ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL

STRUCTURAL ASSESSMENT AND FATIGUE EVALUATION OF A 60-YEAR-OLD LINKSPAN BRIDGE IN NORTHEAST OF ENGLAND

M.Sc. THESIS

Emre USTA

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Structural Engineering Programme

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ISTANBUL TEKNİK ÜNİVERSİTESİ ★ LİSANSÜSTÜ EĞİTİM ENSTİTÜSÜ

KUZEYDOGU INGILTERE`DE 60 YILLIK LINKSPAN KOPRUSUNUN YAPISAL DEGERLENDIRMESI VE YORULMA ANALIZI

YÜKSEK LİSANS TEZİ

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Emre USTA, a M.Sc. student of ITU Graduate School student ID 501201042, successfully defended the thesis/dissertation entitled “STRUCTURAL ASSESSMENT AND FATIGUE EVALUATION OF A 60-YEAR-OLD LINKSPAN BRIDGE IN NORTHEAST OF ENGLAND”, which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

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To my precious spouse, Özde, I have always felt the support behind and could have never achieved this master`s degree without your continued support,

To my dearest son, Aren,

FOREWORD

I am humbled to extend my deepest gratitude to those who have been instrumental in its fruition. Firstly, I express my heartfelt appreciation to Asst. Prof. Dr. Kadir Ozakgul, whose unwavering encouragement and support served as the cornerstone of this endeavor. Additionally, I am indebted to Yasutsugu Yamasaki, not only as an esteemed engineering manager but also as a mentor whose guidance was invaluable throughout this process.

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This thesis stands as a testament to the collective support and encouragement of these remarkable individuals, without whom this accomplishment would not have been possible.

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May 2024

Emre USTA
(Civil Engineer)

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ABBREVIATIONS

LAT	: Lowest Astronomical Tide
HAT	: Highest Astronomical Tide
NDT	: Non-Destructive Testing
MT	: Magnetic Particle Testing
UT	: Ultrasonic Testing
FEA	: Finite Element Analysis
1D	: One Dimensional
2D	: Two Dimensional
UB	: Universal Beam Section
RSC	: Rolled Steel Channel Section
LOA	: Length Overall
B	: Breadth
HGV	: Heavy Goods Vehicle
UDL	: Uniform Distributed Load
ULS	: Ultimate Limit States
N	: Number of Cycles

SYMBOLS

$\mathbf{f_y}$	: Yield strength
$\mathbf{f_u}$	: Tensile strength
E	: Elasticity Modulus
γ_m	: Material partial safety factor
γ_s	: Unit weight of steel
$\mathbf{q_w}$	: surface loading
$\mathbf{I_p}$	: Moment of Inertia
$\mathbf{k_p}$	: spring stiffness coefficient of bearings
$\mathbf{k_s}$	: spring stiffness coefficient of buffers
$\mathbf{R_A}$	: Reaction forces
UC	: Utilisation Ratio
N	: Number of cycles
$\Delta\sigma_p$	: Allowable stress
$\Delta\sigma_R$	: Direct Stress range
$\Delta\sigma_c$	: Detail category
γ_{MF}	: Partial safety factor for fatigue strength
γ_{Ff}	: Partial safety factor for stress range
$\Delta\sigma_D$	: Constant amplitude fatigue limit
$\Delta\sigma_L$	: Cut-off limit

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STRUCTURAL ASSESSMENT AND FATIGUE EVALUATION OF A 60-YEAR-OLD LINKSPAN BRIDGE IN NORTHEAST OF ENGLAND

SUMMARY

This thesis aims to investigate the condition of a linkspan bridge by performing structural inspection and structural assessment concluded with the fatigue evaluation based on the collected inspection data. To that end, a linkspan bridge has been selected which is constructed in 1965 and operational in an international port that is located in Northeast of England. The original linkspan structure was suspended from a fixed gantry and subsequently extended two times which is approximately 32m and is supported by four hinged bearings two on bankseat and two on the pontoon end.

Periodical inspections and evaluation of any changes in the structure over time is a noteworthy concern for the port operators as well in terms of asset management. Accordingly, the condition of the structure can be determined based on overall and member-based structural status, thus the residual life.

The structural inspection is conducted in two steps, consisting of visual inspection and thickness readings by using ultrasonic testing device to determine the overall condition of the linkspan bridge and have further idea about the member thicknesses. Considering the continual corrosion resulting with the loss of member sections, a comparison has been made to determine the structural capacity of the link-span bridge based on the existing condition.

For that purpose, a member-based computational models of the linkspan bridge have been produced in SCIA Engineer Analysis software based on the historic information and existing drawings of the bridge.

Original member thicknesses are compared with the reduced member thicknesses to represent the allowance for corrosion. The results have been defined and evaluated for each member type; accordingly, the minimum allowable section thickness was identified.

In addition to the inspections, fatigue analysis is an important method to have an idea of the residual life of the structure based on the actual data. Whilst it is possible to undertake a retrospective analysis of fatigue, which would need to include information on yearly traffic (numbers of vehicles in certain weight bands for example). Therefore, the actual ferry timetables are used for reference and yearly traffic estimation has been made as an exemplar study.

As structures age (and particularly if their condition and thus capacity deteriorates), the accumulating effects of fatigue can be a significant factor in the eventual failure of a structure, as well as one-off overloading, the chances of which increase under a given load as the structure corrodes and thus its capacity gradually reduces.

KUZEYDOGU INGILTERE`DE 60 YILLIK LINKSPAN KOPRUSUNUN YAPISAL DEGERLENDIRMESI VE YORULMA ANALIZI

ÖZET

Bu tez, bir linkspan köprüsünün yapısal durumununun muayene verilerine dayalı olarak yapısal incelenmesini ve yapılan değerlendirmeler sonucunda yorulma analizini de kapsayarak yapı durumunu ve ömrünü araştırmayı amaçlamaktadır. Bu değerlendirmeyi yapmak için İngiltere'nin kuzeydoğusundaki uluslararası bir limanda faaliyet gösteren ve 1965 yılında inşa edilen linkspan köprüsü seçilmiştir.

Orijinal linkspan köprüsü, köprüyü sınırlayan iki dolfen yapısına asılmak suretiyle faaliyet göstermekteydi. Ardından iki kez iyileştirme ve ek yapılarak nihai uzunluk olan yaklaşık 32 metrelik bağlantı köprüsü elde edilmiştir. Bağlantı köprüsü iki adet yüzer duba üzerinde ve iki adet rıhtım bitiminde olmak üzere toplam dört adet noktadan mesnetlenmiştir.

Periyodik denetimler ve yapıda zaman içinde meydana gelen önemli değişikliklerin etkileri sonucunda yapısal değerlendirmenin gerekliliği ve yapı ömrünün belirlenmesi hedeflenmiştir. Buna göre yapılan incelemeler ve analizlerin sonucunda yapının genel ve eleman bazında yapısal durumu, dolayısıyla kalan yapısal ömrü belirlenebilir.

Yapısal inceleme, bağlantı köprüsünün genel durumunu belirlemek ve eleman kalınlıkları hakkında daha fazla fikir sahibi olmak için ultrasonik test cihazı kullanılarak görsel muayene ve eleman kalınlık okumalarından oluşan iki adımda gerçekleştirilmiştir. Eleman bazında kesit kaybıyla sonuçlanan sürekli korozyon dikkate alınarak, mevcut duruma göre bağlantı köprünün yapısal kapasitesini belirlemek için karşılaştırmalı analiz yapılmıştır.

Bu amaçla köprünün geçmiş bilgileri ve mevcut çizimleri temel alınarak linkspan köprüsünün eleman bazında ve global hesaplamalı modelleri SCIA Engineer yapısal analiz yazılımında üretilmiştir.

Tez çalışması kapsamında yapılan incelemeler ve analizler, köprünün yapısal durumunu belirlemek ve ömrünü uzatmak amacıyla yapılacak olası müdahaleler için önemli veriler sunmaktadır. Eleman kalınlıklarının ultrasonik test cihazı ile ölçülmesi, köprünün mevcut durumunu anlamak için kritik bir adımdır. Bu ölçümler, korozyonun etkilerini ve elemanların ne kadar aşındığını ortaya koymaktadır.

Orijinal eleman kalınlıkları belirli ölçüde azaltılarak korozyonlu durumun temsil edilmesi amaçlanmıştır. Belirli bir miktarda eleman kalınlıkları azaltılmış ve orijinal eleman kalınlığı kullanılarak, sonuç her eleman tipi için tayin edilmiştir. Böylece izin verilen minimum bir kesit kalınlığı tanımlanmıştır.

Muayenelere ek olarak, yapının kalan ömrü hakkında daha iyi bir fikir sahibi olmak için yorulma analizi önemli bir yöntem olarak kullanılmaktadır. Bu analiz, gerçeğe yakın veriler kullanılarak yapılmakta olup, yıllık trafiğe ilişkin bilgileri (örneğin, belirli ağırlığa sahip araç sayıları) içermektedir. Bu kapsamda, güncel gemi seferleri ve araç sayıları referans alınarak, örnek bir çalışma olarak yıllık trafik tahmini

yapılmıştır. Yıllık trafik tahminleri ve artışı da dikkate alınarak, köprünün kullanım yoğunluğu ve bu yoğunluğun yapısal elemanlar üzerindeki etkilerini değerlendirmek için yorulma analizi gerçekleştirilmiştir.

Yorulma analizi, köprünün maruz kaldığı tekrarlı yüklemelerin yapısal elemanlar üzerindeki etkilerini değerlendirmekte ve bu yüklemelerin birikimsel etkilerini ortaya koymaktadır. Bu analizler, köprünün kalan ömrünü tahmin etmek için kritik bilgiler sunmakta ve gelecekteki bakım ve onarım ihtiyaçlarını belirlemeye yardımcı olmaktadır.

Yapılar yaşlandıkça ve özellikle yük taşıma kapasiteleri azaldıkça, yorulmanın biriken etkileri yapının nihai göçmesine ve tekrarlı yükler altında artan aşırı yüklemelerde önemli bir faktör olabilir. Bu nedenle, yapısal kapasite de giderek azalmaktadır.

Sonuç olarak, bu tez çalışması, linkspan köprüsünün yapısal durumunu kapsamlı bir şekilde değerlendirerek, yapının güvenli ve verimli bir şekilde kullanılmasını hedefler. Yapılan incelemeler ve analizler, köprünün mevcut durumunu ve gelecekteki performansını anlamak için veriler sağlamakta olup, yapının ömrünü saptamayı amaçlar. Bu kapsamda elde edilen sonuçlar, benzer yapılar için fikir oluşturarak genel olarak liman yapıları hakkında önemli bir referans noktası oluşturabilir.

Yapısal modelleme yapılırken tarihsel linkspan yapısının gerçeğe en yakın şekilde modellenmesi hedeflenmiştir. Yapılan analizlerin sonucunda, boyuna kirişler ve enine kirişlerdeki %10 kesit kaybı ile karşılaştırma varsayımının doğru yaklaşım olduğu saptanmıştır ve sonuçların mevcut duruma.izin verdiği kanıtlanmıştır. %10'dan fazla kesit kaybı olan elemanlarda yapısal değişim gerekirken, tabliye levhası ve mesnetler kritik bulunmamıştır. Yapısal muayeneler sonucunda boyuna kirişlerin enine kirişlere olan bağlantılarının yoğun şekilde korozyona uğradığı saptanmış ve bazı yerlerde bu eleman kalınlığı boyunca ilerlemiştir. Bu elemanlarda onarımların yapılması gerektiği sonucuna varılmıştır. Tabliye levhasındaki korozyon kaybını sayısal olarak tanımlamak, araçların geçişi korozyon durumunu önleyeceğinden oldukça zordur. Bu sebeple tabliyenin kalan kalınlığını doğrulamak için linkspan köprüsü boyunca ultrasonik kalınlık ölçümü düşünülmelidir.

Linkspan köprüsünün yorulma direncinin değerlendirilmesinde 30 yıllık tasarım ömrü boyunca her gün dört gemi seferi yapılacağı varsayımıyla ve köprünün bu yükleme etkisinde olacağı düşünülerek hesaplamalar yapılmıştır. Linkspan köprüsünün ikinci yapılan genişletme sonrasında kurulan mevcut geometrisi, bu değerlendirmeye temel teşkil etmektedir. Linkspan standardında belirtilern yönergelere göre, yorulma ömrü 30 yıl olarak kabul edilmiş olup, mevcut durumdaki yapı davranışıyla örtüşmektedir.

Yorulma hesaplamaları sonucunda, linkspan köprüsünün maruz kaldığı yüklemeler sonucunda oluşan gerilmelerin, eşdeğer nihai izin verilen gerilmelerden daha düşük olduğu ve dolayısıyla yorulma dayanımlarının kabul edilebilir sınırlar içinde olduğu ortaya koyulmuştur. Araç hareketlerinden kaynaklanan döngüsel yüklemenin yüksek frekansı, nispeten küçük gerilme döngüleri ile sonuçlanmakta ve bu nedenle genel yapısal bütünlüğü önemli ölçüde etkilememektedir.

Yorulma değerlendirmesi, seçilen üç detayın öngörülen 60 yıllık yorulma ömrü için yorulma açısından kritik olmadığını belirlemiştir ki bu, son yapılan genişletilmiş yapı temel alınarak yapılmıştır. Bu nedenle, yapının kalan 34 yıl boyunca hizmet etmesi gerektiği sonucuna varılabilir.

Tez kapsamındaki bulgulara ek olarak tavsiyelere de yer verilmiştir. Korozyon oranlarını her bir eleman başına daha iyi anlamak için kapsamlı durum araştırması yapılmalıdır, örneğin her bir eleamnda farklı kalınlık okumaları alınarak ve bugüne kadar olan korozyonun doğrusal mı yoksa daha komplike eğilimde mi olduğu belirlenebilir.

Deniz ve liman yapıları için korozyon önemli bir faktördür. Bu nedenle, önleyici tedbirler yapıların tasarım ömründe hayati bir rol oynar. Yapısal onarımlar ve korozyon koruma önlemleri, linkspan yapısının kalan tasarım ömrü boyunca hizmet vermesini sağlamak için uygun şekilde uygulanmalıdır.

Linkspan yapısının performansını korumak için bir bakım programı da önerilmektedir. Bu program, rutin denetim planı ve bakım programları dahilinde takip edilebilir; planlı bakım ve periyodik olarak yapılan rutin bakım çalışmalarını içerir. Etkin bakım, gözlemlenen kusurlar sırasında alınacak önlemleri tanımlar. Genel bakım, üyelerin değişim çalışmaları ve gerekli dönemini tanımlar. Ana kirişler, enine kirişler, tabliye levhası ve boyuna kirişler için denetimler, su sızma işaretleri ve korozyon kontrolü ile kaplama sisteminin değerlendirilmesini içerebilir. Büyük kusurları ve yorulma çatlaklarını önlemek için yıllık rutin denetimler önerilirken, reaktif bakım gerektiğinde başlatılmalıdır.

Kıyı ve liman yapılarında etkin ve genel bakımlara ek olarak yıllık rutin denetimler de tavsiye edilir. Kıyı kısmında yer alan mesnetler ve ek bölgesisnde yer alan hidrolik tamponlar işlevsellik kontrollerine tabi tutularak yüzey koruma sistemleri de kontrol edilmelidir. Tabliye yüzeyi, araba yolu ve yaya yolu korozyon ve arayüzlerde aşınma için denetlenebilir.

1. INTRODUCTION

Throughout the history of the linkspan structures the functionality requirements evolve and shall fit for both environmental effects and future vessels dimensions. Considering the tidal range and combined with the minimum and maximum service levels, the linkspans shall be designed to fit various geometrical and environmental requirements or modified to fit for purpose. In this context, the design and maintenance of linkspans necessitate careful consideration of factors such as tidal range, service levels, geometrical variations, and environmental conditions. However, ensuring the optimal performance and structural life of these structures requires a multifaceted approach encompassing structural integrity, corrosion protection, fatigue resistance, and sustainability considerations.

1.1 Purpose of Thesis

The purpose of this thesis is to investigate and analyze critical factors influencing the structural integrity and service life of linkspan structures, with a particular focus on the significance of fatigue life estimation and analysis. The following key aspects underscore the importance of this research:

Evolution of Design Criteria and Structural Capacity Assumptions: Over time, design criteria for linkspan structures have undergone revisions and updates in accordance with evolving standards and regulations. These changes directly impact structural capacity assumptions, necessitating a comprehensive examination of their implications on fatigue life and overall performance.

Marine Environment and Corrosion Effect: The marine environment poses significant challenges to linkspan structures, leading to early corrosion and degradation if not adequately addressed. By understanding the consequences of corrosion on structural integrity, this research aims to emphasize the criticality of implementing corrosion protection measures and regular maintenance practices to mitigate adverse effects and ensure prolonged service life.

Regular Maintenance Practices: Effective maintenance plays a pivotal role in preserving the structural integrity of linkspan structures, particularly in combating corrosion-induced deterioration. This thesis aims to highlight the importance of implementing systematic maintenance protocols to prevent corrosion-related issues, thereby enhancing structural resilience, and prolonging the operational lifespan of the linkspans.

Significance of Fatigue Life Estimates and Accurate Analysis: Fatigue failure represents a major concern for linkspan structures, given the cyclical loading conditions they experience throughout their service life. Therefore, accurate estimation and analysis of fatigue life are imperative for assessing structural performance and predicting potential failure modes. By investigating methodologies for fatigue life estimation and conducting rigorous analysis, this research looks to enhance understanding of fatigue behavior and inform strategies for optimizing design, maintenance, and operation.

In essence, this thesis endeavors to address the sophisticated challenges associated with linkspan structures, emphasizing the critical role of fatigue life estimation, corrosion mitigation, and proactive maintenance in ensuring long-term structural integrity and operational reliability amidst evolving design criteria and environmental conditions.

1.2 Literature Review

Structural Life Assessment of the linkspan structures are not quite common compared to other steel highway bridges or footbridges. Therefore, the studies are conducted as per the bridge codes such that the fatigue evaluation is defined in BS 6349-8 – Maritime Structures - Part 8: Code of practice for the design of Ro-Ro ramps, linkspans and walkways.

The design of linkspan structures becomes more of an issue with the collapse of Ramsgate Passenger Walkway in 1994 which has forced the industry and the maritime engineers to discuss and develop the industry guidance for the ship to shore access structures. Chapman (1998) clearly explains the major cause of the accident, which was a design error in the bearing system for the high-level walkway. The implications of having a torsionally stiff passenger walkway spanning onto a floating (and therefore mobile) body were not considered and the stresses in the bearings exceeded their design values by a very large factor, (Osborn, 2010).

Osborn explains that following the Ramsgate incident distinctive design methods are developed by bridge industry and shipping industry which both approaches in a different design method. Bridge industry predicated ultimate limit state approach in design using BS5400 and Eurocodes whereas the shipping industry has developed working-stress approaches. Further in 2007, BS6349-8 adopted both approaches to govern linkspan procurement by the standard.

On the other hand, CIRIA Guidance C518 states the difficulty of assessment of an old linkspan structure and explains that accurate drawings of the facility and surveys shall be conducted to strengthen the reliability of the assessment. The detailed surveys/inspections cover visual survey, thickness measurements of the members, dimensional inspections, non-destructive tests, sampling of materials. Then further assessment shall be conducted to assess if the structure is subjected to repeated loadings which might have caused a reduction in the fatigue life of linkspan.

Kühn explains that the fatigue assessment procedure consists of several phases: Preliminary Evaluation, Detailed Investigation, Expert Investigation and Remedial Measures. The fatigue assessment can be carried out using S/N curves as well as fracture mechanics assessment in which this thesis S/N curve approach is followed.

1.3 Hypothesis

Over the life span of the steel structures corrosion and cyclic loadings are quite important effects that influence the overall performance and structural capacity. Regular maintenances are crucial especially for the marine structures that are subjected to heavy saline environment. Old structures are generally built with the conservative design approach compared to today`s standards and structural sections are bigger. However, the poor welding quality and lack of control procedures plays a vital role in reducing the structural design life of the structures.

The main purpose is to present the current condition of a linkspan structure with regular inspection methodologies and determine the defects which is then followed with building a structural FE model and performing a structural assessment. It is aimed to maintain the functionality of the structure with the minimum repairs whilst ensuring the repair methods provide sufficient capacity to the structure.

The other purpose is to highlight the importance of corrosion protection and regular maintenance regime of the steel structures. It is a vital and low-effort precaution to avoid corrosion.

It is presumed that the linkspan condition of use has changed over time and the defined service life is reached. Therefore, not only the structural inspection and assessment but also fatigue assessment is found necessary to complete the structural integrity. The historic documents studied then followed with a site visit and visual inspection of the bridge. The fatigue life assessment is conducted based on S/N curves assuming that the sufficient safety is achieved which will allow to calculate the remaining fatigue life of the linkspan bridge.

Corrosion fatigue is not taken into consideration with the assessment as it is the combined action of cycling stresses and a corrosive environment. However, it is well known that the fatigue failure is expected to take place in lower stresses and shorter time when the structures are exposed to corrosive environments.

2. LINKSPAN BRIDGE DESCRIPTION

2.1 Purpose

Ship-to-shore Linkspans are steel structures that provide an intermediate link between the shore and the Ro-Ro ships or ferries. The study linkspan is consists of fixed end at the shore and a floating end that sits on a pontoon structure to accommodate the tidal differences which can vary between 0.00m (LAT) to 5.73m (HAT)[1]

Taking into account the tidal differences and alterations in vessel`s draught and trim caused by the loading and unloading, the linkspans are subject to continuous movements and therefore cyclic loading.

In addition to that, as for the nature of the maritime structures, they are subjected to high salt content and are highly susceptible to corrosion so that require special protection to prevent corrosion.

It is aimed to present the actual condition of the linkspan by performing visual inspection followed by ultrasonic testing method to determine the reduced thickness of the members due to corrosion. Thus, two models are prepared with different scenario consisting of as-designed condition and the actual condition to assess the structural capacity.

The fatigue evaluation is performed considering the actual condition of the linkspan by performing time-history analysis. Critical areas are determined based on the vulnerability due to buckling and fatigue critical joints and the assessment is made based on detail category as per BS EN 1993-1-9.

2.2 General Description of Linkspan Structure

Ro-Ro Linkspans are designed for the vehicles to be loaded at the bow or stern of the ships at lower deck levels where the water level variations are considerably high such that to provide access to the ships regardless of the tide level or the position of the ship.

The linkspans consist of two structural elements: 1. Articulated Structure; connecting the shore with a point close to the ship that is moving up and down to take into account

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[1] Highest & lowest predicted tides at North Shields, UK - National Tidal and Sea Level Facility

the water level changes and different ship heights that are berthing in the terminal. 2. The Ramps (or Finger Flaps), mainly caters for movements of the ships, particularly rolling or pitching.

According to BS 6349-8:2007, there are four different types of linkspans that are commonly used in Ro-Ro Terminals:

(a) Mechanically Lifted Linkspan– these are raised and lowered using lifting equipment such as hydraulic cylinders, rope winches or rack and pinion

(b) Pontoon and Link Bridge – the interface with the ship is a restrained pontoon that floats up and down with the tide and is accessed by a link bridge. The floating linkspan is attached to the dock, with its access ramp resting on a pontoon to act as a level bridge between the quay and the vessel.

The floating linkspan transfers the force of the berthing vessel or its ramp to fenders which absorb the impact at the shore end. It also follows tidal variations and water levels in order to provide a smooth path for efficient loading and unloading.

The floating linkspan can be equipped with hydraulically operated ramps to enable it to function with a variety of vessels.

(c) Semi-submersible – its self-weight is supported by a submerged float but the traffic is supported by a connection to the ship (rope or shelf)

(d) Integral Tank – this is similar to pontoon, but the pontoon is rigidly connected to the bridge.

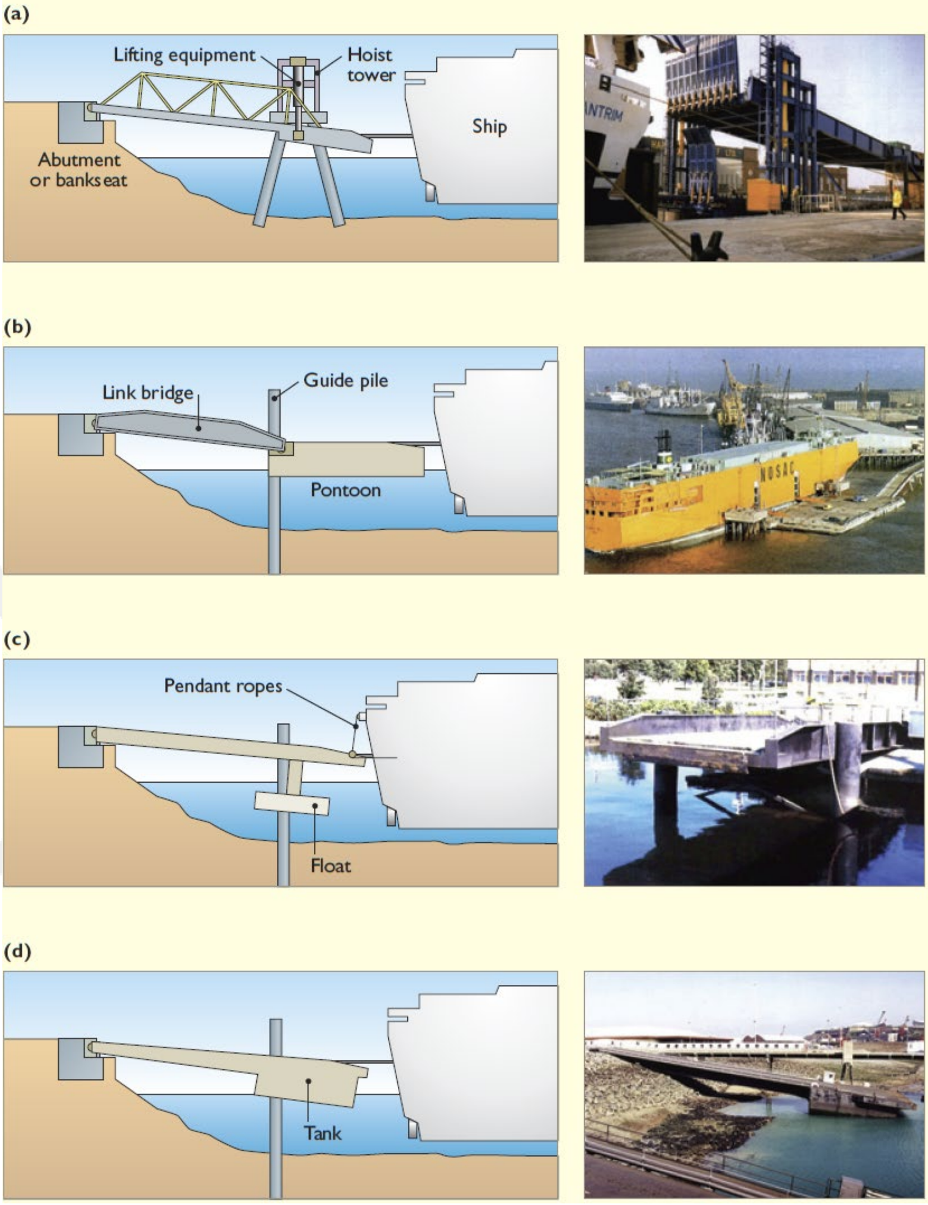


Figure 2.1 : Linkspan Types (Osborn, 2010).

2.3 Bridge Geometry

The linkspan has been designed as a mechanically lifted structure in 1960s and originally suspended from a fixed gantry (Figure 2.2). The linkspan has since been extended on 2No. occasions and the outer end is now supported on a pontoon. Based upon historic photographs, it is assumed that the first extension was in/around 1976 although this is unconfirmed, and no drawings of the works undertaken are available. The second most recent extension was in 1998 and some construction drawings are available for these works.

The linkspan is subjected to general highway traffic as well as port vehicles.

The old linkspan currently has a length of approx. 32.28m, 5.18m between centers of edge beams and is supported by 4No. hinged bearings. To provide lateral resistance, the linkspan is restrained by 2No. dolphins close to the pontoon (Figure 2.3).

Figure 2.2 : Historic photo from seaward end of the original suspended linkspan.

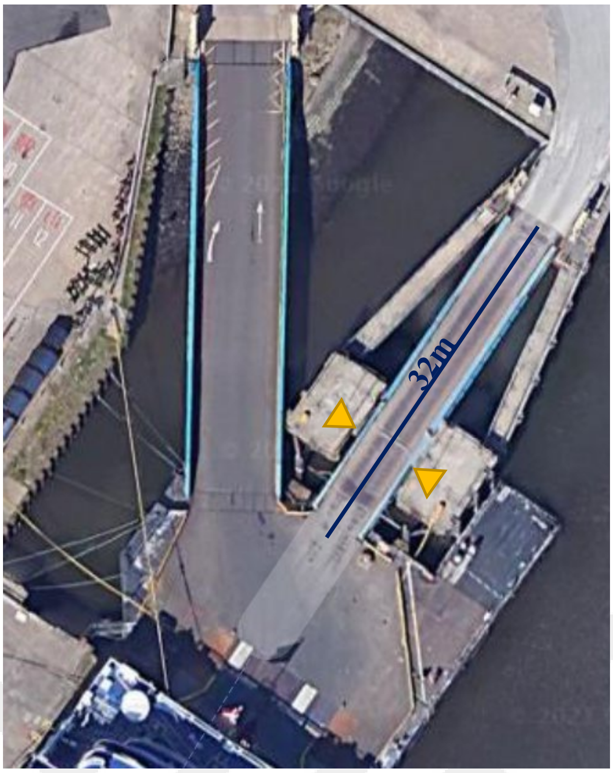


Figure 2.3 : Horizontal support condition of the bridge.

The main load bearing structure consists of 2No. I-section welded plate girders which are connected to each other by transverse cross girders at regular intervals (see Figure 2.4). The cross girders are rolled I-beam sections. The cross girders support twin-channel longitudinal stiffeners with an overlying welded steel deck. As well as supporting the deck structure, the cross girders are welded into full-depth vertical web stiffeners on the main girders which provides discrete lateral U-frame restraints to the compression flange of the main girders in order to resist lateral torsional buckling.

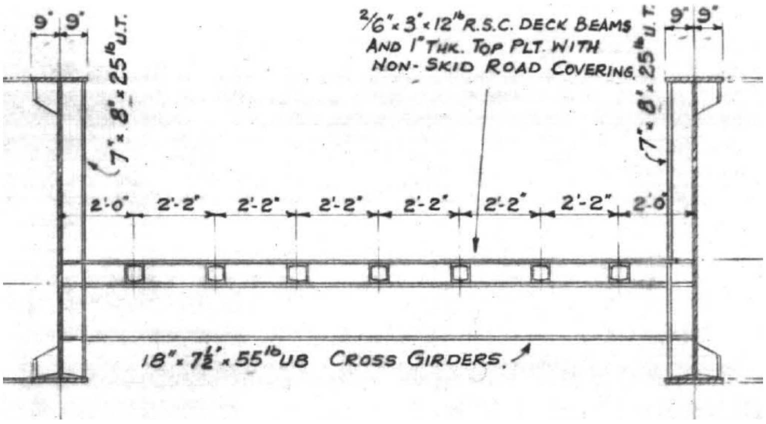


Figure 2.4 : Typical cross section of the linkspan.

The linkspan structure has been modified and extended over the years as the linkspan type has changed from mechanically lifted to pontoon and link bridge. The extensions are further given in Figure 2.5 below.

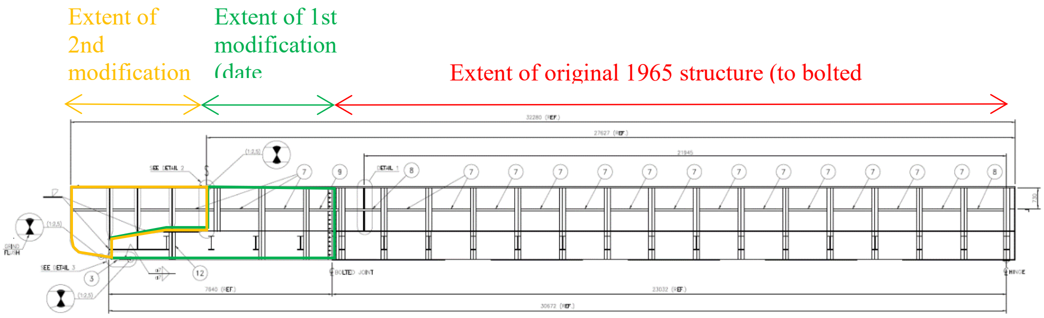


Figure 2.5 : Original and extensions to the linkspan.

3. STRUCTURAL INSPECTION

3.1 Inspection Summary

A visual inspection has been carried out as walkover inspection of topside of the deck and accessible areas and boat inspection for the soffit of the deck and for the stiffeners. The inspections found that the condition of the old linkspan structure is mixed, with the above deck elements generally being in reasonable condition and the below deck elements being in fair to poor condition.

Figure 3.1 : Location Plan.

3.2 Inspection Method

Walk-on inspection has been made to determine the visual defects that are noticeable at first sight which is then followed by the visual inspection by providing access to the soffit of the linkspan deck at both land side and from the pontoon. The mid span of the bridge was inspected when the tide rose such that the hand access was available to have thickness readings from the members.

The visual inspection is carried out starting with the photographic record of the overall linkspan bridge and the structural members, followed with the thickness measurements and UT thickness readings at the spot check and the suspicious points.

It was not found necessary to perform NDT methods like magnetic testing (MT) or ultrasonic testing (UT) on the existing welds after undertaking the visual inspection.

The structural bearings are visually inspected where possible with hand in reach. For the inaccessible areas inspection cameras have been used.

3.3 Inspection Findings

The inspections did identify that there was heavy corrosion of the members and bearings in several areas. A structural assessment of the old linkspan structure was therefore found necessary to assess the structural capacity and criticality of the corrosion which is further explained in the following sections.

3.3.1 Deck plate and member inspection results

The deck plate itself, although heavily corroded and unprotected by a surface coating in general, appeared to be in reasonable condition in general. The corrosion was noted as uniform and appeared to be surface corrosion along the linkspan whereas localised corrosions observed at deck to main girder interfaces.

Figure 3.2 : Overall Condition of Deck Plate – Pontoon End.

Figure 3.3 : Overall Condition of Deck Plate – Abutment End.

Deck plate was originally coated with anti-skid surface where remains can visually be seen at both edges of the road. The passage of vehicles serves to 'polish' the corrosion product off the steel plate and it is therefore possible that the section loss could be more significant than it appears.

A substantial build-up of detritus was noted at the interface between the deck plate and main girder. This encourages corrosion and obscures defects from being visible further shown in Figure 3.4

Figure 3.4 : Main Girder – Deck Plate Connections.

It was observed that the deck soffit has linear surface corrosion where the strips are welded on the deck plate from the top provided in Figure 3.5.

There were also spot corrosions observed on the welds to deck soffit.

Figure 3.5 : Deck Soffit – Corrosion at Strip Locations.

3.3.2 Cross girders and longitudinal stiffeners

Several of the longitudinal deck stiffeners are in a poor condition with through thickness corrosion evident in places. This defect is significant and will lead to a local reduction in the carrying capacity of the deck.

Figure 3.6 : Highly Corroded Longitudinal Stiffeners and Moderate Corrosion on Cross Girders at Abutment End.

Figure 3.7 : Highly Corroded Cross Girder at Pontoon End.

Figure 3.8 : General Attack Corrosion at Bottom Flange of Main Girder and Localised Corrosion at Cross Girders – Extension Area.

A defects map has been prepared to present the factual situation of corrosion levels observed on cross girders and longitudinal stiffeners shown in Figure 3.9. Red areas indicate heavy corrosion level, significant through thickness corrosion result with section loss. Amber areas indicate the moderate localised corrosion and green areas are the low level spot corrosions where mild surface corrosion is observed.

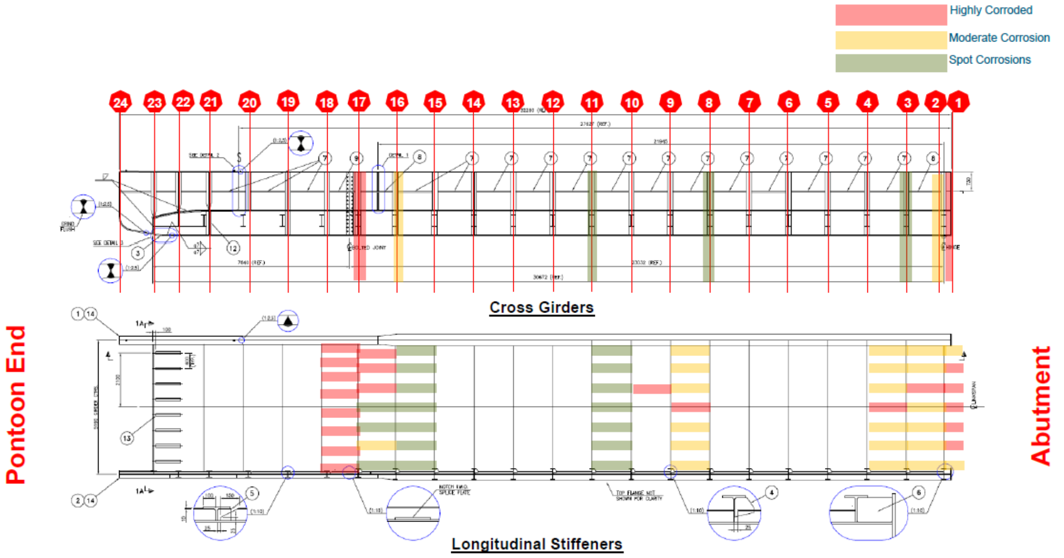


Figure 3.9 : Defects Map.

3.4 Next Phase Inspections

Following the visual inspection, it was observed that the condition of the longitudinal stiffeners and cross girders appeared to be worse than anticipated after the cleaning and blasting works carried out on the linkspan. It is envisaged that after cleaning the surface of the members from debris and contaminated dirt, the defects made visible.

The shot blasting is the application of spreading the grit particles over the steel surface with high pressure. Therefore, expectedly delaminated corrosion layer breaks off making it severe for the member where obvious section losses observed.

The objectives of the inspection were to:

- Visual check the condition of the longitudinal stiffeners and cross girders
- Determine if other defects occurred that are different from the previous situation
- Take thickness readings on the suspected members
- Update the defects map and carry out structural assessment to provide repair solutions

Dimensional measurements carried out to measure the thickness and length for the areas where section losses were obvious as seen in Figure 3.10.

Figure 3.10 : Dimensional Inspection of Longitudinal Stiffeners.

Random UT thickness measurements taken on the areas that were identified with heavy and moderate corrosion as shown in Figure 3.11.

Figure 3.11 : UT Thickness Measurements on Longitudinal Stiffeners.

Following the visual inspections and initial assessment based on the findings, structural defects map has been updated representing the factual situation of corrosion levels and defects as shown in Figure 3.12. The defects map is provided in Appendix B.

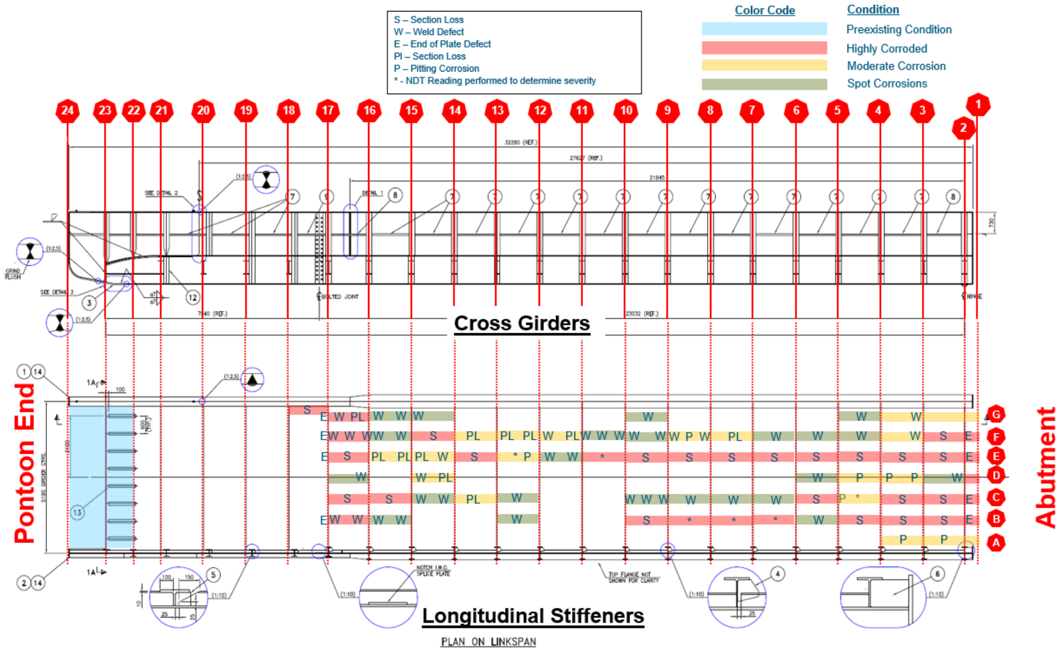


Figure 3.12 : Defects Map Indicating Corrosion Levels.

Thickness readings are made for the locations where the section losses were not obvious on members which are listed for cross girders and longitudinal stifffeners in Table 3.1 and Table 3.2 respectively.

Table 3.1 : Thickness Readings on Cross Girders.

Description		Nominal Thickness	Measured Thickness
2 | A-G	Web	10	8.9
	Flange	16	15.4
3	Web	10	9.7
	Flange	16	15.8
8	Web	10	9.6
	Flange	16	15.9
11	Web	10	9.8
	Flange	16	16.0
16	Web	10	9.0
	Flange	16	16.0
17	Web	10	8.5
	Flange	16	14.8

Table 3.2 : Thickness Readings on Longitudinal Stiffeners.

Description		Nominal Thickness	Measured Thickness
C | 4-5	Web	5.5	4.8
	Flange	10	7.8
B | 4-5	Web	5.5	3.2
	Flange	10	8.0
B | 5-6	Web	5.5	4.8
	Flange	10	8.2
B | 6-7	Web	5.5	5.4
	Flange	10	8.2
B | 7-8	Web	5.5	4.7
	Flange	10	8.0
B | 8-9	Web	5.5	5.1
	Flange	10	8.3
D | 10-11	Web	5.5	4.6
	Flange	10	8.9
C | 17-18	Web	5.5	4.3
	Flange	10	8.0

It was observed that after the grit blasting application at site, the delaminated corrosion layers were broken off making it severe for the member where obvious section losses observed. Especially on the abutment end where the longitudinal stiffeners bear on the end cross girders, there were severe section loss at both the junction of the cross girder and the deck plate.

3.5 Comparison With The Existing Condition

Photographic record is provided in comparison with the initial inspection where the pre-existing condition is shown.

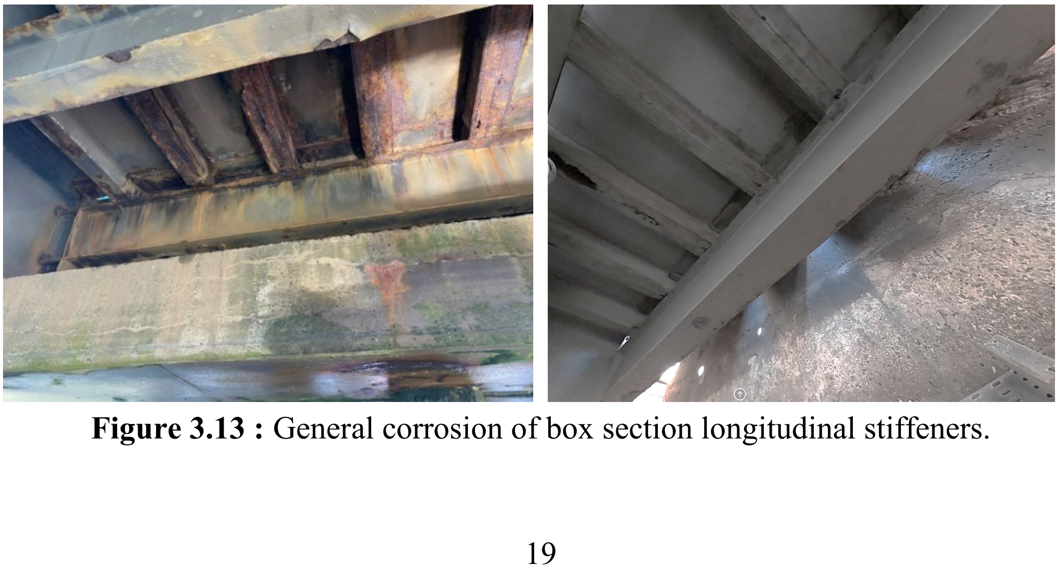


Figure 3.13 : General corrosion of box section longitudinal stiffeners.

Figure 3.14 : General corrosion of box section longitudinal stiffeners.

Figure 3.15 : Corrosion of seam weld on longitudinal stiffeners.

Figure 3.16 : Surface corrosion to transverse beams and longitudinal stiffeners.

Figure 3.17 : Heavier corrosion beneath opening for redundant winching point.

4. LINKSPAN BRIDGE STRUCTURAL MODEL

4.1 Purpose

In the light of the structural inspections, it was identified that there was heavy corrosion at several structural members, and it has been a concern for the structural capacity of the linkspan under the corroded condition.

The objective of the structural assessment is to perform calculations on each of the principal structural elements of the old linkspan in terms of strength and stability according to the current design standards and guidelines.

Structural model of the linkspan has been modelled in SCIA Engineer Analysis software based on the as-built drawings and several cases has been applied to the analysis. The assumption is based on the findings of the structural inspection such that the thickness reduction of the members is applied to the section properties of the members and the analysis has re-run simulating the actual, corroded condition.

Having performed the structural analysis, the capacity of the linkspan is determined and further on time-history analysis made to continue with the fatigue resistance.

The assessment is conducted for the as-built condition of the structure based upon the archive drawings as well as assessing the sensitivity of the results to 10% section loss from corrosion that are based on the inspection findings further detailed in Section 3.4.1. Even though the thickness readings were found sufficient at several locations, it would be prudent to have 10% section loss assumption taking into account the future corrosion that might occur in time.

The fatigue assessments are performed based upon Eurocode-3 in accordance with the Safe Life Method as specified in BS EN 1993-1-9.

Structural connections are excluded from the current assessment.

4.2 Material

4.2.1 Material properties

The steel grade for the original structure (1965) and the second extension is considered as Grade S275 steel. For the purpose of the assessment, it is assumed that steel equivalent to S275 (mild steel) is used throughout (see Table 4.1).

Table 4.1 : Overview of assumed material properties.

Steel Grade	Yield Strength f_y [N/mm^2]	Tensile Strength f_u [N/mm^2]	Elasticity Modulus E [N/mm^2]
S275	275	430	205000

4.2.2 Partial safety factors for materials

BS6349-8 refers to BS 5400-2 for load combinations, where the partial safety factors are identified in BS5400-3 that are used in calculating the design resistance of the structural elements are given in Table 4.2.

Table 4.2 : Overview of material partial safety factors for steel bridges.

Description	γ_m
ULS checks, except situation at below	1.05
Strength of longitudinal stiffeners	1.20 (compression) 1.05 (tension)
Buckling resistance of stiffeners	1.20
Fasteners in tension	1.20
Fasteners in shear	1.10
Friction capacity of HSFG bolts	1.30
Welds	1.20

4.3 Load Actions

4.3.1 Dead loads

The dead loads considered are the self-weight of the bridge structure and the surfacing load on the steel deck.

4.3.2 Dead load of structure

The dead load of the structure is automatically calculated by finite element programme, SCIA Engineer. The software calculates the dead load based upon the following unit weight:

Unit weight steel γ_s = 7850 kg/m3

The web of the main girders is stiffened by transverse and longitudinal stiffeners which are not included in the structural model. In order to account for the dead load of these stiffeners, the unit weight of the main girders is increased by 10%. Similarly, there are various gusset plates welded to the cross girders and the unit weight of the cross girders is increased with 5% to account for this. The corrected unit weights are as follows:

Unit weight steel main girder γ_s = 8650 kg/m^3

Unit weight steel cross girder γ_s = 8250 kg/m^3

4.3.3 Dead load of surfacing

For the assessment, it is assumed that an epoxy layer is applied as surfacing on the steel deck and a uniformly distributed load of 0,2 kN/m^2 is allowed for this. This is applied as a surface load in the model. It should be noted that design checks will be required if a heavier form of surfacing is applied in future.

4.3.4 Traffic loads

The distance between the centre line of the webs of the main girders is 5182mm (17’). The flanges of the main girders in the original section are 457x25mm (18”x1”). The carriageway width of the bridge is determined by the distance between the edge of the top flanges and is approximately 4.74m (see Figure 4.1).

A design in accordance with BS 5400-2 would generally allow for a restraint system (e.g. kerbs or barriers) inboard of the main girders which would provide an offset of 0.6m. In terms of the bridge structure, this tends to reduce the load concentration into the main girder on that side and reduces the utilisation.

However, there is currently no restraint system installed and the main girders are effectively used as the restraint system. For the purposes of the assessment, the vehicles are assumed to be able to manoeuvre up to the inside edge of the upper flange.

Considering that the traffic lane has a width of 3.0m only one traffic passage is considered and the traffic load actions are placed at the most unfavourable location for each structural element.

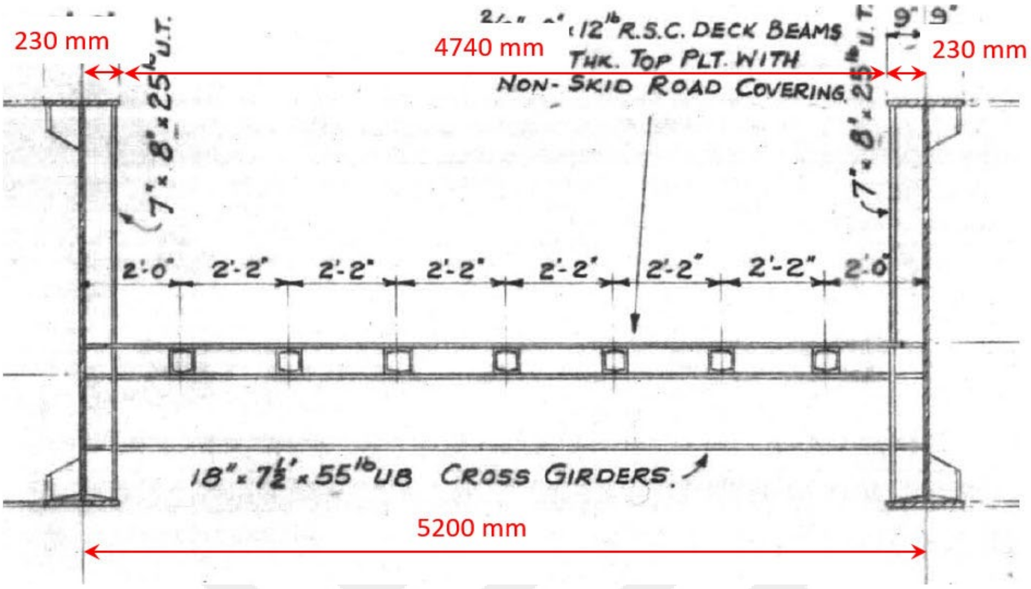


Figure 4.1 : Carriageway width of the bridge.

4.3.4.1 Load model

Loads consist of the representing single vehicles and convoys of vehicles on the linkspan where the live loads are positioned in such a way that the most adverse loading effects will occur on the considered element. CS 454 identifies for the bridges of loaded length less than 36m pedestrian live load to be taken into account as 5kPa uniform load is simultaneously applied to other areas of the deck away from the vehicles.

Main Girder

The wheels are positioned immediately inside the main girder to concentrate the load into that beam (see Figure 4.2). The vehicles are tracked along the deck to give the worst case bending and shear loads – worst case bending generally with maximum load in a central location and worst-case shear generally with maximum load close to one end of the linkspan.

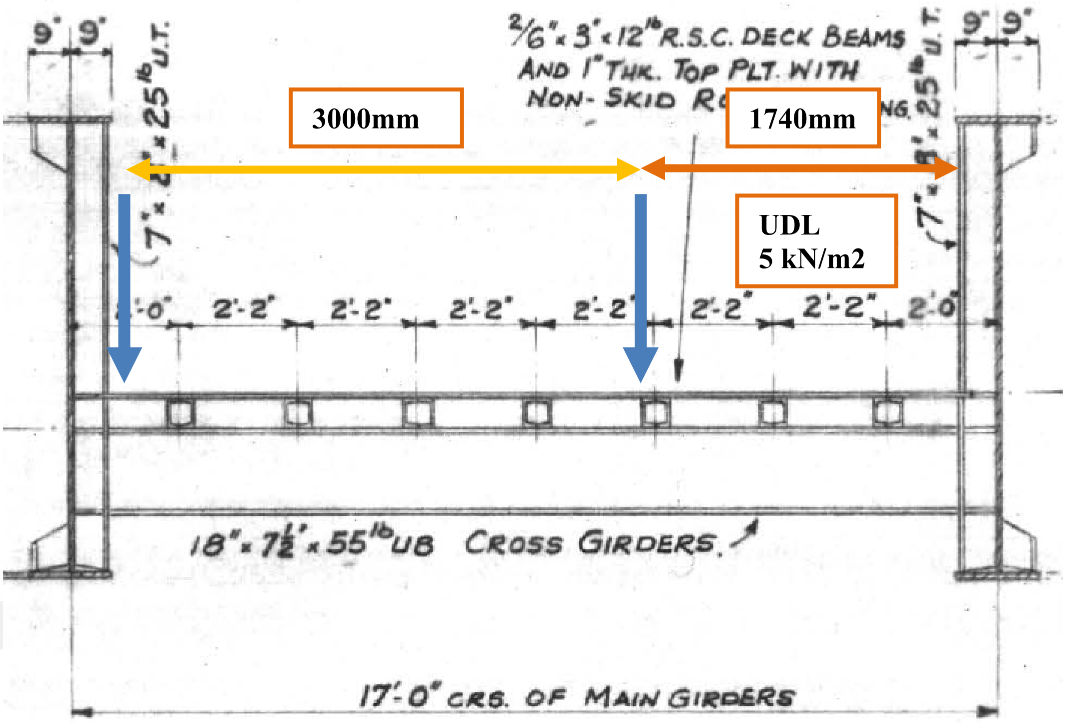


Figure 4.2 : Unfavorable wheel load configuration for main girder.

Cross Girders

The most unfavourable loading configuration for the cross girders is similar to the main girders with the more central row of wheels leading to maximum moment and the row of wheels close to one side leading to maximum shear.

Deck and Longitudinal Stiffeners

Different wheel loading configurations are used to represent the most onerous global and local effects on the deck and stiffeners. For the unfavourable global behaviour, the wheel loads will be positioned around the middle of the bridge and between two cross girders (see Figure 4.3). For the unfavourable local behaviour, one wheel will be located on the stiffeners and the other wheel will be placed between two stiffeners such that the local bending of the deck can be checked.

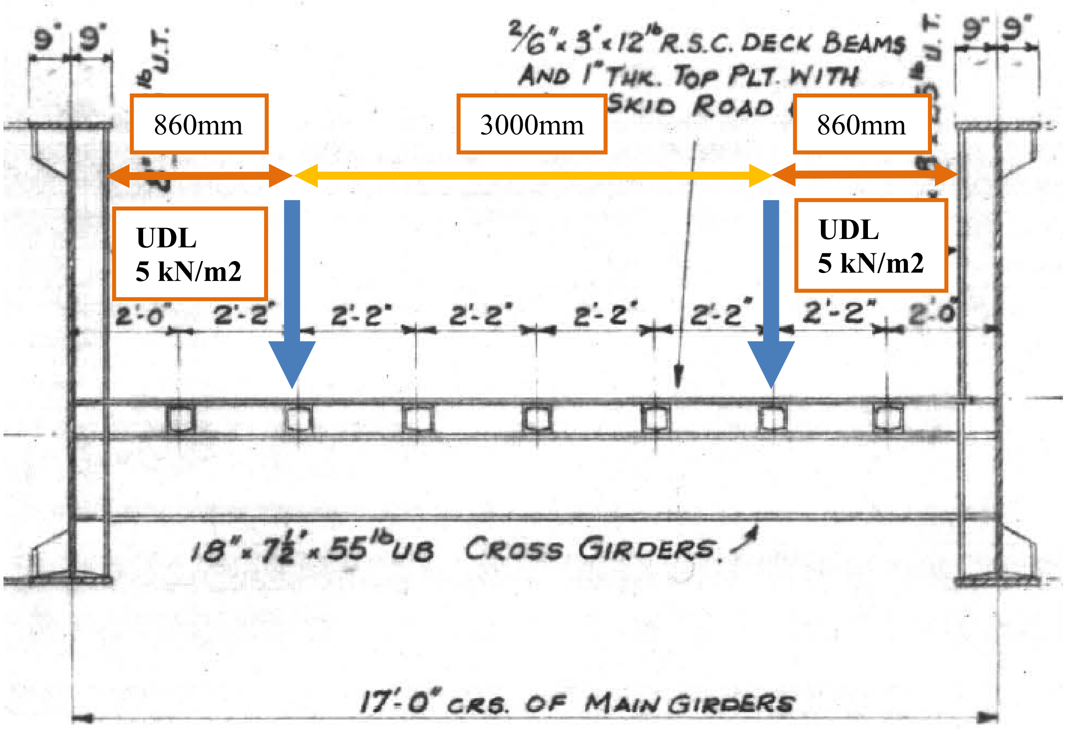
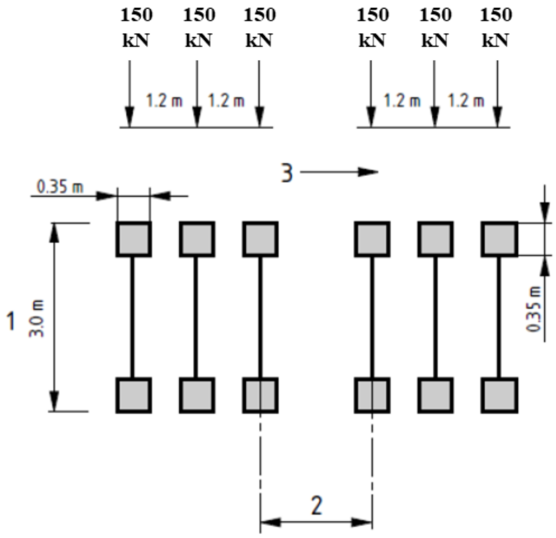


Figure 4.3 : Unfavourable wheel load configuration for deck and longitudinal stiffeners.

4.3.4.2 Special order traffic

The governing special-order vehicle consists of a single 90t SV90 cargo which is based upon properties that the axle load is defined as 150kN to achieve 90t loading.



Key
1 = Outside track and overall vehicle width
2 = Critical of 1.2 m or 5.0 m or 9.0 m
3 = Direction of travel

Figure 4.4 : Axle and load configuration of SV90 vehicle.

The wheel loads are distributed through the deck and surfacing giving a patch load over an area of 0.3625m x 0,3625m. According to this wheel contact area, the wheel load can be converted to a surface loading as in equation (4.1).

$$q_w = {}^{\mathrm{F}}/_{(\mathrm{d} \cdot \mathrm{d})} = {}^{75}/_{(0.3625 \cdot 0.3625)} = 570.75\ \mathrm{kN/m^2} \qquad (4.1)$$

4.3.5 Wind actions

The wind action on the linkspan bridge is determined according to CS 454. For the determination wind action on the bridge, the following assumptions are made:

- Basic hourly mean wind speed: 25 m/s
- Return period: 50 years
- Altitude and direction factor: 1.00
- Upwind distance: 1 km
- Distance to town edge: 3 km
- Height above ground: 5 m
- Topography factor: 1.00

The wind load is determined for the maximum gust speed in the transverse, longitudinal and vertical directions respectively as follows:

- Wind load transversal direction: 143.2 kN
- Wind load longitudinal direction: 35.8 kN
- Wind load vertical direction: 133.8 kN

In reality, the transverse wind load will predominantly act on the web of the main girder while the longitudinal and vertical wind loads will act on the deck. In the structural model, the main girder is modelled as a line element on which it is not possible to assign a surface load. Therefore, the transverse wind load is converted a line load which is applied to in the core of the deck. The bending moments caused by the main girder cantilevering above the deck are also applied to represent the combined effect.

Wind load for model input:

- Wind load in transversal direction
  - Line load: 143.2/32 = 4.48 kN/m
  - Moment: (143.2/(3.49*32))*2.492/2 = 4.0 kNm/m

- Wind load in longitudinal direction
    - Surface load on deck; 35.8/(5.18*32) = 0.21 kN/m^2
- Wind load in vertical direction
    - Surface load on deck; 133.8/(5.18*32) = 0.81 kN/m^2

4.3.6 Seismic actions

The linkspan is not in seismic zone. Hence, the seismic actions will not be governing and are not taken into account in the analysis.

4.3.7 Thermal actions

Thermal actions are not taken into account in this study.

4.3.8 Snow loads

The linkspan is not in operation when covered in heavy snow. Hence, the effects from snow load will not be governing and are not considered further.

4.3.9 Load combinations

The partial load factors for combinations are determined based on CS 454. The assessment is only performed for ultimate limit states where the relevant load combinations are summarised in Table 4.3.

Table 4.3 : Partial factors for load combinations.

Load	Combinations			
	1	2	3	4
Dead Load Steel	1.05	1.05	1.05	1.05
Dead Load Surfacing	1.75	1.75	1.75	1.75
Wind Load	-	1.40	-	-
Traffic Actions (UDL)	1.30	1.10	1.10	1.10
Special Order Traffic	1.10	1.00	1.00	1.00

4.4 Structural model

To complete the structural assessment of the linkspan, a finite element (FE) model was built in SCIA Engineer Analysis software (see Figures 4.5 and 4.6). The model was based upon the details shown in the archive drawings.

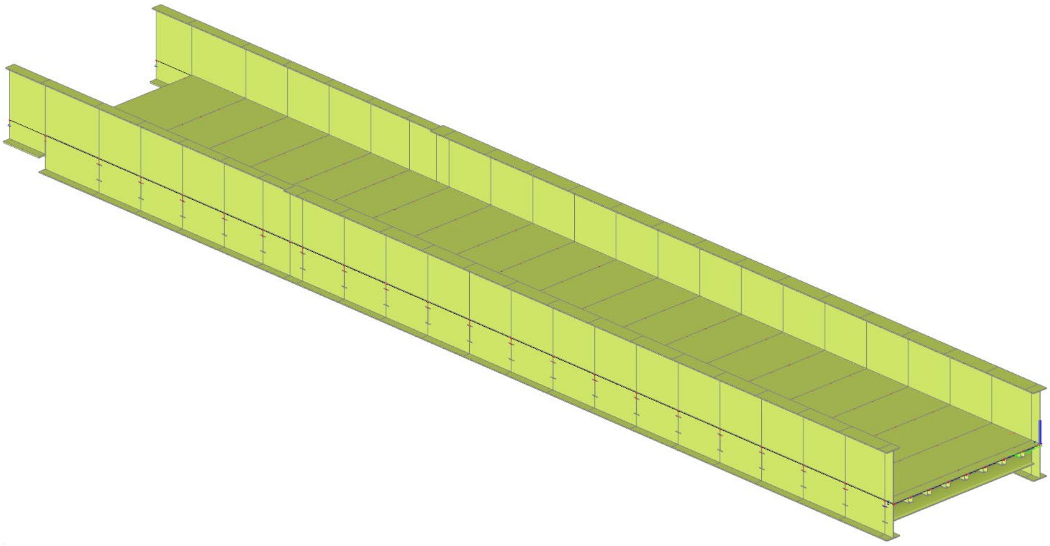


Figure 4.5 : General view of structural model.

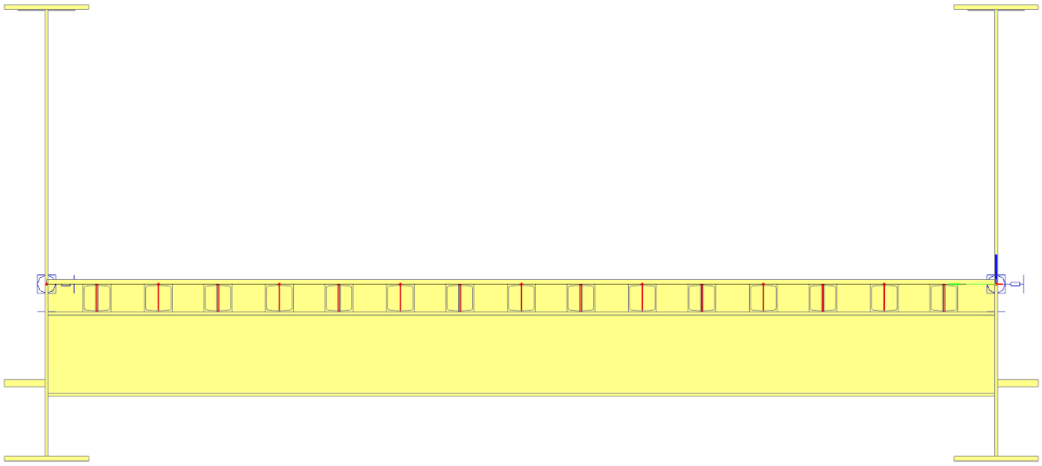


Figure 4.6 : Section view of structural model.

4.4.1 Main girders

The main girders of the bridge consist of I-section plate girders where the web plate is welded to top and bottom flanges. The web plates have a thickness of 15mm and height of 2440 mm. The flanges are made from plates which have a width of 457mm and a thickness of 25 mm. At the pontoon end, the width of the top flange is reduced to 310 mm over a length of approximately 10m where the linkspan was extended.

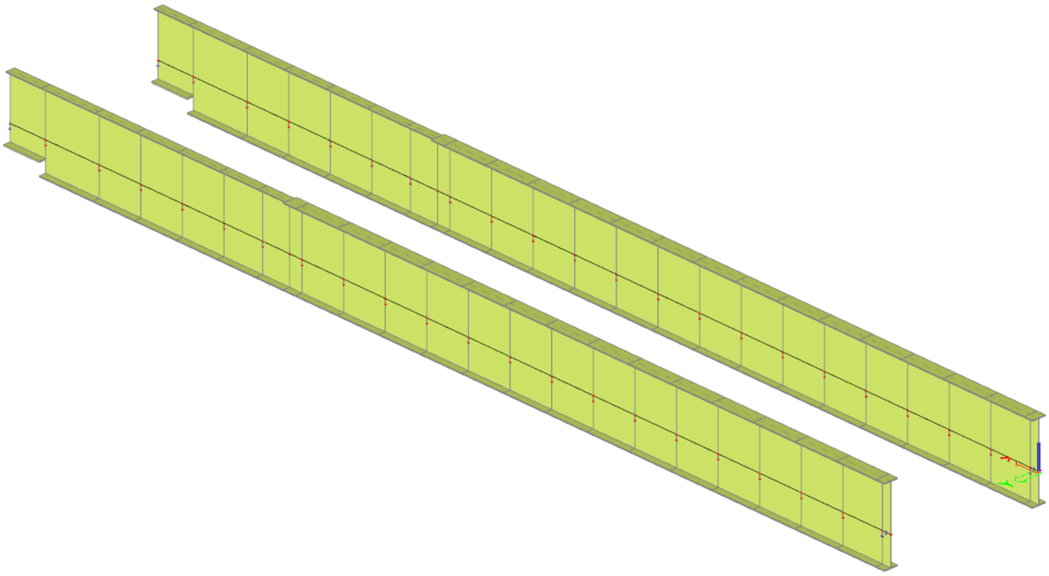


Figure 4.7 : Main Girder of the bridge.

The web of the girder is stiffened by longitudinal and transverse stiffeners. The transverse stiffeners are, made of a T profile while angle profiles are used for the longitudinal stiffeners. These stiffeners are not modelled because they will not affect global behaviour of the girders and their main function is to prevent local buckling.

4.4.2 Cross girders

The cross girders are made from rolled I shape sections equivalent to 457x191x82UB profiles (see Figure 4.8). The spacing of the cross girders is 1525 mm and these girders are connected to the main girders with fully welded moment resistance connections. Similar to the main girder, the cross girders are modelled as 1D bar elements. In order to model the interface with the overlying longitudinal deck stiffeners, the cross girders are offset below the deck stiffeners and connected with rigid connections.

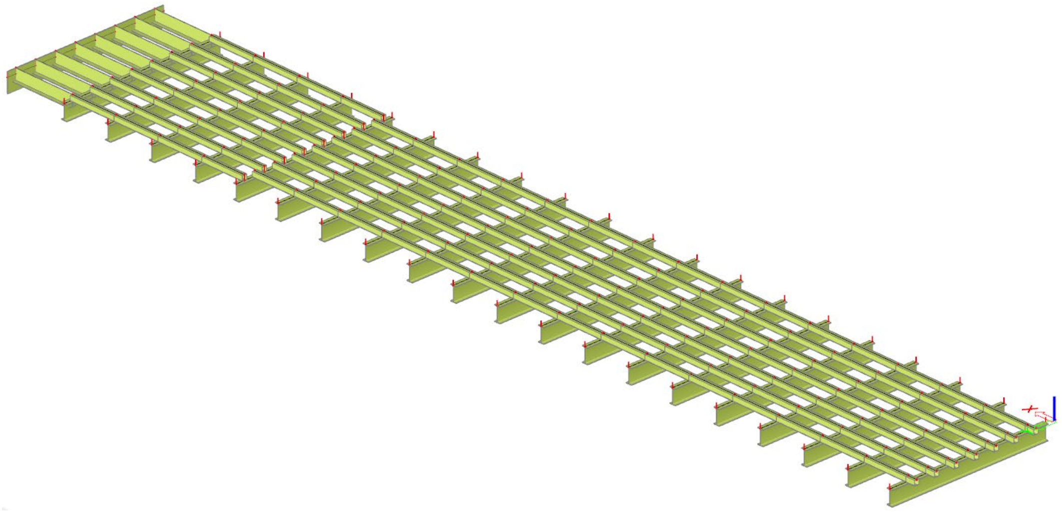


Figure 4.8 : Cross girders of the bridge.

4.4.3 Deck structure

The structure of the bridge deck consists of steel deck plate with underlying longitudinal stiffeners which are welded between the deck and cross girders (see Figure 4.9). The steel deck has a thickness of 25mm which is welded to longitudinal stiffeners as well as to the web of the main girders. The lon-gitudinal stiffeners consist of 2No. toed-in channel sections welded together to form a rectangular hollow section. The stiffener spacing is 660mm.

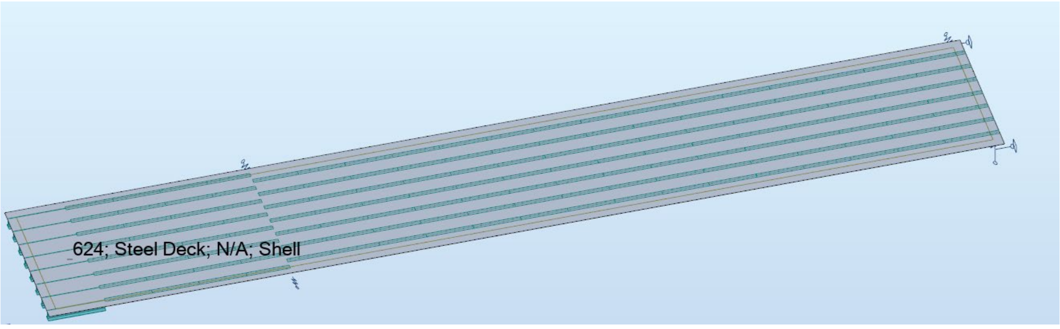


Figure 4.9 : Deck structure and cross section of deck structure.

From the earlier inspections, it can be seen that the longitudinal stiffeners are non-continuous where the linkspan was previously extended (see Figure 4.10). In addition, the longitudinal stiffeners in the extension appear to be a rolled rectangular hollow section. There are no section details available from the archive drawings of the extension so the assessment considers an equivalent section to the lon-gitudinal stiffeners in the original construction. The offset between the two sets of stiffeners is also considered in the model.

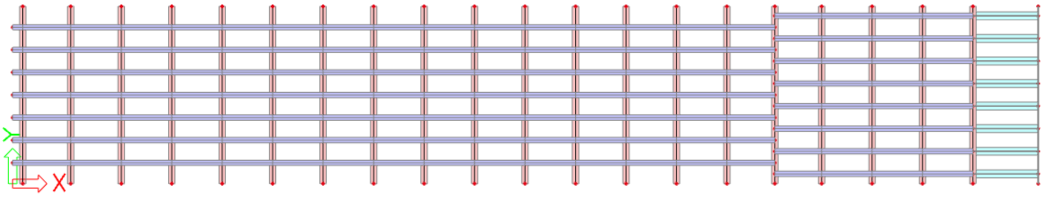


Figure 4.10 : Location adjustment of stiffeners from first extension of the bridge (above) and implementation in model.

During the last extension of the bridge, T sections were used as longitudinal stiffeners at the pontoon end of the bridge. This modification is also implemented in the model (see Figure 4.11).

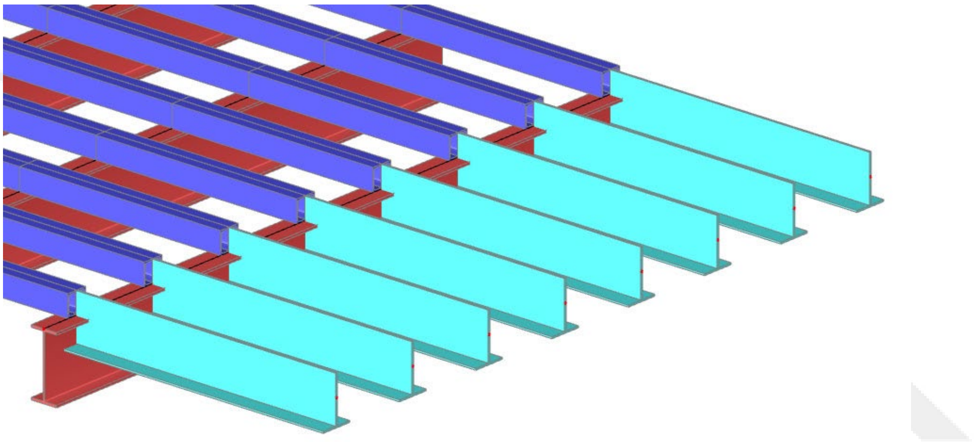
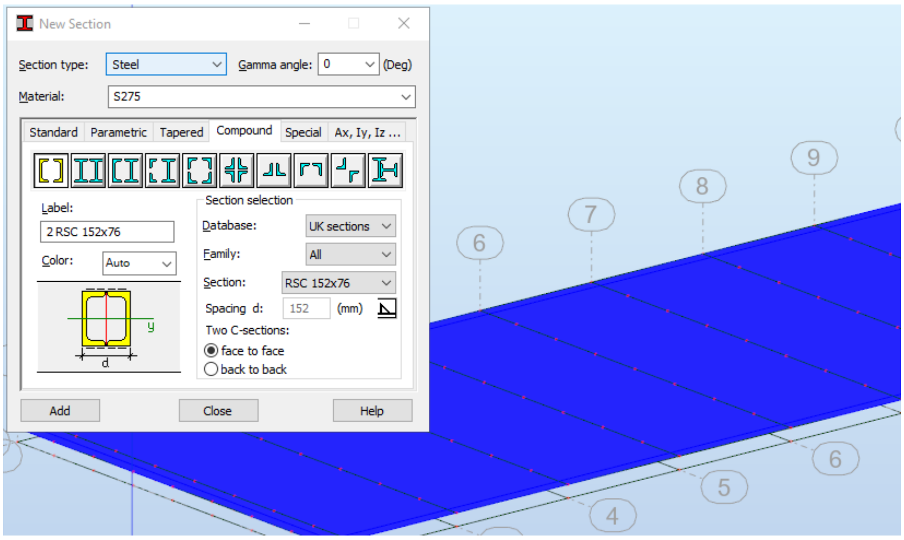


Figure 4.11 : Longitudinal stiffeners after the last extension of the bridge.

4.4.4 Longitudinal stiffeners

Longitudinal Stiffeners are compound box sections consisting of two RSC 152x76 U-profiles facing each other (see Figure 4.12).


New Section

Section type: Steel
Gamma angle: 0 (Deg)
Material: S275

Label: 2 RSC 152x76
Color: Auto

Section selection
Database: UK sections
Family: All
Section: RSC 152x76
Spacing d: 152 (mm)
Two C-sections:
● face to face
○ back to back

Figure 4.12 : Compound box section in model.

4.4.5 Bearings

The linkspan effectively acts as a simply supported beam. At the shore end, each main girder is supported by a hinged support which prevents translation movement. In addition, these supports also provide resistance against in-plane rotation because the bearing is anchor-bolted to the bankseat via vertical hinge plates and a base plate. Moreover, the degree of transverse movement at this support depends upon the stiffness of the vertical plates where the bolt connection between the main girder and base plate occurs. The translation support in this direction is modelled as a spring which has a stiffness equal to the bending stiffness of these plates.

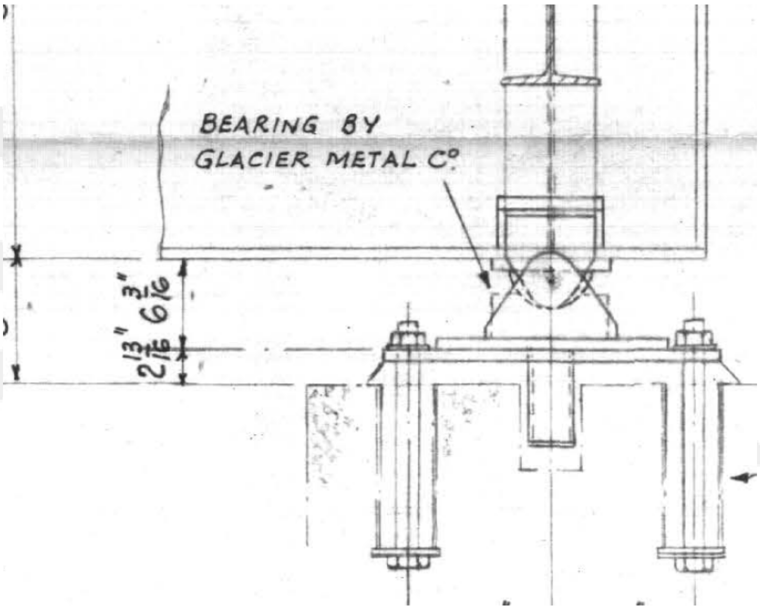


Figure 4.13 : Support condition at the shore end.

Figure 4.14 : Existing condition of bearings at the shore end.

Based upon the drawings of the DC type bearings, the vertical plate has a thickness of t = 15 mm and length of L = 105 mm. The width of these plates is not given in the drawings. The width of the plate is assumed to be similar to the width of the backing plate of the bearings which is given as 9" (229mm) on the drawing. Based on this information, the width of the plate is taken as b = 220 mm. The bending stiffness of the plate is determined as in equations (4.2) and (4.3).

$$I_p = \frac{b \cdot t^3}{12} = \frac{220 \cdot 15^3}{12} = 61875\ mm^4 \tag{4.2}$$

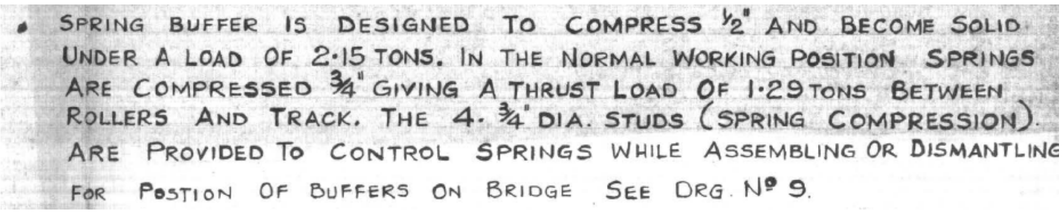
$$k_p = \frac{3 \cdot E \cdot I}{L^3} = \frac{3 \cdot 205000 \cdot 61875 \cdot 10^{-6}}{105^3 \cdot 10^{-3}} = 32.87\ MN/m \tag{4.3}$$

Two plates are applied for a support; therefore, the stiffness of the spring is assigned as 2 x k_p = 65.74 MN/m in the model.

At the pontoon end, the main girders bear directly onto plates of the ponton. The end of the girder is radiused and both rotation and translation in a longitudinal direction are allowed for. In addition, the transverse movement of the bridge is also restrained by the side dolphins where the rotation and other translational movements are allowed for.

Figure 4.15 : Support at the location of dolphin (left) and pontoon (right).

The supports at the location of the dolphins are spring buffers which means that the transverse movement of the bridge is not fully constrained, and the movement allowed depends on the stiffness of the spring. The following information is given on the drawing of the spring buffer;



- SPRING BUFFER IS DESIGNED TO COMPRESS ½" AND BECOME SOLID UNDER A LOAD OF 2·15 TONS. IN THE NORMAL WORKING POSITION SPRINGS ARE COMPRESSED ¾" GIVING A THRUST LOAD OF 1·29 TONS BETWEEN ROLLERS AND TRACK. THE 4· ¾" DIA. STUDS (SPRING COMPRESSION). ARE PROVIDED TO CONTROL SPRINGS WHILE ASSEMBLING OR DISMANTLING

FOR POSTION OF BUFFERS ON BRIDGE SEE DRG. Nº 9.

Figure 4.16 : Extract from historic drawings for spring buffer.

According to this information, the spring buffer has a movement capacity of 12.7 mm (1/2") and thereafter the support reacts as a solid support. As it is stated, an imposed movement is applied to the spring before the installation. The imposed movement is u = 19.05 mm (3/4") and it results a reaction force of F = 12.9kN (1.29 tons). Based on this information, the stiffness of the spring is calculated as in equation (4.4)

$$k_s = \frac{F}{u} = \frac{12.9 \cdot 10^{-3}}{19.05 \cdot 10^{-3}} = 0.677\ MN/m \qquad (4.4)$$

The supports at the location of the dolphins are defined as spring supports with a spring stiffness of k_s.

4.5 Validation of the Structure

The structural model has been validated by comparing the support reactions due to the dead load of the structure from the model and hand calculation. In addition, the deformation behaviour of the bridge is evaluated under uniformly distributed load and asymmetric loading conditions.

Support reactions from the dead load of the structure

The total weight of the structure modelled is calculated as 803.72 kN (Figure 4.17). For the hand calculation, it is assumed that this load is uniformly distributed over the length and width of the structure. The uniformly distributed load is calculated as 803.72 / (32.285 x 5.18) = 4.81 kN/m2 and each main girder will carry half of this load. The line for each girder is q = 4.81 x 5.18 / 2 = 12.46kN.

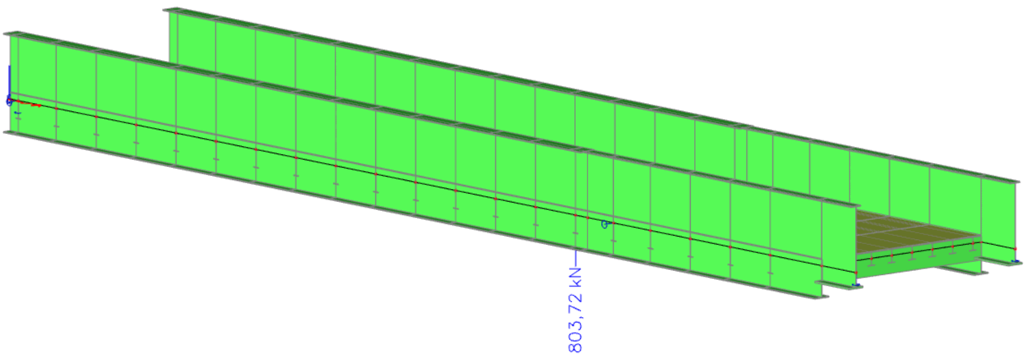


Figure 4.17 : Total weight of the structure from model.

Reaction forces from hand calculation;

$$R_A = R_B = \frac{q\cdot L}{2} = \frac{12.46\cdot 32.285}{2} = 201.1\ kN/m \tag{4.5}$$

The following reaction forces are calculated in SCIA Engineer (Figure 4.18):

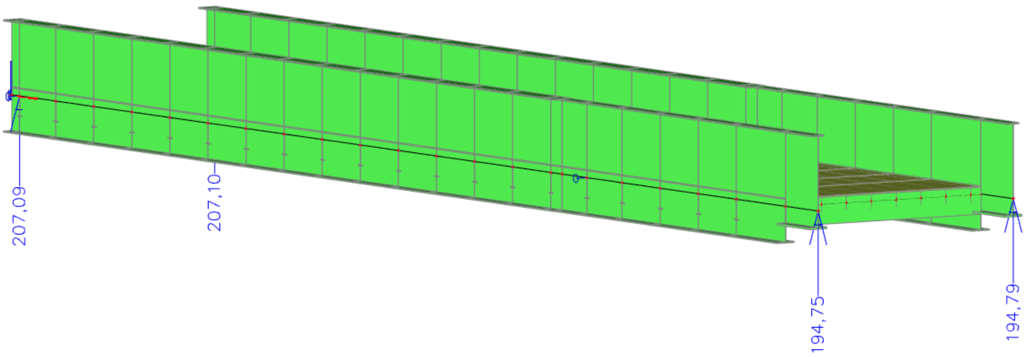


Figure 4.18 : Reaction forces from model for dead load of the structure.

The maximum difference between the hand calculation and model results is 3% which is considered to be a small deviation. This difference can be explained from distribution of the load. At the last part of the bridge, no deck is modelled and therefore the self-weight of the structure will be lower at that end. However, for the hand calculation it is assumed that the self-weight is uniformly distributed over whole length and width of the bridge and this assumption may cause this difference. Nevertheless, the deviation between results is considered acceptable.

5. STRUCTURAL ASSESSMENT

All load actions are assigned to the model and the loads are mainly applied to the deck of the bridge. The wheel loads, wind load, thermal actions are defined in the different load groups and the load cases in these groups are defined as exclusive which means that the governing load case will be found by FEA programme SCIA Engineer. Therefore, these load actions are applied to several locations where the load action can give adverse effect to the structure. By assigning the load actions at different locations and in different directions, all possible adverse scenarios are covered.

The structural assessments are performed in accordance with BS 5400-3 for each structural element of the bridge. The assessment is conducted for the governing section of each element. The assessment procedure for each element is explained in the following sections further.

5.1 Assessment of Main Girder Design

In order to determine the governing cross section of the main girder, the combined Von Mises stress in the main girders is calculated by SCIA Engineer for ULS load combinations (Figure 5.1). The maximum Von Mises stress occurs at the middle of the span as expected and this is part of the original section of the bridge.

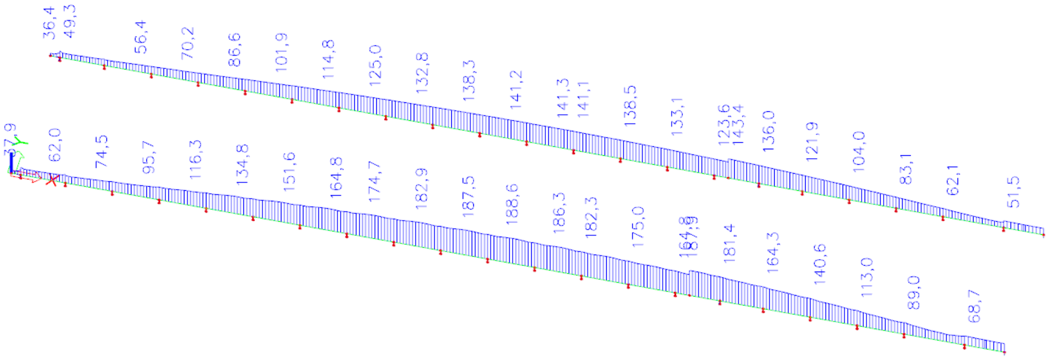


Figure 5.1 : Von Mises stresses in the main girders for ULS load combinations.

Table 5.1 : Utilisation ratios from calculation of the original main girder.

	N_{min}	N_{max}	$V_{y.max}$	$V_{z.max}$	$M_{y.min}$	$M_{y.max}$	$M_{z.max}$
UR	**0.88**	**0.24**	**0.79**	**0.82**	**0.20**	**0.92**	**0.89**

5.2 Assessment of Cross Girder

In order to determine the governing cross girder, the SCIA Engineer model was used to determine the Von Mises stresses in the cross girders for the ULS load combinations (Figure 5.2).

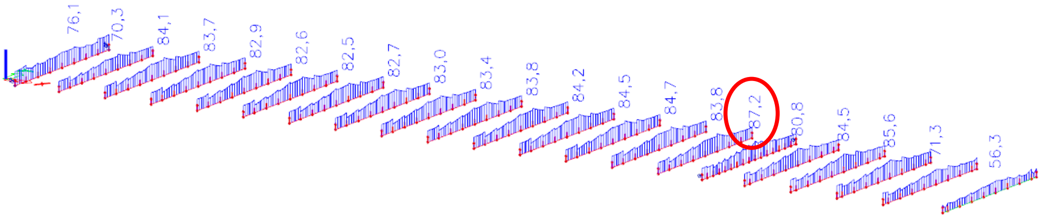


Figure 5.2 : Von Mises stress in the cross girders for ULS load combinations.

As seen during the inspections, some of the cross girders are highly corroded. The structural assessment is performed for the as-built cross girder and for a corroded cross section assuming 10% section loss. The combined utilisation ratios with respect to the maximum value of each action are shown in Table 5.2 below.

Table 5.2 : Utilisation ratios for as-built and corroded cross girders - 10% section loss.

	N_{min}	N_{max}	$V_{y.max}$	$V_{z.max}$	$M_{y.min}$	$M_{y.max}$	$M_{z.max}$
$UR_{original}$	0.04	0.28	0.15	0.20	0.16	0.32	0.33
$UR_{corroded}$	0.02	0.31	0.15	0.22	0.18	0.36	0.37

5.3 Assessment of Longitudinal stiffeners of deck

The governing longitudinal stiffeners of the deck is determined by comparing the Von Mises stress in the sections as calculated by SCIA Engineer for ULS load combinations (Figure 5.3).

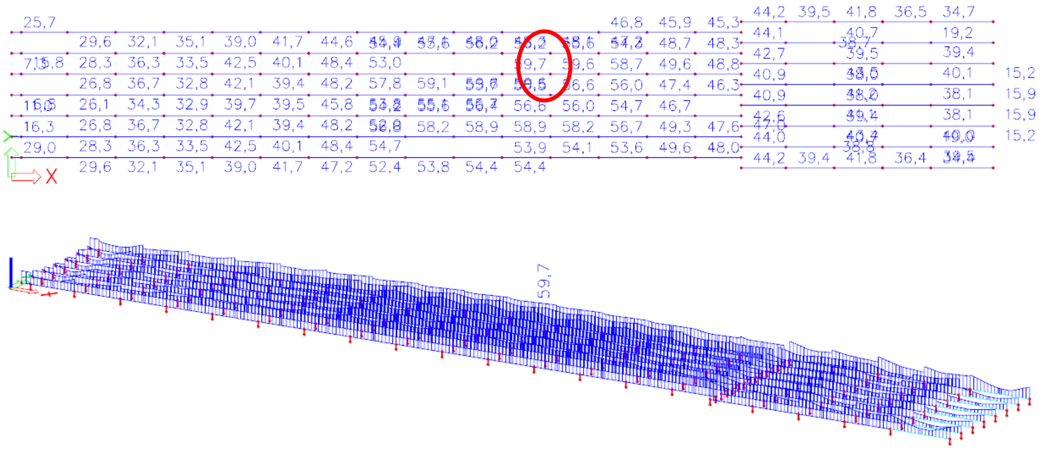


Figure 5.3 : Von Mises stress in the longitudinal stiffeners of the deck.

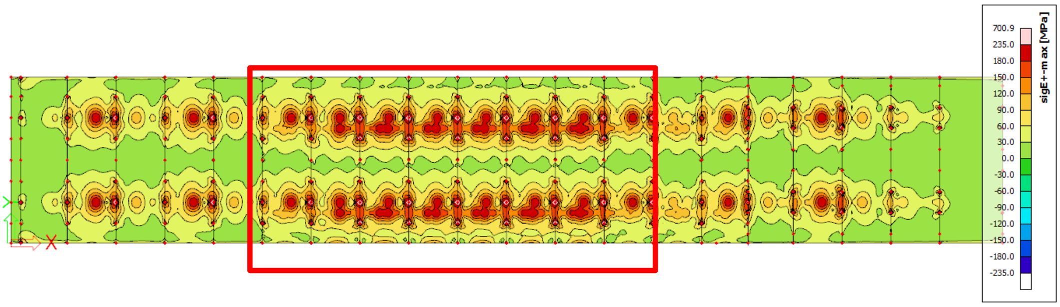
The longitudinal stiffeners of the deck are directly exposed to seawater and some areas were found to be highly corroded during the inspections. The structural assessment is performed for as-built condition and for a corroded cross section assuming 10% section loss. The combined utilisation ratios with respect to the maximum value of each action are shown in Table 5.3.

Table 5.3 : Utilisation ratios for as-built and corroded longitudinal deck stiffeners - assuming 10% section loss.

	N_{min}	N_{max}	$V_{y.max}$	$V_{z.max}$	$M_{y.min}$	$M_{y.max}$	$M_{z.max}$
$UC_{original}$	0.06	0.20	0.16	0.11	0.11	0.23	0.16
$UC_{corroded}$	0.07	0.24	0.19	0.13	0.12	0.27	0.19

5.4 Assessment of Deck

The deck of the linkspan is modelled with 2D plate elements. The cross-sectional check of the deck is completed by comparing Von Mises stresses in the deck for ULS load combinations with the yield strength of the material. Von Mises stresses are determined by SCIA Engineer and the stress distribution is shown in Figure 5.4.



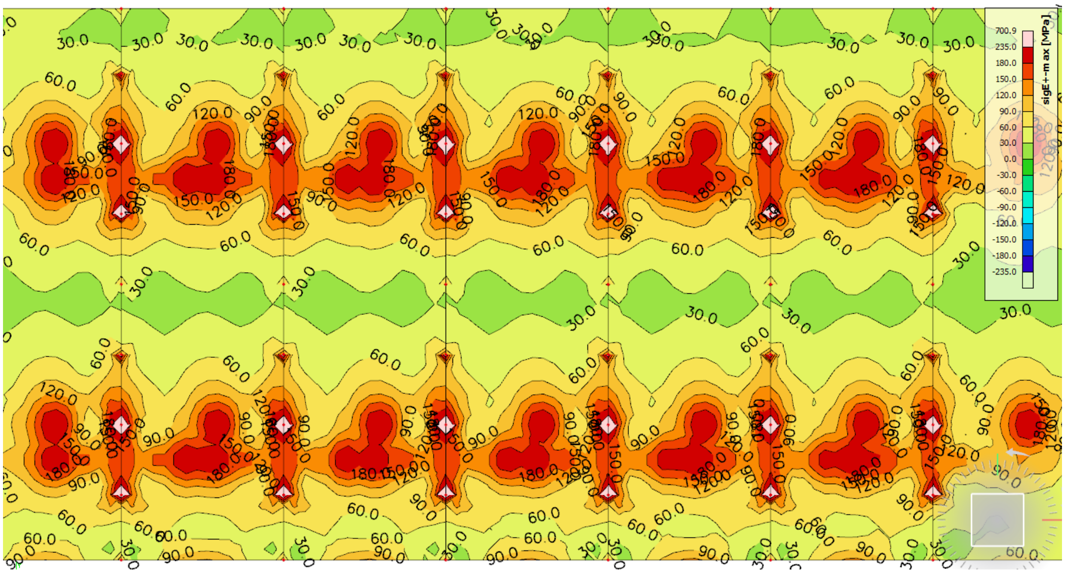


Figure 5.4 : Von Mises stresses in deck plate for ULS load combinations.

The allowable stress is 238MPa and the maximum calculated stress is generally around 180MPa (i.e. 76% utilised).

There are some localised areas where the model shows stresses that are significantly higher than the yield strength of the material. These are generally around the nodes of the underlying 1D elements where it connects to the 2D elements representing the deck. These high stresses can be explained by finite element modelling discontinuities as opposed to real peak stresses and are not considered to be critical. In addition, there are areas of low utilisation in the surround plate that would enable some of the peak stresses to be redistributed. Therefore, it is concluded that the deck plate satisfies the strength requirements.

6. FATIGUE EVALUATION

6.1 Approach

In addition to the challenge of modelling the structure in it actual current condition, as structures age (and particularly if their condition and thus capacity deteriorates), the accumulating effects of fatigue can be a significant factor in the eventual failure of a structure, as well as one-off overloading, the chances of which increase under a given load as the structure corrodes and thus its capacity gradually reduces.

Whilst BS 6349-8 is a newer standard compared to the linkspan, it states “A design life for structural elements within the facility is typically taken as 30 years for the purposes of verifying fatigue endurance.” Our original assessment report did not consider fatigue on the basis that structures of this type are generally heavily engineered so that they are not working at stress levels close to the allowable limit. Adopting a relatively conservative approach to capacity assessment and repair scheduling would ensure that the working stresses do not reach the more significant levels. However, if the structure is to continue to be used in its current condition where the reduced member thicknesses will result in the continued application of higher stresses, the merits of this approach will be lost and thus fatigue failure will become increasingly significant over time.

Whilst it is possible to undertake a retrospective analysis of fatigue, which would need to include information on yearly traffic (numbers of vehicles in certain weight bands for example). Structural models of the Linkspan in its three guises (original plus two number extensions) would be needed, as would allowances for deterioration over time to allow a stress history to be generated.

In the light of above, further analysis could be undertaken to quantify the residual life of the Old Linkspan, the limitations of the work would need to be accepted and due allowance made for potential outcomes of the analysis. Given the criticality of the Linkspan and the inherent uncertainties in the modelling processes, the previously proposed inspection regime would certainly be required as providing some mitigation.

6.2 Fatigue Assessment Procedure

It is intended to assess the fatigue life of the structure by calculating the fatigue damage accumulation within the design life of comparison of the maximum applied stress range with the relevant constant amplitude fatigue limit as is defined in BS 7608:2014.

EN 1993-1-9 provides two different methods for fatigue assessment that are damage tolerant method and safe life method. In this thesis, safe life method is selected where the partial factor for fatigue strength γ_{Mf} is selected as 1.35 based on below Table 6.1 taken from EN 1993-1-9.

Table 6.1 : Recommended values for partial factors for fatigue strength (Table 3.1 in EN 1993-1-9).

Assessment Method	Consequence of Failure	
	Low Consequence	High Consequence
Damage Tolerant	1,00	1,15
Safe Life	1,15	1,35

6.3 Design Life

An assessment has been carried out on the fatigue resistance of the linkspan due to cyclic loading from the daily operation of the link span. The assessment is based on the link span being loaded on and off 4 times per day, every day over a 30 year design life. The linkspan has been extended on 1998 being transformed into current geometry. Therefore the fatigue life of the structure is taken as 30 years in the assessment in accordance with BS 6349-8:2007 considering that the structural behaviour is transformed into present condition.

6.4 Fatigue Load Model

Fatigue load model is created based on the live loads applied on the main girders, cross girders taking into account the SV90 vehicle. The load model is based created based on 17 different Load Cases (130~146) illustrated with some of the moving load in Figure 6.1 below.

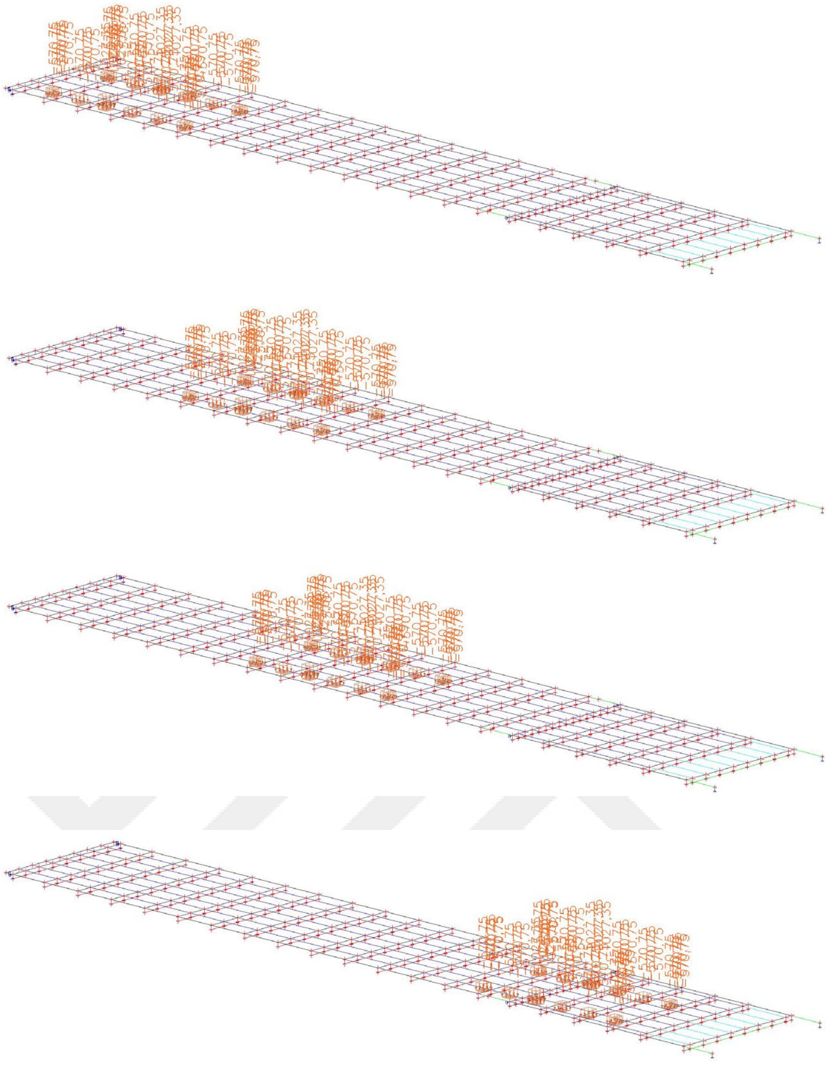


Figure 6.1 : Fatigue Load Model applied on Main and Cross Girder.

6.5 Fatigue Loading

BS6349-8, Clause 7.13 (Fatigue Loading) states that for standard linkspan design with a design life of 30 years to be assumed with 30% of the traffic is being HGV traffic. It is considered within the study that for commercial and private traffic, 4 sailings per day is 60 vehicles per lane total and 60,000 fatigue vehicles per lane per year as shown in Figure 6.2.

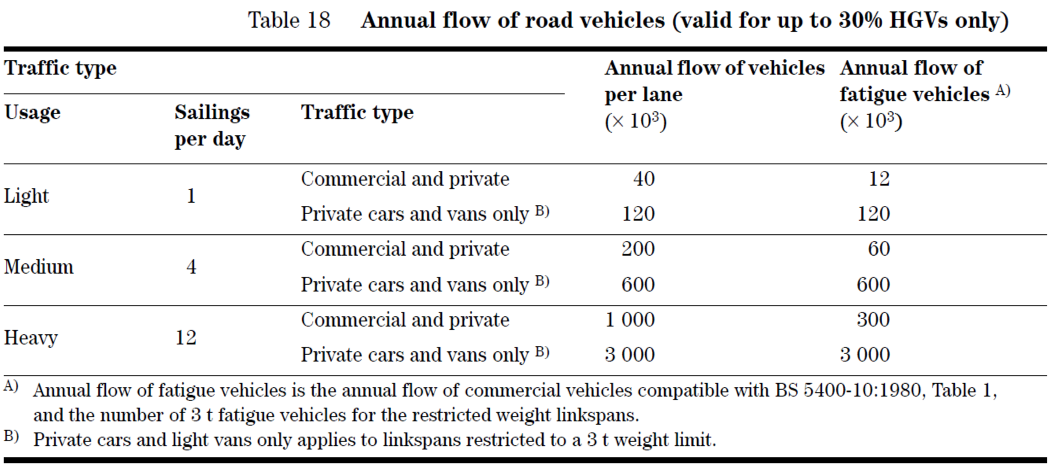


Table 18 **Annual flow of road vehicles (valid for up to 30% HGVs only)**

Traffic type			**Annual flow of vehicles per lane** (× 10^3)	**Annual flow of fatigue vehicles** [A)] (× 10^3)
Usage	**Sailings per day**	**Traffic type**		
Light	1	Commercial and private	40	12
		Private cars and vans only [B)]	120	120
Medium	4	Commercial and private	200	60
		Private cars and vans only [B)]	600	600
Heavy	12	Commercial and private	1 000	300
		Private cars and vans only [B)]	3 000	3 000

[A)] Annual flow of fatigue vehicles is the annual flow of commercial vehicles compatible with BS 5400-10:1980, Table 1, and the number of 3 t fatigue vehicles for the restricted weight linkspans.

[B)] Private cars and light vans only applies to linkspans restricted to a 3 t weight limit.

Figure 6.2 : Annual flow of road vehicles, BS 6349-8.

Considering that the RoRo3 linkspan has only 1 lane, it would be conservative to adopt 60,000 fatigue vehicles per year. Over 60 years, this is 3.6 million lorries.

The assumption of 4 million lorries is taken in this study, based on the vessel sailing and capacity evaluation.

6.6 Classification of the Detail Category and Establishing the Permissible Fatigue Stresses

Looking at the drawings and on site observations, below examples are selected as worst cases for typical details.

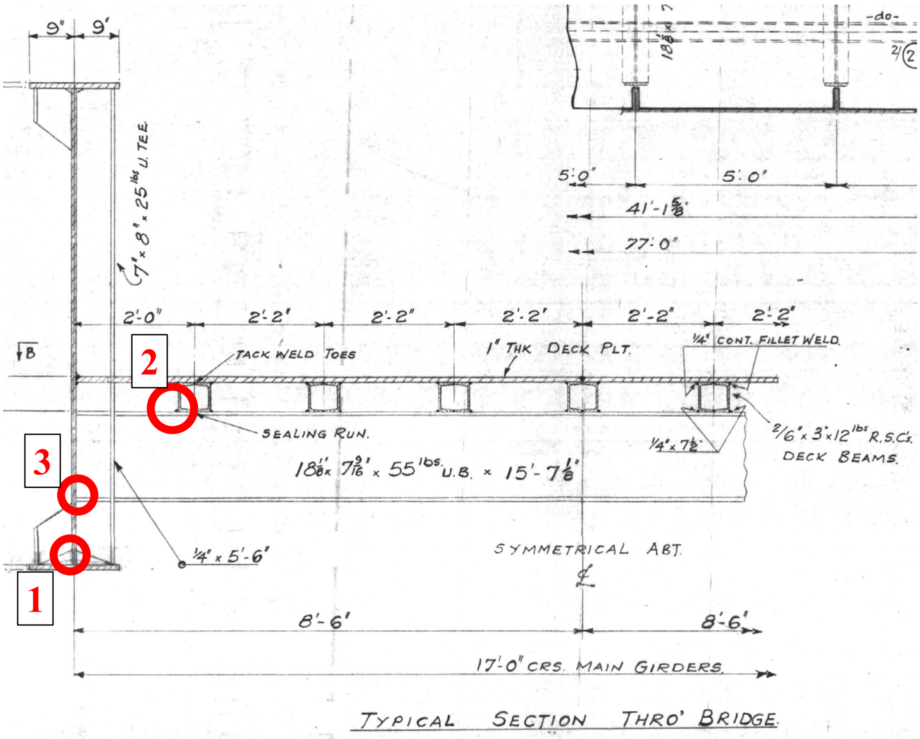


Figure 6.3 : Selected detail categories shown on the linkspan typical section.

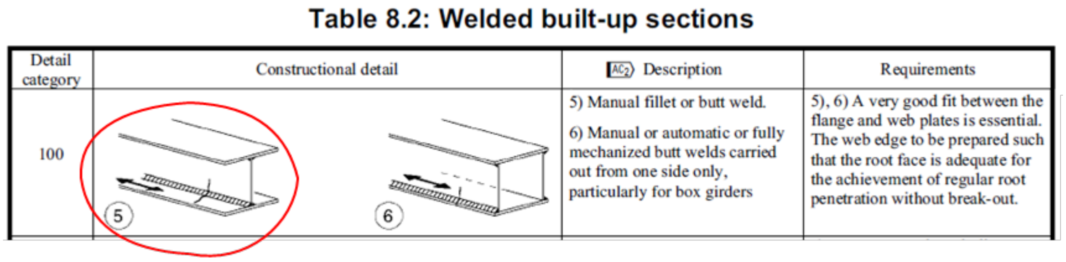


Table 8.2: Welded built-up sections

Detail category	Constructional detail	AC2) Description	Requirements
100		5) Manual fillet or butt weld. 6) Manual or automatic or fully mechanized butt welds carried out from one side only, particularly for box girders	5), 6) A very good fit between the flange and web plates is essential. The web edge to be prepared such that the root face is adequate for the achievement of regular root penetration without break-out.

Figure 6.4 : Selected Detail 1 - Welded built-up sections - Detail Category 100 (Extract from Table 8.2 – EN 1993-1-9).

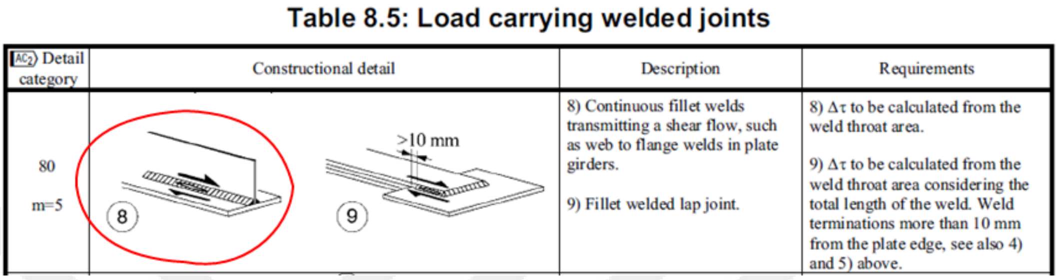


Table 8.5: Load carrying welded joints

AC2) Detail category	Constructional detail	Description	Requirements
80 m=5		8) Continuous fillet welds transmitting a shear flow, such as web to flange welds in plate girders. 9) Fillet welded lap joint.	8) $\Delta\tau$ to be calculated from the weld throat area. 9) $\Delta\tau$ to be calculated from the weld throat area considering the total length of the weld. Weld terminations more than 10 mm from the plate edge, see also 4) and 5) above.

Figure 6.5 : Selected Detail 2 - Load carrying welded joints - Detail Category 80 (Extract from Table 8.5 – EN 1993-1-9).

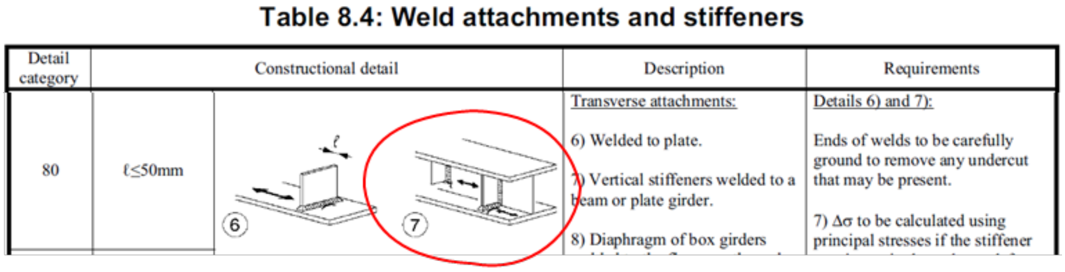


Table 8.4: Weld attachments and stiffeners

Detail category		Constructional detail	Description	Requirements
80	ℓ≤50mm		Transverse attachments: 6) Welded to plate. 7) Vertical stiffeners welded to a beam or plate girder. 8) Diaphragm of box girders	Details 6) and 7): Ends of welds to be carefully ground to remove any undercut that may be present. 7) $\Delta\sigma$ to be calculated using principal stresses if the stiffener

Figure 6.6 : Weld attachments and stiffeners - Detail Category 80 (Extract from Table 8.4 – EN 1993-1-9).

BS EN 1993-1-9, Figures 7.1 and 7.2 are used to establish the permissible fatigue stresses for the various detail categories shown above.

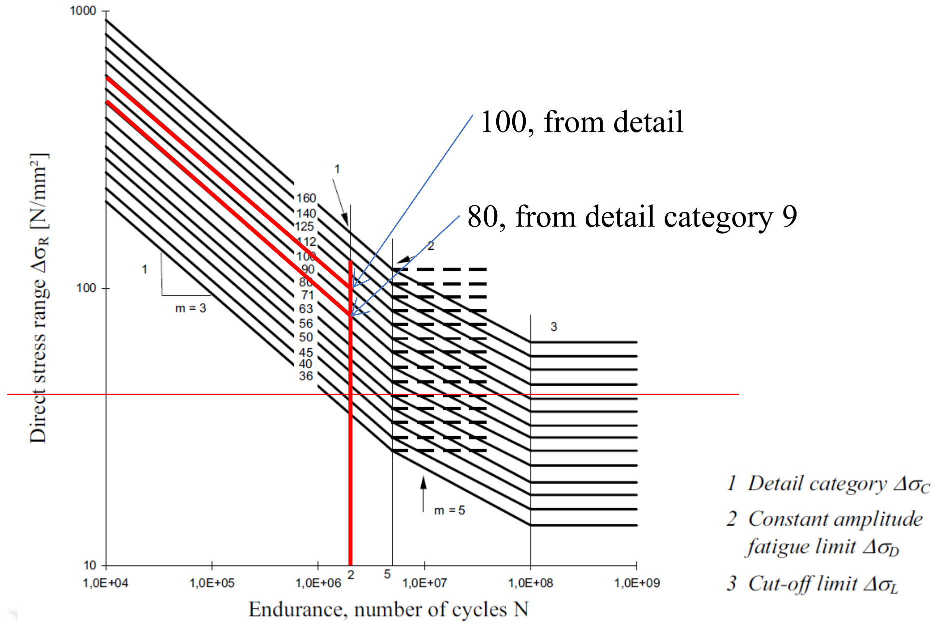


Figure 6.7 : Fatigue strength curves for direct stress ranges, BS EN 1993-1-9:2005.

EN 1993-1-9 defines nominal stress spectra with stress ranges above and below the constant amplitude fatigue limit $\Delta\sigma_D^m$, the fatigue strength should be based on the extended fatigue strength curves and calculated as per the equations (6.1) and (6.2).

$$\Delta\sigma_R^m \; N_R = \Delta\sigma_C^m \; 2 \, x \, 10^6 \qquad \text{with m = 3 for N} \leq 5 \text{ x } 10^6 \qquad (6.1)$$

$$\Delta\sigma_R^m \; N_R = \Delta\sigma_D^m \; 5 \, x \, 10^6 \qquad \text{with m = 5 for } 5 \text{ x } 10^6 \leq \text{N} \leq 10^8 \qquad (6.2)$$

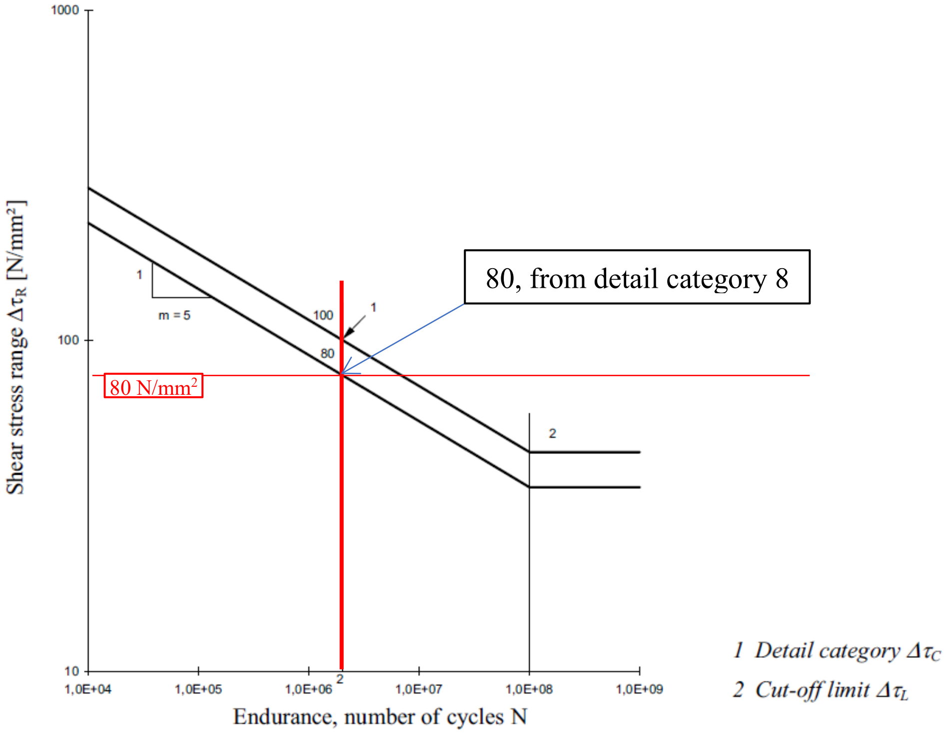


Figure 6.8 : Fatigue strength curves for shear stress ranges, BS EN 1993-1-9:2005.

To obtain an equivalent ultimate allowable stress for fatigue, these figures multiplied by the ultimate partial factor of 1.35 (Safe Life Assessment / High Consequence Failure as per Table 3.1 in EN 1993-1-9). The maximum and minimum results are shown in below Figures 6.9 and 6.10.

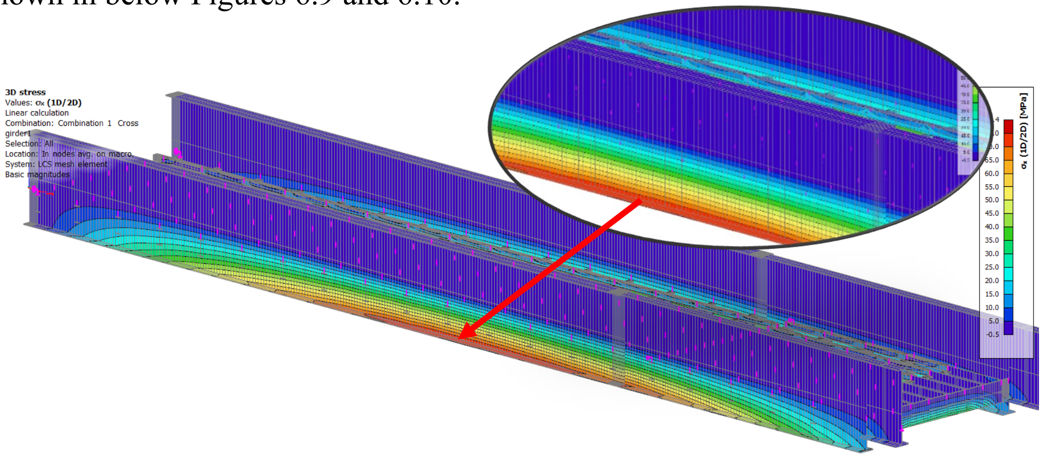


Figure 6.9 : Maximum Stress Results under Fatigue Load Model.

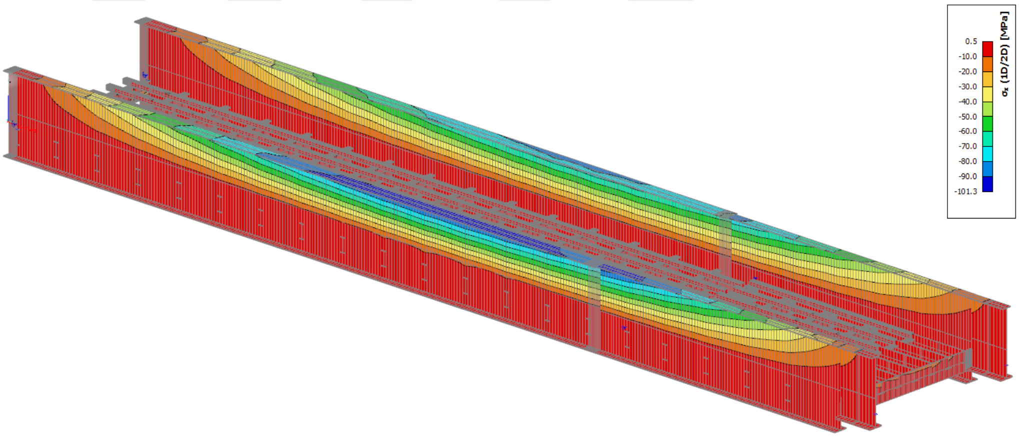


Figure 6.10 : Minimum Stress Results under Fatigue Load Model.

6.7 Evaluation of the Fatigue Stresses

The fatigue stresses are evaluated based on Tables 8.1 and 8.10 of BS EN 1993-1-9:2005 and the constant amplitude stresses are obtained from the S/N curves.

Allowable stresses for the given number of cycles are shown whereas the allowable number of cycles are presented to provide a comparison and for each detail fatigue strength curve plotted.

6.7.1 Detail 1

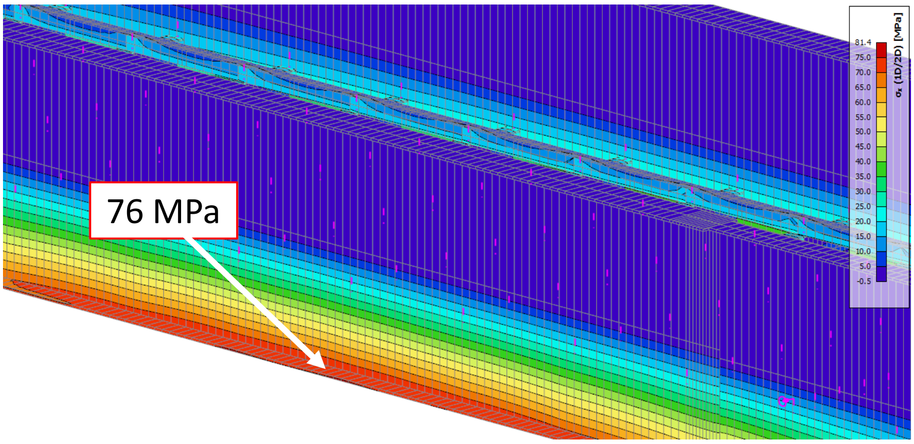


Figure 6.11 : Maximum Stress Results for Detail 1.

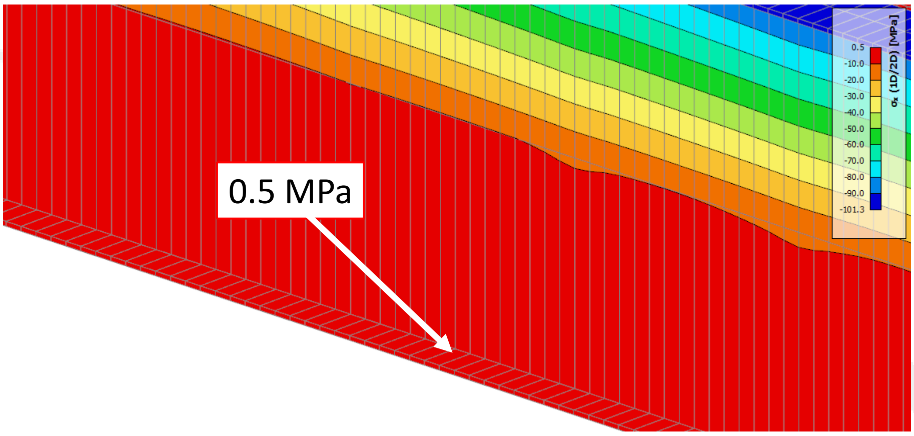


Figure 6.12 : Minimum Stress Results for Detail 1.

Detail Category $\Delta\sigma_c$ = 100 ;

Partial factor for fatigue strength, γ_{MF} = 1.35

Partial factor for stress range, γ_{Ff} = 1.00

Number of cycles of constant amplitude stress, N = 4,000,000

Allowable constant amplitude stress, $\Delta\sigma_R$ = 79.37 N/mm^2

Alternatively,

Constant amplitude stress, $\Delta\sigma_R$ = 75.50 N/mm^2

Allowable number of cycles of constant amplitude stress, N = 4,647,176

Fatigue strength is acceptable.

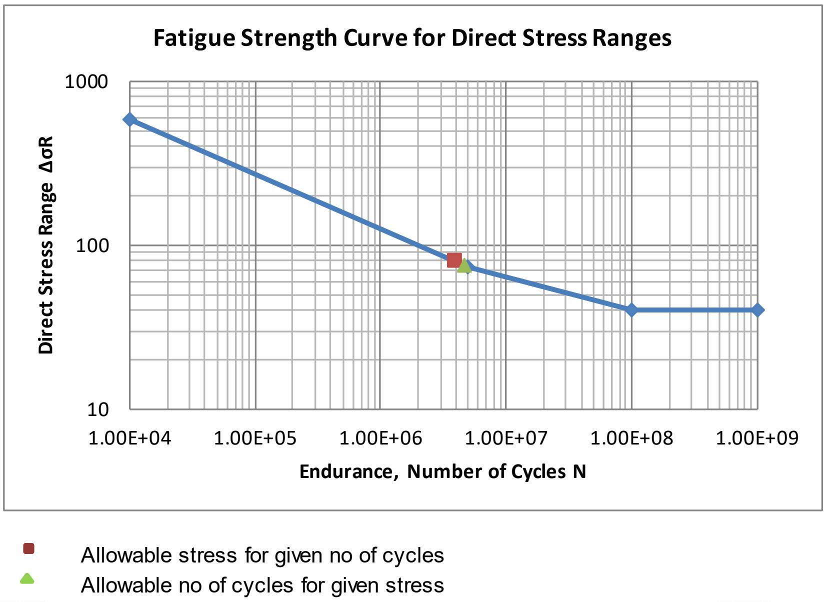


Figure 6.13 : Fatigue Strength Curve for Detail 1.

6.7.2 Detail 2

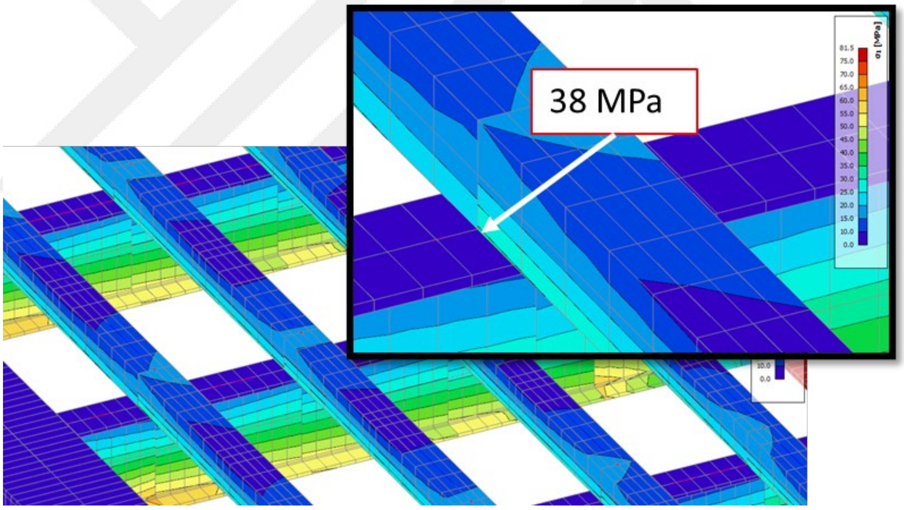


Figure 6.14 : Maximum Stress Results for Detail 2.

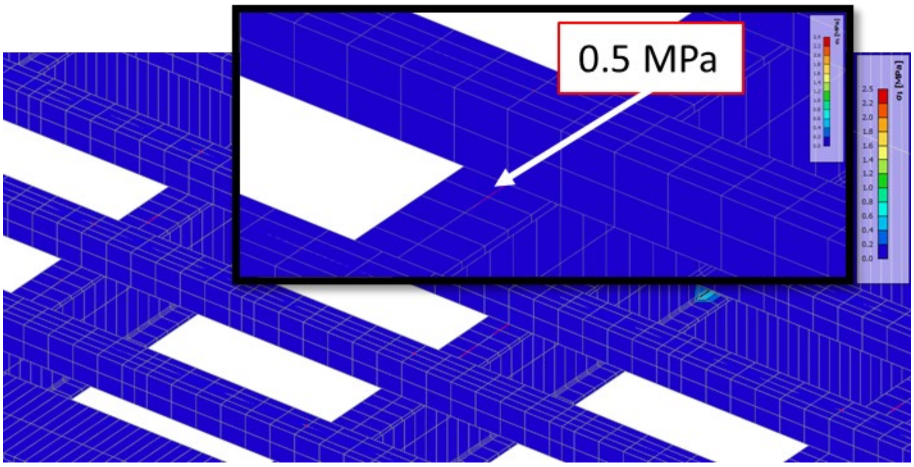


Figure 6.15 : Minimum Stress Results for Detail 2.

Detail Category $\Delta\sigma_c$ = 80 ;

Partial factor for fatigue strength, γ_{MF} = 1.35

Partial factor for stress range, γ_{Ff} = 1.00

Number of cycles of constant amplitude stress, N = 4,000,000

Allowable constant amplitude stress, $\Delta\sigma_R$ = 63.50 N/mm^2

Alternatively,

Constant amplitude stress, $\Delta\sigma_R$ = 37.50 N/mm^2

Allowable number of cycles of constant amplitude stress, N = 48,039,783

Fatigue strength is acceptable.

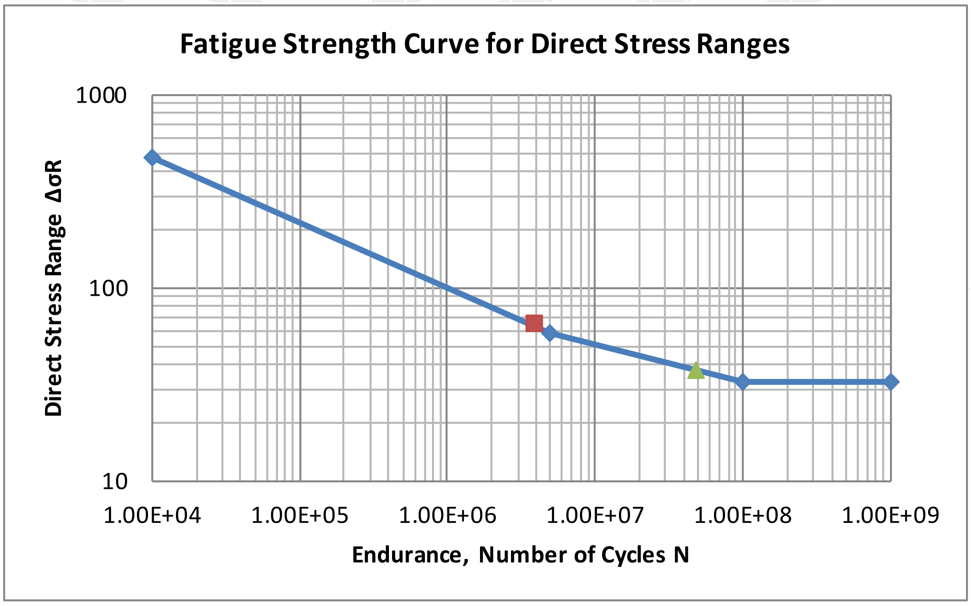


Figure 6.16 : Fatigue Strength Curve for Detail 2.

6.7.3 Detail 3

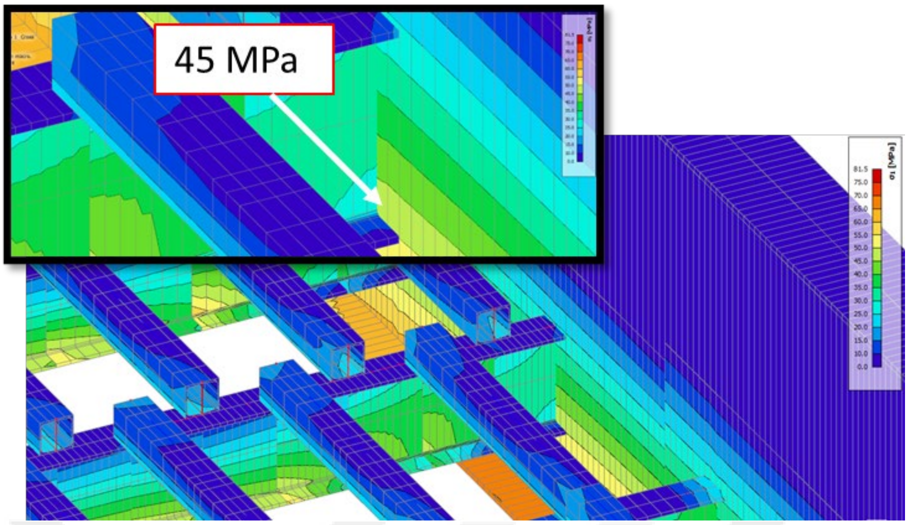


Figure 6.17 : Maximum Stress Results for Detail 3.

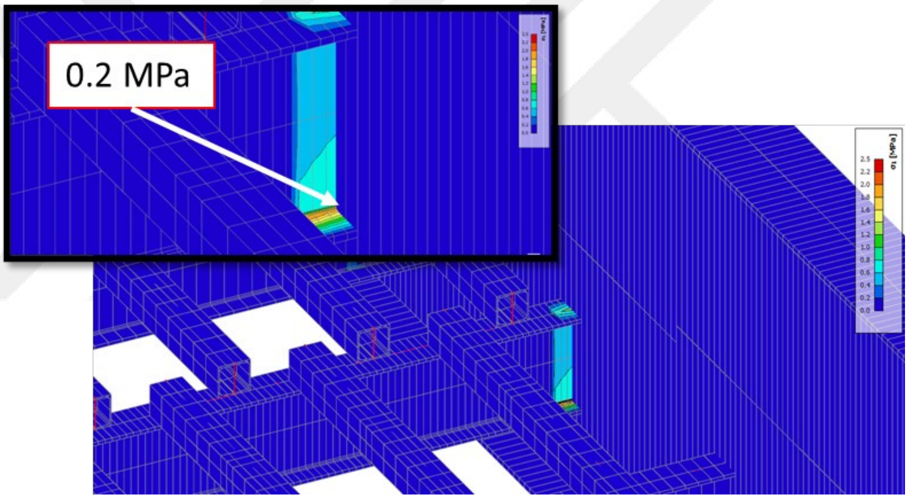


Figure 6.18 : Minimum Stress Results for Detail 3.

Detail Category $\Delta\sigma_c$ = 80

Partial factor for fatigue strength, γ_{MF} = 1.35

Partial factor for stress range, γ_{Ff} = 1.00

Number of cycles of constant amplitude stress, N = 4,000,000

Allowable constant amplitude stress, $\Delta\sigma_R$ = 63.50 N/mm^2

Alternatively,

Constant amplitude stress, $\Delta\sigma_R$ = 44.80 N/mm^2

Allowable number of cycles of constant amplitude stress, N = 19,740,916

Fatigue strength is acceptable.

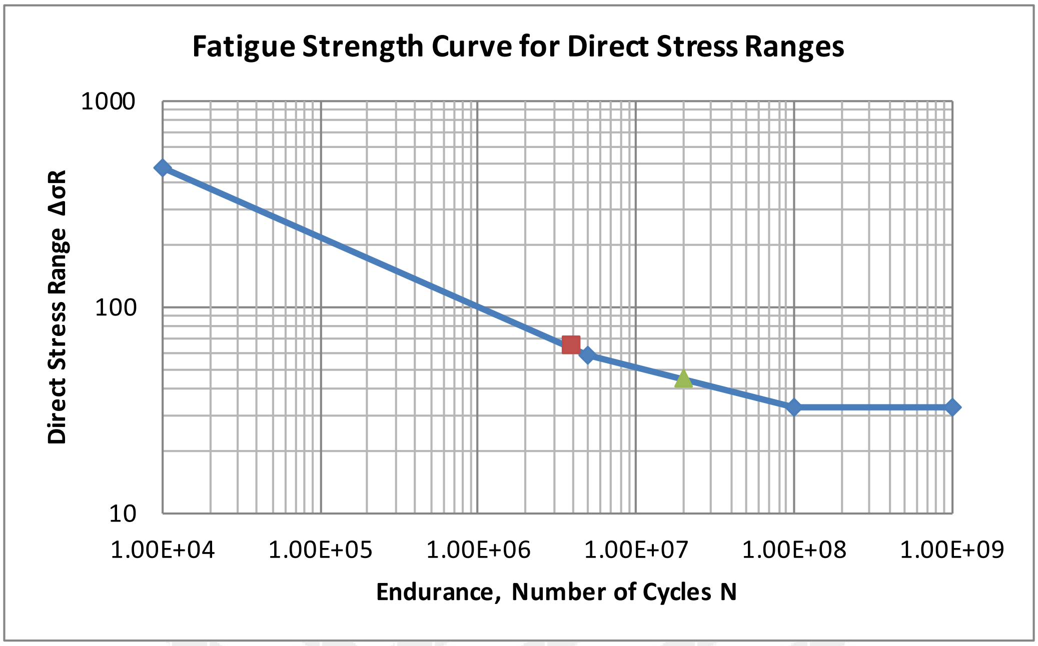


Figure 6.19 : Fatigue Strength Curve for Detail 3.

Equivalent Ultimate Allowable Stress

Welded built up sections	79.37 N/mm^2 > $\Delta\sigma_P$=75.5 N/mm2 (taken from model)
Load carrying welds	63.5 N/mm^2 > $\Delta\sigma_P$ =37.5 N/mm2 (taken from model)
Weld attachment / stiffeners	63.5 N/mm^2 > $\Delta\sigma_P$ =44.8 N/mm2 (taken from model)

From this it can be seen that the direct stresses obtained form model are lower than the equivalent ultimate allowable stresses and therefore fatigue strengths are found acceptable.

It is seen from above that the cyclic loading from vehicle movements have a much higher frequency which the stress cycle is considerably small and therefore not significant.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary and Discussions of Results

Significant work on structural modelling has been done including fatigue analysis and accurate modelling of historical linkspan structure.

Utilisations proved that the comparison study allows for the 10% section loss on the longitudinal stiffeners and cross girders. More than 10% section loss would require structural replacement on those members whereas the deck plate and bearings are not found critical.

The longitudinal stiffeners to the cross girders are heavily corroded and in places this has gone through thickness. Repairs are required to these elements.

The corrosion loss to the deck plate is hard to quantify as the passage of vehicles removes the build-up of corrosion product. Ultrasonic thickness measurement should be considered in order to confirm the thickness of the deck remaining.

Fatigue evaluation has determined that the selected three details are not fatigue critical for the envisaged 60 years fatigue life which is based on latest extended structure. Therefore, it can be concluded that the structure should serve for the remaining 34 years.

Monitoring of structures approaching the end of their service life. Although this has predominantly been for suspended deck structures rather than Linkspans, similar principles would apply.

7.2 Future Work and Recommendations

Extensive condition survey should further be performed to understand the corrosion rates per member e.g., by taking several thickness readings on each member and consideration as to whether corrosion to date has been linear or more complex.

Corrosion is an important factor for the marine structures. Thus the preventive measures play a vital role on the design life of the structures.

Structural repairs and corrosion protection measures shall properly be applied to ensure that the linkspan structure serves its remaining design life.

A programme of maintenance is recommended to safeguard the performance of the asset into the future which can be followed within a routine inspection plan and maintenance schedules. Minor planned maintenance includes the routine maintenance works that are done periodically. Reactive Maintenance defines the actions to be taken whilst defects observed. Major Maintenance outlines the period and required replacement works of the members.

For the main beams, transverse beams, deck plate, and longitudinal stiffeners, inspections can include checking for signs of water ingress and corrosion, as well as assessing the coating system. Routine inspections are recommended annually to prevent major defects and fatigue cracking whereas reactive maintenance should be initiated as needed. Transition flaps at the shore and river ends require similar inspections and maintenance tasks, with routine inspections also recommended annually.

Bearings on the abutment and spring buffers undergo functionality checks, with attention given to their condition and coating systems.

Surfacing on the deck, including carriageway and walkway can be inspected for signs of water ingress, corrosion, and wear at interfaces. Routine inspections are advised, with maintenance tasks such as litter collection, interim repair of potholes, and setting up warning signs as necessary.

As part of structural repairs works, whole linkspan structure is grit blasted and shop primed to be able to measure thickness based on the defects map. Random thickness readings are taken from longitudinal stiffeners, cross girders and deck plate.

Like-a-like replacement has been made for the sections that do not satisfy the minimum 10% section loss criteria.

Some photos are provided in Figures 7.1, 7.2, 7.3 and 7.4 below showing the initial, interim blasting and primer application, fitup of new members and final condition after the replacement of the longitudinal stiffener members.

Figure 7.1 : Initial Condition of the Cross Girder and Longitudinal Stiffeners.

Figure 7.2 : Blasted and Shop Primed Condition of the Cross Girder and Longitudinal Stiffeners.

Figure 7.3 : Fit-up and welding of new longitudinal stiffeners.

Figure 7.4 : Final Condition of the Cross Girder and Longitudinal Stiffeners after Like-a-like replacement.

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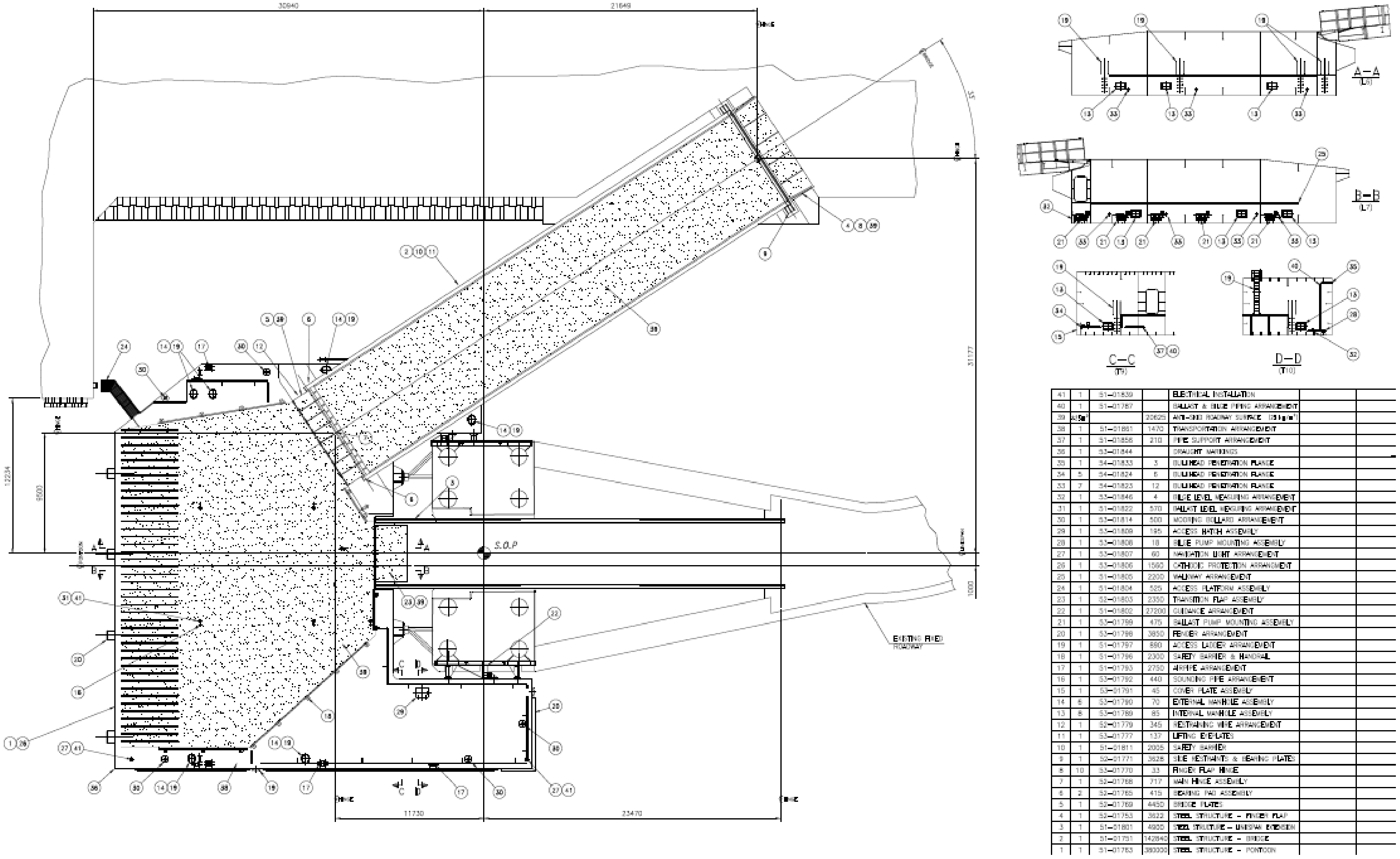
[1] **Url-1** https://ntslf.org/tides/hilo Highest & lowest predicted tides at North Shields, UK - National Tidal and Sea Level Facility

APPENDICES

APPENDIX A: Linkspan Drawings

APPENDIX B: Structural Inspection Defects Table

APPENDIX A: Linkspan Drawings



41	1	51–01839		ELECTRICAL INSTALLATION		
40	1	51–01787		BALLAST & BILGE PIPING ARRANGEMENT		
39	A(5m²)		20625	ANTI-SKID ROADWAY SURFACE (25 kg/m²)		
38	1	51–01861	1470	TRANSPORTATION ARRANGEMENT		
37	1	51–01856	210	PIPE SUPPORT ARRANGEMENT		
36	1	53–01844		DRAUGHT MARKINGS		
35	1	54–01833	3	BULKHEAD PENETRATION FLANGE		
34	5	54–01824	6	BULKHEAD PENETRATION FLANGE		
33	7	54–01823	12	BULKHEAD PENETRATION FLANGE		
32	1	53–01846	4	BILGE LEVEL MEASURING ARRANGEMENT		
31	1	51–01822	570	BALLAST LEVEL MEASURING ARRANGEMENT		
30	1	53–01814	500	MOORING BOLLARD ARRANGEMENT		
29	1	53–01809	195	ACCESS HATCH ASSEMBLY		
28	1	53–01808	18	BILGE PUMP MOUNTING ASSEMBLY		
27	1	53–01807	60	NAVIGATION LIGHT ARRANGEMENT		
26	1	53–01806	1560	CATHODIC PROTECTION ARRANGMENT		
25	1	51–01805	2200	WALKWAY ARRANGEMENT		
24	1	51–01804	525	ACCESS PLATFORM ASSEMBLY		
23	1	52–01803	2350	TRANSITION FLAP ASSEMBLY		
22	1	51–01802	27200	GUIDANCE ARRANGEMENT		
21	1	53–01799	475	BALLAST PUMP MOUNTING ASSEMBLY		
20	1	53–01798	3850	FENDER ARRANGEMENT		
19	1	51–01797	880	ACCESS LADDER ARRANGEMENT		
18	1	51–01796	2300	SAFETY BARRIER & HANDRAIL		
17	1	51–01793	2750	AIRPIPE ARRANGEMENT		
16	1	53–01792	440	SOUNDING PIPE ARRANGEMENT		
15	1	53–01791	45	COVER PLATE ASSEMBLY		
14	6	53–01790	70	EXTERNAL MANHOLE ASSEMBLY		
13	8	53–01789	85	INTERNAL MANHOLE ASSEMBLY		
12	1	52–01779	345	RESTRAINING WIRE ARRANGEMENT		
11	1	53–01777	137	LIFTING EYEPLATES		
10	1	51–01811	2005	SAFETY BARRIER		
9	1	52–01771	3628	SIDE RESTRAINTS & BEARING PLATES		
8	10	53–01770	33	FINGER FLAP HINGE		
7	1	52–01768	717	MAIN HINGE ASSEMBLY		
6	2	52–01765	415	BEARING PAD ASSEMBLY		
5	1	52–01769	4450	BRIDGE PLATES		
4	1	52–01753	3622	STEEL STRUCTURE – FINGER FLAP		
3	1	51–01801	4900	STEEL STRUCTURE – LINKSPAN EXTENSION		
2	1	51–01751	142840	STEEL STRUCTURE – BRIDGE		
1	1	51–01763	380000	STEEL STRUCTURE – PONTOON		

Figure A.1 : General Arrangement Layout of Berth.

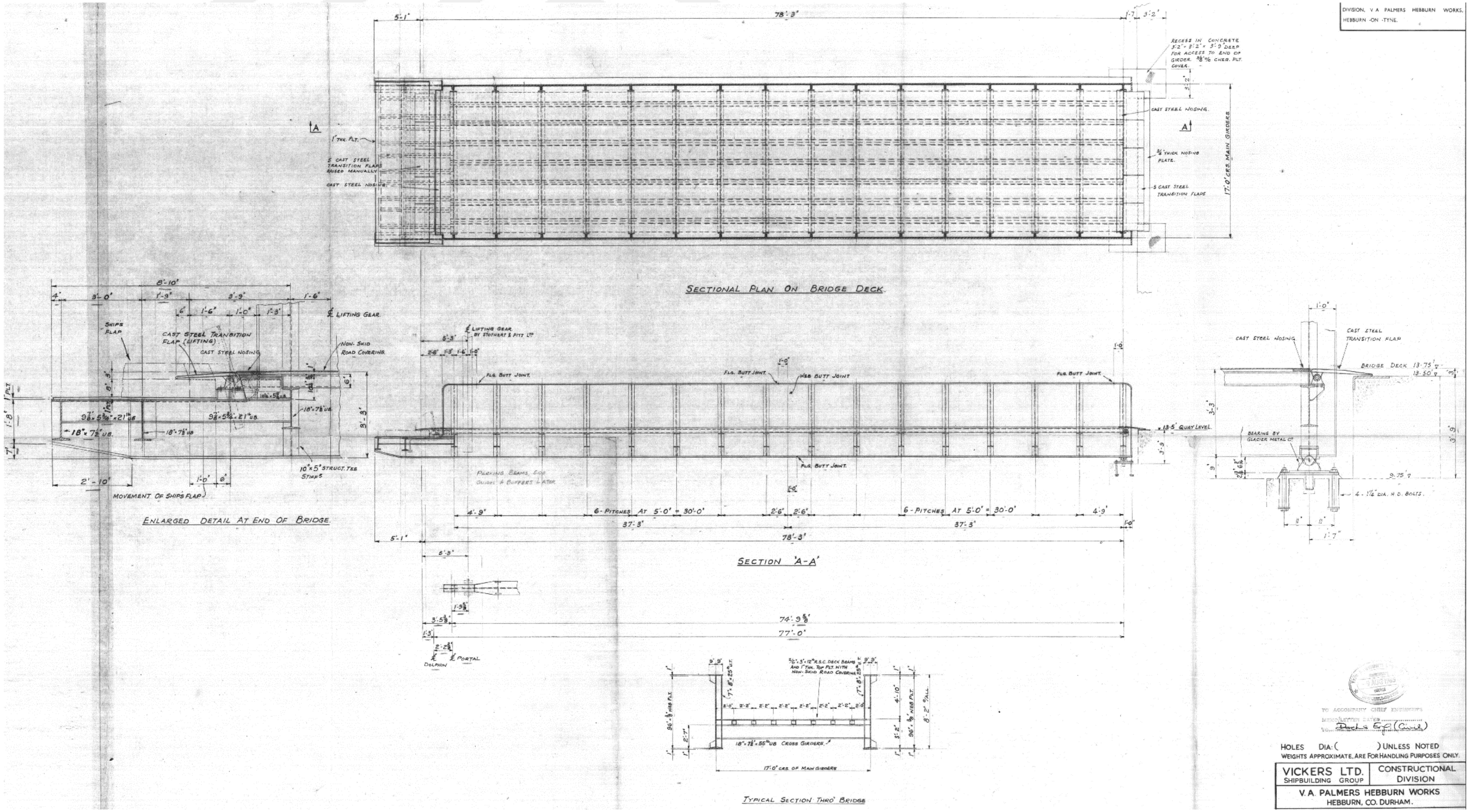


Figure A.2 : General Arrangement of Bridge – Before Extension.

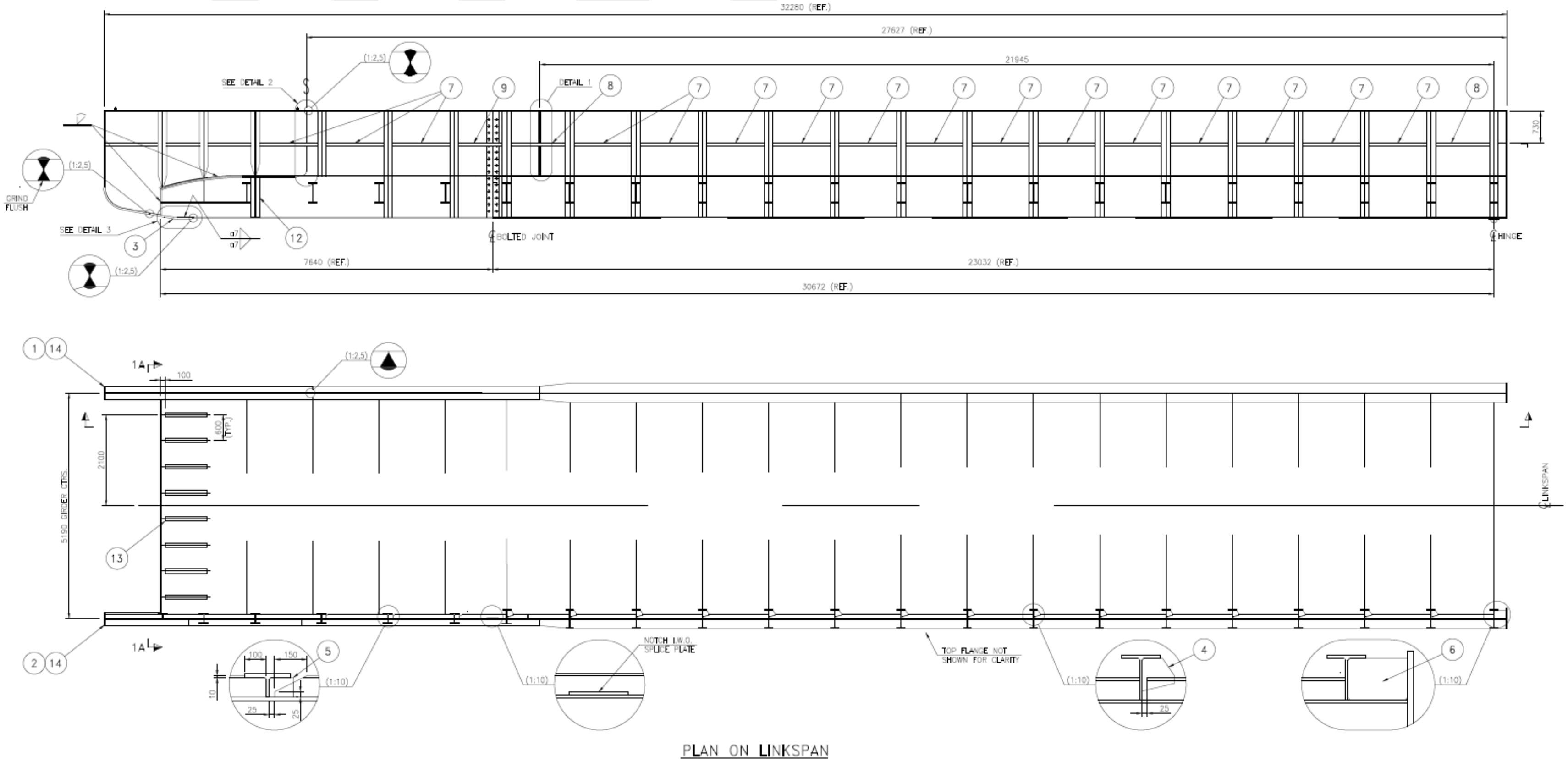


Figure A.3 : General Arrangement of Bridge – Linkspan Extension.

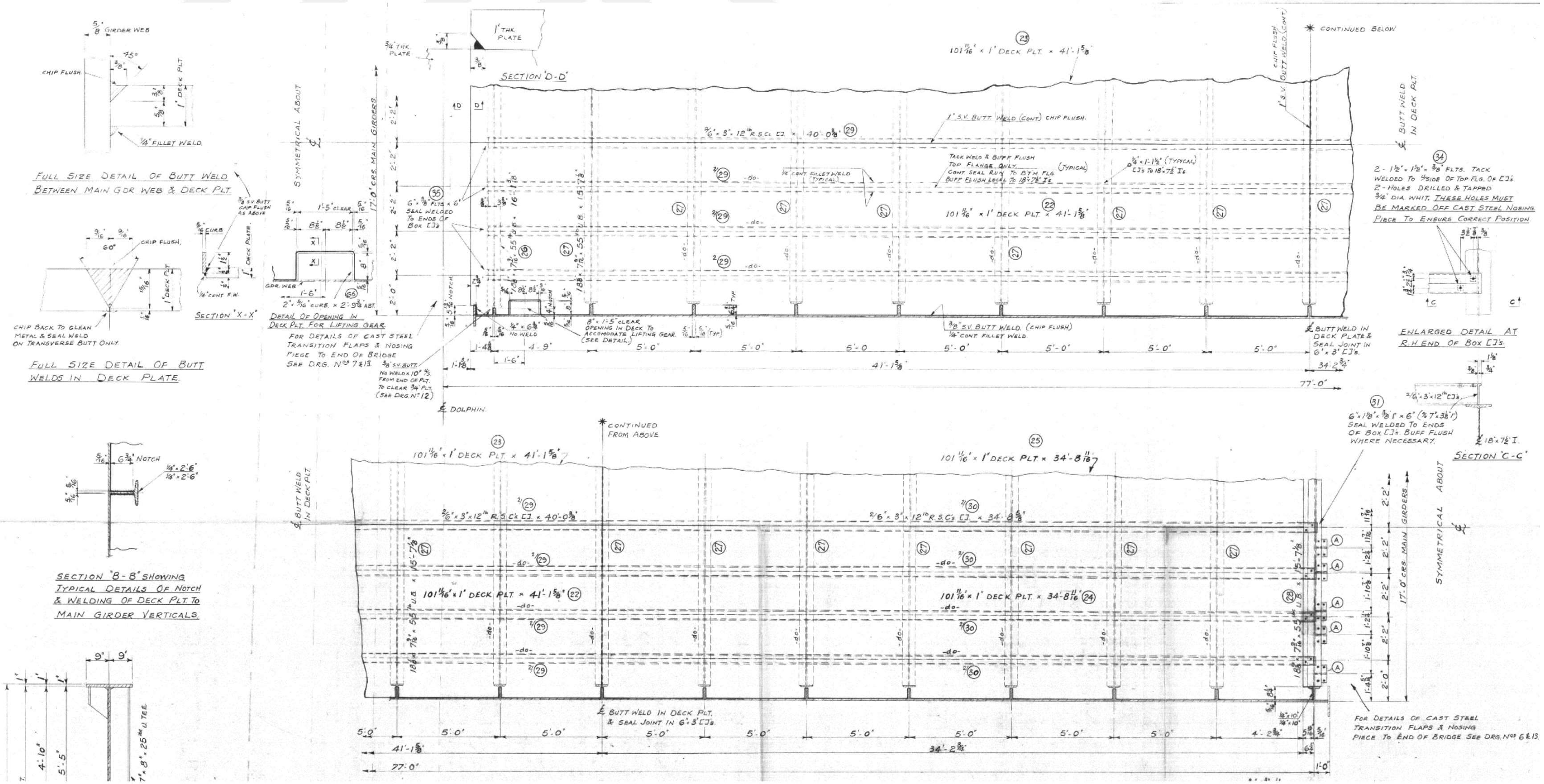


Figure A.4 : Main Girders.

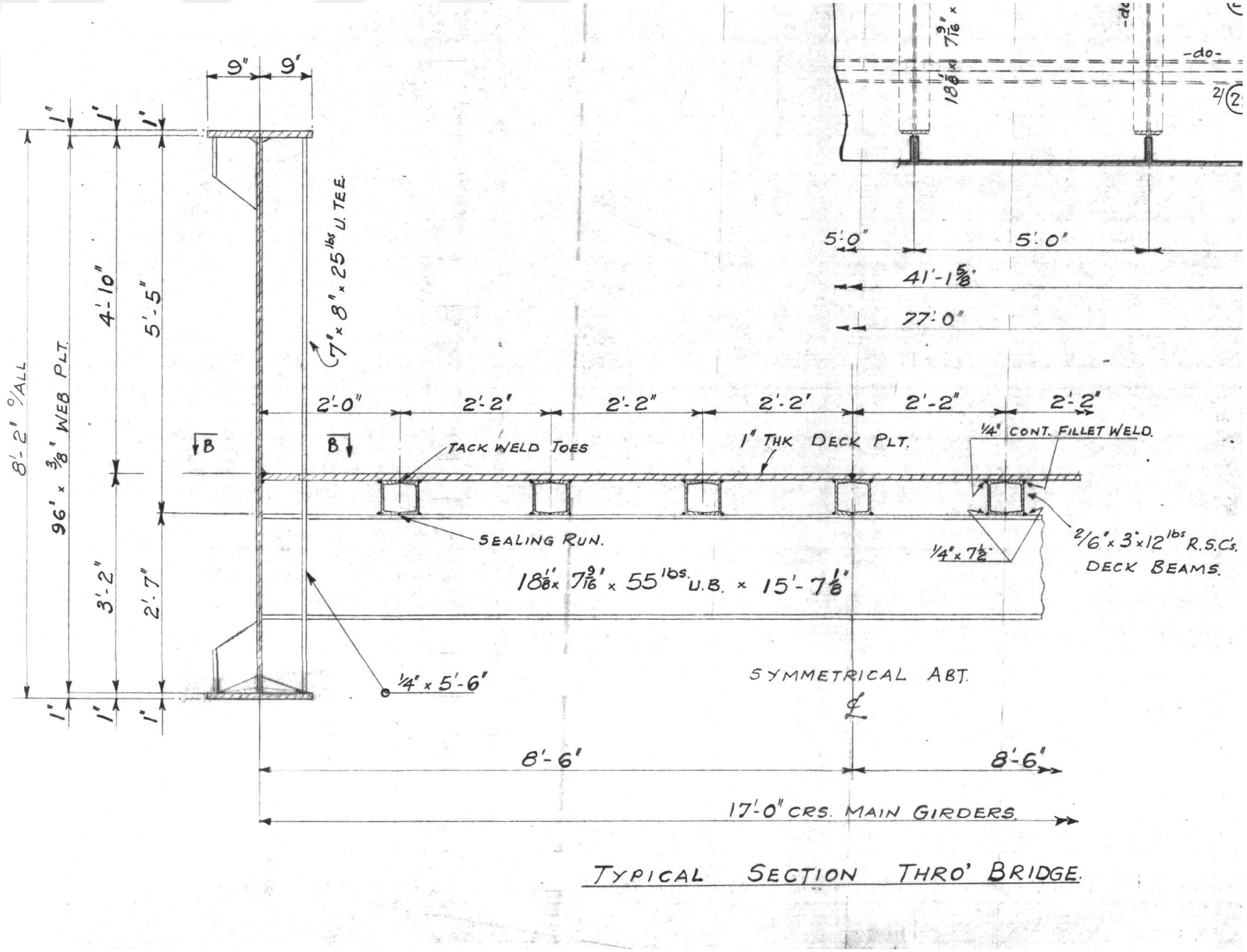


Figure A.5 : Typical Section of Old Linkspan – Before Extension.

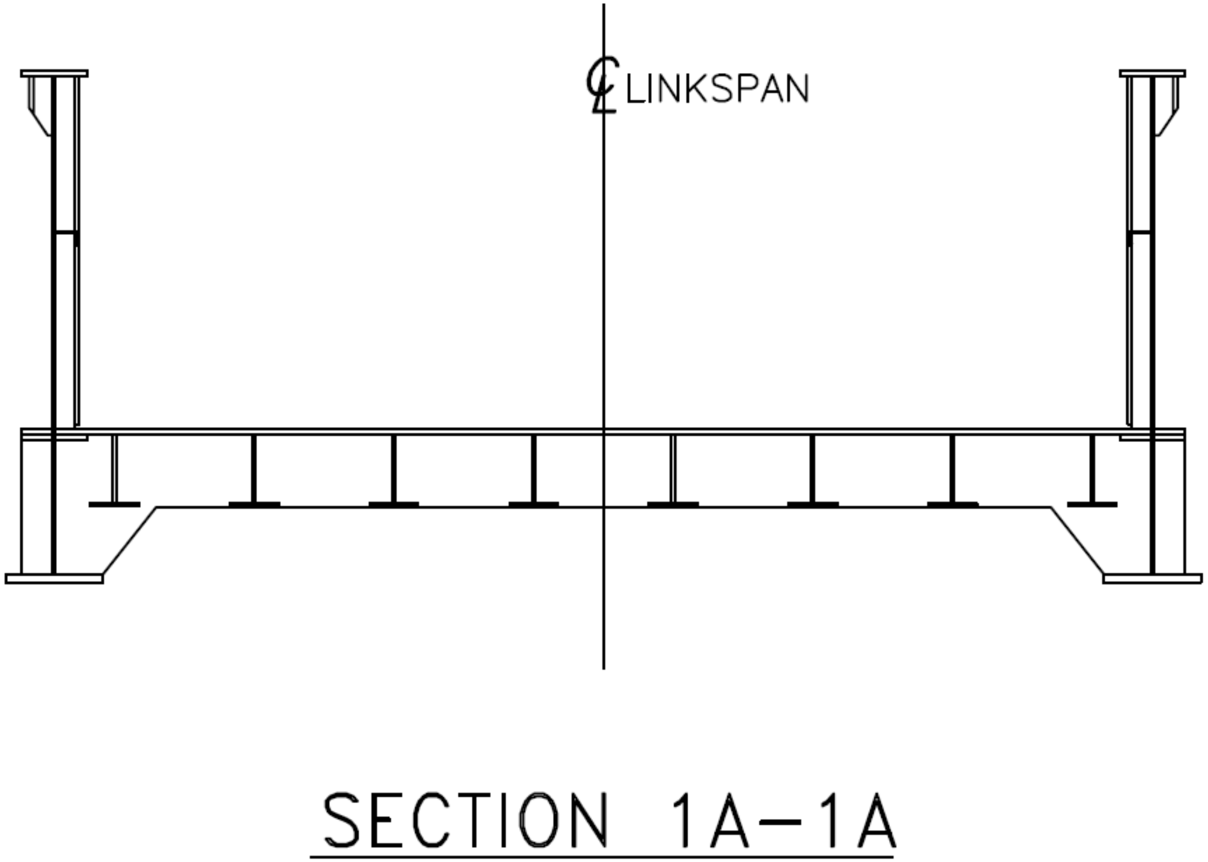


Figure A.6 : Typical Section of Extended Linkspan.

APPENDIX B: Structural Inspection Defects Map

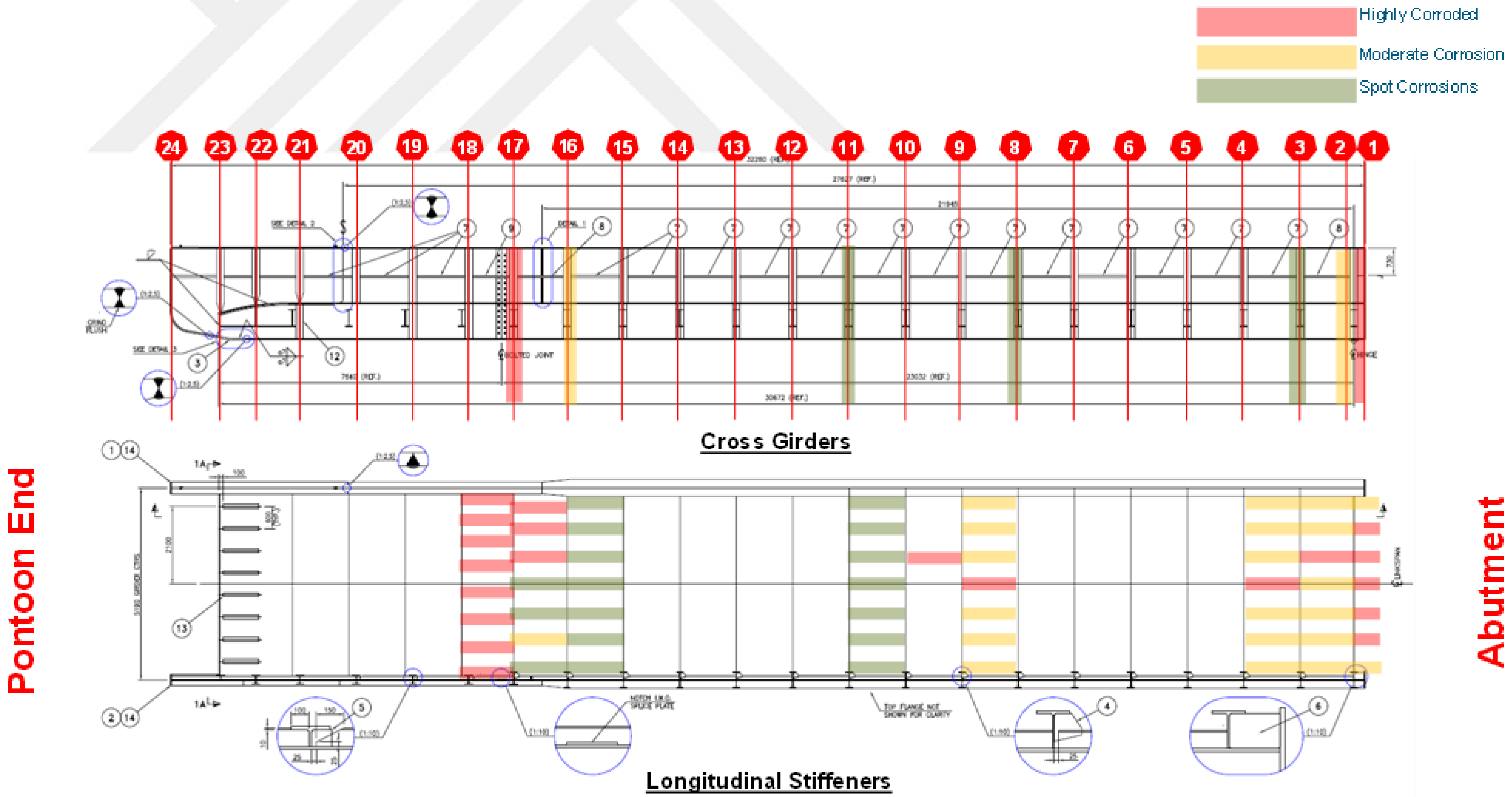


Figure B.1 : Structural Inspection Initial Defects Map.

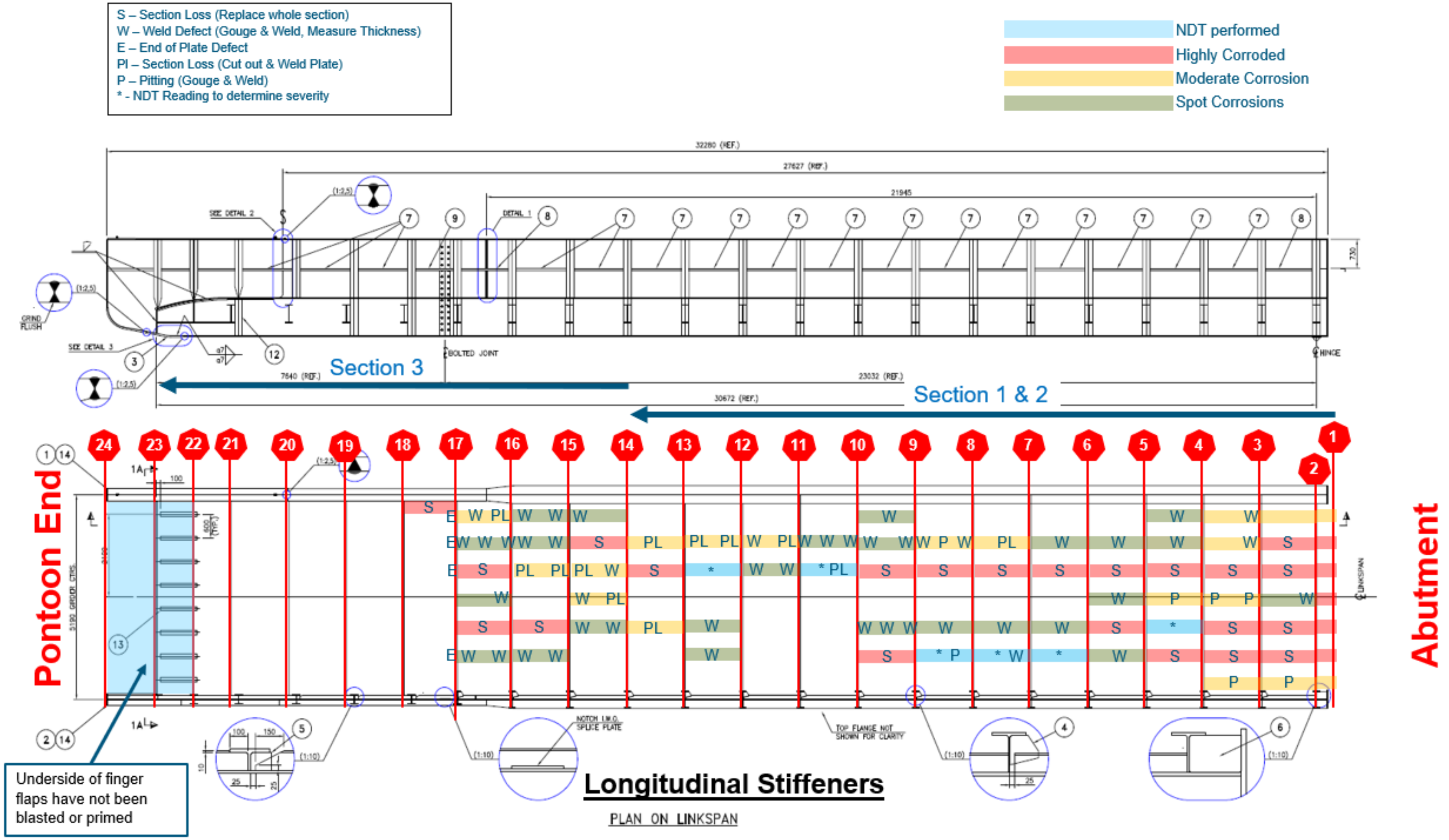


Figure B.2 : Structural Inspection Findings Updated Defect Map.

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