10	250	20

Table 9: Dimensions of deck construction

First step is to determine the center of gravity of the deck. The CoG depends on W, the width of the section that is taken into account. When the CoG is known and also the neutral axis it is possible to determine the maximal point load. For the first situation this is given by:

$$F_{max\ clamped} = \frac{\sigma \cdot I \cdot 8}{L \cdot y}$$

Error! Use the Home tab to apply Kop 7;Appendix to the text that you want to appear here.-1

For the second situations is the force given by:

$$F_{max\ imposed} = \frac{\sigma \cdot I \cdot 4}{L \cdot y}$$

Error! Use the Home tab to apply Kop 7;Appendix to the text that you want to appear here.-2

where
- σ is the bending stress
- *I* is the moment of inertia about the neutral axis
- *L* is the length of the section
- *y* is distance to the neutral axis in which the stress is calculated

For different values of W the forces for both the situations are given in table below together with the max load on that section according to the global deck pressure of 10mT/m^2.

W [mm]	$\boldsymbol{F_{max\ clamped}}$ [mT]	$\boldsymbol{F_{max\ imposed}}$ [mT]	$\boldsymbol{F_{global\ on\ section}}$ [mT]
250	113	56	15
500	128	64	30
750	135	67	45
1000	139	69	60
1250	142	71	75
1500	143	72	90
1750	145	72	105
2000	146	73	120
2250	147	73	135
2400	147	73	144

Appendix F Report about Local floor pressure at HCN

Huisman is a company operating globally with extensive experience in the design and manufacturing of heavy construction equipment for world's leading on and offshore companies. The facility in Fujian Province (Xiamen area) in China is founded to facilitate customers in this region and to increase the overall production capacity. The production facility has been fully operational since April 2007 and delivers a significant contribution to the overall Huisman engineering and production capacity. Huisman China (HCN) is a production facility with a total area of 284,000 m^2. Part of the terrain is the first quayside owned by an international company in China that is opened up in 2012. This quayside is specially constructed by Huisman to increase production area and be self-reliant for loading and unloading.

Owing to the opening of the new quayside in 2012 a large storage area is obtained. In combination with requests of clients to store their products passed the delivery date, the amount of products currently stored have rapidly increased. During production and storage of the products, additional support structures (SS) are needed which results in a big stock. This stock exists out of two types of SS; workshop supports (WSS) and spreader boards (SB). The use of these two types of SS can be split up into mainly two categories; distribution of the floor pressure and supporting of the product. At the moment almost 1250mT of steel is used to distribute the pressures to the floor. HCN assumes that the max allowable global floor pressure is equal to the max allowable local floor pressure. This global floor pressure is determined by a local design institute; “Fujian Architectural & Light Textile Design Institute” and is based on a soil survey, ‘Report for geology survey of dream hall’ [1] performed at the quayside. (The soil survey of the soil beneath the storage yard is not available. Therefore it is assumed that the soil beneath the quayside and storage yard is the same. The foundation on top of the upper soil layer is not the same for the storage yard and quay side.) During this survey samples of all the different ground layers are taken. For all the layers, different tests are performed to determine the properties of that layer. When the results are known, the max allowable floor pressure is determined.[3] The tests and calculations that are necessary to determine the max allowable floor pressure are according the Chinese code “Load code for the design of building structures” [13].

To be able to determine the correctness of the max allowable local floor pressure used at the moment first the foundations of the SYIII and QS are discussed. After that two models, one model based on footing design and one model based on slab-on-grade pavement design, will be constructed. These models can calculate the max allowable local floor pressures at both the areas at the site of HCN. The limitations of the models will be discussed afterwards followed by the results.

Foundation Storage Yard Phase III and Quayside

The foundation of the SYIII exists out of a detritus layer with cast-in-situ concrete structures on top. The cast-in-situ structures have dimensions 5000x5000x250mm (the thickness of 300mm in the figure in not correct). The compressive strength of the concrete has a value of 30MPa and a flexural strength of 3.0MPa. There are no reinforcements used.

The foundation of the QS exists out of a layer of detritus followed by a lean concrete layer with a precast concrete block C50 on top. Main function of the lean concrete layer is to ensure a good distribution of the loads. The lean concrete has a compressive strength of 15MPa. The compressive strength of the detritus is for both the quay side and the storage yard equal to about 120KPa.

What is missing in the section view of the QS is a sand layer of about 5cm between the precast concrete block and the lean concrete layer.

[3] The design institute wasn't willing to provide information about the calculations report of the foundation because that is classified information.

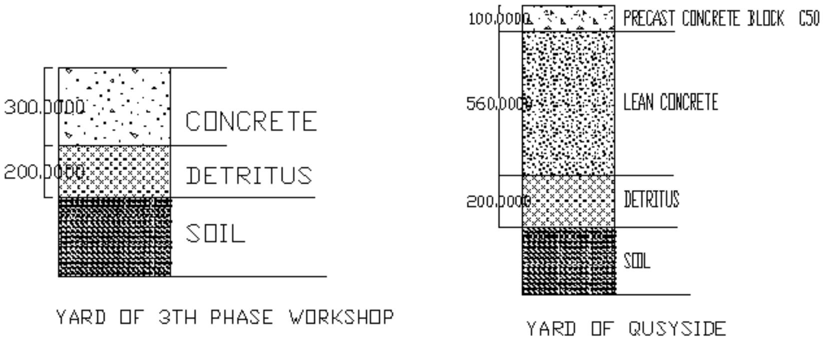

Figure 27: Foundation storage yard phase III and quayside

Calculation Models

The model used to determine the punch shear and beam shear is based on footing calculations. The cast-in-situ structure represents a square footing which is concentrically loaded by a support structure under loading. This support structure can be seen as the column. [14]

To determine the bending stresses in the concrete, a model based on Westergaard's stress equations is used. [15] Westergaard's equations are used in slab-on-grade pavement design. It calculates the max allowable wheel load. This report is however about floor pressure due to loaded supports and not due to wheel loads. That's why this model provides only an approximation.

When the footing/ground slab is concentrically loaded and a uniform distribution of soil pressure is assumed, the max local allowable floor pressure is equal to:

$$q_{local} = A * q_{all}$$

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where A is the surface area of the ground slab and q_{all} is the max global allowable floor pressure. Filling in the dimensions of the footing and the global floor pressure, results in the possible allowable local floor pressure on top of the footing. If this local floor pressure is applicable depends, besides the bearing pressure of the soil, also on stresses in the concrete footing, the surface area on which the local floor pressure acts and the position where the local pressure engages to the footing.

Punch and beam shear

In case of a local floor pressure on a small surface area of the footing instead of the max allowable global floor pressure over the entire footing, large soil pressure is present under the footing which leads to high shear stresses. The shear capacity of the concrete must equal or exceeds the critical shear force produced by the local floor pressure

$$V_u \leq \Phi V_c$$

Error! Use the Home tab to apply Kop 7;Appendix to the text that you want to appear here.-4 where Φ is the strength reduction factor. [4] This is the case for both the punching shear force and the beam shear force.

The model that will be analysed is given in Figure 28. C_1 and C_2 are the dimensions of the ground surface of the support, L, B and h are the dimensions of the footing, d is the effective depth of the footing and P_d is the dead load which is applied to the support.

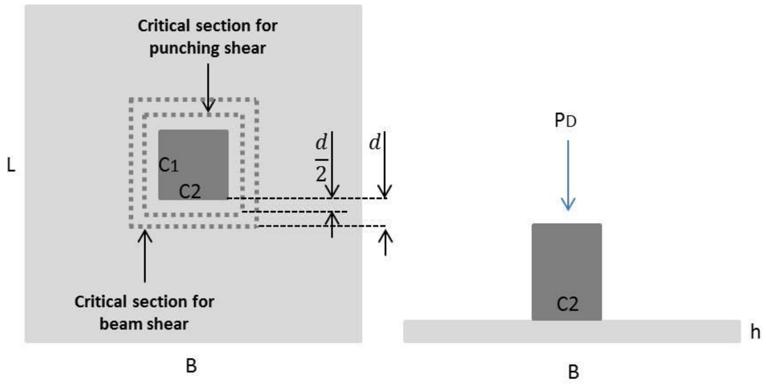

Figure 28: Critical shear sections

The critical section for punching shear is located at distance d/2 from the support structure. The effective depth is equal to the thickness of the slab minus 75mm. [5] The critical punching shear force can be evaluated using the following equation:

$$V_u = q_{all}[L * B - (C_1 + d)(C_2 + d)]$$

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Punching shear force resisted by concrete V_c is given as the smallest of:

$$V_c = 0.53\sqrt{f_c'}\left(1 + \frac{2}{\beta}\right)\lambda b_o d$$

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$$V_c = \lambda\sqrt{f_c'}\, b_o d$$

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[4] (The ACI code (2005) requires reduction factors of 0.85 for sand lightweight aggregate concrete (SLWAC) and 0.75 for all light weight concrete (ALWAC).)
[5] According to ACI Code 15.7, depth of footing above reinforcement is not to be less than 15cm for footings on soil. Noting that 7.5cm of clear concrete cover is required if concrete is cast against soil.

appear here.-7

$$V_c = 0.27\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\, b_o d$$

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where

- f'_c is the compressive strength of the concrete used after 28days
- β is the length width ratio of the concrete structure
- λ is the factor accounting for concrete density
- b_o is the perimeter of the critical section for punching shear
- α_s is a parameter [6] equal to 40

As is said before, the shear capacity of the concrete must equal or exceeds the critical shear force produced by the local floor pressure.

Besides punching shear, beam shear can also form a problem. The critical section for beam shear is located at distance d from the support structure. The beam shear force is given by:

$$V_u = q_{all}\, B\left[\left(\frac{L - C_2}{2}\right) - d\right]$$

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for shear in the B direction and:

$$V_u = q_{all}\, L\left[\left(\frac{B - C_1}{2}\right) - d\right]$$

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for shear in the L direction. Same can be done for the beam shearing force resisted by concrete, for the B direction:

$$V_c = 0.53\sqrt{f'_c}\, Bd$$

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[6] These equations are normally used for footings. In case of interior footings parameter is equal to 40, for edge and corner footings the parameter is 30 and 20.

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L direction is:

$$V_c = 0.53\sqrt{f_c'}\,Ld$$

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For $V_u \leq \Phi V_c$, there is no problem.

Bearing strength

All the forces applied at the support structures must be transferred to the soil by bearing on concrete. Bearing on concrete for the footing may not exceed the concrete bearing strength. The footing could fail by crushing the concrete under the support structure. The bearing capacity is:

$$\Phi P_n = \Phi(0.85\,f_c' A_1)\sqrt{\frac{A_2}{A_1}} \leq 2.0\Phi(0.85\,f_c' A_1$$

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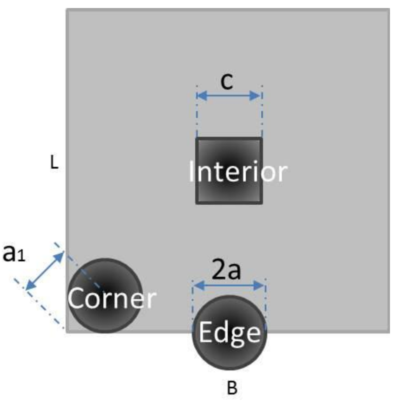
where

- f'_c is the compressive strength of the concrete used after 28days
- A_1 is the cross-sectional area of the contact surface between the support structure and the slab
- A_2 is the area of the lower base of the largest frustum of a pyramid having for tis upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal

When the bearing strength is however bigger than the local load it doesn't matter if the bearing strength is not according to equation 6-11.

Bending stress

Now the beam shear and punch shear are discussed it is time to have a look at the bending stress in the concrete. The model is based on Westergaard's stress equations. Westergaard derived a lot of equations to determine the maximum bending stress for three different loading conditions (interior, edge and corner), see Figure 29. Over the years, also additions and simplifications are made to make it more easily to use the equations. The equations derived by Westergaard are used to design slab-on-grade pavement. It is assumed that the slab is of infinite or semi-infinite dimensions. The equations are based on wheel load applied to the slabs, for wheel loads it can be assumed that the pressure is uniformly distributed over the area of a small circle with radius a.



Besides the uniformly distributing of the pressure over a circle area for all the three loading conditions, there is also an equation for the uniformly distributing of the pressure over a square in case of interior loading. In the following it is shown that simplified Westergaard's equations (all with a circular pressure distribution) approach the elaborated Westergaard's equations [15], [16], [17].

Figure 29: Loading conditions elaborated Westergaard's equations

Interior loading

The interior loading conditions are valid in case the support is located at a "considerable distance from the edges". When the load is applied some distance from the edges of the slab, the critical stress in the concrete will be in tension at the bottom surface. This tension is greatest directly under the centre of the loaded area.

Maximum bending stress in case of the ordinary theory for a square is equal to

$$\sigma_i = \frac{3P(1+\mu)}{2\pi h^2} * (\ln\left(\frac{2l}{c'}\right) + 0.5 - \gamma) + \frac{3P(1+\mu)}{64h^2}\left(\frac{c'}{l}\right)^2$$

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where

- P is the applied load
- E is the young's modulus
- μ is the Poisson's ratio, which is equal to 0.15
- h is the thickness
- k is the modulus of subgrade reaction
- c is the side length of square load
- l is the radius of relative stiffness equal to:

$$L_{rs} = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}}$$

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- c' is $(e^{\frac{\pi}{4}-1}/2^{0.5})c$, which is equal to 0.573804c.
- γ is Euler's constant

In case the square area is approached by a circular area with radius a, the equation becomes:

$$\sigma_i = \frac{3P(1+\mu)}{2\pi h^2} * (\ln\left(\frac{2l}{a}\right) + 0.5 - \gamma) + \frac{3P(1+\mu)}{64h^2}\left(\frac{a}{l}\right)^2$$

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where a is equal to:

$$a = \sqrt{\frac{c^2}{\pi}}$$

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Maximum bending stress according to the simplified Westergaard's calculation, which is also based on a circular area, is equal to:

$$\sigma_i = \frac{0.316P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 1.069\right]$$

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in this equation is b the radius of the resisting section. For the situation at HCN this radius is always equal to a.

Edge loading
When the load is applied at a point along an edge of the slab, the critical tensile stress is at the bottom of the concrete, directly under the load. For the edge the ordinary theory based on a semicircle with the force acting on the centre of the circle the maximum bending stress is equal to

$$\sigma_i = 0.529(1 + 0.52\mu)\left(\frac{P}{h^2}\right)[\log_{10}\left(\frac{Eh^3}{ka^4}\right) - 0.71$$

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Maximum bending stress according to the simplified Westergaard's calculation is given by

$$\sigma_e = \frac{0.572P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 0.359\right]$$

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Corner loading
The critical stress in case of the load applied at the corner of the slab is tension at the top surface of the slab. In case of corner loading there is no simplified equation available. According to Westergaard the maximum bending stress in case of corner loading can be determined by:

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a_1}{l}\right)^{0.6}\right]$$

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Whereby a_1 is the distance to point of action resultant along corner angle bisector, which is given by

$$a_1 = a\sqrt{2}$$

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Comparison of the equations

In Table 10 the results of the equations discussed above are listed for loading of the supports on the storage yard. Al the values listed in the table are based on supports with a floor contact area of $1m^2$.

It can be noted that there are no big differences between the multiple equations for the same loading condition.

		Max local load [mT] for multiple k-values x 10000 [kN/m^3])						
	Method	*k*=1	*k*=1.5	*k=2*	*k=2.5*	*k=3*	*k=3.5*	*k=4*
Interior	Ordinary theory for square area	19	20	21	22	23	23	24
	Ordinary theory for circular area	18	20	21	22	22	23	23
	Simplified for circular area	19	20	21	22	23	24	24
Edge	Ordinary theory for semi-circular area	14	15	17	18	19	20	21
	Simplified for circular area	14	15	17	18	19	20	21
Corner	Westergaard's theory for circular area	18	21	26	30	34	39	46

Table 10: Comparison of max allowable local floor pressures Westergaard's equations

Limitations

The model described above for determining the max allowable local floor pressure has some limitations. Punch shear force and beam shear force are based on footing calculations:

- In case of footings, the column is always attached to the footing. In case of HCN, the supports are not attached to the cast-in-situ structures.
- Footings are always beneath the surface and therefore there is always some type of soil on top of the footing. In case of HCN, the cast-in-site structures form the floor surface.
- At the quay side, the foundation exists out of one big lean concrete layer. To perform shear calculations the concrete layer is divided in squares of 5 by 5 meter.

The Westergaard's equations also have a few limitations:

- The vague definition of '*Removed considerable distance'*. For this study it is assumed that a considerable distance is equal to a distance of 1000mm between the center of the support to the edge.
- It is assumed that the slab is of infinite or semi-infinite dimensions. The slab is only 5000 by 5000mm, while the supports have at least a contact area with the slab of 1000 by 1000mm. This means that the slab is not infinite at all.
- The Westergaard's equations make use of the modulus of subgrade reaction (k-value). This modulus is always a constant for the footing. From the soil survey at the quay side it can be seen that there are a lot of different layers in the soil. For the calculations at HCN it is assumed that the upper two layers (backfilling stone and backfilling sand) have the biggest impact on this k-value. Those two upper layers have at least a depth of 9 meters for the whole quayside; an

average of these two values is used. This also means that the detritus layer of 200mm for both the storage yard and quay side is not taken into account during the calculations. Detritus has however a positive effects on the foundation, this results in an underestimation which is accepted.

Results

With use of the models explained above the following results are obtained for the max allowable local floor pressure on the storage yard and quay side.

Storage yard

The cast-in-situ concrete structures (L x B x h = 5000x5000x250mm) have compressive strength of 30MPa at 28days [7] and flexural strength of 3MPa. The cast-in-situ concrete structures are not reinforced and are not mutually attached. The support structures have cross sectional dimensions C_1 x C_2 which is equal to 1000x1000mm. The max allowable global floor pressure of the storage yard is 5T/m^2, this result in a max allowable local floor pressure per concrete structure of 125T/m^2

$$P = A * q_{all} = 125\text{T/m}^2$$

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This is however only the max allowable local floor pressure if no stresses in the concrete are exceeded.

The products which are stored at Huisman can be seen as dead loads. Dead loads have safety factor of 1.2. So it is possible to concentrically load a support structure on top of a storage yard ground slab with a load of 104T if critical punching force, beam shear force, bearing capacity and bending stresses are not exceeded.

Punch and beam shear

The critical punching shear force can be evaluated using the following equation

$$V_u = q_{all}[L * B - (C_1 + d)(C_2 + d)] \approx 117\text{T}$$

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with q_{all} = 5T/m^2, L = B = 5000mm, $C_1 = C_2$ = 1000mm and d = 250-75mm = 175mm.

Punching shear force resisted by concrete V_c is given as the smallest of:

$$\Phi\, V_c = \Phi\, 0.53\sqrt{f_c'}\left(1 + \frac{2}{\beta}\right)\lambda b_o d \approx 548\text{T}$$

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[7] The concrete slabs at the storage yard are much older, probably 5 or 6 years old. This means that the compressive strength can be multiplied by a factor of 1.35. The results obtained in this report should however always be valid, also when new slabs are placed. Therefor this strength ratio is not used.

$$\Phi\, V_c = \Phi\, \lambda \sqrt{f_c'}\, b_o d \approx 345\text{T}$$

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$$\Phi\, V_c = \Phi\, 0.27 \left(\frac{\alpha_s d}{b_o} + 2\right) \lambda \sqrt{f_c'}\, b_o d \approx 325\text{T}$$

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with Φ =0.75, f_c' = 30MPa, β = L/B = 1, λ = 1, b_o = 2(C_1+d) + 2(C_2+d) = 4900mm and α_s = 40.

$$\mathrm{V_u} \leq \Phi \mathrm{V_c} \rightarrow 117 \leq 325 \qquad \text{Ok.}$$

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The slab and the support structure are both square, so beam shear force has to be evaluated for only one direction. In case the slab or support structure is not square, both the directions have to be evaluated.

The beam shear force is given by

$$V_u = q_{all}\, B \left[\left(\frac{L - C_2}{2}\right) - d\right] \approx 46\text{T}$$

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The beam shearing force resisted by concrete is given by

$$\Phi\, V_c = \Phi\, 0.53 \sqrt{f_c'}\, B d \approx 194\text{T}$$

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here.-30

$$V_u \leq \Phi V_c \rightarrow 46 \leq 194 \quad \text{Ok.}$$

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Bearing strength

The bearing capacity is

$$\Phi P_n = \Phi(0.85\, f_c' A_1)\sqrt{\frac{A_2}{A_1}} \leq 2.0\Phi(0.85\, f_c' A_1) \approx 3903\text{T} \leq 3903\text{T} \qquad \text{Ok.}$$

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It seems a little bit coincidental that the bearing on concrete equals the concrete bearing strength. However when looking at this equation and the values of A_1 and A_2 it becomes clear that the result is okay; for a cast-in-situ thickness of 250mm in combination with the used support, the values for A_1 and A_2 become respectively $1m^2$ and $4m^2$. For the terrain at HCN the bearing strength is however not that important because the total load on top of one concrete structure is at most 125T which is a pittance in comparing with the bearing strength.

Bending stress

Last possible critical stress which can give problems in the concrete slabs is the bending stress. As discussed in the previous chapter are there three loading conditions (interior, edge and corner). The loading conditions for these three conditions depend a lot on the soil settlement and thus the k-value.

As already discussed in the limitation of the model, the k-value is based on the upper two soil layers. These two layers have at least a depth of 9.2meters for the entire quayside. So it assumed this is also the case for the storage yard. According to the design institute, the k-values for the backfilling stone and backfilling sand are respectively 20000 – 40000kN/m^3 and 10000-40000kN/m^3. In Table 11 the max allowable local floor pressures are given for the three loading conditions for varying k-values.

Al the values listed in the table are based on supports with a floor contact area of $1m^2$.

Max local load [mT] for multiple k-values x 10000 [kN/m^3])							
	k = 1	*k* = 1.5	*k* = 2	*k* = 2.5	*k* = 3	*k* = 3.5	*k* = 4
Interior	19	20	21	22	23	24	24
Edge	14	15	17	18	19	20	21
Corner	18	21	26	30	34	39	46

Table 11: Max allowable local floor pressures at the storage yard for the three loading conditions

Conclusion

The critical punch shear force and the beam shear forces are even higher than the possible total loading of one concrete structure so they will not lead to failure of the concrete structure.
The critical stress, when determining the max allowable local floor pressure, is the bending stress. The maximal allowable bending stress depends on the modulus of subgrade reaction (k-value). To obtain the right k-value for the soil beneath the storage yard it is therefore strongly recommended to perform a test which will determine this value. This test is described at the end of this report.

Quay side

For the quay side the same model is used for the punch shear and beam shear as is used for the storage yard. In the limitations in the previous chapter it is already discussed that the lean concrete layer at the quay side is divided into plates of 5000 by 5000mm to perform punch and beam shear calculations. Because of the thick lean concrete (C15) layer (560mm) the punch shear and beam shear resisted by the lean concrete layer will be that big that the punch and beam shear due to the local floor pressure will never be exceeded. Therefore the max allowable local floor pressure will be based on the bending stress in the concrete layer.

Bending stress

As is said before is the lean concrete layer just one big layer. This results in only one loading condition, the interior loading condition. For the calculation of the bending stress only the concrete layer is taken into account, the detritus layer, the sand layer and the precast concrete blocks are ignored.

	Max local load [mT] for multiple k-values x 10000 [kN/m^3])						
	k = 1	*k* = 1.5	*k = 2*	*k = 2.5*	*k = 3*	*k = 3.5*	*k = 4*
Interior	74	78	81	83	86	88	90

Table 12: Max allowable local floor pressures at the quay side for the interior loading condition

Conclusion

At the moment the max allowable local floor pressure at the quayside is 5mT/m^2. According to models where the punch shear, beam shear and bending stress in the lean concrete layer (with ignoring the other layers) are calculated, the local floor pressure can be increased significantly. The bending stress will be the critical stress in the concrete layer. The bending stress is for the lowest k-value, equal to 10.000kN/m^3, not exceeded until a local load higher than 74mT/m^2 is applied. Of course the max allowable global floor pressure of 5mT/m^2 may not be exceeded.

The precast concrete blocks have a compressive strength of 50MPa, so this doesn't give any problems. Point of attention is however the 50mm sand layer between the concrete blocks and the lean concrete layer. This layer may result in sagging of the precast concrete blocks. The sand layer is probably also the reason for the current waving of the quayside floor surface.

Test to determine k-value

The modulus of subgrade reaction is of importance for the determination of the max allowable local floor pressure at HCN. At the moment this value is based on the two upper soil layers beneath floor at the quayside. For the storage yard and the quay side it is however very practical to obtain a real k-value determined by a test.

According to an article excerpted from the PCA publication Concrete Floors on Ground (EB075.03) [18]:

*" The k-value is measured by plate-loading tests taken on top of the compacted subgrade (or subbase, if used). A general procedure for load testing is given in ASTM D 1196, **"Standard Test Method for Nonrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements".** This method provides guidance in the field determination of subgrade modulus with various plate diameters. Design of Slabs on Grade (ACI 360R) is specifically oriented to the determination of modulus of subgrade reaction using a 760-mm, diameter plate and gives more detailed information on test methods using this size plate. This plate is loaded to a deflection not greater than 1.25 mm, and the k-value is computed by dividing the unit load by the deflection obtained. "*

With the k-value obtained by this test it is possible to determine the exact max allowable local floor pressure.

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Appendix G Support Structure Design phase

In order to come up with a final redesign of the SS first different concept designs should be considered. The complete SS exist at the moment out of two separate support components; the WSSs and the SBs. The structure of the redesign will however be different. The redesign will be composed of one support beam on top with a permanent fixed support towards the floor, from now on column support.

New SSs with different sizes will be designed; SSs possible to transport with one SPMT and SSs that should be transported with two SPMT's. The redesigned SSs should be able to carry the construction during storage and during transport. The surface area that is in contact with the floor should make it possible to distribute the product load to the floor according to different floor pressure scenarios.

The redesign should fulfil some important requirements:

- Stability of the stored products must be ensured at all times.
- The redesign must be resistant to two types of loading; SWL acting in the middle of the beam and a SWL uniformly distributed over the beam.
- It should be possible to place a second level of SS on top of the redesign.

The redesign of the SS is subdivided into two parts; the design of the support beam and the design of the column support. For both parts different options are evaluated together with all their possible failures.

Support beam

The support beam is subjected to a downwards pointed SWL perpendicular to the support beam. Due to this loading, the support beam can collapse due to bending stress, shear stresses and lateral torsional buckling (LTB.

The support beam should have a length of at least 7meter (drive through width of SPMT is 5.5meter); this length depends on the dimensions of the support columns in combination with the floor plates. The support columns will be evaluated in the next section. For the cross section of the support beam different sections are possible; an I-beam is most preferable to prevent the beam from bending. Disadvantage of an I-beam is the possibility occurrence of LTB. Therefore it is much better to use a box section beam instead of an I-beam.

It is required that a second level SS can be built on top of the support beam. Therefore the surface area of the top side of the support beam should not be too small. Combining all these possible sections results in a box section existing out of two or three I-beams (Figure 30); I-beams are good against bending and it is possible to make the upper flange wide enough for the second level SS. The box section should give enough stiffness against LTB.

The dimensions of the I-beams depend on stresses in the section, but it also depends on the design choices for the columns and at the plate material that is at stock at HCN. After evaluation of the stock in the '*FUJIAN Huisman administration system*' it becomes clear that the material that will be used for the support beam will be steel52 or Q345D which both have a yield stress starting from 345MPa. There are quite a number of plate thicknesses in stock, so that will not cause any bottlenecks. Most of the plates have a length of 12000mm and a width of 3000. This should be taken into account to minimize the scrap.

In Microsoft Excel two models are constructed, one model which evaluated the SS which should be transported by one SPMT and one model to evaluated the SS which should be transported by two SPMTs. The models are evaluated by use of hand calculations. With use of this model it becomes very easy to compare different/multiple I-beam section, different SWLs and different lengths of the support beam. The model calculates the maximum stresses in the support beam (bending stress and shear stress). For the calculation of the stress and the LTB, it is assumed that the support beam is simply supported on both ends of the beam (Figure 31a). In case of the SS that need to be transported with use of two SPMTs it is assumed that the support beam is clamped (no deflection and angular movement) at the position where it is connected to the middle column structure (Figure 31b).

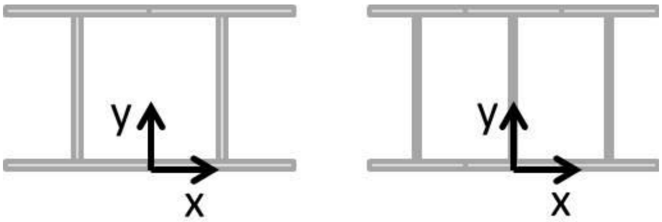

Figure 30: Two types of box section

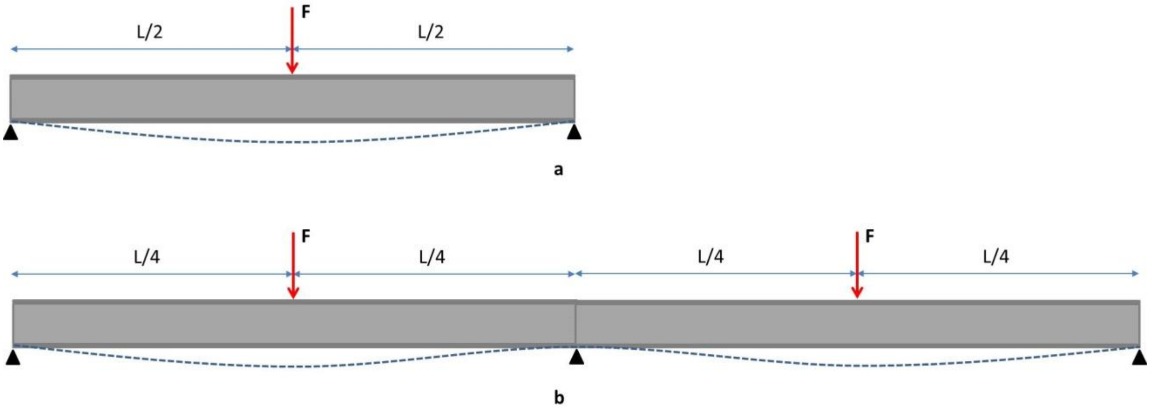

Figure 31: Deflection of the two models.

Calculation of stresses

For the deflection of the support beams of Figure 31 it is assumed that the force F engages in the center of gravity.The deflection of the beam, the max bending stress in the upper flange and the max bending stress in the bottom flange have the highest value at the middle of the beam. They are given by:

$$v = \frac{F \cdot L^3}{48 \cdot E \cdot I_x}$$

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$$max\ \sigma_{bend\ fl.\ up} = \frac{M \cdot y}{I_x}$$

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$$max\ \sigma_{bend\ fl.\ bot} = \frac{M \cdot y}{I_x}$$

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With taken into account the shear stress due to force F, which is also applied at the middle of the beam, the maximum stresses in the beam can be calculated. Shear stress is calculated by:

$$\tau = \frac{F \cdot Q}{I_x \cdot t}$$

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For the support beam that should be transported with two SMPTs is becomes a little bit more elaborated. According to beam calculations, 'VergeetMeNietjes' listed in 'Toegepaste Mechanica deel3' [1], the maximum moment will occur in the beam at location L/2. This moment will occur when SWL F is located 0.207*L from both ends of the beam. The difference in max moment in case the load is acting on L/4 from the end of the beam is however negligible small that this point is used in further evaluation of the beam.

With use of a Finite Element Method described in 'An Introduction to the Finite Element Method' [2], it is possible to calculate the reaction force (A_y, B_y and C_Y) in each of the three supporting points, see Figure 32. A stiffness matrix for the support beam can be constructed to calculate the reaction forces. The forces are applied to the beam at L/4 from both ends of the beam. The reaction forces A_y and C_Y become equal to 5F/16 and B_y is equal to 22F/16. This is only the case when the load is action in point L/4 and 3L/4. In case the entire beam is uniformly distributed the reaction forces A_y and C_Y become equal to 3Q/5 and B_y is equal to 10Q/8. For the maximum safety working distributed load of 200mT (400mT in total for the whole beam), the reaction force in B_y is equal to 250mT. The combination of the bending stress at the point where the beam is supported by the middle column and the shear force B_y acting in that point results in the maximum stress point for the double support beam.

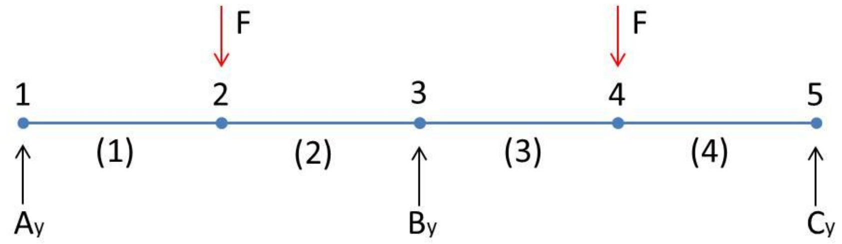

Figure 32: 2-Dimensional beam structure loaded in plane.

For every SWL, combination of I-profiles and length of the support beam the stresses in the beam and the mass of the beam can be calculated.

The maximum stress in the beam is calculated with use of the Von Mises stress criterion:

$$\sigma = \frac{1}{\sqrt{2}} \cdot \sqrt{[2 \cdot \sigma_{bend}^2 + 6 \cdot \tau^2]}$$

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Lateral torsional buckling model

Beams that are exposed to a very high load can fail due to LTB. Within HCN there is no regular model to calculate if the beam will collapse due to LTB. According the ‘*Calculation manual*’ [1] chapter 4.3.4 are tubes stiff enough to resist LTB. Remarkable is however the stiffeners that used at the current SB to prevent LTB. Since there is no information available about calculations, the actual LTB is calculated according a model [4] constructed by *Queen’s University Belfast* which is a public research university. This model is based on steel design according to Eurocode 3.

Beams without continuous lateral restraint are prone to buckling about their major axis, this mode of buckling is called lateral torsional buckling (LTB), see Figure 33.

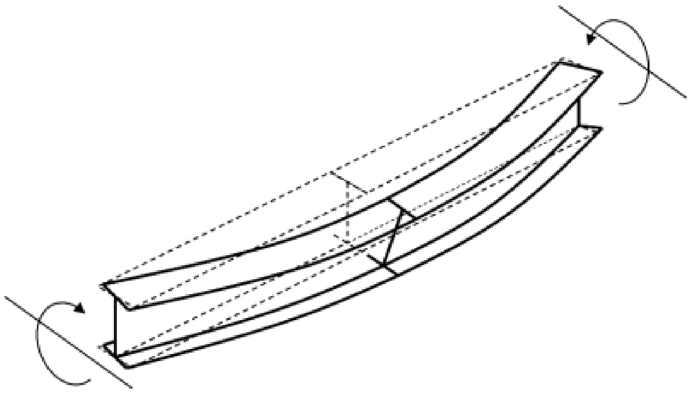

Figure 33: Lateral torsional buckling of a simply-supported 'I' beam under constant bending. [5]

The support beams showed in Figure 31 has no lateral restraints. Calculations beneath will show if lateral restraints are needed.

When the cross section is not bisymmetrical (upper and bottom flanges are not per definition the same) the equation for the LTB becomes elaborated. The design moment must be less than the design buckling resistance moment:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1.0$$

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$$M_{b,Rd} = \chi_{LT} \cdot W_y \cdot \frac{f_y}{\gamma_{M1}} \qquad \text{where } \gamma_{M1} = 1.0$$

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Section modulus W_y depends on the classification of cross section. The support beam has a class 3 cross-section because the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses which can reach the yield strength. [5] The section modulus is equal to the elastic section modulus.

The yield strength f_y depends on the steel grade and nominal thickness of element. For this particular case this is equal to 345MPa. (Steel grade = S355, nominal thickness of element is between 16 and 41mm.)

The reduction factor χ_{LT} depends on the buckling curve, the section modulus, the yield strength and the critical moment according to:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \lambda_{LT}^2}}$$

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where

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$

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α_{LT} depends on the buckling curve. The cross section is a combination of welded I sections where the height of the profile is smaller than twice the width of one I-beam flange. This gives α_{LT} a value of 0.49.

$$\lambda_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}}$$

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with

$$M_{cr} = C_1 \frac{\pi^2 \cdot E \cdot Iy}{(kL)^2}\left[\sqrt{\left(\frac{k}{k_w}\right)^2 \cdot \frac{I_w}{I_y} + \frac{(kL)^2 \cdot G \cdot J_t}{\pi^2 \cdot E \cdot I_y} + \left(C_2 \cdot z_g\right)^2} - C_2 \cdot z_g\right]$$

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- L is length of the beam
- E is young modulus
- G is shear modulus
- I_y is the second moment of area about the weak axis
- J_t is the torsion constant
- I_w is the warping constant
- k and k_w are effective length factors equal to 1.0
- z_g is the distance between the point of load application and the shear centre.
- C_1 and C_2 are coefficients. For transverse loading they are based on the moment diagram; C_1 = 1.348 and C_2 = 0.630.

Buckling of web profiles

Last possible failure mechanism for the support beam is buckling of the web due to the point loading. It is very hard to calculate this by hand, therefore the buckling of the web is analysed in '*IMEP 2*'.

Column structure

For the support columns beneath the support beam there are multiple designs possible. For the column it is possible to use a tube section or an I-beam profile. The dimensions of the tube depend on the material on stock, for the I-beam there are less restrictions since the variety of plates on stock is large.

For the column structure distinction can be made between placing the column structure perpendicular to the support beam or under a certain angle. Advantage of the column structure under a certain angel is that it can result in more stability. Another reason to place the column structure under a certain angle is to reduce the length of the beam; same stability as perpendicular columns, but the floor plates are located beneath the columns. A big disadvantage is however the large stresses that will occur in the welds. These stresses can be reduced by use of ribs. In that case the length of the support beam also has to be increased (Figure 34); column structures placed perpendicular to the support beam will be used.

Figure 34: Options for orientation column structure.

To make the SS more stable it is advisable to support the support beam with use of two columns under a certain angle instead of a column structure that is placed perpendicular to the floor surface. When the columns are placed under a certain angle the two columns can connected by use of a truss bar or by placing one big floor plate beneath the column structure. Since the truss bar result in high stresses in the columns and because one floor plate connected to both columns will have more stiffness, the redesign will make use of one big floor plate. All the different options can be seen in the morphologic overview, Table 13.

Since two types of SS are designed (single SPMT and double SPMT), distinction has to be made between the loading on the column structures of the models. In case of the SS which will be transported with one SPMT the maximum load on top of one column structure is equal to the safety working point load until the safety working point load is exceeds half the SWL of the SPMT. In that case the maximum load on the column structure is equal to half the SWL of the SPMT. For the double support beam this is different as already discussed in the previous section. For the two outer column structures this load is the same as for the single support beam. For the middle column structure the load is higher; the maximum vertical loading on the column structure is equal to 75 percent of the maximum distributed SWL on top of the entire support beam.

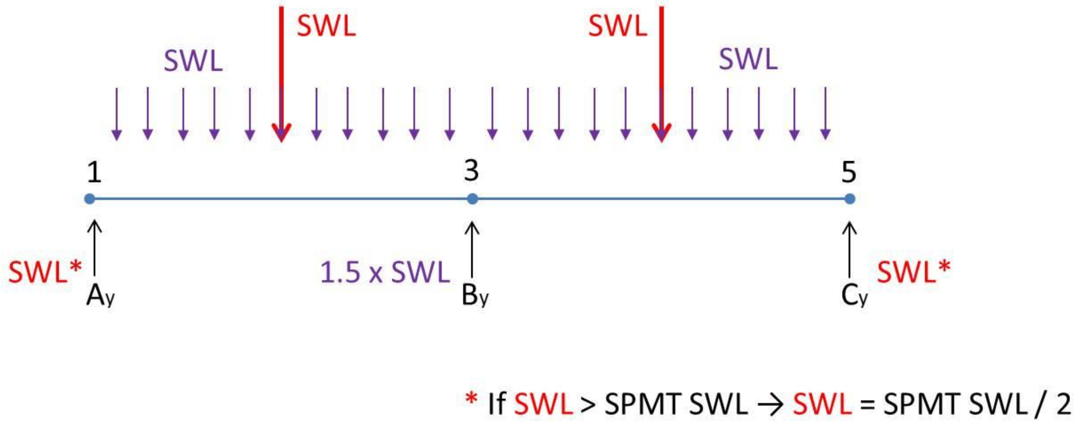
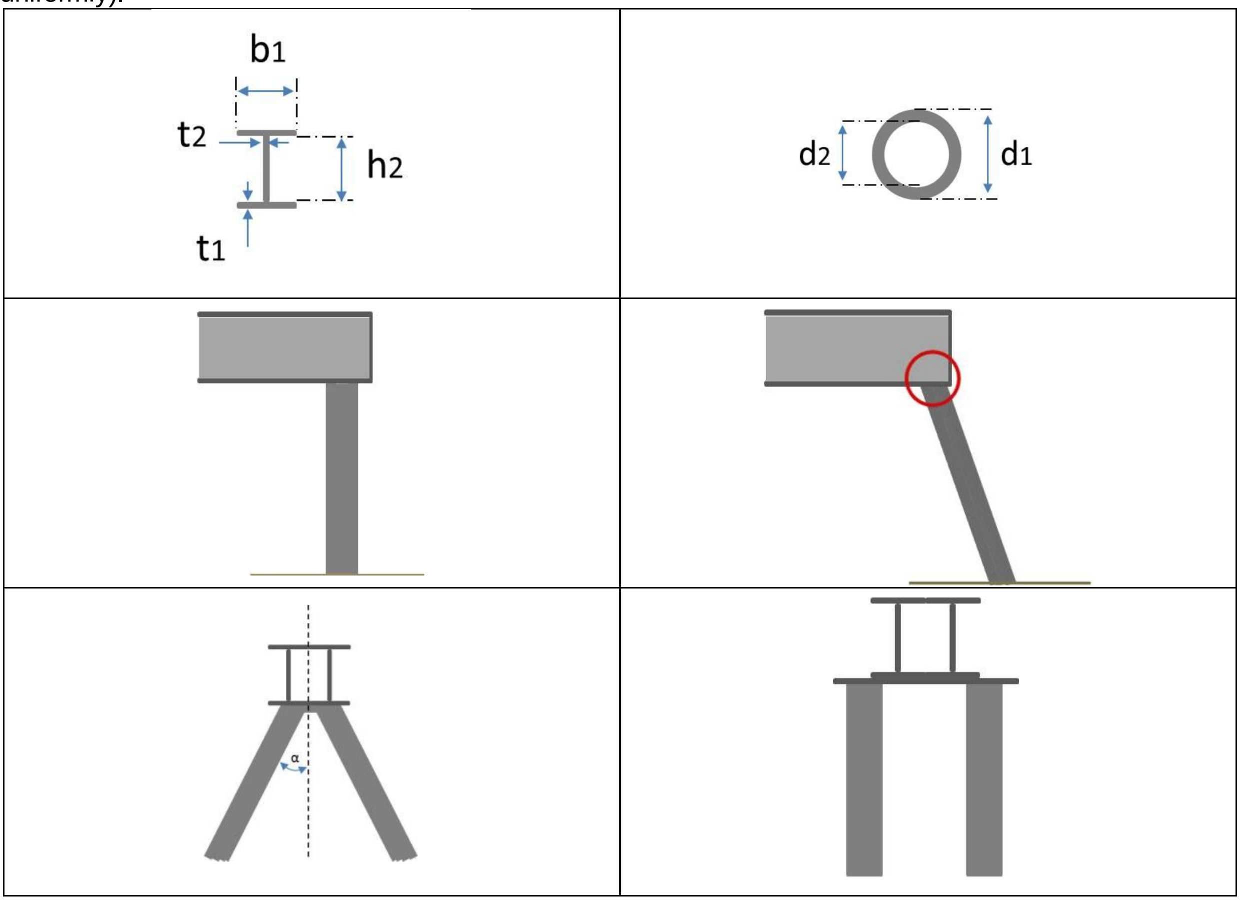


Figure 35: Loading of the double support beam.

In Table 13 it is already shown that each column structure will exist out of two columns. Due to the waving of the floor surface on the QS and due to the forces acting on the product due to wind, it is possible that the forces are not always divided equally over the two columns. It is assumed that in worst case scenario the force distribution between the columns is 40 to 60 percent.

The base plates, which will distribute the pressure to the floor, are connected to the columns and in addition ribs are connected between the column and base plate in order to increase the load distribution of the base plate (this way it can be assumed that the load distribution of base plates to floor is uniformly).



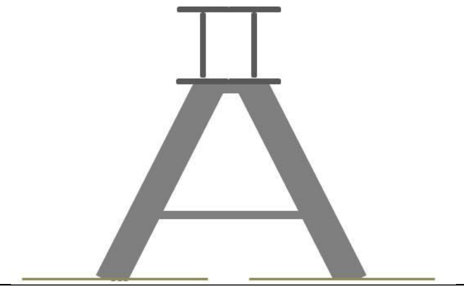
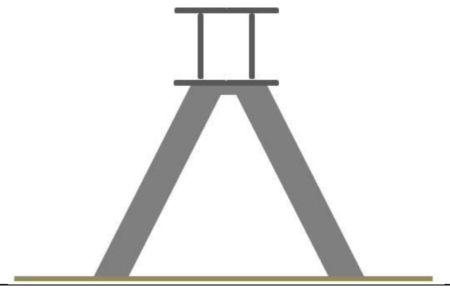
	

Table 13: Morphologic overview

Calculation models

The columns of the column structure can fail due to buckling or axial pressure. To determine the critical buckling force and the axial stress in the column for multiple profiles the forces on the support column are rewritten in a rotated coordinate system (Figure 36):

$$(F_{column})_u = F_{column} \cdot \cos(\alpha) \qquad (F_{column})_v = F_{column} \cdot \sin(\alpha)$$

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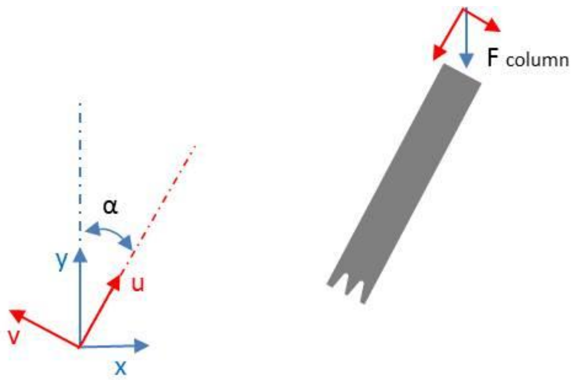

Figure 36: Support column rotated coordinate system

The axial force may not exceed the critical buckling load which is given by:

$$P_{critical} = \frac{\pi^2 \cdot E \cdot I_{column}}{4 \cdot L_{buckling}}$$

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In Figure 37 two possible buckling options are displayed for the columns. The critical buckling force of the column will be calculated according the first option, to make sure no buckling will occur.

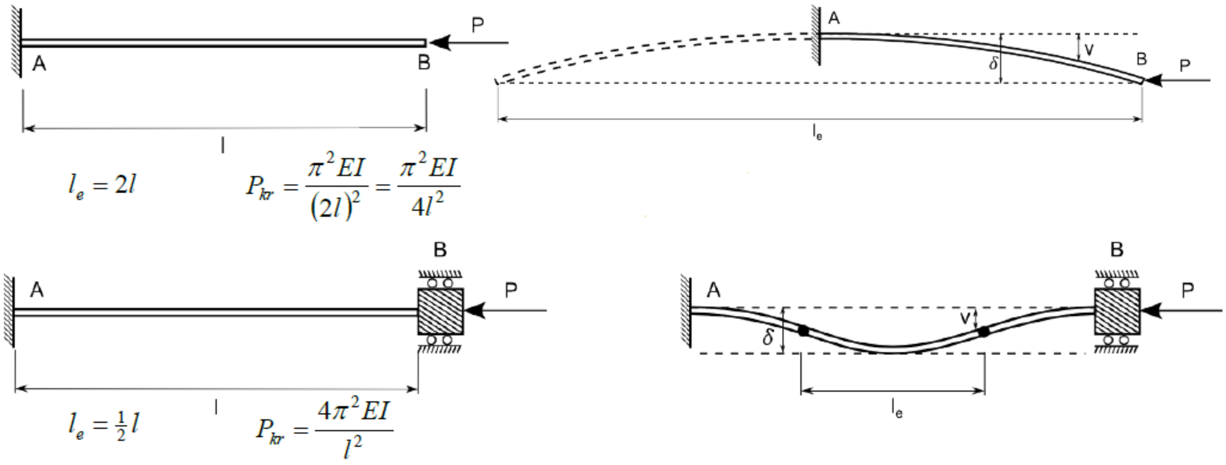

Figure 37: Buckling options. [6]

The normal stress in the column due to the axial force is equal to:

$$\sigma_{axial} = \frac{(F_{column})_u}{A_{column}}$$

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Appendix H Internship Assignment

Support structures redesign

1. Introduction

Huisman is a privately owned company operating globally with extensive experience in the design and manufacturing of heavy construction equipment for world's leading on- and offshore companies. With engineering and production capacity in The Netherlands, Czech Republic and China, Huisman is generating close to 100,000 m^2 of total production surface. The new to build production facility, which should be operational in 2014, will increase this production surface even more. Together with the local sales, engineering and service supports in Australia, Norway, Brazil, Singapore, Slovakia and the USA a dedicated service team of skilled professionals is always able to provide advice and service support before, during and after installations and delivery.

The facility in Fujian Province (Xiamen area) in China is founded to facilitate customers in this region and to increase the overall production capacity. The production facility has been fully operational since April 2007 and delivers a significant contribution to the overall Huisman engineering and production capacity. Huisman China (HCN) is a production facility with a total area of 284,000 m^2. Part of the terrain is the first quayside owned by an international company in China that is opened up in 2012. This quayside is specially constructed by Huisman to increase production area and be self-reliant for loading and unloading.

Owing to the opening of the new quayside in 2012 a large storage area is obtained. In combination with requests of clients to store their products passed the delivery date, the amount of products currently stored have rapidly increased. During production and storage of the products, additional support structures are needed which results in a big stock of support structures. This intern assignment will focus on these support structures.

Figure 38: Site of HCN with EMAS AMC in front of Quayside

2. Description

The support structures at HCN, representing a total stock equal to 2.600mT (about 5.2 million euro's), are used to support products during manufacturing in the workshops and during storage on the quayside. The usage of the support structure can be divided between distribution of the floor pressure and supporting of the product.

The report "Reduction of support structures" provides a good basis for the assignment but could use further expansion; further investigation should give more insights in the actual and total reduction of stock that is possible within HCN.

During this assignment it is recommended to design the support structures with keeping the SPMTs in mind. The costs associated with the SPMTs have, compared with the costs that are associated with the support structures, more impact on HCN. A support structure that will increase the efficiency of the SPMT can therefore maybe lead to a higher reduction of costs.

The following topics/requirements have to be taken into account during the internship:

1. All the different transportation scenarios (SPMTs located next to each other and in front of each other) result in different type of support structures. In consultation with the Management team of HCN, the Mechanical Engineering department and the workers in the workshop it should be possible to come up with a support structures that will increase the efficiency of the SPMTs and also lead to a significant reduction of stock. These two improvements will lead to significant savings.

   The SPMT will result in requirements for the max loading of the support structure, for the width of the support structure and for the height of the support structure.

2. There is a wide range of products at HCN and each product is stored on its own way. It should be possible to store the product with use of the new support structure.

3. During the new design phase of the support structures, the validity of the 'floor pressure report at HCN' has to be taken into account.

4. In further investigation the possibilities to build up a complete support structure out of separate components has to be evaluated. With use of separate components it is easier to modify the complete support structure with respect to the requirements for the products. This way is it also possible to connect two complete support structures to each other. Separate components also results in less storage space for the support structures in case they are not in use by HCN.

   Since the mass of the support structure should be kept as low as possible the connection between the different components can cause difficulties.

5. During the design of the support structures attention has to be paid to the floor plates. It must be ensured that the floor plates will distribute the pressure to the floor as uniformly as possible.

6. The products are often transported with use of a transport barges. These transport barges result in requirements in the form of local deck pressure. Investigation of this deck pressure can give an outcome to this problem.

The assignment is a combination between Civil and Mechanical Engineering.

3. Final Presentation

The student is expected to present the findings and proposal in two ways:

1. A written report with detailed findings of the research, and a detailed layout of the proposed improvements and changes, and implementation plan.

2. A verbal presentation with accompanying PowerPoint and any other diagrammatic & written material as needed, to the Top Management team and related key members of the organisation that are impacted by the proposal.

Figure 39: Store out configuration reel cradle

References Appendices

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