

University of Wales Trinity Saint
David
School of Engineering

Technical Feasibility Assessment of a
Modular Fabrication Transport
System for Heavy Project-Shop
Manufacturing

*A Modular Approach to Material Handling
within Ledwood's Heavy Fabrication
Operations*

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Joshua Axon

BEng Mechanical Engineering

2218372



This thesis is the result of the author's original research. It has been composed by the author and has not been previously submitted for examination which has led to the award of a degree:

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1 Introduction

1.1 Context, Background & Abstract

Ledwood Mechanical Engineering Ltd, (LMEL) operates within engineer-to-order heavy fabrication sectors including energy, renewable and nuclear construction. Fabrication activities at the Waterloo facility are currently organised using a project-shop manufacturing model, where bespoke assemblies progress through shared resources according to evolving project requirements rather than fixed production flow. While this structure provides flexibility, it also contributes to elevated work-in-progress (WIP), reactive prioritisation and reduced production stability under increasing schedule and regulatory pressures. This report therefore investigates whether a Fabrication Transport Module (FTM) could act as a modular handling and control system to improve manufacturing flow, material handling and operational visibility within LMEL's fabrication Environment.

1.2 Project Aim & Objectives

To design and technically evaluate a Fabrication Transport Module (FTM) to improve manufacturing flow, handling efficiency, and operational control within Ledwood Project-shop environment.

1.2.1 Project Objectives

1. Literature Review and Current-State Analysis

Establish the theoretical and industrial basis for the FTM through a review of modularisation, Lean production, and relevant industrial case studies, alongside analysis of Ledwood's existing fabrication operations to identify bottlenecks, handling inefficiencies, sequencing failures and cause of elevated work-in-progress (WIP).

2. Functional and Structural Requirements Definition

Translate the FTM concept into measurable engineering requirements, including payload capacity, lifting geometry, safety constraints, centre-of-gravity control, and compliance with applicable structural standards such as BS EN 1993 (Eurocode 3).

3. Design Development

Develop the FTM through concept generation, option selection, CAD modelling, and preliminary finite element analysis, progressing the concept into a defined engineering system.

4. Structural Validation and Verification

Assess the structural performance of the FTM using analytical calculations and finite element analysis under representative loading conditions to verify structural adequacy, safety, and operational suitability.

1.3 Scope and Limitations

This project focuses on the conceptualisation and structural design of an FTM intended for batch handling of pipe spools and fabricated assemblies within Ledwood's Waterloo facility, with applicability across the wider business.

Scope includes:

- Definition of functional and structural requirements for lifting, handling, and transport.
- Concept generation, selection, and detailed design of a representative heavy-lift cage.
- Structural validation using analytical methods and finite element analysis.
- Qualitative evaluation of impacts on manufacturing flow, logistics, safety, and indicative cost considerations.

Scope excludes:

- Full-scale prototype fabrication or on-site trials.
- Integration with detailed logistics or digital tracking systems.
- Detailed cost modelling or production planning.

The project is therefore positioned as a technical feasibility study, demonstrating engineering design competence while aligning with Lean manufacturing principles. A provisional project plan is provided in Figure 1.

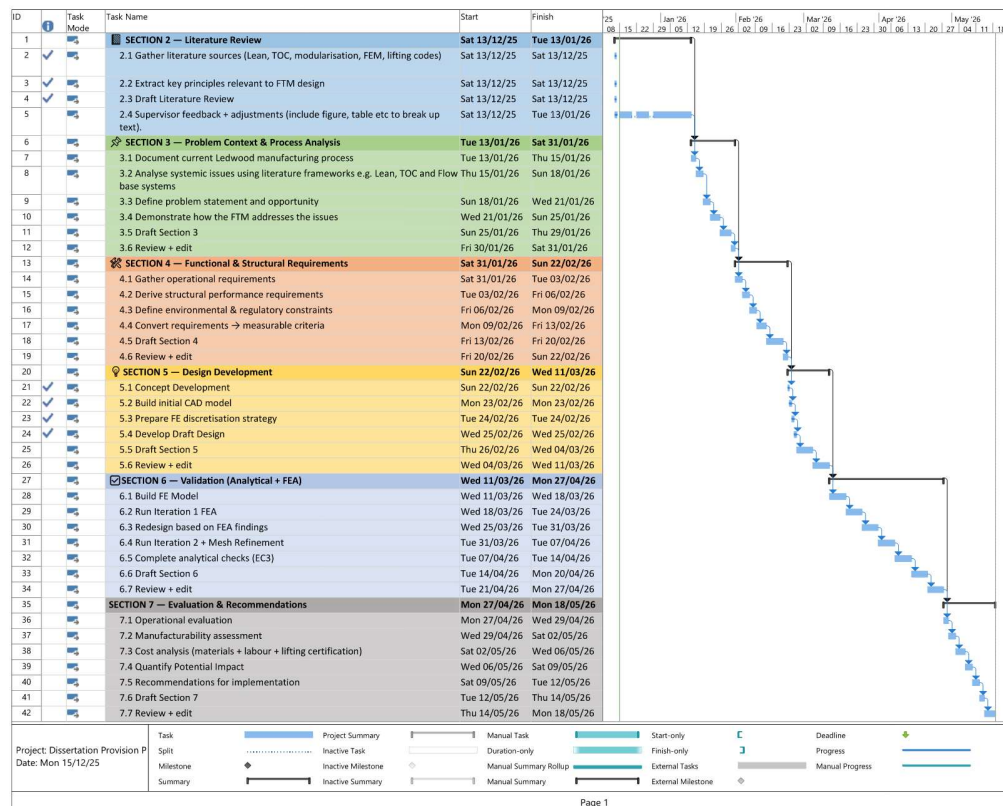


Figure 1 - Dissertation Programme and Work Breakdown Structure Gantt Chart. Source: Created by document owner.

2 Literature Review and Industrial Review

Increasing complexity and regulatory pressure within engineer-to-order infrastructure projects are placing greater demands on fabrication contractors to deliver predictable manufacturing performance (Department for Energy Security and Net Zero , 2025). As a result, traditional project-shop environments are increasingly challenged by high variability, unstable workflow and elevated work-in-progress (WIP). These conditions have driven growing interest in modularisation and flow-control strategies capable of improving operational stability without removing the flexibility required for bespoke fabrication (BERR & HSE , 2008; Braun, 2021).

2.1 Project-Shop Fabrication in Engineer-to-Order Heavy Industry

A project shop is a form of production and organisational structure in which work is arranged around discrete projects rather than a continuous flow of standardised products. It is well suited to one-off or highly customised outputs, where priority is placed on meeting individual project requirements and delivery milestones rather than maximising repeat efficiency (Braun, 2021). As illustrated in Figure 2, project-shop production contrasts with alternative production methods in both structure and scalability. While effective at low volumes and high variability, the comparison highlights that as production volume increases, the limitations of the project-shop model become more pronounced. In this context, continued reliance on a pure project-shop approach results in repeated “sprints” to complete each project, rather than the sustained, predictable flow characteristic of more mature production systems, indicating the need for a transition toward an alternative or hybrid production method as volumes grow (Roser, 2017; Slack, et al., 2022).

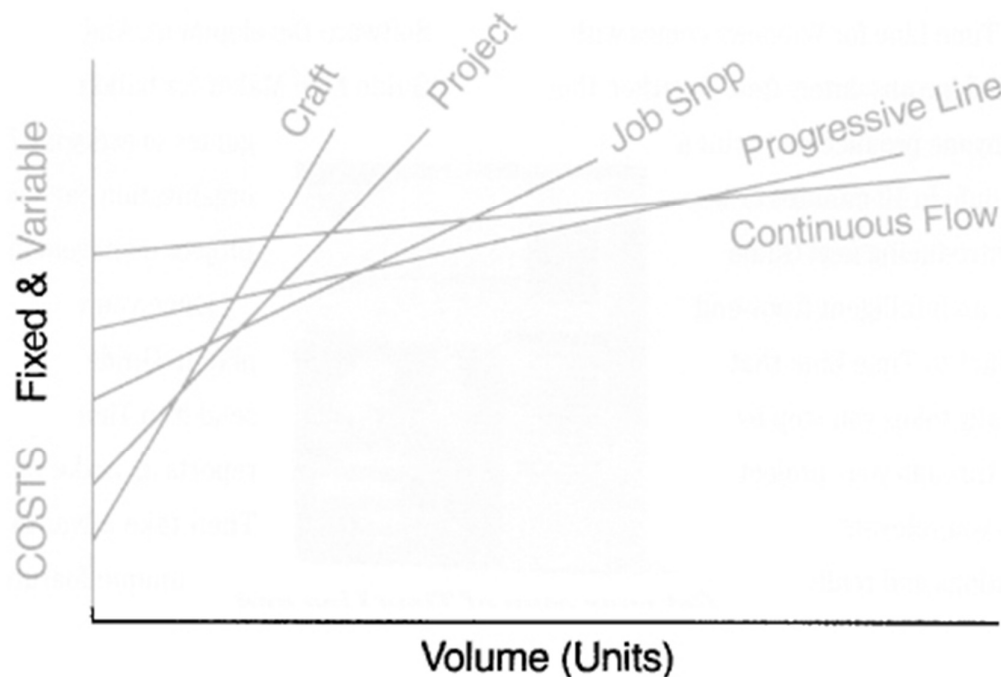
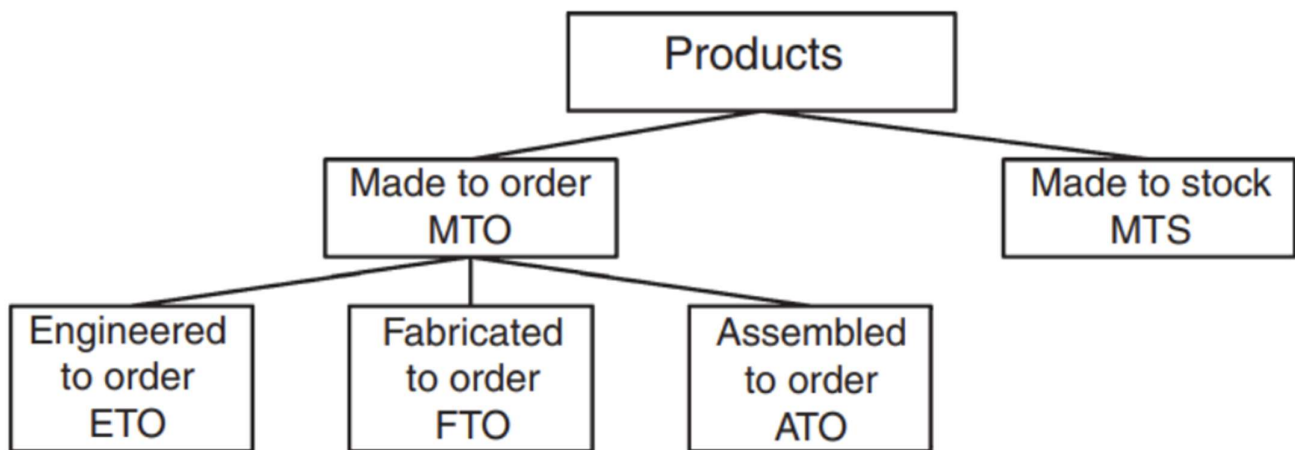


Figure 2 - Cost-Volume Break-Even Curve for Alternative Mode of Manufacturing. Source: Adapted from (Webster, 1993).

At Ledwood's Waterloo operations work is governed primarily by project delivery logic: fabrication is executed through work packages tied to programme milestones, evolving engineering definition, regulatory controls and installation readiness constraints. The underlying system is therefore not optimised around repetitive routing efficiency, but around absorbing and resolving the inherent uncertainty associated to Engineer-to-Order (ETO) / Fabricated-to-Order (FTO) delivery. Project-oriented organisation such as Ledwood are common in construction and large engineered products, where manufacturing should be thought of as a network of competing interfering projects via the same shared constraints rather than a defined and clear manufacturing line (Webster, 1993).

Figure 3 – Type of Manufacturing Product Types. Source: Adapted from (O'Brien, et al., 2008)



While project shops and job shops can both exhibit high variety and low volume, project shops stand unique their environment is dominated by project uncertainty and risk; incomplete scope definition, late design maturation, shifting priorities driven by project access, and the need to preserve traceability and compliance as work progresses is more acute in project shops compared to job shops. Which has variable routing of many small jobs across work centres where the aim is to sequence competing order through shared machines. Production priorities are frequently dictated by project state and emergent site demand rather than stable production sequencing (Graves, 2008; Zall, et al., 1994).

The challenge therefore becomes how project shops can achieve predictable flow despite unavoidable uncertainty and variability without relying on expediting or excessive WIP buffer (Goldratt, 1997; Goldratt, 1992).

2.2 Why traditional Project-Shop Fabrication is Increasingly Misaligned

Historically, project-shop fabrication has been a rational response to Made-To-Order (MTO) markets. It enables firms to accept bespoke work, adapt to changing requirements and mobilise skilled trades against unique geometries and tolerances. However, the direction of the wider sector is increasingly demanding predictable delivery even for bespoke and regulated work (Davies, et al., 2016). This becomes all the more pressing when it comes clear the future role that these manufacturing system will play in green infrastructure build out (Welsh Senedd, 2018; Slack, et al., 2022).

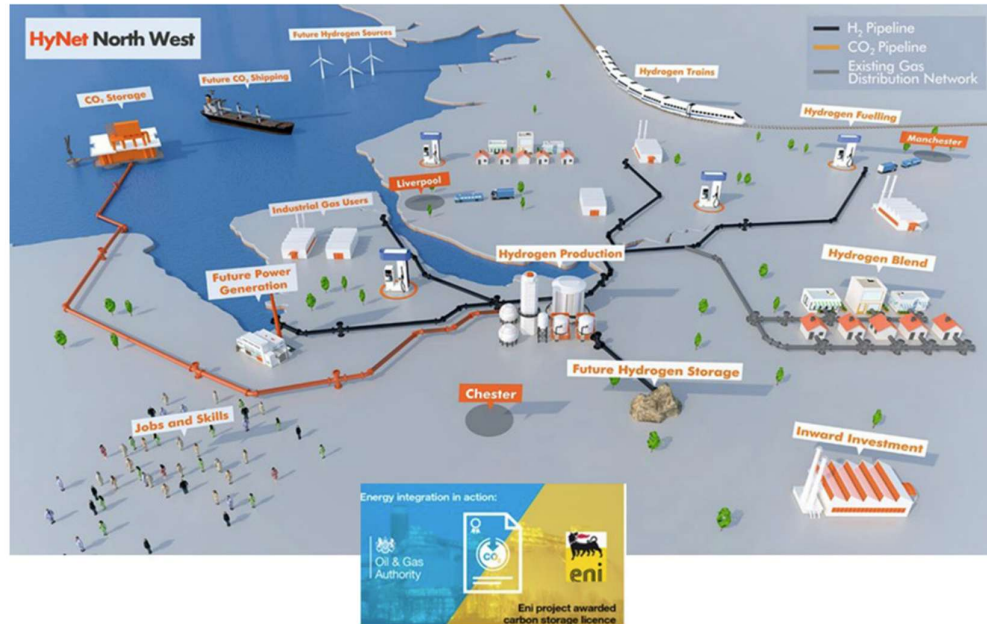


Figure 4 – Map of HyNet Project of Green infrastructure build out. Source: (HyNet, 2024).

Figure 4 illustrates the scale and interconnected nature of future low-carbon infrastructure programmes such as HyNet North West. While presented at a system level, each element shown within the network (e.g., hydrogen production facilities, pipelines, storage systems, compressor stations, CO₂ transport infrastructure and industrial interfaces) ultimately consist of large volumes of fabricated steelwork, pipework, pressure vessels and systems as well as modular assemblies produced within ETO or project shop manufacturing environments.

The transition toward low-carbon infrastructure is unlikely to reduce reliance on project-shop fabrication; rather, it is expected to intensify it. Major infrastructure sectors such as nuclear, hydrogen, carbon capture and offshore wind continue to depend on bespoke fabricated assemblies characterised by evolving design information, site-specific interfaces and strict regulatory requirements. Consequently, requiring; certainty, traceability and quality control commonly associated with repetitive manufacturing systems (Davies, et al., 2016).

This pressure is further amplified within government-supported or strategically significant infrastructure programmes, where delays, cost overruns and compliance failure attract increased public and commercial scrutiny (Flyvbjerg, 2017). As expectations extend across the wider supply chain, traditional project-shop environments such as Ledwood's Waterloo facility increasingly struggle to maintain predictable flow under conditions of high uncertainty.

A common response within project-shop systems is to absorb variability through elevated work-in-progress (WIP), early work release and reactive expediting. Rather than gating production against confirmed downstream demand, fabrication activities are frequently progressed “just in case” they are later required, creating completed-but-undelivered inventory that acts as a buffer against schedule uncertainty (Jorgensen & Emmitt, 2008; De Meyer, et al., 2006).

While this approach may preserve short-term responsiveness, it also generates persistent queues, increased handling demand and reduced production stability. The result is a manufacturing environment that appears highly active, yet becomes progressively more difficult to govern, predict and control (Petropoulos, et al., 2025).

2.3 Structural Dynamics of Project Shops: Variability, Maturity, and Systemic Instability

Why are Project Shops so exposed to high level of uncertainty and risk? This exposure does not arise primarily from poor management practice, but from the projects environment inability to eliminate uncertainty and variability. Contrasting for example with high-volume manufacturing systems which progressively standardise, stably route processes through repeatable processes (Carvalho, et al., 2014; Brytebuild, 2024). Project shops inherently don’t have this luxury, bespoke integration is key to their business model, as a consequence this limits their ability to remove variability (Morgan & Nyenje, 2021). Incomplete information and evolving requirements are common and have to be carried by production and managed dynamically, this leads to reset in production system maturity in each project cycle as the goal post are shifted to meet changing standards, requirements and goals. This compounds a project shops ability to “learn” or continuously improve as each project feels like restarting and re-inventing the system and any previous lessons learnt feel abstract or are difficult to draw upon as each project feels and is unique (Aapaoja & Haapasalo, 2014). Unlike higher volume production systems where the initial awkward exploration phase gives way to much more optimised exploitation phase of the system as showcased in Figure 5. This is why project shops often exhibit poor operational performance or excellence (Bresen, et al., 2003).

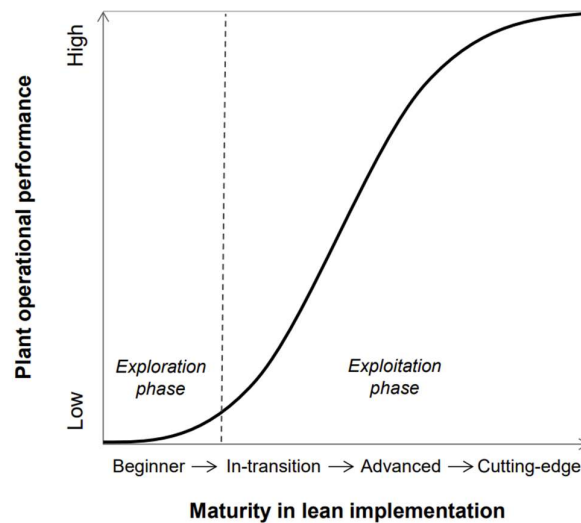


Figure 5 – Relationship between system maturity and operational performance. Source: Adapted from (Netland & Ferdows, 2016)

A second structural driver of risk within project shop is multi-project contention for shared constrained resources, often resulting in volatile “feast and famine” capacity cycles (Steyn, 2002). These shared constraints create workflow interference, stop-start production behaviour, and frequent reprioritisation as scheduling becomes increasingly driven by external project pressure rather than internal production logic (Sanchez, et al., 2023). The consequence being that work is executed less through stable production rhythm and more through reactive coordination, often described operationally as “organised chaos” (Hayes & Wheelwright, 1979).

Collectively, these characteristics position project shops as risk-absorbing systems within the wider project delivery model, buffering site-driven uncertainty through the fabrication cycle itself (Atli & Krystallis, 2025). Importantly, this instability does not originate solely from fabrication activities, but can also arise from upstream factors such as delayed engineering release, procurement constraints, incomplete design information, material shortages, and ineffective planning. Regardless of the source, fabrication operations are frequently required to absorb these disruptions in order to protect downstream installation and commissioning activities (Petroopoulos, et al., 2025).

In practice, variability is commonly buffered through elevated work-in-progress (WIP), flexible sequencing, early release of work packages, and reactive decision-making. While these mechanisms may preserve short-term delivery capability, they simultaneously embed instability within the production system as disruption and schedule churn propagate between projects (Garcia-Lopez & Fischer, 2016).

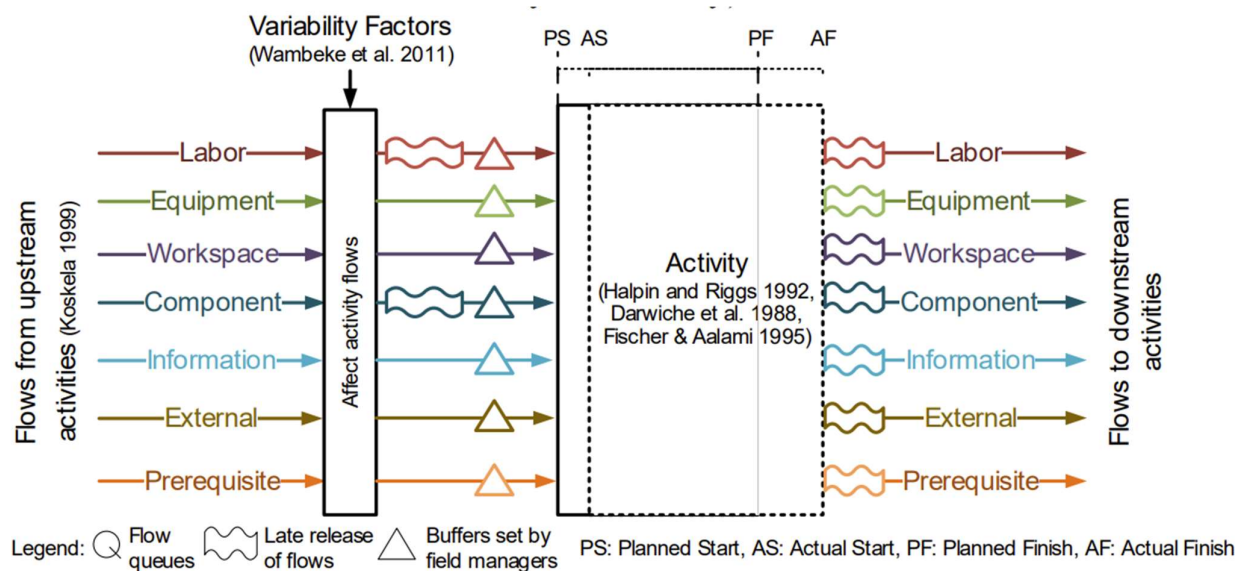


Figure 6 – A conceptual model of Construction workflow showcasing how upstream disruption flows to downstream instability via late release. Source: Adapted from (Garcia-Lopez & Fischer, 2016).

Figure 6 conceptualises this relationship, illustrating how upstream disruption propagates downstream through late information release and reactive workflow behaviour, ultimately generating instability within construction and fabrication operations.

Making incremental improvement difficult, deprioritised as operational survival becomes the dominant priority. This lack of innovation, coupled with increasing regulatory scrutiny and delivery pressure, often results in systems evolving only sufficiently to remain compliant rather than genuinely competitive (Schwierking & Anantatmula, 2015).

2.4 The Resilience of Project Shops and the Emergence of Hybrid Project–Flow Systems

Despite the structural challenges, the project shops presents distinct advantages in absorbing uncertainty and enabling complex delivery in spite of their inherent inefficiencies they continue to dominate ETO and FTO heavy fabrication environments (Halse, 2015). Their persistence reflects the distinct advantages they offer in absorbing uncertainty that cannot be decoupled or fully removed from project environments (Kjersem & Junge, 2016). Project shops are uniquely capable of progressing work in the absence of certainty, allowing projects to advance despite upstream uncertainty that would otherwise stall delivery entirely in other production systems (Gosling, 2011).

This capability is particularly critical in regulated and first of a kind (FOAK) context, such as nuclear new build, hydrogen infrastructure, and carbon capture systems. In such environments, full standardisation and early design freeze are often impractical, as compliance requirements, interfaces, and acceptance criteria evolve as the project progresses this inertia is inherited to project delivery of FOAK (MIT, 2018; Hafele, 1981). The loosely defined and flexible nature of projects shops allows them to absorb this inertia and push delivery. Furthermore, their reliance on skilled craft labour and local problem-solving supports and bespoke integration in ways that highly standardised flow systems cannot readily replicate until much later in their production system maturity (Ford, 2015; EASE, 2025).

However, while these strengths explain why project shop survive as a practice, they do not negate the growing misalignment between traditional project-shop structural behaviour and contemporary delivery expectations. Increasing scrutiny, tighter installation windows and reduce tolerance for rework and demand for greater predictability and stability, even in bespoke regulated work (Maqbool, et al., 2023; Love, et al., 2025). The central questions therefore is not whether project shops should be replaced but whether their performance can be meaningfully improved without undermining their core ability to absorb uncertainty.

Evidence from both industry practice and the literature suggests that wholesale application of lean or flow-based manufacturing principles is neither feasible nor effective in project-shop environments (Tantardini & Staudacher, 2012). Attempts to eliminate variability entirely, enforce uniform takt or impose rigid pull systems across all activities typically fail when confronted with immature design, regulatory change and shared resource contention. Instead, improvement efforts have increasingly focused on hybrid project-flow systems such as that shown in figure 7, which selectively introduce flow-based control mechanisms where variability can be contained, while preserving project-based flexibility elsewhere (Mirzaei, et al., 2025).

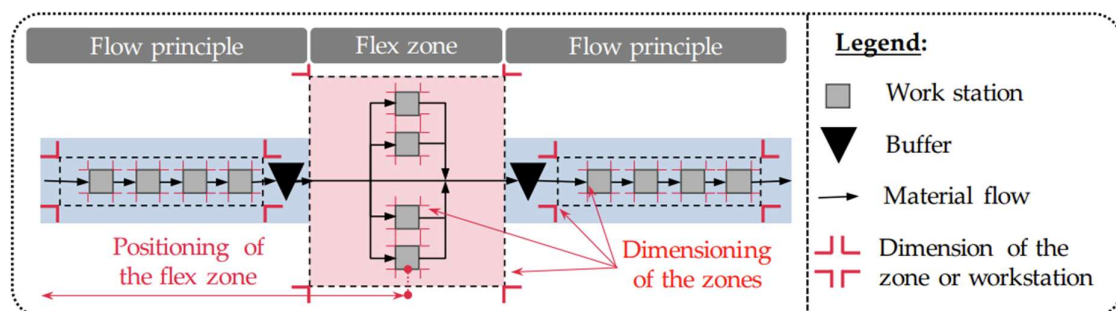


Figure 7 – Hybrid Structure as a combination of flow and job shop principles using 'flex-zones'. Source: Adapted from (Bleckmann, et al., 2025).

These emerging approaches indicate that project shops are not inherently doomed to inefficiency. Rather, that implementing rigid pull systems across all activities and wholesale application of lean or flow based principles aren't feasible, as it has to be understood that variability and some level of risk will always be present within the system it cannot be wholly eliminated (Zheng & Carvalho, 2016; Apaolaza, et al., 2020). But using mechanisms such as modularisation to standardise interfaces rather than the product content, decoupling points to isolate upstream variability from downstream execution, and selective use of pull between set stages of production routing e.g. cutting, Fit-up, Welding, Painting. Collectively these interventions aim to reduce work-in-progress, dampen priority churn, and improve predictability without attempting to suppress the inherent uncertainty around project delivery, and re-define project shop place in project memorandum nor its capabilities (Gepp, et al., 2016; Wikner, 2014).

2.5 System Control in Project Shops: Modularisation & Constraints

Efforts to improve project-shop performance increasingly recognise that variability cannot be eliminated but must instead be strategically contained and governed (De Meyer, et al., 2002). Consequently, improvement efforts focus not on standardising products or enforcing uniform production rhythms, but on implementing system-level control mechanisms that stabilise execution while preserving the flexibility required for ETO delivery (Perrin, et al., 2016). Key approaches include modularisation of interfaces rather than product content, the use of decoupling points as control boundaries, and constraint-led planning methods such as Theory of Constraints and Drum Buffer Rope (Gronvald, et al., 2025).

In project-shop environments, modularisation is most effective when applied to interfaces, logistics envelopes, and control artefacts rather than internal product geometry. Although fabricated components remain bespoke, the upstream and downstream interfaces they pass through are often consistent (Gronvald, et al., 2025). For example, fabricated pipework and structural assemblies must ultimately progress through common post-fabrication activities such as painting, galvanising, transport and dispatch.

By introducing a transportation module that standardises handling, staging and dispatch interface, a stable release and sequencing logic can be established around an otherwise variable production environment (Filipe & Pimentel, 2023). Work is prioritised around completion of the defined module rather than continuous initiation of new components, naturally constraining WIP and reducing accumulation of partially completed assemblies (Steele, 2025).

Within LMEL operations, fabricated items are additionally constrained by the logistics envelope of a standard flatbed trailer. Defining the module around this dominant transport constraint transforms it into a repeatable unit of control that reflects the practical limitations of the wider system (Wortmann, et al., 1997; Goldratt, 1997).

Standardised interfaces also function as decoupling points, separating unstable upstream project activities from controlled downstream execution (Cannas, et al., 2019). By discretising delivery into modular units, project shops can absorb schedule churn and reprioritisation without destabilising workshop operations. This reduces uncontrolled WIP growth, rework and traceability loss while allowing production to remain responsive to project-driven change (AutoDesk, 2020).

In this configuration, modular deliveries provide a stable and defensible scope of work, enabling project shops to reconcile project-driven uncertainty with controlled execution without excessive buffering or reactive expediting (Aapaoja & Haapasalo, 2014).

2.6 Ledwood's Fabrication Transport Module (FTM) as a System-Level Intervention

The selection of the Fabrication Transport Module (FTM) was driven by analysis of operational behaviour observed within project-shop environment, particularly the accumulation of completed but undelivered assemblies. Figure 8 therefore highlights that the primary constraint within the system was not solely fabrication capability, but the inability to effectively control the transition between fabrication completion, transport, dispatch and site installation. This distinction is significant because conventional improvement approaches within project-shop environment frequently focus on increasing fabrication productivity, labour utilisation or workshop efficiency, despite the underlying instability originating from the logistics and dispatch interface.

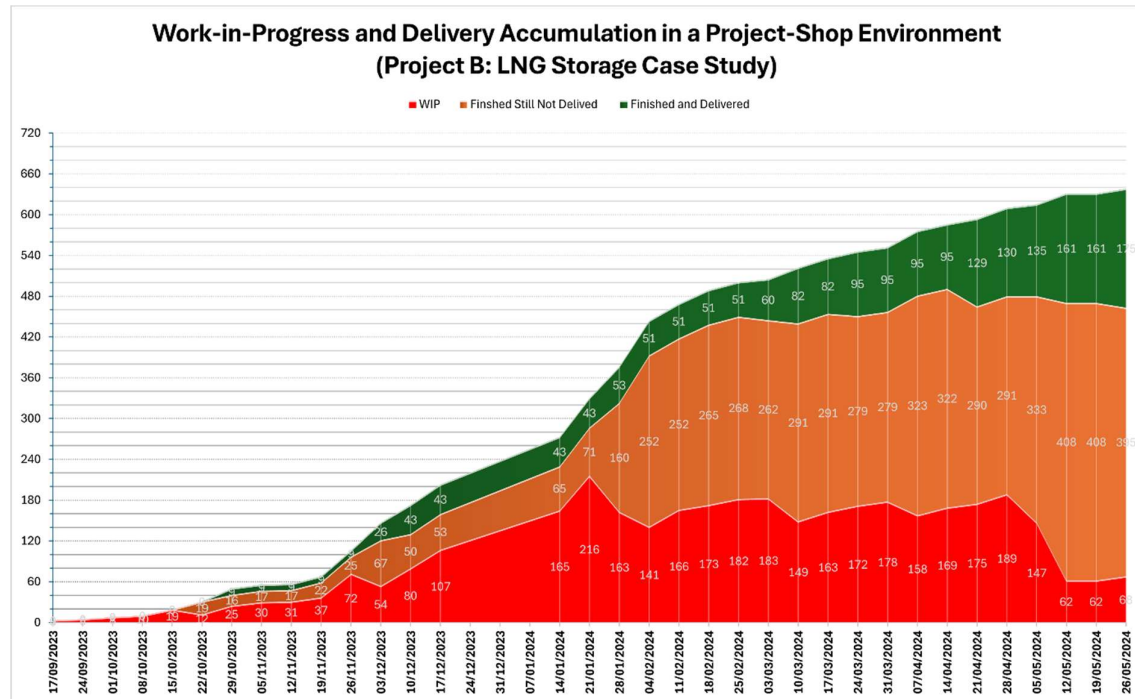


Figure 8 – Cumulative WIP, Completed and Delivered Pipe Spools Over Project “B” Lifecycle Chart for Ledwoods Project.
Source: created by document owner.

The FTM represents a system-level intervention rather than a discrete item of lifting or handling equipment. While the FTM performs an essential physical role in supporting the movement and transportation of fabricated assemblies, its primary contribution lies in how it restructures the control logic of the production system. Specifically, the FTM would function as both a decoupling mechanism and a unit of control, enabling variability to be contained and managed rather than allowed to propagate unchecked through the workshop (Phipps, 2025).

The challenges and interventions explored in this review have implications that extend beyond Ledwood and are indicative of broader structural issues facing UK heavy fabrication sector. As the UK transitions toward regulated, capital-intensive green infrastructure buildout strategies the competitive basis of domestic fabrication capability is shifting (High Peak Steels News, 2025). Cost competitiveness driven by labour cost is increasingly losing relevancy in highly regulated sectors;

instead, competitive advantage is emerging around delivery reliability, regulatory compliance and systems integration (Turkmen, 2024). In these sectors value is not created solely through low fabrication cost, but through the ability to deliver complex assemblies predictably, traceably, and in alignment with tightly controlled installation and commissioning windows (Mirzael, et al., 2021). Failure to do so propagates risk and cost downstream, often dwarfing any marginal savings achieved through cheaper labour. Consequently, fabrication systems that can reliably integrate with regulated project environments become strategically significant assets rather than interchangeable suppliers (Government Office for Science, 2013).

Ultimately, the future competitiveness of Ledwood within the context of the UK heavy fabrication sector is likely to rest not on becoming the lowest-cost producer, but on becoming the most dependable partner in regulated project delivery (OneAdvanced PR, 2024).

3 Problem Analysis and Current State Assessment

3.1 Weld Repair as an Indicator of Production Instability

One of the most direct and quantifiable consequences of production instability in project-shop environments is increased defect rates in finished welds. Weld defect rates are particularly sensitive to system conditions, as it is strongly influenced by schedule compression, expediting pressure, and reactive production behaviour (Nepal & Park, 2006). Project A, a petrochemical brownfield upgrade, provides a representative case study. As inefficiencies accumulated and programme pressure increased, LMEL introduced a productivity incentive scheme to recover lost schedule. While this improved throughput in the short term, it also shifted focus away from right-first-time quality. As shown in Table 1, individual weld repair rates increased from less than 1% to 10%, with the project-wide repair rate reaching 5.5%.

Table 1 – Comparison of individual welder repair rates derived from radiographic testing results on Project A.

Welder ID	No Welds	Sum of RAD Accept	Sum of RAD Reject	Repair Rate based on Overall Number of Welds	Repair rate based on Number of Tested Welds.
L947	284	211	31	10.92%	12.81%
L942	100	72	6	6.00%	7.69%
L939	723	538	53	7.33%	8.97%
L597	428	324	62	14.49%	16.06%
L1290	301	224	27	8.97%	10.76%

The resulting impact was significant, generating approximately 3,496 additional labour hours and an estimated direct labour cost of £63k based on conservative blended labour rate shown in Appendix A. This estimate captures only direct repair labour and excludes secondary impact such as additional NDT, schedule delay, disruption to installation sequencing, and the lost opportunity cost of diverting skilled welders from productive work. These indirect impacts are likely to be substantial given that NDT was typically conducted four to six weeks after weld completion, meaning defects were often identified long after fabrication activities had progressed.

Importantly, the increase in repair activity cannot be attributed to greater technical complexity or reduced workforce competence. Rather, it demonstrates the effect that unstable system conditions can have on quality performance and project profitability. Weld repair therefore provides a measurable indicator of economic penalty associated with unmanaged variability within project-shop environments.

Analysis of Project records indicated the underlying driver of the schedule pressure on Project A was not fabrication performance itself, but a high volume of late engineering changes, with the project experiencing in excess of 1,000 drawing revisions throughout its lifecycle.¹ This directly reflects the characteristics discussed in Section 2.3, where project shops act as risk-absorbing system for wider project uncertainty. In this case, variability originating from engineering and project control propagated into fabrication, where it manifested as schedule compression, productivity incentives, and ultimately increased weld repair. The cost of this instability was therefore not removed from the project but transferred into the fabrication system.

3.2 WIP as a System Problem

Figure 9 illustrates a typical snapshot of WIP within Ledwood Mechanical Engineering Pembroke Fabrication facility. The image shows a large volume of partially completed pipe spools, fittings and assemblies distributed across the workshop floor, occupying fabrication bays, access routes, and laydown areas simultaneously. While each individual item presents a legitimate project scope, there collective presence indicates a production system operating with a high level of concurrent, unfinished work rather than a controlled flow of completed assemblies.

In project-shop environments, elevated WIP is not merely an inventory condition but a structural indicator of unresolved uncertainty, priority churn, and unstable release logic. Excess WIP increases handling, obscures production status, fragments traceability, and amplifies the cost of disruption when priorities change. As such, WIP provides a tangible and measurable proxy for system instability and serves as an appropriate point for assessing the current-state performance of Ledwood's fabrication operations.



*Figure 9 - Picture of Ledwood Workshop B during Project "B" showcasing the huge number of Work-In-Progress (WIP).
Source: Photo taken by document owner.*

¹ Based on confidential project record reviewed by the document owner.

4 Functional and Structural Design Requirements

4.1 Purpose

The purpose of this chapter is to translate the findings of the Section 2 and 3 into an engineered solution for the proposed FTM. Previous chapters identified elevated WIP, unstable workflow, and inefficiencies at the interface between fabrication, dispatch, and site installation as key sources of instability within project-shop environments.

Within LMEL analysis indicated that completed assemblies frequently accumulate faster than they can be dispatched and installed. The FTM is therefore conceived not simply as a lifting and transport frame, but as a modular unit of control intended to standardise handling, staging, transport, and release activities.

A key finding of the literature review was the effective project-shop interventions should be centred around system constraints. For LMEL, the dominant constraint is the geometric envelope of a standard articulated flatbed trailer. Consequently, the overall dimensions of the FTM are governed primarily by integration with existing transport infrastructure rather than structural optimisation alone.

The object of this chapter is therefore to develop and verify an FTM concept that:

- Operates within the geometric constraints of a standard UK articulated flatbed trailer (approximately 13.6m x 2.90m) (Low Loaders Europe , 2026).
- Accommodates the maximum credible transport payload.
- Maintains adequate strength and stiffness during lifting and transport.
- Remains practical to manufacture and deploy within existing fabrication operations.

The resulting design assesses whether a modular transport frame can function as both a structural lifting device and a system-level control mechanism for improving flow within a project-shop manufacturing environment.

4.1.1 Design Methodology Overview

The design process follows a structured, stage-gated workflow progressing from requirement definition through concept development, structural assessment, and iterative refinement. To govern this process, Pugh's Product Development Process was selected as the overarching design methodology, as illustrated in Figure 10. While alternative frameworks such as the VDI 2221 systematic approach or Ulrich and Eppinger's product development model offer comparable stage structures, Pugh's model was favoured for its explicit integration of a concept screening and selection matrix, which provides a transparent, criteria-driven basis for evaluating competing design alternatives a critical requirement given the structural and regulatory constraints of this project. Although Pugh's model has recognised limitations, particularly its relative linearity in early stages and its reduced prescriptiveness during detailed analysis phases, these are mitigated here through the deliberate incorporation of iterative feedback loops between the analytical and refinement stages, reflecting the cyclical nature of structural optimisation (Takai & Ishii, 2021; Frey, et al., 2009).

In accordance with Pugh's framework, Sections 4.2-4.4 address the early design phases of Product Design Specification (PDS) development, classification within relevant codes and standards, and concept generation and selection. Sections 4.5-4.8 correspond to the embodiment and analytical design phases, where the finite element (FE) model is developed, boundary conditions are screened, and governing load cases are applied to establish baseline of structural performance. Sections 4.9-

4.12 represent the iterative and detailed design stages, including optimisation of the primary frame, development of welding and fabrication methodologies, verification of lifting padeyes and consideration of material selection and fatigue behaviour. Collectively, this reflects the iterative loop between analysis and refinement central to Pugh's model, ensuring that the final design solution is both compliant, applicable to the standards and fit for purpose within the operational constraint at Ledwood Fabrication Yard and Construction Sites.

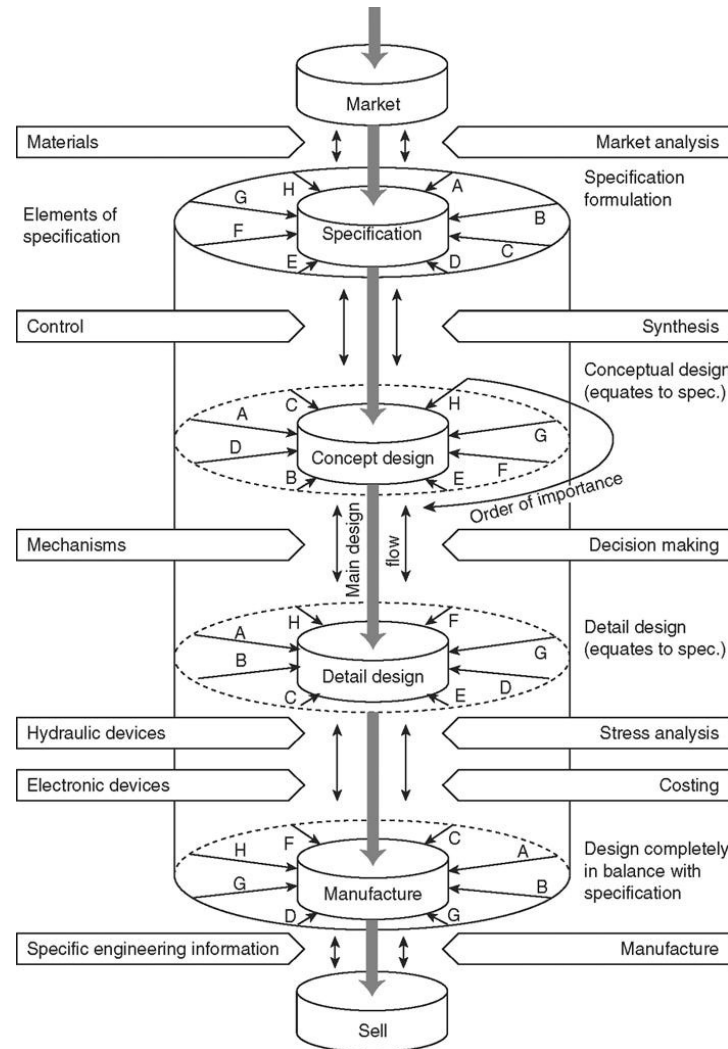


Figure 10 – Product Development Process Diagram. Source: (Pugh, 1990).

4.2 Product Design Specification

To ensure traceability between the research findings and the developed design, a concise PDS was established. The purpose of this PDS is not to define a complete commercial product specification, but to provide a clear set of engineering requirements against which the proposed FTM frame can be assessed.

The specification is directly informed by the constraints and operational needs identified in Sections 2 and 3. In particular, the geometric envelope of a standard 6-axle flatbed trailer represents the primary fixed boundary condition and therefore governs the maximum permissible footprint of the FTM. Similarly, variability in fabricated module geometry, lifting practices within heavy construction

projects, and the requirement to safely transport high-mass assemblies informed the capacity, handling and performance requirements defined below.

Only requirements that can be explicitly verified through analysis, calculation, or design review are included, ensuring the PDS remains proportionate to the scope of the dissertation and aligned with the subsequent structural assessment methodology.

Table 2 - Product Design Specification for FTM frame.

Requirement	Target / Specification	Rationale	Verification Method
1.1 Maximum Overall envelope	Overall length, width and height to fit within usable deck envelope of a standard 6-axle flatbed trailer as well meet standard height restrictions for roads and bridges.	Identified as the primary fixed system constraint governing feasible transport solutions	Design to be no larger than 13.6m x 2.9m and 2m high.
1.2 Maximum payload capacity	44Te design payload.	Represents the upper limit of payload that can be transported without classification as an abnormal load under National Highways requirements. It therefore also defines the maximum standard dispatch size for Ledwood; any item exceeding this threshold would require specific transport planning and controls.	Global FE analysis under representative load cases up to 44Te.
1.3 Lifting Interface	Provision of 4-6 lifting padeyes rated to carry 44Te.	As the overall envelope could, in theory, be loaded in an infinite number of configurations, each padeye is conservatively rated for the full design load. In practice, the load cannot be carried by a single padeye as the lift arrangement requires multiple lifting points to maintain stability.	padeye calculations to ASME BTH-1 / API RP 2A for minimum design load of 44Te.
1.4 Transport restraint interfaces	Provision of integrated tie-down / lashing points compatible with standard ratchet straps.	Ensures secure restraint of the payload prior to lifting of the frame.	Load-securing calculations to EN 12195-1 (or equivalent) to derive required restraint forces for braking/cornering and uplift; confirm number/position/angles of lashings.
1.5 Load positioning	Payload represented as distributed patch load with allowance for eccentric placement	Accounts for variability in payload geometry and non-uniform weight distributed.	Parametric FE load cases with varied load locations.

1.6 Handling Conditions	Frame to withstand lifting, set down and transport in quasi-static conditions	Reflects typical construction handling scenarios including crane lifting and placement.	Combined payload load cases including dynamic amplification factors.
1.7 Design codes and Standards	Structural members assessed to Eurocode 3; lifting elements assessed to ASME BTH-1 and API RP 2A-WSD principles.	Dual approach ensures compliance with both structural steel design practice and lifting appliance requirements.	Code-based calculations and load factors.
1.8 Strength requirement	Member stresses to remain within allowable limits defined by Eurocode 3 (EC3) and emphasised through lifting-specific checks.	Ensures structural integrity under worse credible loading scenarios.	FE stress utilisation review and member checks to EC3.
1.9 Fatigue Assessment	Not normally governing for the proposed duty, but design to avoid fatigue-sensitive details at primary lifting and load-introduction regions; if repeated service use is intended, assess critical welded details for the anticipated duty cycle in accordance with BTH fatigue provisions / recognised lifting-detail fatigue practice.	The frame is intended for intermittent lifting, transport and set-down rather than high-cycle continuous service. Fatigue is therefore unlikely to govern the global design, but local welded details at padeyes, attachments, cross-member junctions and restraint points may be susceptible to crack initiation under repeated handling cycles, particularly where stress concentrations or load reversals occur.	Carry out Fatigue Stress Assessment to ASME BTH-1 2017.
2.0 Deflection Limit	Global deflection limited to maintain stable support surface and prevent load shifting.	Excessive deformation may introduce slope or instability of transport payload.	FE displacement assessment and comparison with defined EC3 serviceability criterion and additional movement risk assessment informed deflection limit.
2.1 Global stability	No unacceptable twist or roll of the frame under combined loading.	Required to maintain load stability and safe lifting behaviour.	Evaluation of deformed shape and torsional response in FE results.
2.2 Weight efficiency	Frame Weight to be minimised while maintaining required stiffness and strength.	Supports ease of handling and transport efficiency without compromising safety.	Iterative optimisation using FE results and section rationalisation. Achieve a High capacity load-to-weight ratio ~20:1 (Load to Self-Weight).
2.3 Manufacturability	Use of commonly available UK sections with practical weld access and assembly sequence.	Ensures the design is realistically fabricable within a heavy fabrication workshop environment.	Section selection review and constructability assessment.

The PDS defined in Table 2 provides a clear link between the operational constraints identified in earlier chapters and the engineering checks undertaken in the remainder of this design study. Each requirement is directly mapped to a corresponding verification method, ensuring that the subsequent finite element modelling, calculations for padeyes and connections, and iterative member optimisation collectively demonstrating compliance with the stated design objectives.

4.3 Design Category / Class, Relevant Standard and Statutory Requirements

The FTM functions as a structural steel frame that is subject to both lifting operations and transport loading. As such, its design is governed not only by conventional structural steel design standards but also by regulatory requirements applicable to lifting accessories and work equipment. This section defines the design classification, applicable standards, and statutory framework adopted for the structural assessment presented in this chapter.

4.3.1 Structural Design Basis

The FTM primary steelwork is verified to Eurocode 3 (EN 1993-1-1) (BSI Standards Publication, 2022) using EN 1990 partial factor method, reflecting its function as a structural transport frame subject to fabrication, handling, lifting, transport and storage actions. Lifting is then treated as a transient design situation within the Eurocode framework for global member verification. This approach provides the basis for assessing members under combined axial force, bending moment and shear, together with overall stability and serviceability performance.

Serviceability considerations are included to limit global deflection and torsional distortion, ensuring that the frame provides a stable support surface for transported modules and does not introduce unacceptable geometric change during lifting operations.

The Eurocode verification follows EN 1990 load combinations with partial factors and EN 1993-1-1 resistance factors, adopting Nationally Determined Parameters from the UK National Annex (BSI Standards Publication, 2022). For ultimate limit state (ULS) STR/GEO combinations, permanent and variable actions are factored using;

$$\gamma_G = 1.35(\text{unfacturable})$$

$$\gamma_G = 1.00(\text{favourable})$$

$$\gamma_Q = 1.50(\text{variable action})$$

The governing variable action is taken as the lifting / handling action, defined as the payload together with equivalent inertial effects arising from hoisting and slewing the structure.

Steel member resistances are evaluated in accordance with EN 1993-1-1 using;

$$\gamma_{M0} = 1.0 \text{ for cross-section resistance,}$$

$$\gamma_{M1} = 1.0 \text{ for member buckling resistance, and}$$

$$\gamma_{M2} = 1.25 \text{ for fracture-related failure modes where applicable.}$$

4.3.2 Statutory and Regulatory Framework

The design and intended use of the FTM falls within the scope of UK legislation governing lifting operations and work equipment. The following statutory requirements are considered:

- **LOLER (Lifting Operations and Lifting Equipment Regulations 1998)**, which requires that lifting equipment is of adequate strength and stability, is appropriately marked with its safe working load, and is subject to suitable examination and Testing.
- **PUWER (Provision and Use of Work Equipment Regulations 1998)**, which requires that work equipment is designed and constructed so as to be suitable for its intended use and safe to operate.

Consequently, the design philosophy adopts the essential health and safety requirements relevant to lifting accessories, including structural integrity, prevention of unintended load release, and provision for inspection.

4.4 Initial Concept Development

The initial concept for the FTM frame was developed to satisfy the geometric, handling and capacity requirements defined in the PDS while maintaining compatibility with existing fabrication and transport practices. The concept adopts a modular steel frame arrangement comprising primary longitudinal beams, transverse cross-members. A sketch of which can be seen in Figure 11 below.

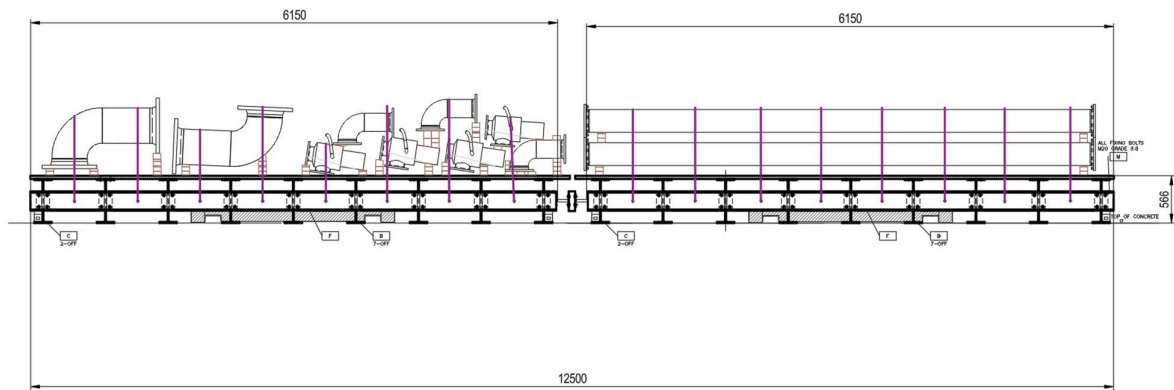


Figure 11 - FTM Concept Sketch fully loaded. Created by document owner.

4.5 Initial Concept Verification

Eurocode 3 / EN 1993-1-1 (BSI Standards Publication , 2022) adopts a limit state design approach to assess structural members subjected to combinations of axial force, bending moment and shear. The method verifies that the design effect of action E_d does not exceed the design resistance R_d of the member or connection. Design verification is carried out by checking relevant limit states, primarily the Ultimate Limit State (ULS), which ensures adequate strength, stability and safety against failure.

A fundamental requirement is:

Equation 1 – Verification criterion for design resistance

$$E_d \leq R_d$$

E_d Design Effect of Actions includes:

- axial force N_{Ed}
- bending moment M_{Ed}
- shear force V_{Ed}
- combinations of these

R_d Design Resistance these come from:

- Material strength f_y
- Cross-section properties
- Partial safety factors γ_M

4.6 Model Build and Baseline Assumptions

The following baseline assumptions were adopted for the initial concept verification of the structural frame. At this stage of the design process, the objective is to obtain a reliable understanding of the global structural behaviour, load paths, and member force distribution while maintaining a manageable level of modelling complexity. Simplifications are there introduced where appropriate to balance computational efficiency with engineering realism. Autodesk Inventor is used to carry out the initial frame analysis, this converts frame components such as the structural members into 1D beam elements, creating faster simplified simulations compared to standard 3D FEA.

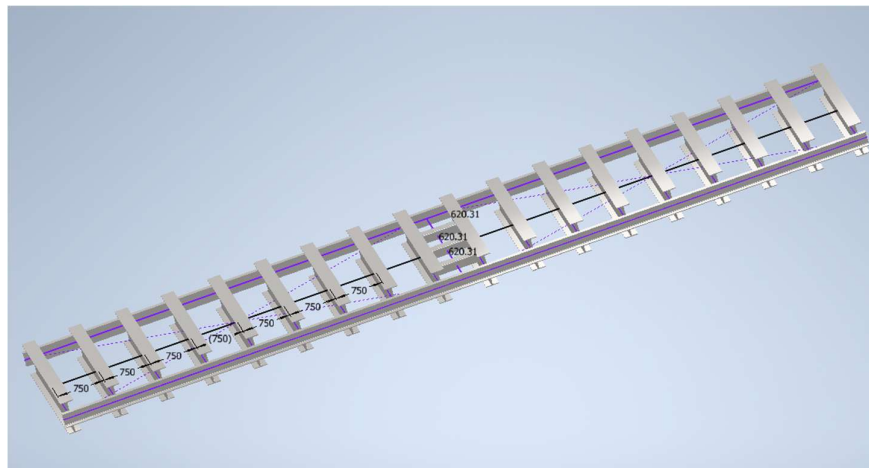


Figure 12 - Initial Frame Concept Modelled in Inventor. Created by document owner

Figure 12 showcases 1D elements used to represent the frame within the software. Although the members are visually represented using three-dimensional profiles corresponding to the selected structural section. The underlying finite element representation is defined only by a line element along the members centroidal axis. The cross-sectional properties are not mapped as physical geometry but are instead assigned through the section properties stores within the element metadata.

4.6.1 Connection Idealisation

The frame was modelled as a free-standing structure, with no artificial restraints applied along its base or perimeter. Instead, fixed support constraints were applied exclusively at the lifting points, representing the locations where the structure is engaged by slings during lifting operations.

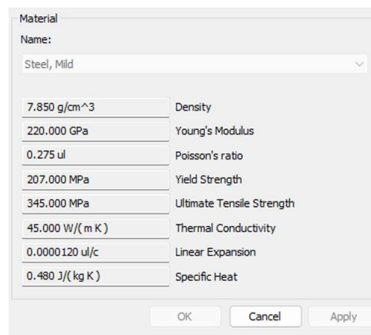
This approach provides a more realistic representation of the governing load case for the structure, where:

- Loads are transferred through discrete lifting points rather than distributed supports
- The frame is free to deform globally under load
- Internal force distribution develops naturally based on structural stiffness rather than imposed restraint conditions

By avoiding over-constraint, this modelling strategy eliminates artificial stiffness and reduces the risk of underestimating member forces or deflections. The resulting behaviour more accurately reflects the true load path and structural response during lifting, which is considered the critical design condition for the frame.

4.6.2 Material Property Simplification

All members within the analysis model were assigned generic mild steel properties rather than specific structural grades. This provides a consistent basis for concept-level assessment of load paths, stiffness, and member behaviour without reliance on a final material specification. As the structural grades considered for the final design possess yield strengths equal to or greater than those assumed in the model, the resulting stress utilisations are conservative. Designs demonstrating acceptable performance under these assumptions are therefore expected to exhibit equal or greater capacity when verified using the final specified materials.



Material	
Name:	
Steel, Mild	
7.850 g/cm ³	Density
220.000 GPa	Young's Modulus
0.275 ul	Poisson's ratio
207.000 MPa	Yield Strength
345.000 MPa	Ultimate Tensile Strength
45.000 W/(m K)	Thermal Conductivity
0.0000120 ul/c	Linear Expansion
0.480 J/(kg K)	Specific Heat
OK Cancel Apply	

Figure 13 – Steel Frame Mild Steel Material Properties. Taken from Model in Inventor. Created by Document owner.

4.6.3 Static Load Representation

The frame is assumed to be in static equilibrium, and dynamic effects are not explicitly modelled. Instead, environmental actions such as wind loading and imposed loads are pointed to in section 5.1 of Future Work recommendations.

This approach aligns with common practice in Eurocode-based structural design, where complex loading conditions are simplified into static load cases during early design stages. By applying appropriate load factors and combinations in accordance with EN 1990 and EN 1991 (BSI, 2018).

This allows the analysis to capture the governing structural behaviour without introducing unnecessary modelling complexity.

4.7 Implications of Modelling Assumptions

The implications of the assumptions made for the initial concept verification introduce a number of simplifications which influence the accuracy and interpretation of the results. These simplifications are considered appropriate for the concept design stage. However, the results of the analysis should be primarily interpreted in terms of global member forces, bending moments and deflections, rather than detailed local stress states.

4.8 Define Actions (Loads)

Actions are the loads applied to the structure. Eurocode 3 classifies them as:

Table 3 – Classification of principal structural action categories considered within the FTM structural design basis, including permanent, variable, environmental and accidental loading actions.

Type of Action	Examples
Permanent G	Self-Weight of Steel members, cladding, equipment
Variable Q	Live loads, maintenance loads, crane loads
Environmental	Wind, snow, temperature
Accidental	Impact, explosion

For the purposes of concept verification analysis, the variable actions were divided into two primary load groups:

- Payload loads imposed by equipment, assemblies and pipework supported on the frame.
- Lifting loads generated during crane handling of the frame and payload assembly.

4.8.1 Variable Actions (Q) – Payload Load from Transported Assemblies

Payload loads represent the variable actions generated by fabricated assemblies placed onto the transport frame during fabrication, storage, and transport operations. Due to the variability in geometry, support conditions, and centres of gravity of transported assemblies, a deterministic load distribution cannot be defined. Accordingly, conservative idealised loading arrangements are adopted at concept design stage to envelope the most adverse global structural effects.

In practice, assemblies bear onto time dunnage positioned on the deck plate (Figure 14), producing localised patch loads. These loads are transferred through the deck into the supporting universal beams (UBs). For global analysis, these patch loads are conservatively idealised as concentrated vertical point loads applied at beam midspan, representing the least favourable bending condition for a simply support member and neglecting any beneficial load distribution through the deck plate.



Figure 14 – Example of Loaded Pipe Lorry showing dunnage beneath assemblies creating localised patch loads on the deck.
Source: REN Group Document IS.STD.060

The design payload capacity of the frame is defined as 44 tonnes, corresponding to the upper operational limit prior to abnormal transport classification. Applying EN 1990 load factors in Appendix B, this results in a factored global vertical load of:

$$647.46kN$$

4.8.1.1 Load Distribution Assumption

Analysis of over 1000 fabricated components (Figure 15) shows that the majority fall within the 0-20% length category relative to the frame, indicating that loading is predominately localised rather than uniformly distributed.

To represent this behaviour, a 75/25 load distribution is adopted as a data-informed upper bound for eccentric loading:

- 75% of total load applied to one side of the frame
- 25% applied to the opposite side

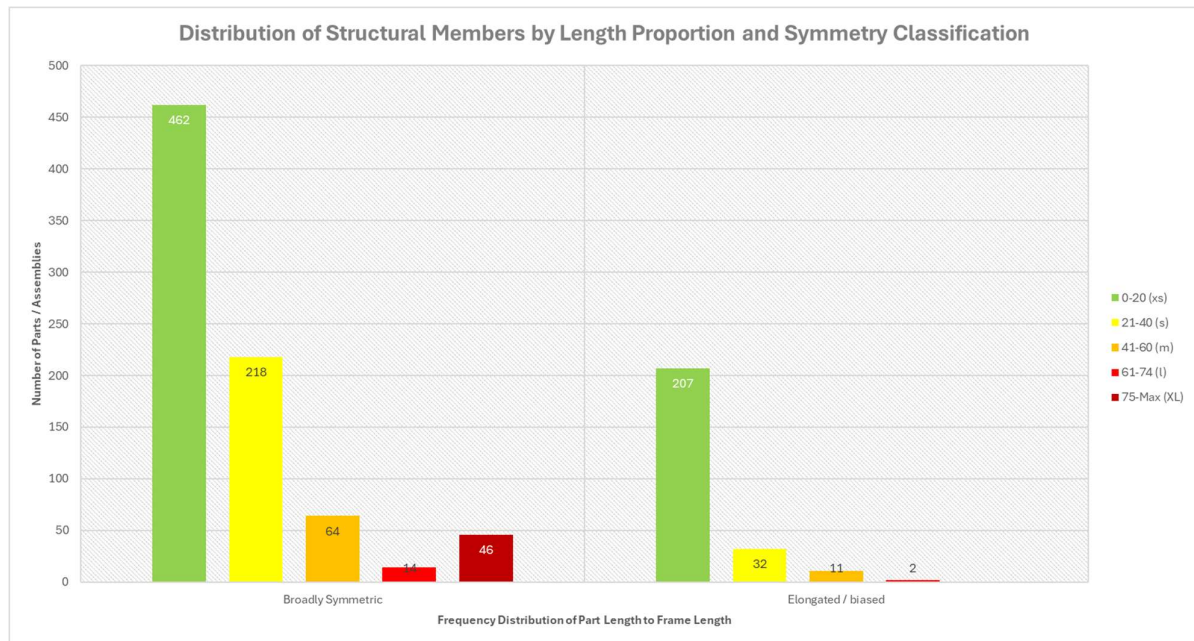


Figure 15 – Distribution of structural member lengths relative to frame length, highlighting the predominance of short, localised load components and limited full-span loading. Source: document owner.

The 75/25 distribution further represents a credible worse-case condition within normal lifting operations, incorporating the effects of eccentricity arising from asymmetry and non-uniform placement. Beyond this threshold, loading conditions would fall outside of accepted rigging practice. Guidance from LEEA (Lifting Equipment Engineers Association, 2024) states that loads should be balanced and stable prior to lifting, with centre of gravity positioned directly beneath the hook.

4.8.1.2 Explicit Load Application to Model

The global payload is distributed to the transverse members as discrete point loads:

- Total load: 647.46 kN
- Split by eccentricity:
 - Heavily loaded side: $0.75 \times 647.46 = 485.60$ kN
 - Lightly loaded side: $0.25 \times 647.46 = 161.87$ kN

Each side of the frame contains 9 transverse supporting members (18 total), therefore:

- Load per beam (heavy side):
 $485.60 / 9 = 59.96$ kN per beam
- Load per beam (light side):
 $161.87 / 9 = 17.99$ kN per beam

For global analysis, each of these is applied as a point load at the midspan of the corresponding transverse beam, resulting in:

- 9-point loads on the heavily loaded side.
- 9-point loads on the lightly loaded side.

This idealisation intentionally maximises bending effects and provides a conservative representation of localised loading behaviour.

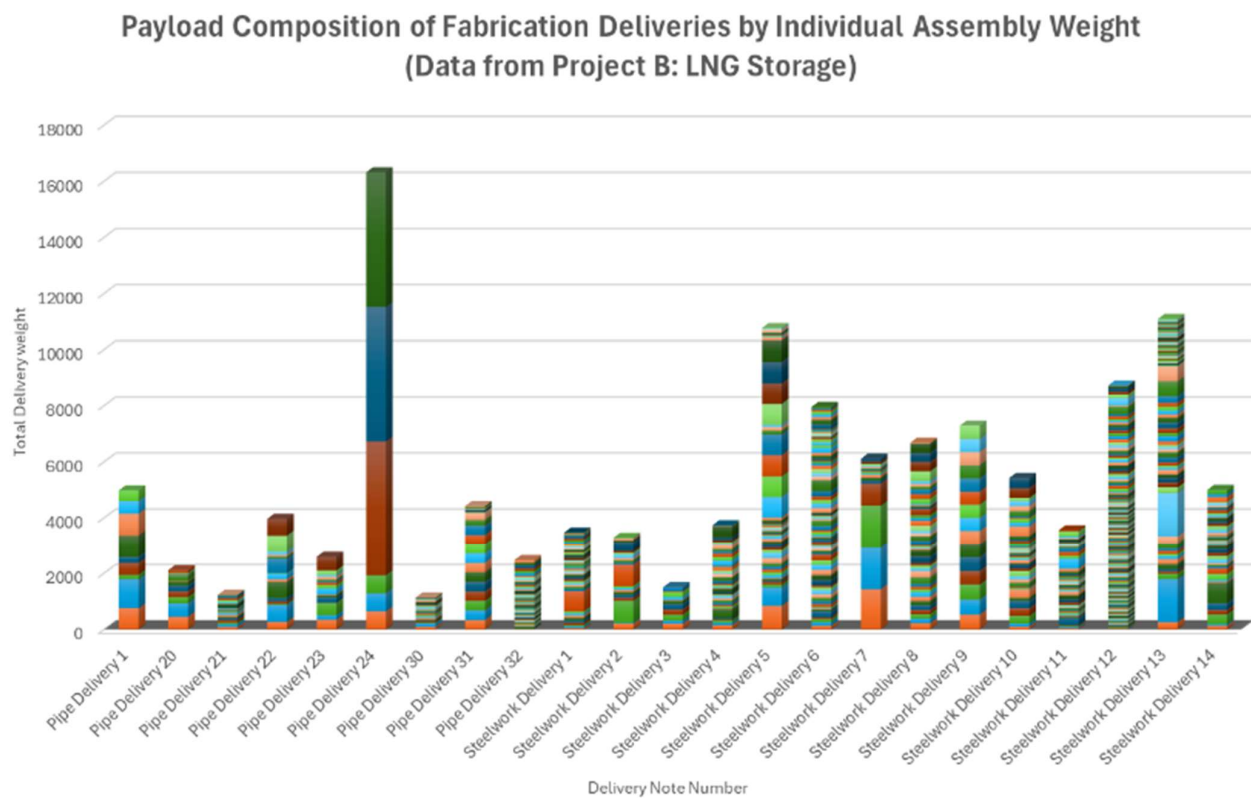
4.8.2 Operational Context and Conservatism

The 44-tonne design payload represents an upper-bound envelope rather than a typical operating condition. Project data (Figure 16) shows that actual delivery weights are significantly lower, with a maximum observed delivery weight of approximately 16 tonnes (~36% of design capacity).

The adopted loading model is therefore intentionally conservative and reflects:

- Uncertainty in future payload configurations.
- Localised load introduction via dunnage.
- Potential eccentric placement of assemblies.
- The requirement to demonstrate structural robustness across a wide range of service conditions.

Figure 16 – Composition of pipework and structural steel deliveries for Project B. Each coloured segment represents an individual fabricated assembly, with the cumulative height of each column indicating the total delivery weight.



4.8.3 Variable Actions (Q) – Lifting Actions

A second significant category of variable action considered in the analysis is the loading generated during lifting operations. Unlike the payload case, where the frame is supported from below through its deck and primary supporting members, the lifting load case represents a handling condition in which the combined weight of the frame and payload is transferred through discrete lifting points positioned around the perimeter of the structure.

During lifting, the total lifting weight acts vertically downward through the centre of gravity of the frame and its supported payload, while the supporting crane actions are introduced through the sling arrangement connected to lifting points. As a result, the frame is subjected to not only the vertical forces but to local bending and torsional effects arising from the eccentricity between the lifting points and the centreline of supporting members.

For the purposes of the analysis, the lifting actions were derived from the design lifted mass of the frame and payload combination.

The lifting forces acting at each lifting point were determined from the sling geometry. Where inclined sling legs are used, the force in each sling exceeds the simple vertical share of the load, as only the vertical component of the sling tension contributes to supporting the lifted weight. Accordingly, sling tension increases as the sling angle reduces from the vertical. The relationship can be expressed as:

Equation 2 – Relationship between sling tension, lifted load and sling angle for symmetrical two-leg lifting arrangement.

$$T = \frac{W}{n \cos \theta}$$

where:

- T = sling tension
- W = total lifted weight
- n = number of sling legs sharing the load
- θ = sling angle measured from the vertical

A maximum sling angle of 45° was adopted for this analysis to represent a practical upper-bound lifting condition consistent with standard rigging practices. While industry standards such as ASME B30.9 cite 30° as the absolute minimum allowable angle, it is treated as a critical limit rather than a standard operating procedure.

At a 30° angle, sling tension effectively doubles (reaching 200% of the load share), often triggering the requirement for a Critical Lift Plan. Furthermore, angles below 45° introduce significant horizontal (crushing) forces that necessitate the heavy downrating of hardware like eyebolts and shackles.

In a field environment, 45° serves as the representative threshold for general operations; most qualified riggers will proactively re-rig or adjust the hitch to maintain a steeper angle to ensure a higher factor of safety and protect the structural integrity of the load.

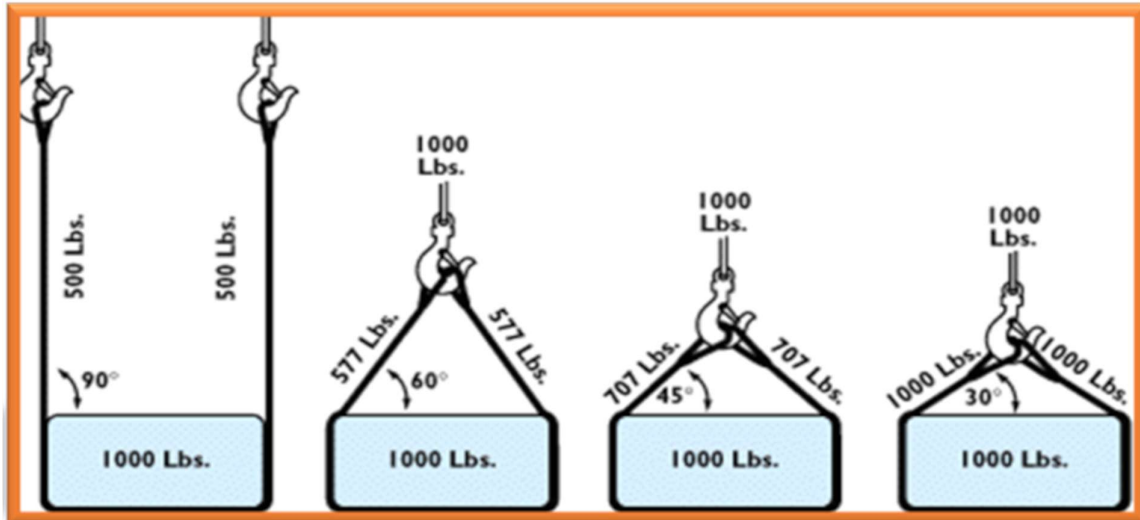


Figure 17 – Relationship between Sling Angle and Individual Leg Tension for a 1,000 lb Load. Source: (Collins, 2023)

Assuming that the front and rear lifting regions are supported by two lifting points, the 75/25 load bias gives a vertical share of 16,500 kg per front lifting points and 5,500 kg per rear lifting points. Converting these masses to forces gives:

$$W_{Front} = 16500 \times 9.81 = 161.87 \text{ kN}$$

$$W_{Rear} = 5,500 \times 9.81 = 53.96 \text{ kN}$$

Using the 45° sling angle, the corresponding sling tensions are:

$$T_{Front} = \frac{161.87}{\cos 45} = 228.92 \text{ kN}$$

$$T_{Rear} = \frac{53.96}{\cos 45} = 76.31 \text{ kN}$$

These tensions were then resolved into their vertical and horizontal components. At a sling angle of 45°, the horizontal and vertical components are equal in magnitude, such that:

Equation 3 – Resolution of sling tension into horizontal and vertical force components for a 45° sling arrangement.

$$F_V = F_H = T \times \sin 45$$

Therefore, for the front lifting points:

$$F_{VF} = F_{HF} = 228.92 \times \sin 45 = 161.87 \text{ kN}$$

and for the rear lifting points:

$$F_{VR} = F_{HR} = 76.31 \times \sin 45 = 53.96 \text{ kN}$$

For concept-stage verification, the lifting arrangement was treated as an equivalent static load case. Although additional dynamic effects may arise during hoisting, take-up of slack, slewing, or lowering, the use of a conservative sling angle and eccentric load introduction provides an appropriate basis for initial structural assessment of the frame under handling conditions.

4.9 Initial Concept Analysis

Following the definition of loading and boundary conditions, an initial global structural analysis was performed to assess load distribution, stiffness, and overall structural adequacy. The structural loads derived and calculated in Appendix B and section 4.8.1 & 4.8.3 are used as the basis of a worse case load scenario.

The results indicate that the longitudinal Parallel Flange Channel (PFC) members, which form the primary load-carrying elements along the length of the frame, are critically undersized. Peak normal stresses of approximately 880 MPa were observed (Figure 18), significantly exceeding the yield strength of typical structural steel (e.g. S275 / S355), indicating a clear failure of the initial load path design. Stress concentrations were predominately located along these longitudinal members, confirming that the structure was not effectively distributing loads into the transverse framing system.

In contrast the transverse members exhibited comparatively moderate stress levels, suggesting that their capacity was underutilised.

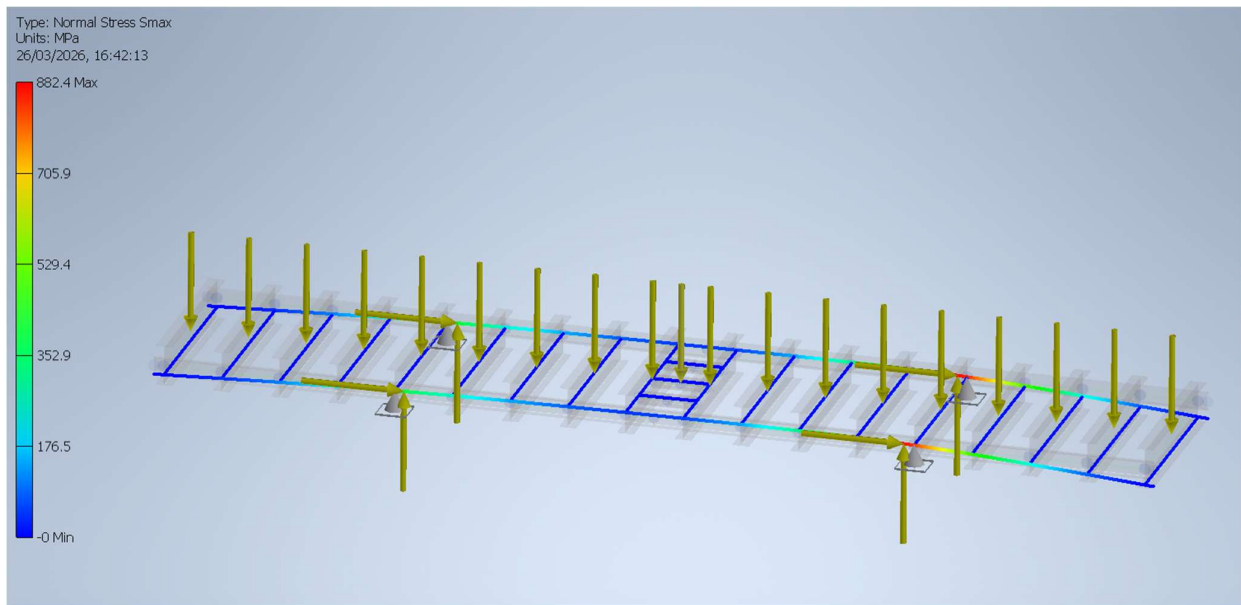


Figure 18 – Initial Concept: Normal Stress Distribution showing peak stresses (~880 MPa) concentrated in longitudinal PFC members, indicating inadequate load distribution and structural failure. Created by document owner.

Global displacement behaviour further reinforced this conclusion. Maximum deflections of approximately 218mm were recorded (Figure 19), with only regions directly aligned with lifting points showing limited displacement. This indicates that the structure behaves as a poorly coupled system with insufficient stiffness to transfer loads efficiently across the frame.

Such levels of deformation are unacceptable for lifting operations, where excessive deflection can result in:

- Load redistribution and instability,
- Increased sling force variation,
- and loss of geometric control during lifting.

Therefore, the initial concept fails to satisfy both ultimate limit state (ULS) and serviceability requirements, and a redesign is required to establish more efficient global load path, redistribute stresses away from overstressed longitudinal members, and significantly increase structural stiffness.

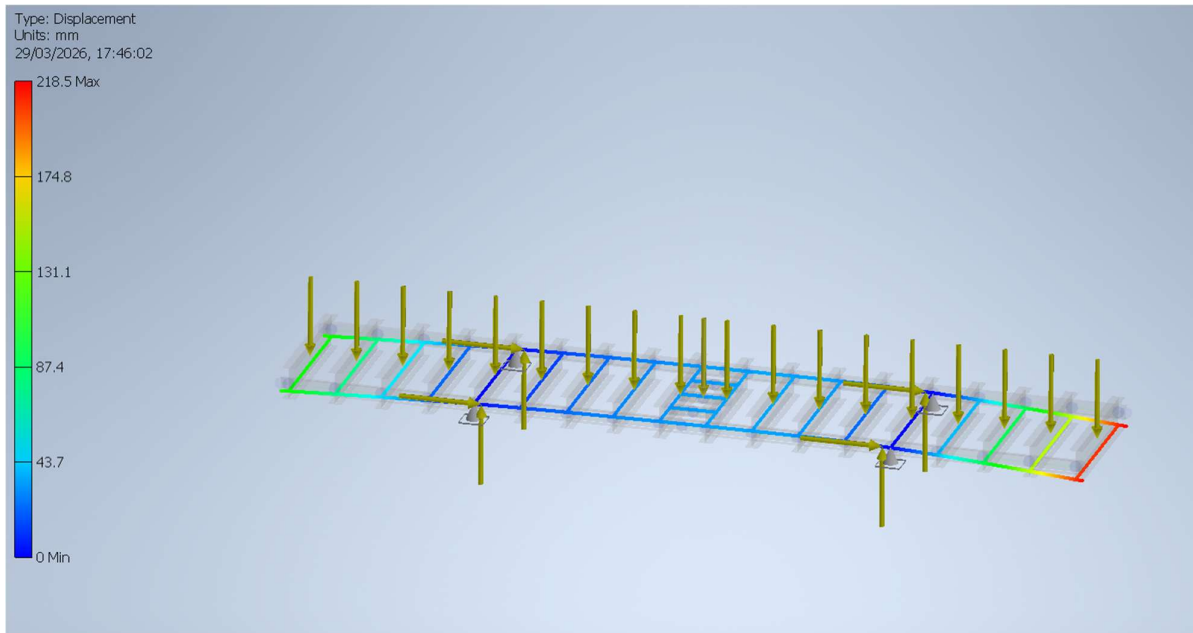


Figure 19 – Global displacement plot showing excessive deflection (~218mm), with deformation localised away from lifting points, indicating poor structural stiffness and ineffective load transfer. Created by Document Owner.

4.9.1 Design Iteration 2 – Structural Reconfiguration

The first redesign iteration aimed to address the identified deficiencies by improving both load distribution and global stiffness, while maintaining structural simplicity.

To achieve this, the longitudinal PFC members were replaced with a closed box section profile, increasing both bending and torsional stiffness. Closed section provides superior resistance to torsion and lateral instability compared to open sections, making them more suitable for lifting structures subjected to eccentric loading (Hughes, et al., 2011). These changes are shown in Figure 20 and 21.

Additionally, a dual longitudinal beam arrangement was introduced, consisting of:

- an upper longitudinal member connected to the top flange of transverse beams, and
- a lower longitudinal member connected via a triangulated bracing system.

This configuration creates a truss-like structural behaviour, increasing the effective depth of the system and significantly improving bending resistance without requiring a proportional increase in material (Brett & Rushton, 1990). The triangulated bracing system was implemented using angle sections to (Steel Construction Institute , 2010; Steel Construction Institute , 2015):

- provide efficient axial load paths,
- restrain flange rotation of transverse members,
- and enhance global torsional rigidity.

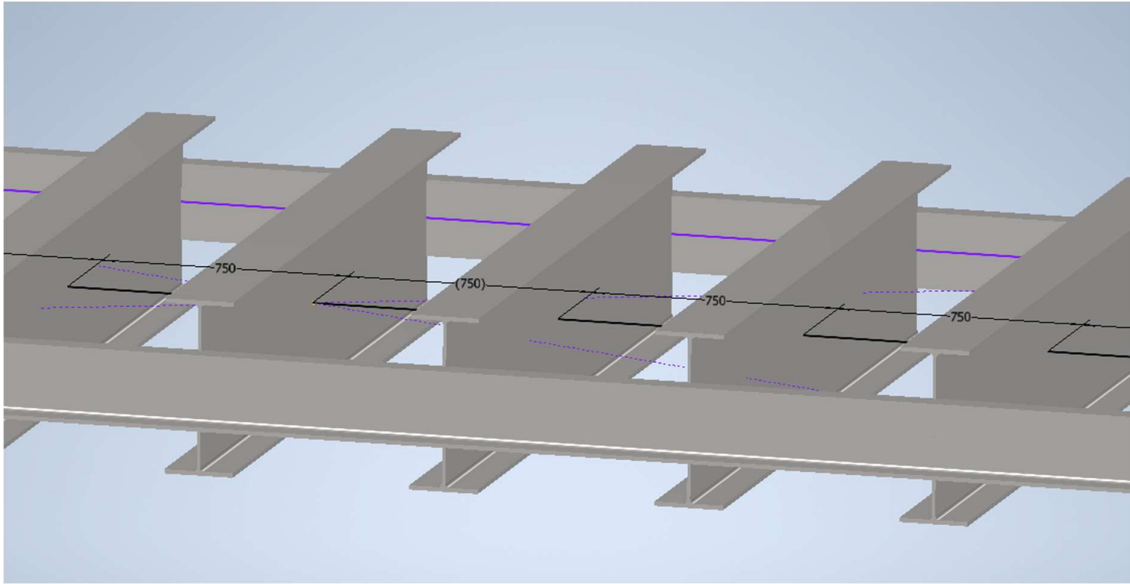


Figure 20 – Original PFC Longitudinal beam design. Created by document owner.

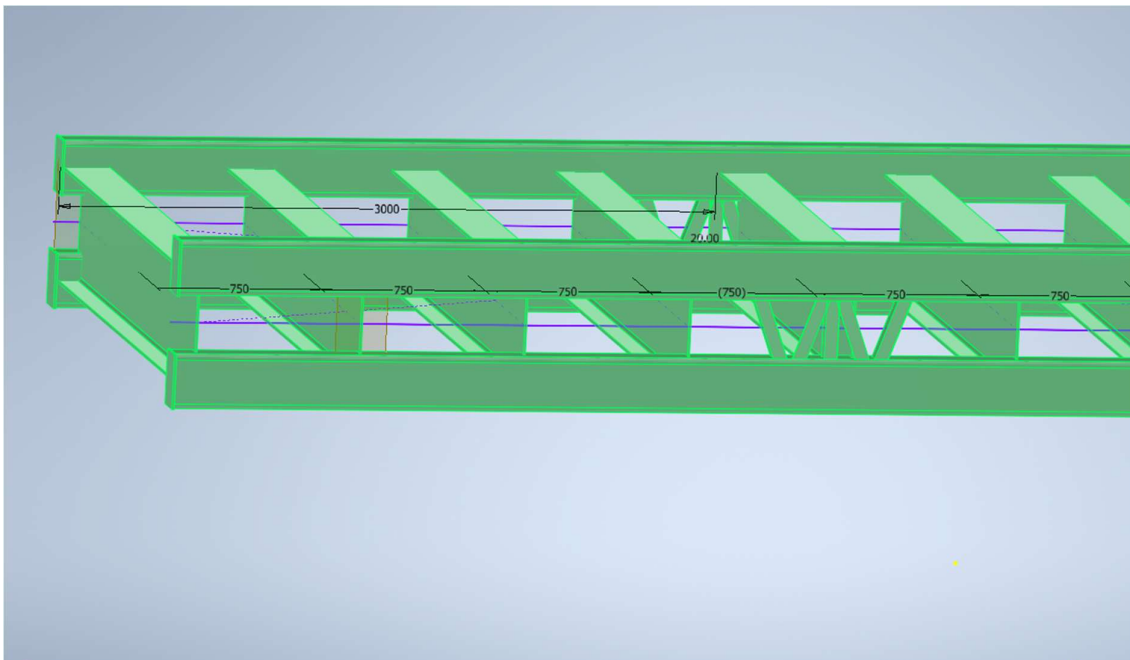


Figure 21 – Design iteration 2: Structural Arrangement. Reconfigured frame geometry incorporating dual longitudinal members and triangulated bracing to improve load distribution and global stiffness. Created by document owner.

To further reduce deflection, an intermediate bracing system was introduced across the span. A regular bracing pattern was selected to reinforce central regions of maximum deflection while maintaining robustness against variable loading conditions. While more advanced optimisations techniques (e.g. topology optimisation) could further refine the layout, a deterministic bracing arrangement was adopted to ensure reliability across a wide range of practical loading scenarios shown in Figure 22.



Figure 22 – Design iteration 2: Intermediate bracing system. Reconfigured bracing system on transverse members to increase stiffness and reduce global deflection behaviour. Created by document owner.

4.9.2 Design iteration 2 analysis

The revised structure was analysed under identical loading and boundary conditions. The results demonstrate a significant improvement in structural performance (shown in Figures 23 & 24):

- Maximum normal stresses reduce from approximately 880 MPa to 67 MPa, representing a ~12x reduction.
- Peak stresses remained localised around lifting points, which is expected due to load introduction effects.
- Maximum deflection reduced from ~218mm to ~3mm, representing a ~70x increase in stiffness.

This confirms that the redesign structure successfully establishes more efficient load paths for longitudinal members and significantly improves global rigidity, however noted on the stress contour map is still very low utilisation or load pathing to the transverse members indicating that these large heavy members are still underutilised and not significantly contributing to global frame stability.

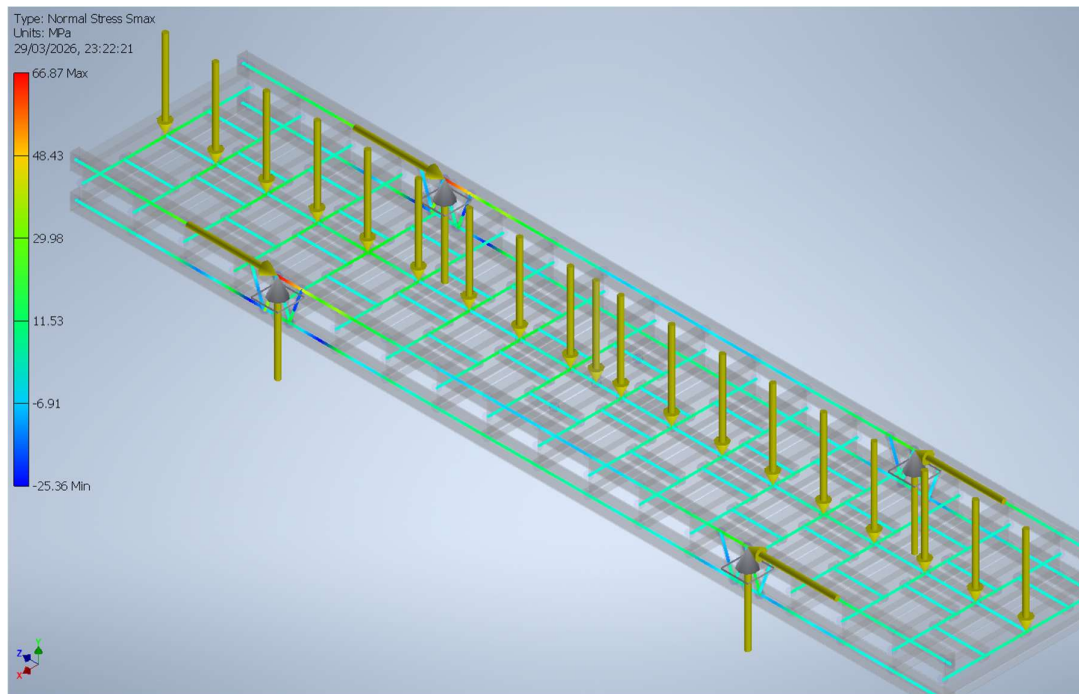


Figure 23 – Design iteration 2: Reduce stress distribution (~67 MPa max) demonstrating effective load redistribution and removal of overstressed longitudinal members. Created by document owner.

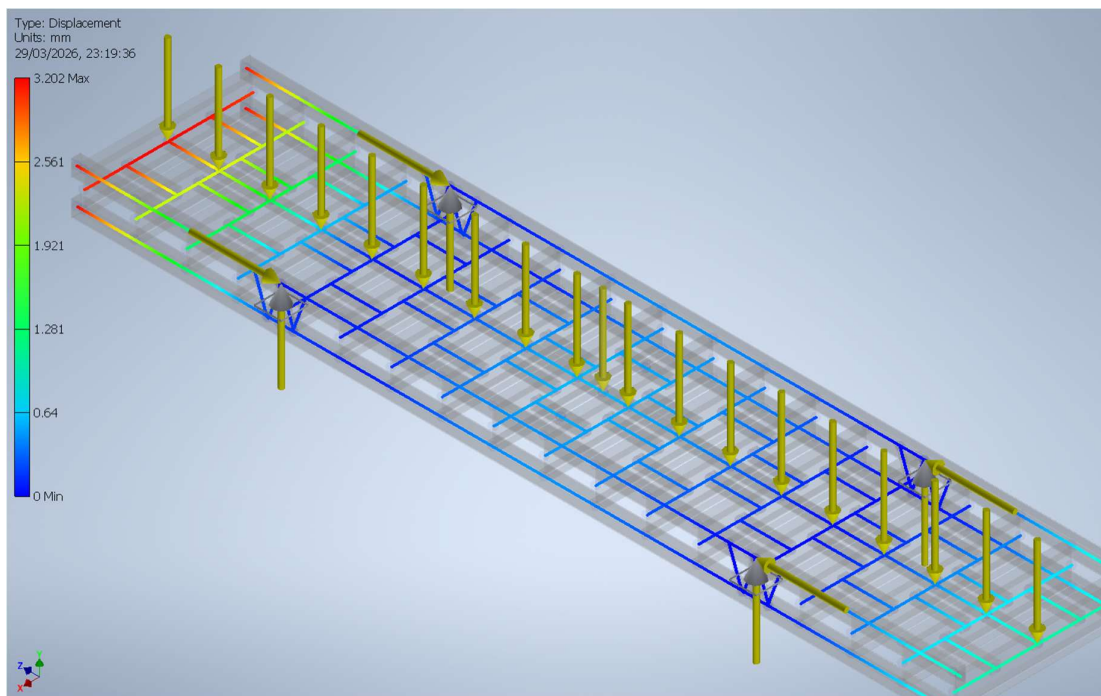


Figure 24 – Design iteration 2: Reduce Global displacement plot. Created by document owner.

To quantify the improvement to structural stiffness the below relationship was used:

Equation 4 – Relationship between applied force, displacement and structural stiffness based on linear elastic behaviour.

$$k = \frac{f}{\delta}$$

Where:

- f = Global applied load (N)
- δ = Maximum deflection (mm)

The results were then normalised by the self-weight to assess structural efficiency and shown in the below table 4.

Table 4 – Comparison of structural stiffness, self-weight and lifting efficiency between the initial and revised FTM design iterations.

Design	Stiffness (N/mm)	Mass (kg)	Stiff/Weight Ratio	Lift-to-self-weight Ratio
Initial Concept	6914.42	2156.030	0.327	20.4:1
Design iteration 2	471830.11	8335.38	56.60	5.27:1

While the redesign significantly improves structural performance, this is achieved at the cost of a substantial increase in mass. The structure now weighs approximately 8.3Te, resulting in a lift-to-self-weight ratio of approximately 5:1 when compared against the design payload of 44Te, compared to the target 20:1 defined in the design specification.

This indicates that, although structurally efficient in terms of stiffness, the design is materially inefficient, and further optimisation is required to reduce mass while maintaining adequate performance.

4.9.3 Design iteration 3 – Weight Optimisation

Following iteration 2, it was identified that although the structure achieved a substantial improvement in stiffness and stress reduction, this achieved at the expense of a significant increase in self-weight.

Therefore, the objective of the third iteration was to reduce structural mass while maintaining adequate strength and stiffness, ensuring compliance with Eurocode 3 ultimate and serviceability limit state requirements.

A key principle adopted in this stage was improvement of global member utilisation. In structural system, efficient designs are characterised by a relatively uniform stress distribution, where members contribute proportionally to load resistance (Allwood & Moynihan, 2014). Conversely, regions of low stress indicate excess material and opportunities for weight reduction. Therefore, the redesign aimed to:

- eliminate underutilised material,
- redistribute loads more effectively across the structure, and
- align member capacity more closely with demand.

To achieve this, a fundamental reconfiguration of the frame was undertaken.

Firstly, the number of large transverse primary beams was reduced and relocated to the perimeter of the structure, forming a stiffer outer load-carrying frame. This change reflects the observation from previous analysis that peak stresses consistently occurred at load introduction points, particularly near lifting locations. By strengthening the perimeter, the design better aligns structural capacity with these critical regions.

Secondly, the internal structure was redefined to act as a secondary load distribution system, replacing heavy primary members with lighter closed section member (box sections). The members provide sufficient local stiffness and improve torsional resistance while significantly reducing overall mass compared to open or heavily sized primary sections.

Additionally, a cross-bracing system was introduced to replace portions of the intermediate bracing used in iteration 1 and 2. The use of triangulated bracing enables loads to be transferred primarily through axial forces rather than bending, which is a more material-efficient load-carrying mechanism (Steel Construction Institute, 2010). This allows for reduced intermediate bracing while maintaining comparable structural stiffness.

The redesign a transition from a stiffness-dominated solution to a load-path-driven solution, where structural efficiency is achieved through geometry and force flow rather than increase material usage (these changes can be seen in stress contour plot figures 25 & 26).

4.9.4 Design Iteration 3 – Analysis

The analysis of iteration shows that the revised frame structurally viable at a global level, but with a change in behaviour. Maximum normal stress increased to approximately 199.7 MPa, compared with approximately 66.9 MPa in the previous iteration. Likewise, maximum displacement increased to approximately 16.84mm, compared with approximately 3.20mm previously. These changes indicate that the weight saving has been achieved by allowing a greater proportion of the structure to participate structurally rather than suppressing deformation through excessive member size.

The self-weight of iteration 3 is 5453.4 kg, representing a reduction of approximately 34.6% relative to the previous redesign and in fact. This improves the lift-to-self-weight ratio to approximately 8.07:1, which, although still below the 20:1 target, is a substantial improvement.

The results were then normalised by the self-weight to assess structural efficiency and shown below in table 5.

Table 5 – Comparison of structural stiffness, self-weight and lifting efficiency between the three FTM design iterations.

Design	Stiffness (N/mm)	Mass (kg)	Stiff/Weight Ratio	Lift-to-self-weight Ratio
Initial Concept	6914.42	2156.030	3.207	20.4:1
Design iteration 2	471830.11	8335.38	56.60	5.27:1
Design iteration 3	89714.96	5453.402	16.45	8.07:1

Overall, Iteration 3 demonstrates that the previous redesign was overly conservative, achieving structural performance primarily through increased material rather than efficient load transfer. By removing underutilised members and reconfiguring the frame around more direct, braced load paths, a substantial reduction in self-weight has been achieved while maintaining acceptable global structural behaviour.

This iteration represents a deliberate trade-off between stiffness and mass, where a controlled increase in stress and displacement is accepted in order to improve the lift-to-self-weight ratio. The resulting design moves closer to a balanced structural solution, in which strength, stiffness, and weight are considered concurrently rather than in isolation.

However, the results also indicate that achieving the target lift-to-self-weight ratio of 20:1 may not be feasible under the current modelling assumptions and material selection without peaking normal stresses. The use of conventional structural steel sections limits the achievable strength-to-weight efficiency, and further reductions in mass would likely require either higher-strength materials or more advanced structural optimisation. Both approaches introduce additional complexity and cost, which may conflict with the practical and economic constraints of the design brief.

Therefore, Iteration 3 can be considered a rational compromise solution within the defined constraints, while also highlighting the limitations of the current design space and the need to balance structural performance with material efficiency and constructability the results of the analysis can be seen below in figure 25 and 26.

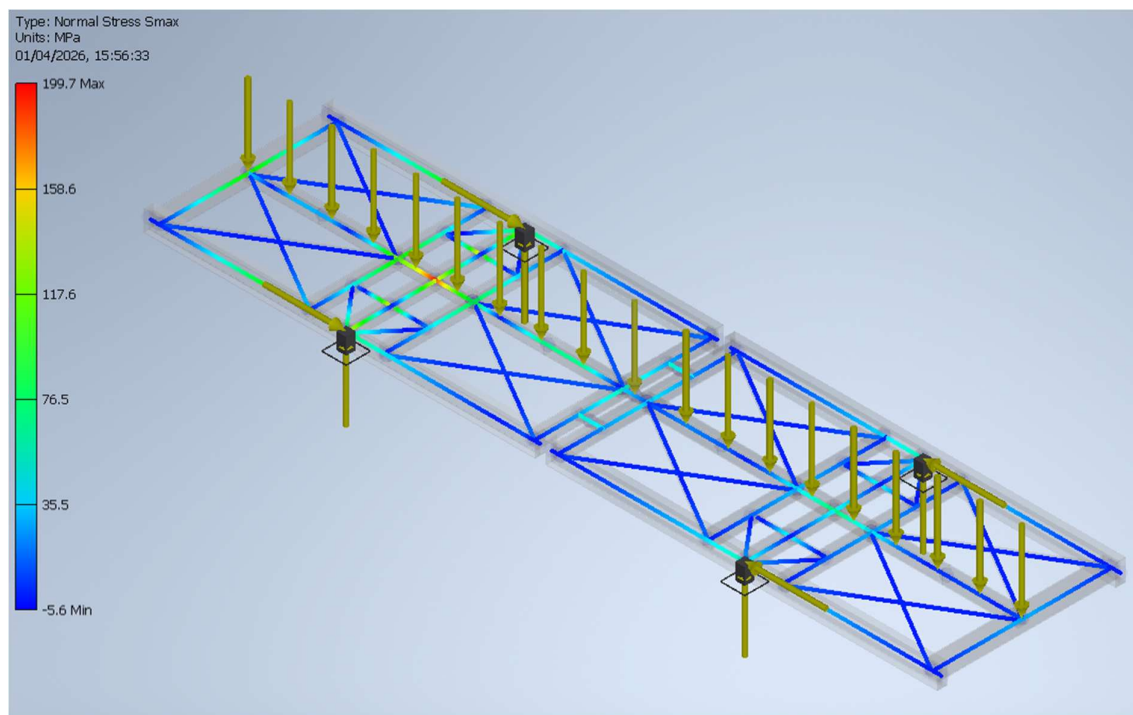


Figure 25 – Design iteration 3: normal stress distribution. Created by document owner.

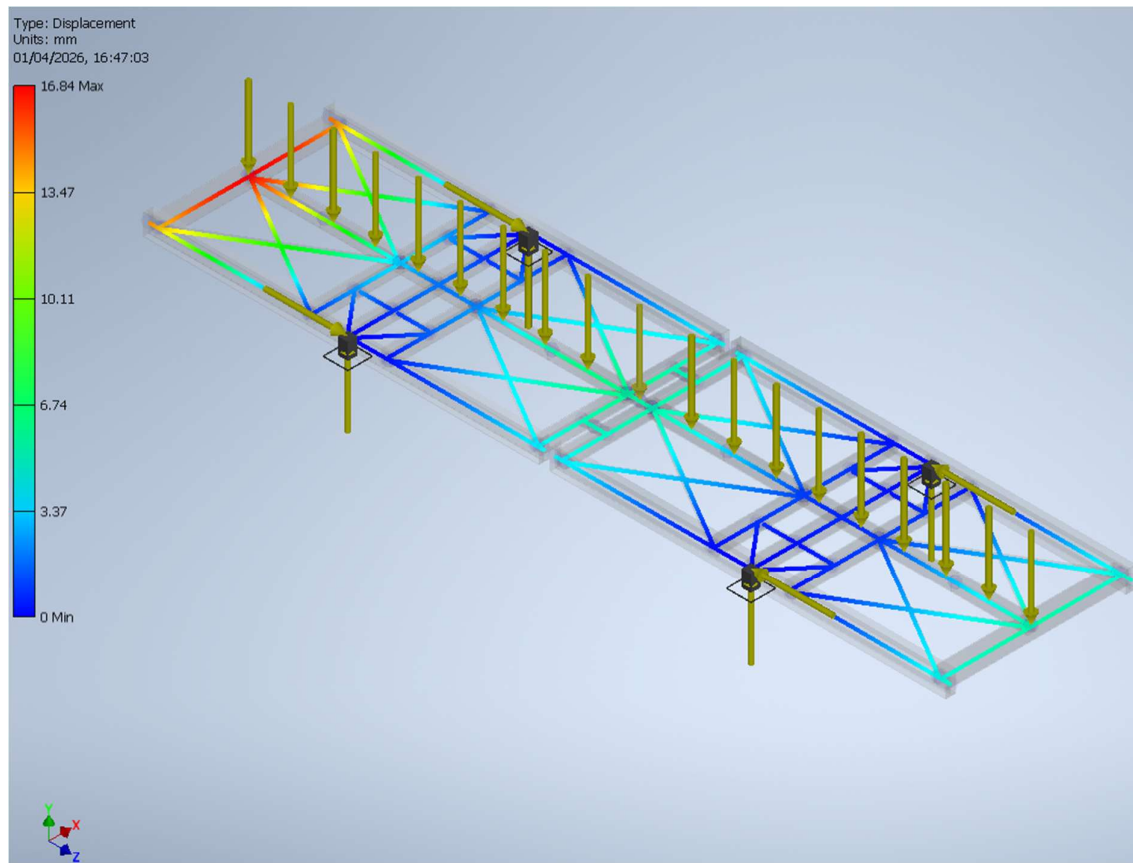


Figure 26 – Design iteration 3: global displacement. Created by document owner.

4.10 Transition to Eurocode 3 Member Verification

The finite element analyses presented in the preceding sections were used primarily as a global screening and comparative design tool to understand structural behaviour, identify inefficient load paths, and guide progressive refinement of the frame geometry. In this context, contour plots of normal stress and displacement were used to compare design iterations and to identify heavily loaded regions, underutilised members, and trends in global stiffness.

However, these plots do not in themselves constitute a code-compliant proof of structural adequacy. While useful for visual interpretation of force flow and relative demand, stress contours do not replace formal member verification in accordance with BS EN 1993-1-1. Eurocode 3 design requires members to be assessed using defined resistance models, stability considerations, and interaction relationships for the governing combinations of axial force, shear force, and bending moment (BSI Standards Publication, 2022; CSI, 2016).

Accordingly, the next stage of the assessment uses the principal member force results obtained from Iteration 3 as the basis for formal design verification. These internal actions are extracted from the global frame analysis and used to carry out member-level checks in accordance with Eurocode 3. In this way, the finite element model is used appropriately not as a standalone design check, but as a means of generating the internal force effects required for standards-based verification (Cook, et al., 2002).

To ensure a consistent and structured approach, the verification process follows the procedural workflow outlined in Figure 27, based on the requirements of BS EN 1993-1-1. This workflow defines the sequence of checks required to assess structural adequacy under combined loading conditions.

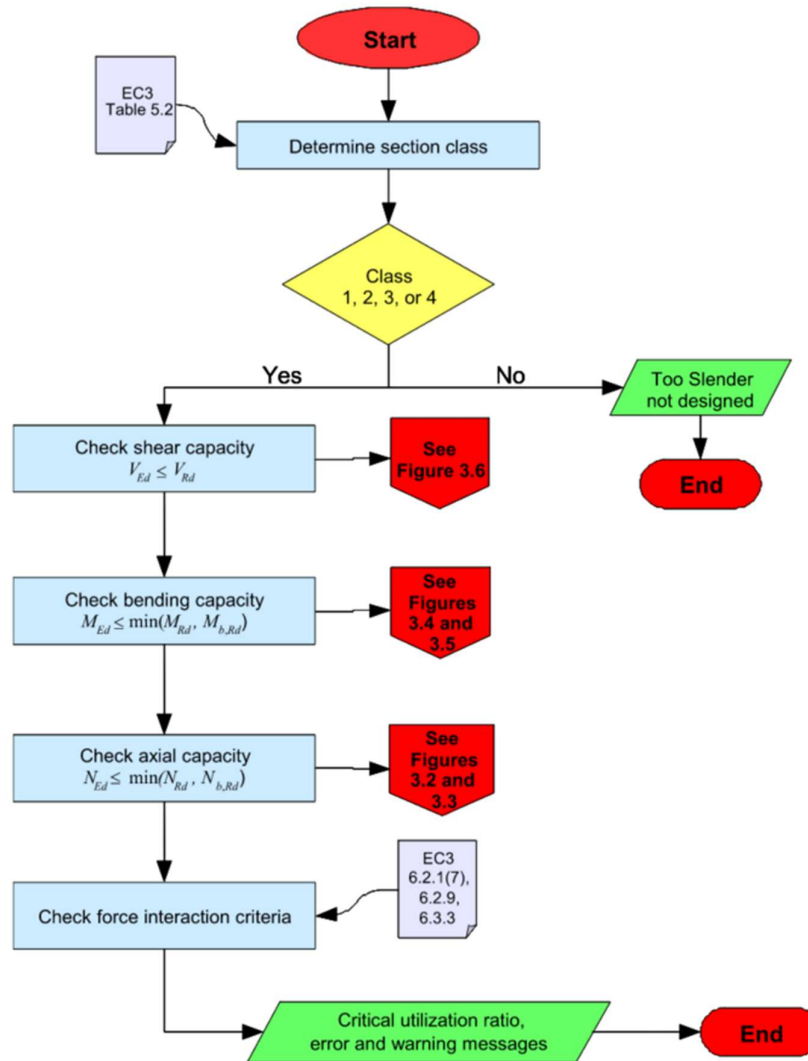


Figure 27 – Eurocode 3 member verification workflow adopted for structural assessment. Adapted from (CSI, 2016).

Global sway and second-order (P-Δ) effects have not been explicitly considered in the analysis. This assumption is considered appropriate for the concept-stage assessment due to the relatively high stiffness and low global slenderness of the frame, as indicated by the limited lateral deflections observed in the finite element analysis (BSI Standards Publication, 2022; Gardner & Nethercot, 2005).

The structure is intended for short-duration lifting and transport operations rather than permanent installation. Accordingly, serviceability considerations, particularly deflection and geometric stability during handling, govern the design. The frame does not exhibit significant sway behaviour under the applied loading, and second-order effects are therefore considered negligible in accordance with BS EN 1993-1-1.

The verification process begins with cross-section classification, which determines the applicable resistance models this is done in Appendix C. Class 1 to Class 3 sections are suitable for standard resistance checks, whereas Class 4 sections require effective section properties due to local buckling susceptibility.

Following classification, members are verified for:

- Shear resistance
- Bending resistance
- Deflection limits

Where members are subjected to combined loading, Eurocode 3 interaction criteria are applied to verify the combined effects of axial force and bending moment. This approach links the global finite element analysis to local member verification, ensuring analytical justification and Eurocode compliance. Then following the workflow shown in Figure 27 these calculations are then carried out in Appendices D to F using the results of iteration 3 members.

4.10.1 Verification Results and Utilisations

The beam-element output provides the total member-axis torsional moment in the form of $M_{x,max}$, but does not separately resolve the Saint-Venant and warping torsional components required for a full Eurocode torsion verification. Since for $T_{Ed} = T_{t,Ed} + T_{w,Ed}$ and for open UB sections the division between these components depends on member restraint, section properties, and loading distribution, $T_{t,Ed}$ cannot be derived directly from the reported total torsional moment alone. By contrast, for closed sections such as RHS members, warping effects are generally negligible and may reasonably be taken as $T_{w,Ed} \approx 0$ such that $T_{Ed} \approx T_{t,Ed}$ (Trahair, 1993). For the longitudinal UB member, the reported torsional moment is significant, with $M_{x,max} = 214kN/m$. Accordingly, although the member has been assessed for axial force, shear, bending, and lateral-torsional buckling, it has not been fully verified for torsional resistance within the scope of the present study. The structural adequacy of this member should therefore be regarded as provisional pending a dedicated torsion and warping assessment, as outlined in Section 5.2 (future works).

For the RHS members, beam-element results indicate negligible axial force and major/minor-axis bending moments. As closed hollow sections are not susceptible to lateral-torsional buckling in the same manner as open sections, verification is limited to cross-section classification and the relevant resistance checks (BSI Standards Publication, 2022). RHS 150x100x10 is governed primarily by shear, with bending and torsional effects negligible (Appendix E, Figure 34), while RHS 250x150x12.5 is governed by combined shear and torsion and is therefore verified for shear resistance and torsional shear stress (Appendix F, Figure 35).

Summary Check Table to Eurocode 3 BS EN 1993-1-1					
Section Size	Check Type	Design Action (Ed)	Resistance (Rd)	Utilisation (Ed/Rd)	Status
UB 356x171x67	Shear (V)	103.139kN	676.98kN	15.20%	Pass
	Bending (M)	211.42kNm	429.55kNm	49.21%	Pass
	LT Buckling	211.42kN	411.51kNm	51.37%	Pass
	Axial (N)	1.58 kN	~very large	~0.00	Not explicitly verified in the check
	Combined (N+M)	-	-	~0.51 (governed by bending)	Not explicitly verified in the check
RHS 150x100x10	Deflection (δ)	15mm	32.5mm	N/A	Pass
	Shear (V)	11.04kN	522.64kN	<1%	Pass
	Bending (M)	0.46kNm	76.68kNm	<1%	Pass
	LT Buckling	N/A	N/A	N/A	See section Appendix E section 12.4
	Axial (N)	0.676N	~very large	~0.00	Not explicitly verified in the check
	Combined (N+M)	-	-	~0.01 (governed by bending)	Not explicitly verified in the check
	Torsion Shear (T_V)	11.04kN	517.75kN	<1%	Pass
RHS 250x150x12.5	Deflection (δ)	5.81mm	1mm	N/A	Fail
	Shear (V)	114.9kN	1157.2kN	<1%	Pass
	Bending (M)	0.031kNm	262.64kNm	<1%	Pass
	LT Buckling	N/A	N/A	N/A	See section Appendix F section 13.5
	Axial (N)	9.7N	~very large	~0.00	Not explicitly verified in the check
	Combined (N+M)	-	-	~0.01 (governed by bending)	Not explicitly verified in the check
	Torsion Shear (T_V)	114.9kN	876.88kN	<1%	Pass
RHS 250x150x12.5	Deflection (δ)	16mm	6.3mm	N/A	Fail

Figure 28 - Summary of Eurocode 3-member verification results for the principal frame members, showing design actions, calculated resistances, utilisation ratios, and serviceability performance.

The summary presented in Figure 28 demonstrates that all members satisfy the relevant ultimate limit state (ULS) resistance checks for the actions extracted from the beam model. The UB member remains below 52% utilisation in both bending and lateral torsional buckling, indicating adequate strength and stability under the applied design loading.

For the RHS members, axial force and conventional flexural demand are negligible, while shear and torsional effects remain well within the available section resistance. It is important to note that the load case adopted for this analysis represents a conservative worst-case scenario, with concentrated point loads applied to maximise the response in the primary, long-span members. As a result, the secondary RHS members experience relatively low stress levels. This provides additional confidence that these members possess sufficient reserve capacity for alternative or undefined load cases where load redistribution may increase demand in secondary elements (BSI, 2005).

Although several members exhibit very low utilisation ratios, this does not necessarily indicate an inefficient design. The larger section sizes were intentionally selected to improve global stiffness and reduce both local member deflection and overall frame deformation during lifting and transport conditions. Given the variable and non-standardised nature of the intended loading arrangements, the structure has been conservatively designed to accommodate a wide range of potential loading scenarios and future operational requirements. Consequently, the low stress utilisation observed within a number of members reflects the robustness and adaptability of the design, rather than excessive overdesign (The American Society of Mechanical Engineers, 2017).

However, both RHS members fail to satisfy the adopted deflection criteria, with calculated displacements of 5.81 mm and 16 mm exceeding the allowable limits. This indicates that the governing design criterion for these members is serviceability (stiffness) rather than ultimate strength.

It should be noted that the deflection limits applied are based on Serviceability Limit State (SLS) criteria typically derived from structural building codes, where failure is defined in terms of loss of function such as excessive deflection affecting the operation of finishes, cladding systems, or alignment-sensitive elements (BSI, 2005).

The direct application of such criteria to a structural lifting frame may not be fully appropriate, as the functional requirements differ significantly from those of permanent building structures. For lifting operations, serviceability is more closely related to factors such as load stability, alignment during lifting, and control of deformation to prevent unintended load redistribution or rigging issues (The American Society of Mechanical Engineers, 2017; BSI, 2018).

Therefore, further assessment is required to determine whether the adopted deflection limits are representative of the actual performance requirements of the lifting frame. An evaluation is required for an acceptable methodology and appropriate serviceability criteria this is expended upon further in Section 5.3 (Future Work).

4.11 Welding, Manufacturability and Fatigue Loading

The FTM was developed in accordance with a fabrication-first philosophy, recognising that manufacturability is a critical design consideration within project-shop environments. Preference was given to standard structural sections, simple welded connections, and arrangements that minimise fit-up complexity and fabrication hours. The design therefore seeks to balance structural performance with practical workshop execution, reducing manufacturing risk while ensuring compatibility with existing LMEL fabrication processes and capabilities.

4.11.1 Weldability Material Selection

In the fabrication of lifting equipment, assessment of structural steel weldability is critical. Grades such as S275 and S355 are commonly selected because they provide a favourable balance between strength, ductility and weldability. Their relatively low carbon content reduces hardenability and limits the formation of brittle martensitic microstructures within the weld heat-affected zone during rapid thermal cycling. This reduces susceptibility to hydrogen-assisted cold cracking, particularly when combined with suitable welding consumables, controlled heat input, appropriate preheat where required, and good fabrication controls (Lancaster, 1999; Kou, 2003).

For lifting equipment, ductility and toughness are particularly important due to dynamic loading, impact loading and fatigue effects. S355 is therefore often preferred where higher strength is required while maintaining good weldability. Suitable sub-grades such as S355JR or S355J2 should be selected based on service temperature and toughness requirements. These grades provide verified Charpy V-notch impact properties, reducing the risk of brittle fracture under low-temperature or dynamic loading conditions (British Standards Institute, 2019).

To further reduce the risk of fabrication or in-service cracking, material certificates should be reviewed to confirm chemical composition and carbon equivalent value (CEV). It is recommended that the CE(IIW) value is maintained below approximately 0.45 where practical in line with established International Institute of Welding guidance (Lucas, et al., 2000).

Where this value is not directly provided on the material certificate, it may be calculated using the International Institute of Welding (IIW) carbon equivalent relationship shown in Equation 5 (IIW , 1988).

Equation 5 – International Institute of Welding (IIW) Carbon Equivalent (CE) equation used to assess the weldability and hydrogen cracking susceptibility of steels based on chemical composition.

$$CE(IIW) = \%C \frac{\%Mn}{6} + \frac{\%Cr + \%Mo + \%V}{5} + \frac{\%Ni + \%Cu}{15}$$

A low carbon equivalent indicates reduced hardenability and improved weldability. This may support the reduction or elimination of preheat requirements, subject to confirmation against the applicable welding standard, material thickness, joint restraint, consumable hydrogen classification, heat input and approved WPS requirements (BSI , 2001). Example material certificates are provided in Appendix G, with relevant WPS examples included in Appendix H, critically for the above reason these feature no-preheat requirements reducing fabrication complexity further.

For fabrication, arc welding processes such as Manual Metal Arc welding and Metal Active Gas welding are considered suitable due to their proven use in structural steel fabrication, adequate mechanical properties and practical deposition rates. Tungsten Inert Gas welding, while capable of producing high-quality welds, is not considered practical for the primary structural welds due to its lower deposition rate, reduced productivity and higher fabrication cost for the weld volumes required (Cary & Helzer, 2005).

4.11.1.1 Required Weld Volumes

For weld specification and joint design, a key consideration is the number of loading cycles the structure is expected to experience throughout its operational life. This assessment is necessary to determine whether full-strength groove welds are required, or whether fillet welds between overlapping members are sufficient. Section 4.12 further evaluates the anticipated duty cycle of the FTM and demonstrates that, although the structure will undergo repeated loading during lifting, transport and set-down operations, the overall fatigue demand is relatively low when compared to continuously cycled lifting appliances or crane structures (The American Society of Mechanical Engineers , 2017).

Despite the comparatively low fatigue classification, the frame will still be subjected to dynamic loading conditions associated with heavy transported loads, road-induced vibration, lifting impacts and localised load redistribution. Consequently, joint selection has been based not only on static strength requirements, but also on fatigue resistance, weld quality, inspect ability and practical manufacturability (Maddox, 1991).

For primary structural load paths and highly stressed connections, complete or partial joint penetration groove welds are preferred to ensure reliable force transfer and minimise stress concentrations (Salmon, et al., 2009). However, due to the geometry of Universal Beam (UB) sections, it is not always practical or economical to achieve full penetration welds at all secondary member intersections, particularly where Rectangular Hollow Sections (RHS) frame members overlap with the UB web region. The restricted access and thin web geometry make full penetration welding difficult to execute and inspect reliably (The Steel Construction Institute, 2014).

Accordingly, a hybrid jointing philosophy is proposed. Where secondary RHS members frame into primary UB members, partial joint penetration (PJP) groove welds combined with supplementary fillet

welds are recommended. The prepared RHS member end is bevelled to permit adequate weld penetration and root fusion while avoiding excessive heat input into the thinner UB web section (Malik, 2016). Where access permits and stresses are highest, complete joint penetration (CJP) welds may still be utilised, particularly at highly loaded lifting interfaces and primary bending connections.

This approach provides an appropriate balance between structural performance, fatigue resistance, fabrication practicality and manufacturing cost, while remaining consistent with common heavy structural fabrication practice (Lancaster, 1999; BSI, 2001; Kou, 2003).

A full set of fabrication GA and part assembly drawing should be developed to use the methodology developed here in this section this pointed to in future works section 5.4 (Future Work).

4.11.2 Inspection and Testing

Upon completion of fabrication, all welds shall undergo 100% visual inspection (VT) in accordance with BS EN 1090-2. The structure is recommended to be fabricated to Execution Class EXC3 due to its lifting function, transportable nature, and the potential consequences of structural failure during lifting or transport operations, consistent with the consequence classes and service categories defined within BS EN 1990 (BSI, 2005). EXC3 provides an enhanced level of fabrication control, inspection, traceability, and quality assurance appropriate for safety-critical load-handling structures (BSI, 2018).

In addition to VT, supplementary non-destructive testing (NDT) is recommended to verify weld integrity. A minimum of 10% ultrasonic testing (UT) is proposed for groove welds, including partial penetration welds where examination is practicable, with particular focus on primary load paths and lifting connections.

Magnetic particle inspection (MPI) is also recommended to detect surface-breaking defects, including hydrogen-assisted cracking. Inspection hold times should comply with the delayed inspection requirements of BS EN 1090-2 Table 23 (Figure 29).

Table 23 — Minimum hold times

		Hold time (hours) ^a	
If preheat is applied in accordance with Method A of EN 1011-2: 2001, Annex C			
Weld size (mm) ^b	Heat input Q (kJ/mm)	S275 – S460	Above S460
a or s ≤ 6	All	Cooling period only	24
6 < a or s ≤ 12	≤ 3	8	24
	> 3	16	40
a or s > 12	≤ 3	16	40
	> 3	24	48
If preheat is applied in accordance with Method B of EN 1011-2: 2001, Annex C			
Weld size (mm) ^b		S275 – S690	Above S690
a or s ≤ 20		Cooling period only	24
a or s > 20		24	48

^a The time between weld completion and commencement of NDT shall be stated in the NDT report. In the case of “cooling period only” this will last until the weld is cool enough for NDT to commence,

^b Size applies to the nominal throat *a* of a fillet weld or the nominal material thickness *s* of a full penetration weld. For individual partial penetration butt welds the governing criterion is the nominal weld depth *a*, but for pairs of partial penetration butt welds welded simultaneously it is the sum of the nominal weld throats *a*.

Figure 29 – Table 23 of BS EN 1090-2 minimum hold times prior to NDT. Adapted from BS EN 1090-2.

4.11.3 Maintenance

The primary long-term maintenance concerns associated with the FTM structure are fatigue-related cracking, local weld degradation and corrosion-induced deterioration arising from repeated lifting, transport and environmental exposure throughout the operational life of the frame.

To ensure continued structural integrity and safe operation, it is recommended that the frame is subjected to periodic inspection and recertification in accordance with the requirements of LOLER. As a minimum, a detailed visual inspection should be undertaken every 12 months by a competent third party to verify that no significant deformation, weld cracking, corrosion, mechanical damage or local overstressing has occurred during service (Health and Safety Executive , 1998).

Particular attention should be given to:

- lifting points and padeye regions,
- highly stressed welded connections,
- areas susceptible to water or debris retention,
- transport restraint locations,
- and interfaces between primary and secondary structural members.

In addition to routine visual examinations, it is recommended that periodic non-destructive testing (NDT) is undertaken at extended inspection intervals of approximately 2–3 years, depending on operational usage and duty cycle. Magnetic Particle Inspection (MPI) or Dye Penetrant Testing (PT) should be used to detect surface-breaking defects that may not be visible during standard visual inspections, particularly fatigue cracking initiated at weld toes, geometric discontinuities or highly stressed local details (Maddox, 1991).

This inspection philosophy provides a practical balance between operational safety, inspection cost and long-term structural reliability for a transportable lifting structure subjected to repeated dynamic loading conditions.

4.12 Padeye Design and Verification

The next stage of the design assessment focused on the verification of the lifting padeyes, which are required to safely transfer the applied lifting load from the frame into the rigging system during lifting operations.

The assessment was carried out in accordance with ASME BTH-1:2017 and ASME B30.20:2018. These standards were selected because they are specifically applicable to below the hook lifting devices, which precisely describes the function of the fabrication frame and its integrated lifting attachments. ASME BTH-1 provides the design and strength verification requirements for the below the hook devices, including the relevant failure modes under the applied design loads. ASME B30.20 complements this by defined the operational and safety requirements associated with the use of the below the hook lifting devices.

For structural and Mechanical lifting devices as per section 20-1.2.2 of B30.20-2018 to ASME BTH-1 Design Category B, this means that a Nominal Design Factor $N_d = 3.0$ for allowable stress equations. Additionally, ASME BTH-1 requires the selection of a service (fatigue lift criteria) based upon the below table (Figure 30).

Table 2-3-1 Service Class

Service Class	Load Cycles
0	0–20,000
1	20,001–100,000
2	100,001–500,000
3	500,001–2,000,000
4	Over 2,000,000

Figure 30 - ASME BTH-1:2017 service class Table definitions based on the expected number of load cycles experienced during the operational life of the lifting device. Adapted from ASME BTH-1:2017.

To understand what service class is representative for the use case of FTM it's important to properly define a "cycle" and an operational life for the FTM. In this use case a cycle could be broadly defined a:

1. Attach rigging
2. Lift the load/frame
3. Transport /reposition
4. Set down
5. Remove load

That = approximately 1 cycle.

ASME BTH-1 does not prescribe a fixed calendar-based design life for below the hook lifting devices. Instead, the standard characterises design life through cumulative load cycles and operational duty classification. Therefore, an additional reference was required to establish an indicative operational working life for the Fabrication Transport Module (FTM).

BS EN 1990 provides indicative design working life categories for structural systems. Table 2.1 of the standard identifies replaceable structural components, such as gantry girders and bearings, as having an indicative working life of 10-25 years. As the FTM is intended to operate as a repeatedly lifted structural device subjected to comparable operational loading conditions, this range was considered an appropriate benchmark for establishing the intended service life of the system.

To provide a conservative upper-bound assessment for fatigue exposure and cumulative operational demand, a design working life of 25 years was adopted for the FTM.

Table 2.1 - Indicative design working life

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures ⁽¹⁾
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures
(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.		

Figure 31 – Indicative design working life categories and associated service life expectations for structures in accordance with BS EN 1990. Adapted from BS EN 1990, Table 2.1.

Based on these assumptions, the approximate lifetime number of load cycles may be estimated using the following relationship:

Equation 6 – Relationship used to estimate cumulative lifetime lifting cycles based on operational frequency and design working life.

$$\text{Total Design Cycles} = \text{Lifts per day} \times \text{Days in use per year} \times \text{Operational Design life}$$

For the purpose of the assessment, a lifting frequency range of 1-10 lifts per day was considered. The upper value of 10 lifts per day provides a conservative allowance for periods of high utilisation, where the frame could potentially be repositioned approximately once per hour during a 10-hour working shift, given the size, weight and intended payload of the FTM, it is considered unlikely that the frame would be lifted more frequently than this under normal operational conditions.

An annual usage period of 285 days per year was adopted to represent a conservative working-year allowance, accounting for weekly rest periods, annual leave, shutdowns and periods where the frame would not be in active use. Combined with the adopted 25-year operational design life, the upper-bound lifetime cycle estimate is therefore:

$$10 \times 285 \times 25 = 71,250 \text{ cycles}$$

This estimated lifetime cycle count falls within the range for ASME BTH-1 Service Class 1, which covers 20,001 to 100,000 load cycles. The FTM has therefore been classified as Service Class 1. As detailed in Appendix I, this classification carries a comparatively moderate design factor relative to higher service classes, meaning that less severe strength reduction factors are applied in the subsequent resistance and capacity calculations. This is significant as it reflects the moderate operational demand placed on the FTM over its service life, and ensures that the design remains appropriately conservative without being unnecessarily penalised by reduction factors associated with higher-cycle fatigue classifications.

4.12.1 Padeye Design and Geometry

The lifting lug geometry was developed using an iterative strength-based sizing approach to balance rigging compatibility, structural resistance and fabrication practicality (The American Society of Mechanical Engineers, 2017). The initial geometry was selected based on shackle and pin dimensions,

minimum edge distances, available plate thicknesses and manufacturability considerations, while minimising eccentricity and local stress concentrations.

As defined in Section 4.8.1 and Appendices B and C, the governing lifting load occurs under a biased loading condition. To ensure safe lifting regardless of orientation, each padeye was designed for the worst-case lifting load of 44 Te.

To align with standard rigging hardware availability, the geometry was developed around a 55 Te shackle with a 74 mm hole diameter to suit a standard 70 mm pin, maintaining the approximate 6% clearance ratio recommended by DNV 2.7-1. Where smaller shackles are required, welded bosses may be introduced to maintain acceptable pin-to-hole clearance and bearing performance (DNV.GL, 2017).

An initial lug thickness of 55 mm was selected to provide sufficient bearing resistance and out-of-plane stiffness while remaining compatible with shackle jaw dimensions (American Institute of Steel Construction , 2022). Remaining geometric proportions were then developed relative to the hole diameter to provide adequate resistance against net section rupture, shear tear-out and bearing deformation, with final verification carried out in accordance with ASME BTH-1 failure mode checks (The American Society of Mechanical Engineers , 2017).

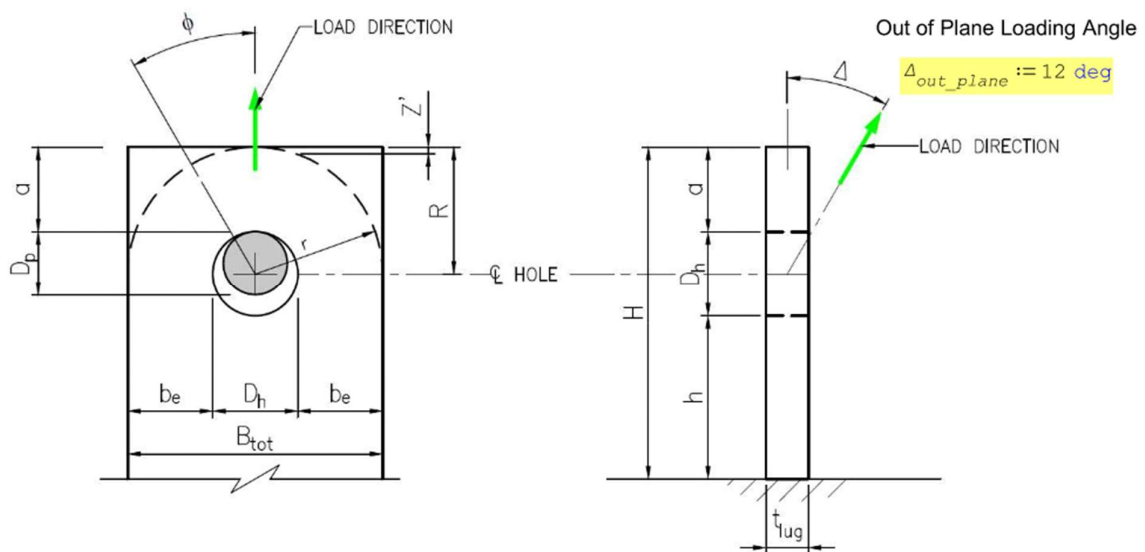


Figure 32 – Preliminary Lifting Lug Geometry. Adopted from the ASME BTH-1 static design assessment, showing the principal geometric parameters, load direction and nominal out-of-plane loading allowance. Adapted from Lifting Lug Calculation Methodology.

4.12.2 Padeye Analysis and Results

The preliminary lifting lug geometry shown in figure 32 was assessed against the static requirements of ASME BTH-1-2017 Design Category B using a nominal design factor of $N_d = 3.0$.

The assessment considered the principal failure mechanisms associated with padeye behaviour under lifting loads shown in Figure 33.

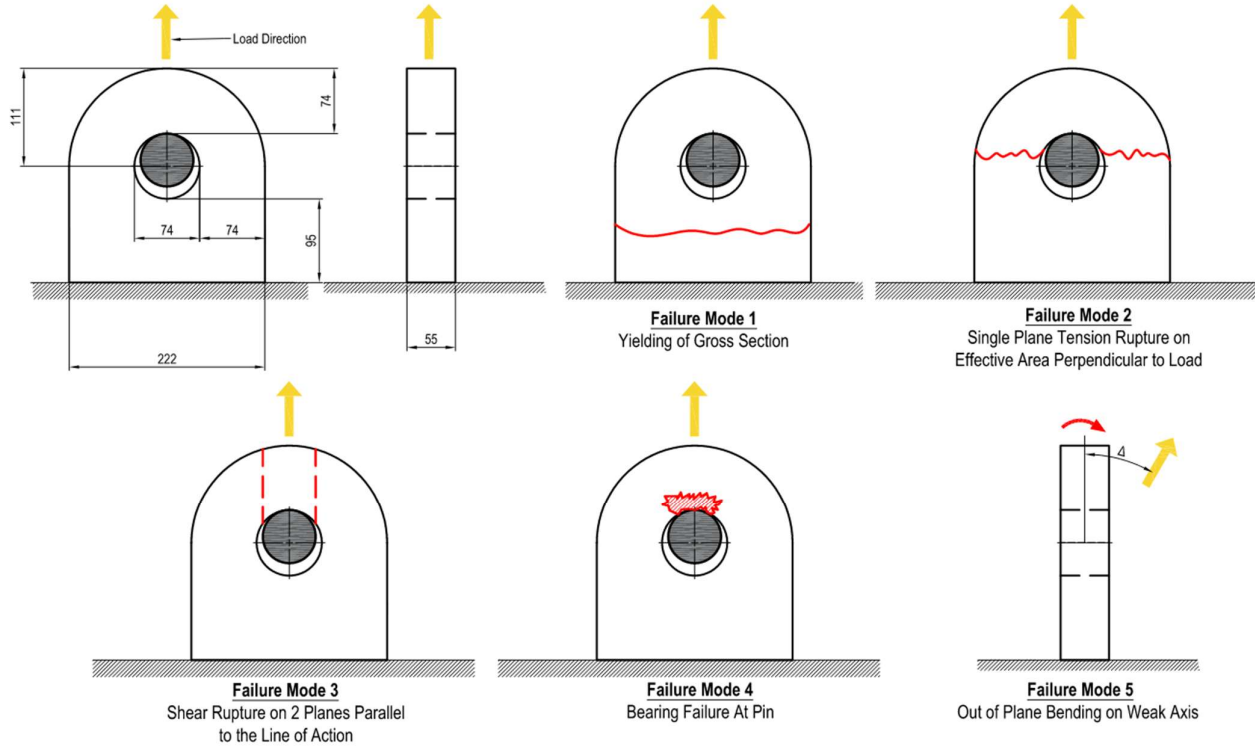


Figure 33 – Failure modes assessed for the lifting lug under axial loading, including gross section yielding, tensile rupture, shear rupture, bearing failure, out of plane bending. Created by document owner.

The assessment utilised S355 low-carbon fine grain structural steel due to its favourable balance of strength, toughness, weldability and ductility for welded lifting applications (British Standards Institute, 2019). The selected material satisfies the steel classifications referenced within ASME BTH-1 Appendix A Section A-4.5 and is well suited to heavily restrained welded lifting details where localised stress concentrations are expected.

The lifting lug was assessed against a design lifting load of as per appendix B (unfactored) of this report:

$$P_{design} = 431.49kN$$

Corresponding to the 44Te rated lifting capacity of the Fabrication Transport Module.

4.12.3 Interpretation of Results

The calculated allowable resistances are done in appendix I each assessed failure mode are summarised in below table 6.

Table 6 – Padeye Verification Summary Table. Created By Document Owner.

Padeye Check Verification to ASME BTH-1-2017 Summary Table				
Failure Mode	Design Action (Unfactored)	Design Resistance	Utilisation	Status
Yield of Gross Section	432kN	1204.38kN	35.80%	Pass
Single Plane Tension Rupture on Effective Area Perpendicular to Load	432kN	577.94kN	74.75%	Pass
Shear Rupture on 2 Planes Parallel to the Line of Action	432kN	310.56kN	139%	Fail
Bearing Failure At Pin	432kN	1025.49kN	42.13%	Pass
Out of Plane Bending on Weak Axis	432kN	1135.96kN	38.03%	Pass

The analysis demonstrates that the preliminary lug geometry possesses adequate resistance against gross yielding, local bearing, and out-of-plane bending actions. However, the governing failure mechanism was identified as shear rupture on the planes parallel to the line of action.

4.12.4 Engineering Interpretation

The governing shear rupture mechanism indicates that the effective shear area surrounding the pin connection is insufficient to safely transfer the required lifting load, this insight is gained from analysis of the equations used to determine resistance taken from appendix I and shown below in equations 7 to 9.

Equation 7 – Determination of the effective shear-plane reduction parameter z' for lifting lug shear rupture assessment in accordance with ASME B30 Equation C-2.

$$z_{prime} = r - \sqrt{r^2 - \left(\frac{D_{pin}}{2} \times \left(\sin(\Phi_{loading\ angle}) \right) \right)^2} = 0.37mm$$

Equation 8 – Calculation of the effective shear area surrounding the lifting pin based on lug geometry and shear-plane dimensions. Adapted from AISC 360-16.

$$A_{sf} = 2 \times \left(a - z_{prime} + \frac{D_{pin}}{2} \right) \times t_{lug} = 11949.3mm^2$$

Equation 9 – Calculation of allowable shear rupture resistance for the lifting lug based on effective shear area and material ultimate strength. Adapted from AISC 360-16.

$$P_{allow\ shear} = \frac{0.6 \times F_{u\ lug} \times A_{sf}}{\Omega_{shear}} = 310.56kN$$

The analysis provided a clear basis for refinement of the padeye geometry through several possible design modifications, including:

- Increased effective shear area,
- Increased lug thickness.
- Revised geometric proportions
- Or the introduction of local reinforcement such as cheek plates if required.

Of the available options, increasing the effective shear area by increasing the material above the lifting hole was selected as the preferred design iteration. Increasing the lug thickness was not considered efficient, as the existing 55mm plate thickness had already demonstrated adequate performance against the remaining assessed failure modes, including gross yielding, bearing and out of plane flexure. Similarly, the introduction of cheek plates would have increased fabrication complexity through additional cutting, fit-up and welding operations.

Increasing the height of the lug was therefore considered the least invasive and most practical design modification, as it maintained a relatively simple fabrication methodology. The revised geometry could still be profile cut directly from plate while retaining only a single primary welded interface between the lug and the main frame structure.

By rearranging the governing shear rupture equations, the required increase in effective shear area for the 44Te design lifting load could be determined shown in Equations 10 and 11.

Equation 10 – Rearranged shear rupture resistance equation used to determine the required effective shear area for the lifting lug under the applied design load.

$$A_{sf} = \frac{P_{Allow\ shear} \times \Omega_{shea}}{0.6 \times F_{u\ lug}} = 22005.6mm^2$$

Substituting this required shear area into the geometric relationship for shear plane gives:

Equation 11 – Rearranged geometric relationship used to determine the required material height above the pin to achieve the necessary effective shear area.

$$a = \left(\frac{A_{sf} + 2 \times (Z_{prime}) - 2 \times \left(\frac{D_{pin}}{2} \right)}{t_{lug}} \right) \div 2 = 199.42mm$$

This demonstrated that the material above the pin required and increase of approximately ~125mm relative to the original design iteration in order to achieve adequate resistance against shear plane rupture.

Although this represents a significant increase in lug height, corresponding to approximately 2.7 times the original material above the pin, the modification remained practical from both a structural and fabrication perspective. The revised geometry preserved the simplicity of the original concept while substantially increasing the available shear transfer area without introducing additional welded reinforcement components.

5 Recommendations for Future Work

5.1 Wind Loading

Wind loading was not explicitly included within the baseline frame analysis because the FTM is intended for use across multiple locations and no single site-specific wind velocity could be defined at concept stage. However, as wind may act concurrently with gravity loading during outdoor handling and temporary storage, future design verification should include a global horizontal wind load case in accordance with BS EN 1993-1-1. These should then be used to inform, operational restrictions such as maximum allowable wind speeds during lifting and handling.

5.2 Torsional Assessment

The high torsional moments observed in certain longitudinal UB members (up to 214 kN·m) are believed to be influenced by the simplified method used to introduce lifting forces into the model. As the vertical and horizontal sling force components were applied at midspan locations, further investigation is required to determine whether a load application method more representative of the actual padeye arrangement would reduce these torsional effects and alter the governing member actions. Should significant torsional moments remain following refinement of the loading arrangement, a methodology should be developed to separate the total torsional moment into its Saint-Venant and warping components to enable a complete Eurocode 3 torsional resistance assessment.

5.3 Deflection Criterion

As discussed in Section 4.10, several secondary RHS members did not satisfy the adopted deflection criterion. However, conventional building-based serviceability limit state (SLS) limits may not be directly applicable to transportable lifting structures, where serviceability is governed more by load stability, geometric control and restraint behaviour than architectural performance.

Future work should therefore establish an application-specific deflection criterion based on operational performance during lifting and transport activities. This should consider the relationship between structural deflection, payload rotation, frictional resistance, restraint effectiveness and eccentric load positioning. Excessive member deflection may induce local rotations and horizontal force components capable of reducing payload stability where frictional resistance is exceeded.

Further work should also include:

- Load-securing calculations in accordance with BS EN 12195-1.
- Verification of tie-down and lashing point capacities.
- Assessment of braking, cornering and uplift load cases.
- Wider investigation of eccentric and asymmetric payload arrangements.

Collectively these assessments would improve confidence in the operational stability, restraint effectiveness and long-term serviceability performance of the FTM and satisfy the requirements missed out in current report shown in PDS table 2 in section 4.2 point 1.4 and 1.5.

5.4 Fabrication Drawings

The welding methodology and inspection philosophy developed in Sections 4.11.2 & 4.11.3 should be expanded into a complete fabrication package suitable for workshop manufacture. This would include detailed fabrication drawings identifying weld symbols, joint preparations, fit-up requirements, inspection locations and anticipated weld volumes.

Further work should also assess weld sequencing, distortion control methodology, temporary fabrication support and inspection accessibility for ultrasonic and magnetic particle testing. This would allow the concept design to progress toward a fully manufacturable and code-complaint lifting structure.

5.5 Cost to Implement, Operating Costs and ROI

The present study focused primarily on technical feasibility and structural performance. However, further work is required to assess implementation cost, operational expenditure and long-term return on investment (ROI).

Based on Appendix J, the estimated raw material cost of a single FTM is approximately £11,886 with a predicted self-weight of approximately 5.4 tonnes. Using an indicative fabrication allowance of 40 man-hours per tonne and an internal fabrication rate of approximately £40/hour, the estimated fabrication cost is approximately £8,725 per frame. This gives an initial estimated manufacturing cost of approximately £20,000 per unit excluding coating and commercial overheads.

These values should be regarded as preliminary order-of-magnitude estimates only.

Future work should therefore focus on:

- Fleet sizing and utilisation rates.
- Yard storage and handling requirements.
- Transport logistics and return-load efficiency.
- Lifecycle maintenance and inspection costs.
- Whole-life operational cost modelling.

Phased implementation strategies should also be considered to allow operational benefits and utilisation rates to be validated progressively before large-scale deployment.

Particular consideration should be given to the potential use of return-load transport arrangements, whereby empty FTMs could be returned to the fabrication facility using existing inbound logistics movements. This may significantly reduce transport costs and improve utilisation efficiency across the operational lifecycle of the system.

Consideration must also be given to the engineering assurance and certification requirements associated with operational deployment of the design. At present, neither LMEL nor the document owner holds professional Liability Insurance (Error & Omissions Insurance) suitable for formally certifying or signing off the structural design for operational use.

5.6 Commercial Impact

Although this study primarily focused on engineering and structural design, the findings suggest broader commercial applications within project-based fabrication environments. The proposed FTM concept demonstrates how standardised transport and handling interfaces may improve workflow stability, reduce unnecessary handling and support more predictable fabrication sequencing.

Future work could investigate integration of fabrication workflow data with commercial performance metrics including:

- Yard utilisation and loading management.
- Labour productivity tracking.
- Cash-flow forecasting.
- Margin protection and commercial risk analysis.

Poor fabrication control can propagate directly into commercial consequences including reduce labour efficiency, subcontracting requirements, delayed progress, increased WIP and extension-of-time claims. Future development could therefore incorporate production-flow metrics and earned-value methodologies to better quantify the relationship between fabrication instability and commercial performance (Love, et al., 1999).

Such a framework could support improved fabrication prioritisation, forecasting accuracy, logistics planning and investment justification within project-shop fabrication environments.

5.7 Protective Coatings and Future Development

Due to the overall size and welded configuration of the structure, hot-dip galvanising of the complete ass is not considered practical within currently available UK galvanising facilities. Sectionalising the frame for galvanising would introduce additional fabrication complexity and post-treatment rework requirements.

Future work should therefore properly assess suitability for hot-dip galvanising / sectionalisation or focus on development of suitable protective coating specification, including surface preparation, coating selection, inspection requirements, repair methodology and long-term durability under transport and outdoor exposure conditions.

High-visibility coating systems may additionally improve visual management, inspection visibility and asset identification throughout the operational life of the frame.

6 Conclusion

This study investigated the design and structural assessment of a Fabrication Transport Module (FTM) intended to improve manufacturing flow within Ledwood Mechanical Engineering's project-shop fabrication environment. The research was motivated by operational challenges including elevated work-in-progress (WIP), unstable sequencing, handling inefficiencies and reactive expediting commonly associated with engineer-to-order heavy fabrication systems.

The literature review demonstrated that many of these issues are structural characteristics of project-shop manufacturing environments rather than purely managerial shortcomings. Existing research suggests that fully flow-based manufacturing systems are often impractical within highly variable fabrication environments. Instead, hybrid project-flow approaches seek to improve operational stability through modularisation, controlled interfaces and selective implementation of pull-based principles.

In response, a Product Design Specification (PDS) was developed to define the functional, structural and operational requirements of the proposed FTM system. A structural steel transport frame was subsequently design and assessed using finite element analysis, Eurocode 3 member verification and ASME lifting-device assessment methodology.

The final design demonstrated adequate global structural performance under the representative loading conditions considered. Primary members satisfied the adopted Eurocode strength requirements, while detailed padeye assessment identified shear rupture as the governing local failure mechanism and informed subsequent geometric refinement.

The study also highlighted the importance of manufacturability, weldability, inspection philosophy and maintenance considerations within heavy engineering fabrication environments. Material selection, welding methodology and inspection requirements were therefore incorporated into the overall design philosophy to ensure the concept remained practically manufacturable while maintaining appropriate structural integrity and quality assurance standards.

A significant outcome of the study was the recognition that conventional building-based serviceability criteria may not be directly applicable to transportable lifting structures. Instead, operational stability, restraint effectiveness and load migration behaviour may provide a more representative basis for future deflection criteria development.

Although the study demonstrated the technical feasibility of the proposed FTM concept, the work was limited to conceptual and preliminary structural verification. Detailed fabrication design, prototype manufacture, physical testing, transport restraint verification and advanced fatigue assessment remain areas for future development.

Overall, the research demonstrates that the proposed FTM represents a technically viable and operationally relevant system-level intervention for improve manufacturing flow within project-shop fabrication environments. Rather than eliminating variability entirely, the concept provides a mechanism for containing and managing variability through modular handling, controlled interfaces and improve logistical stability.

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8 Appendix A – Estimated Labour-Hour Impact of Increased Weld Repair Rates.

To illustrate the scale of inefficiency introduced by a rise in weld defects, consider a conservative set of assumptions aligned with typical LMEL pipe-spool fabrication:

8.1 Assumptions (realistic industrial values)

- Average welds per 6-inch spool: 8–12 (use 10 for convenience)
- Typical man-hours per weld (fit-up + weld + visual): 4–6 hours
 - Use 5 hours as an average baseline
- Typical repair time for a defective weld (grind-out + reweld + NDT): 6–10 hours
 - Use 8 hours average
- Baseline repair rate (LMEL normal performance): <1%
- Observed peak repair rate per some individuals under job-shop pressure: ≈10%
- Global repair rate on the project: ≈5.5%

8.2 Calculation — labour-hour impact per 100 welds & project-wide **Baseline scenario (<1% repair rate)**

- Defective welds: 1 weld per 100
- Repair labour: $1 \times 8 \text{ hrs} = 8 \text{ hours}$

Job-shop pressure scenario (~5.5% repair rate)

- Defective welds: 5.5 welds per 100
- Repair labour: $5.5 \times 8 \text{ hrs} = 44 \text{ hours}$

Potential additional labour due to increased defects

- $44 - 8 \text{ hrs} = 36$ additional labour hours per 100 welds

This represents a 450% increase in repair workload relative to LMEL's normal performance level.

Global Repair Rate on the project

- Defective welds on the project: 437 welds
- Repair labour: $437 \times 8 \text{ hrs} = 3496 \text{ labour hours}$

8.3 Cost Impact Estimate

- Estimated labour cost: £18/hr
- Total cost of repair effort:

$$3,496 \text{ hrs} \times £18/\text{hr} = £62,928$$

This figure reflects direct labour only. It excludes indirect costs such as:

- schedule delays
- additional NDT mobilisation
- overtime premiums
- extended supervision
- lost opportunity cost (diverted welders)
- disruption to installation sequences
- overhead absorption loss

As such, £62.9k is a conservative minimum estimate of the cost impact associated with elevated weld defects driven by job-shop inefficiencies.

9 Appendix B – Design Load for UB

Total design load is 44Te, reaction force will be 44Te divided by each beam and times by 9.81 gravity to turn into Newtons.

$$F_k = 44000 \times 9.81 = 431.64kN$$

Because these are variable actions partial safety factor of 1.5 is used. Therefore, design load for the UB is:

$$Fd = \gamma_F \times F_k = 1.5 \times 431.64 = 647.460kN$$

10 Appendix C – Section Classification

In design iteration 3 the following sections were used:

- UB 356x171x67 (Universal Beam)
- L60x60x10 (Equal Angle)
- RHS150x100x10 (Rectangular Hollow Section)
- RHS250x150x12.5 (Rectangular Hollow Section)

All steel is set as being S355 due it being one of the most common structural grades available on the market.

10.1 Determine ε

For S355 Steel and $t_f \leq 40mm$, f_y (yield strength) = $355N/mm^2$

(taken from table 3.1 of EN 1993-1-1 2022)

$$\varepsilon = \sqrt{\frac{235}{f_y}} = 0.81 \text{ Cl 5.2.5.2}$$

10.2 Define Stress ratio

The stress ratio is determined using the maximum and minimum normal stresses extracted from the frame analysis. Results are taken from the left-hand side of the frame, where peak stress conditions were observed.

For each member type, the governing loading condition is defined using the worst-case combination of axial force and bending moment derived from the analysis. These values are then applied conservatively to all members of the same section type for the purposes of classification and resistance checks.

Where multiple load effects act simultaneously, their combined influence is considered in accordance with Eurocode 3 requirements. However, where the worst-case stress condition arises from a dominant single action rather than a combined case, this condition is adopted directly, as it represents the most onerous demand on the section. This approach ensures that section classification reflects the most critical stress distribution (pure compression, moderate bending, or high bending) without unconservative downrating from less severe combined cases.

CASE	α_c	MEANING
PURE COMPRESSION	1.0	Whole flange in compression
MODERATE BENDING	~0.5–0.8	Partial compression
HIGH BENDING (NEAR NEUTRAL AXIS)	small (<0.5)	Only a small part is compressed

10.2.1.1 UB 356x171x67 (Universal Beam)

$$\alpha_c = \frac{\sigma_{min}}{\sigma_{max}} = \frac{198.839}{199.681} = 0.996$$

(outstand flange largely in compression)

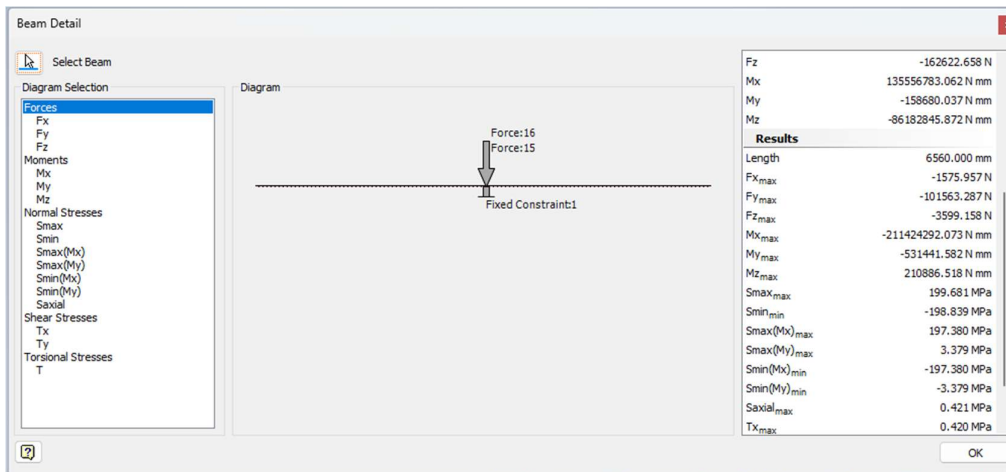


Figure 34 – Beam force and stress resultants for UB 356x171x67 (Universal Beam) worst case (Frame Iteration 3) created by document owner.

10.2.1.2 RHS150x100x10 (Rectangular Hollow Section)

$$\alpha_c = \frac{\sigma_{min}}{\sigma_{max}} = \frac{28.223}{28.815} = 0.979$$

(outstand flange largely in compression)



Figure 35 – Beam force and stress resultants for RHS150x100x10 (Rectangular Hollow Section) worst case (Frame Iteration 3) created by document owner

10.2.1.3 RHS250x150x12.5 (Rectangular Hollow Section)

$$\alpha_c = \frac{\sigma_{min}}{\sigma_{max}} = \frac{192.371}{193.909} = 0.992$$

(outstand flange largely in compression)

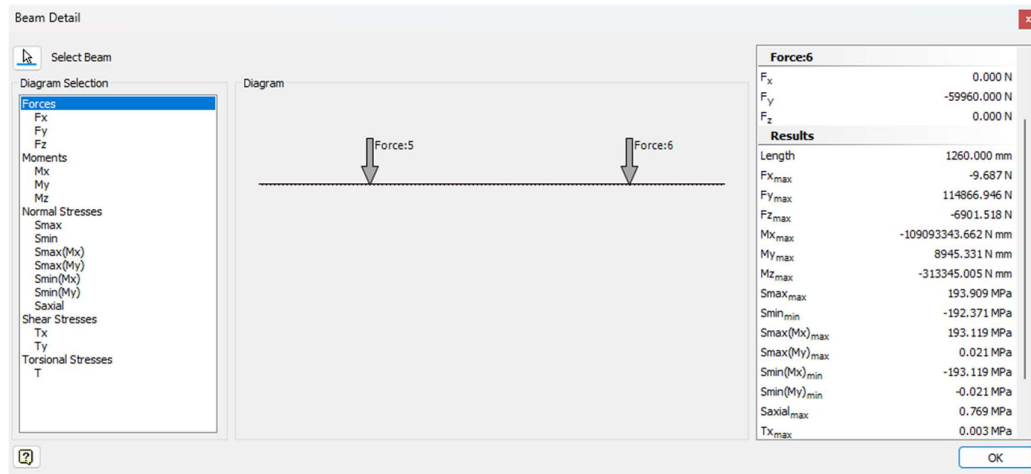


Figure 36 – Beam force and stress resultants for RHS250x150x12.5 (Rectangular Hollow Section) worst case (Frame Iteration 3) created by document owner.

10.3 Classification of Outstand Flanges

- UB 356x171x67 (Universal Beam) $c_f / t_f = 4.58$ Class 1
- RHS150x100x10 (Rectangular Hollow Section) $c_f / t_f = 7.00$ Class 1
- RHS250x150x12.5 (Rectangular Hollow Section) $c_f / t_f = 7.56$ Class 2

$$\text{Class 1} = c_f / t_f \leq 9\epsilon = 7.29$$

$$\text{Class 2} = c_f / t_f \leq 10\epsilon = 8.1$$

Values taken from Steel Construction Institute 'Blue Book'.

- L60x60x10 (Equal Angle) = N/A (Member in tension not acting like beam no considerations for local buckling required).

Table 7.4 — Maximum width-to-thickness ratios for compression parts of outstand flanges

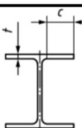
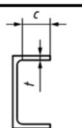
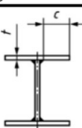
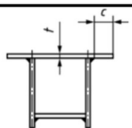
Outstand flanges			
			
Rolled sections		Welded sections	
	Part in pure compression	Part subject to bending and axial force	
Stress distribution in parts (compression positive)			
Class 1	$c/t \leq 9 \epsilon$	$c/t \leq \frac{9\epsilon}{\alpha_c}$	$c/t \leq \frac{9\epsilon}{\alpha_c \sqrt{\alpha_c}}$
Class 2	$c/t \leq 10 \epsilon$	$c/t \leq \frac{10\epsilon}{\alpha_c}$	$c/t \leq \frac{10\epsilon}{\alpha_c \sqrt{\alpha_c}}$
Stress distribution in parts (compression positive)			
Class 3	$c/t \leq 14 \epsilon$	$c/t \leq 21 \epsilon \sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

Figure 37 – Width-to-thickness limits for outstanding flanges (Eurocode 3 / BS EN 1993-1-1 2022, Table 7.4)

10.4 Classification of elements (Web)

Ratios for Local Flange Buckling

- UB 356x171x67 (Universal Beam) $c_w / t_w = 34.2$
- RHS150x100x10 (Rectangular Hollow Section) $c_w / t_w = 22.0$
- RHS250x150x12.5 (Rectangular Hollow Section) $c_w / t_w = 17.0$

$$\text{Class 1} = \frac{c_w}{t_w} \leq 72\epsilon = 58.32$$

All Section Webs are Class 1

Values taken from Steel Construction Institute 'Blue Book'.

- L60x60x10 (Equal Angle) = N/A (Member in tension not acting like beam no considerations for local buckling required) as per (Owens & Davison, 2012) reaccommodation's for X-Brace members.

10.5 Classification of Section

Table 7 – Eurocode 3 cross-section classification results for the principal FTM structural members.

Section Size	Flange Classification	Web Classification	Overall Section Classification
UB 356x171x67 (Universal Beam)	1	1	1
RHS150x100x10 (Rectangular Hollow Section)	1	1	1
RHS250x150x12.5 (Rectangular Hollow Section)	1	2	2

11 Appendix D – UB 356x171x67 Universal Beam Design Checks

11.1 Shear Resistance (EN 1993-1-1:2022 Section 8.2.6)

Height of web $h_w = 311.6mm$ values from SCI 'Blue Book'

Web thickness $t_w = 9.1mm$ values from SCI 'Blue Book'

Shear Area for I and H Section $A_v = 3303 mm^2$ cl.8.2.6.3

Design Shear Resistance $V_{pl,Rd} = \frac{A_v \left(\frac{f_y}{\sqrt{3}} \right)}{\gamma_{M0}} = 676.98kN$ eq (8.23)

11.2 Shear Buckling (EN 1993-1-5:2006 Cl.5)

Shear buckling resistance for web is verified according to Section 5 of EN 1993-1-5:

$$\frac{h_w}{t_w} > 72 \frac{\varepsilon}{\eta}$$

Web ratio for Shear Buckling $\frac{h_w}{t_w} = 34.24$

Factor for Shear Area $\eta = 1.0$ Cl.5.1 (2) / BS-EN NA

Shear Buckling Limit $72 \frac{\varepsilon}{\eta} = 56.32$

Shear Buckling Limit not hit therefore shear buckling can be ignored.

11.3 Bending Resistance (EN1993-1-1: 2022 Section 8.2.8)

Design Shear Action $V_{Ed} = 101.56kN$

Shear Resistance Verification Check $V_{Ed} \leq \eta_V V_{pl,Rd}$ eq (8.35)

$$\eta_V = 0.5 \text{ eq (8.35 NOTE)}$$

Shear Plastic Resistance $\eta_V V_{pl,Rd} = 338.49kN$

Therefore, shear force effect on bending moment resistance can be ignored

Class 1 section, therefore use plastic modulus $W_{pl} = 1\,210\,000mm^3$ values from SCI 'Blue Book'

Design bending resistance $M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = 429.55kN/m$ eq (8.20)

11.4 Design Buckling Resistance (EN1993-1-1: 2022 Section 8.3.1.1)

Young's Modulus of Elasticity $E = 210\,000 N/mm^2$ Cl 5.2.5 (1)

Poisson's Ratio $\nu = 0.30$ Cl 5.2.5 (1)

Shear Modulus of Elasticity $G_s = \frac{E}{2(1+\nu)} \approx 81\,000 N/mm^2$ Cl 5.2.5 (1)

Buckling Length $L_{cr} = 6560mm$ Taken from Figure 28

Characteristic value of the bending moment resistance $M_{y,Rk} = W_y f_y = 429.55 \text{ kN/m}$ eq (8.11)

Non-dimensional slenderness ratio $\bar{\lambda}_{LT} = \sqrt{\frac{M_{y,Rk}}{M_{cr}}} = 0.657$ eq (8.80)

Depth to width ratio for buckling curve $\frac{h}{b} = 2.08$

Buckling curve for h/b ratio $\text{Buckling curve} = b$ Table 8.4 / BS-EN NA

Imperfection factor for buckling curve b $\alpha_{LT} = 0.34$ Table 8.5 / BS-EN NA

Factor of bending moment distribution $f_M = 1.0$ Table 8.6

Radius of gyration about the relevant axis $i = 39.9 \text{ mm}$ values from SCI 'Blue Book'

Relative Slenderness for weak axis flexural buckling $\bar{\lambda}_z = \frac{L_{cr}}{i} \times \frac{1}{93.9\epsilon} = 0.78$

Intermediate factor for

reduction factor for LT $\Phi_{LT} = 0.5 \left[1 + f_M \left(\left(\frac{\bar{\lambda}_{LT}}{\bar{\lambda}_z} \right)^2 \alpha_{LT} (\bar{\lambda}_z - 0.2) \right) + \bar{\lambda}_{LT}^2 \right] = 0.792$ eq (8.82)

Buckling reduction factor for LT $\chi_{LT} = \frac{f_M}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - f_M \bar{\lambda}_{LT}^2}} = 0.958$ eq (8.81)

Design Buckling Resistance $M_{b,Rd} = \chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}} = 411.51 \text{ kN/m}$ eq (8.79)

11.4.1 Critical Buckling Moment (NCCI SN003b-EN-EU)

Second Moment of area about the weak axis $I_z = 13\,600\,000 \text{ mm}^4$ values from SCI 'Blue Book'

Torsion Constant $I_t = 557\,000 \text{ mm}^4$ values from SCI 'Blue Book'

Warping Constant $I_w = 412\,000\,000\,000 \text{ mm}^4$ values from SCI 'Blue Book'

Length of Beam between Lateral Supports $L / L_{cr} = 2370 \text{ mm}$ derived from FE model

C1 Factor $C_1 = 1.0$ value from table 3.1 derived from figure 32

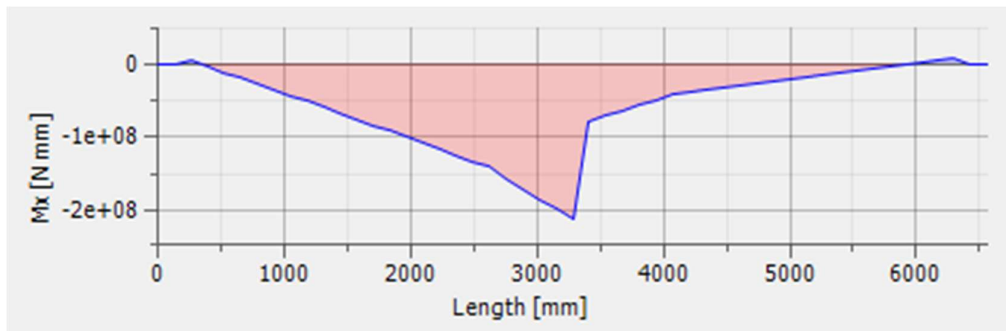


Figure 38 – Bending moment diagram along the member length.

Critical Buckling Moment $M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G_s I_t}{\pi^2 E I_z}} = 994.577 \text{ kNm}$ NCCI SN003b-EN-EU eq 3

11.5 Deflection Limit (BS EN 13001-3-1:2012+A2:2018)

Effective Span

$$L = 6500mm \text{ Taken from Figure 28}$$

Deflection Limit

$$\delta_{Lim} = \frac{L}{200} = 32.5mm \text{ Table 13}$$

12 Appendix E – RHS 150x100x10 (Rectangular Hollow Section)

Design Checks

Design checks on this member are very limited due to that fact that it's noted that design actions for axial force, bending moments and torsion are negligible relative to section capacity.

12.1 Shear Resistance (EN 1993-1-1:2022 Section 8.2.6)

Shear Area for Section

$$A_v = 2550 \text{ mm}^2 \text{ cl.8.2.6.3}$$

Design Shear Resistance

$$V_{pl,Rd} = \frac{A_v \left(\frac{f_y}{\sqrt{3}} \right)}{\gamma_{M0}} = 522.64 \text{ kN} \text{ eq (8.23)}$$

12.2 Shear Buckling (EN 1993-1-5:2006 Cl.5)

Rectangular and Square Hollow Sections are not included within shear buckling provisions as their closed-section geometry provides inherent restraint to plate buckling.

12.3 Bending Moment Resistance (EN1993-1-1: 2022 Section 8.2.5)

Class 1 section, therefore, use plastic modulus

$$W_{pl} = 216\,000 \text{ mm}^3 \text{ values from SCI 'Blue Book'}$$

Design bending resistance

$$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = 76.68 \text{ kNm} \text{ eq (8.20)}$$

12.4 Design Buckling Resistance (EN1993-1-1: 2022 Section 8.3.1.1)

Rectangular and square hollow sections are not included in Table 8.4 for lateral torsional buckling curve selection, as their closed-section geometry provides high torsional stiffness and restrains lateral displacement of compression zones. Consequently, classical lateral torsional buckling is not a governing failure mode for these sections, and no reduction factor χ_{LT} is required in accordance with EN 1993-1-1 guidance.

12.5 Deflection Limit (BS EN 13001-3-1:2012+A2:2018)

Effective Span

$$L = 200 \text{ mm} \text{ Taken from Figure 28}$$

Deflection Limit

$$\delta_{Lim} = \frac{L}{200} = 1 \text{ mm} \text{ Table 13}$$

12.6 Torsional Shear Stress (BS EN 1993-1-1:2022 Section 8.2.7)

Design Torsional Shear Resistance

$$V_{pl,T,Rd} = \left[\sqrt{1 - \frac{\tau_{t,Ed}}{1.25(f_y/\sqrt{3})/\gamma_{M0}}} \right] \times V_{pl,Rd} = 517.75 \text{ kN} \text{ eq (8.33)}$$

13 Appendix F – RHS 250x150x12.5 (Rectangular Hollow Section)

13.1 Shear Resistance (EN 1993-1-1:2022 Section 8.2.6)

Shear Area for Section

$$A_v = 5646 \text{ mm}^2_{\text{cl.8.2.6.3}}$$

Design Shear Resistance

$$V_{pl,Rd} = \frac{A_v \left(\frac{f_y}{\sqrt{3}} \right)}{\gamma_{M0}} = 1157.2 \text{ kN}_{\text{eq (8.23)}}$$

13.2 Shear Buckling (EN 1993-1-5:2006 Cl.5)

Rectangular and Square Hollow Sections are not included within shear buckling provisions as their closed-section geometry provides inherent restraint to plate buckling.

13.3 Bending Moment Resistance (EN1993-1-1: 2022 Section 8.2.5)

Class 1 section, therefore, use plastic modulus

$$W_{pl} = 739\,850 \text{ mm}^3_{\text{values from SCI 'Blue Book'}}$$

Design bending resistance

$$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} = 262.64 \text{ kNm}_{\text{eq (8.20)}}$$

13.4 Design Buckling Resistance (EN1993-1-1: 2022 Section 8.3.1.1)

Rectangular and square hollow sections are not included in Table 8.4 for lateral torsional buckling curve selection, as their closed-section geometry provides high torsional stiffness and restrains lateral displacement of compression zones. Consequently, classical lateral torsional buckling is not a governing failure mode for these sections, and no reduction factor χ_{LT} is required in accordance with EN 1993-1-1 guidance.

13.5 Deflection Limit (BS EN 13001-3-1:2012+A2:2018)

Effective Span

$$L = 1250 \text{ mm}_{\text{Taken from Figure 28}}$$

Deflection Limit

$$\delta_{Lim} = \frac{L}{200} = 6.3 \text{ mm}_{\text{Table 13}}$$

13.6 Torsional Shear Stress (BS EN 1993-1-1:2022 Section 8.2.7)

Design Torsional Shear Resistance

$$V_{pl,T,Rd} = \left[\sqrt{1 - \frac{\tau_{t,Ed}}{1.25(f_y/\sqrt{3})/\gamma_{M0}}} \right] \times V_{pl,Rd} = 876.88 \text{ kN}_{\text{eq (8.33)}}$$

14 Appendix G – Material Certificates

26100 Cremona - Italia
Via Acquaviva, 3 - Zona Porto Canale
Tel. +39 0372 4091
Fax +39 0372 413170
www.arvedi.it

Cap.Soc. 24.000.000 EUR i.v.
Società soggetta all'attività di
direzione e coordinamento di
Finarvedi S.p.A.
Società con Socio Unico
R.I. CR 00113630198
R.E.A. CR 90188
Mecc. Est. CR 000296
C.F. e Part. IVA IT 00113630198

Arvedi Tubi Acciaio

Inspection certificate EN 10204/3.1

AB2

Number 1005864 **date** 19/03/2026 **Messrs** NATIONAL TUBE STOCKHOLDERS LTD

Our order 0211306380 / 20
Customer order 53834
date 12/02/2026
DALTON INDUSTRIAL ESTATE

Consegna 0221686713
delivery 0000206396 date 19/03/2026
YO7 3H DALTON, THIRSK, NORTH YORKSHIRE
GB

Description TUBE NORMALIZED
Standard EN 10210-1 ATA
Steel grade S355J2H/NH
Dimensions SGM 250X250X12,5X10000 EN10210-S355J2HZ3
Tubes 28 **MT** 280,00 **KG** 24.950,00

Chemical analysis

	C	Mn	Si	P	S	Al	Cr	Ni	Mo	Cu	Sn	V	Nb	Ti	B	N
min		0,8500				0,0150										
max	0,2200	1,7000	0,5500	0,0350	0,0350		0,3600	0,5500	0,1300	0,3900		0,1400	0,0600	0,0400		0,0220
3155602A	0,1890	0,9370	0,1710	0,0121	0,0035	0,0295	0,0763	0,0627	0,0124	0,1900		0,0012	0,0115	0,0019		0,0056
3155602A	C+Mn/6+(Cr+Mo+V)/5+(Ni+Cu)/15									<=0,43		0,3800				

Mechanical and technological tests

Works Nr.	Heat	Lot	Tubes	Rm (MPa)	REH (MPa)	A(%)	Rs/R	Fm (N)
1 1145246	3155602A		28	470	355	22,0		
				557	388	29,00		

Impact test

Code	°C	Limit (J/cm2)	Value	Mean	Sample dim.	Position
1 KV	-20	40	214	204	10	pipe body - longitudinal

Test	Standard	Required	-	Result
Non destructive test	EN ISO 10893-2	E4	%	100 POSITIVE
Dimensional inspection	EN 10210-2		LOT	1 POSITIVE
Flattening test	EN ISO 8492		LOT	1 POSITIVE
Visual	EN 10210-1		%	100 POSITIVE

Remarks Surface status according EN 10163-3 Class C Sub Class 3

Figure 39 – Sample 3.1 Inspection Certificates for S355J2/NH material hot rolled CHS. Provided by National Tube Stockholders Limited.



Tata Steel UK Limited
20" H.F.I Mill
Brenda Road
Hartlepool Cleveland UK TS25 2EG

Telephone: + 44 (0) 1429 266611
Email: tubesalessUK@tatasteeleurope.com

INSPECTION CERTIFICATE

EN10204 Type 3.1

Customer
NATIONAL TUBE STOCKHOLDERS LTD
DALTON INDUSTRIAL ESTATE
DALTON, THIRSK
NORTH YORKSHIRE
YO7 3HE

CE UK
0343 0038
24 21
Quality Management System
Approved to ISO 9001

Date 15/03/26 **Z02**
Cert No. 280/2612/0002 **A03**
Del. Note
Page No. 01 **of** 02
Tata Steel Ref.No. Sales **BZB460343** Works **4130**
Customer Order No. **P/O: 53614** **OF 14/JAN/26**

Product Description
CELSIUS HOT FINISHED WELDED STEEL RECTANGULAR HOLLOW SECTIONS TO EN 10210 : 2006 GRADE S355NH AND DUAL CERTIFIED WITH S355J2H INCLUDING OPTIONS 1.2, 1.4 + 1.5. SILICON CONTENT 0.15-0.25%, CARBON EQUIVALENT 0.43 MAX. CORNER RADIUS 2T MAX. SURFACE CONDITION IN ACCORDANCE WITH EN10163-3 CLASS C SUB-CLASS 3.

FPC CERTIFICATE NUMBER 0343/CFR/LRQ0840080/A/B
FPC CERTIFICATE NUMBER 0038/CFR/LRQ0840080/A/B

L=Longitudinal		T=Transverse		B=Body		W=Weld		KV=Charpy V-Notch		HV=Hardness Vickers(10Kg Load)		DT=Drop Weight Tear Test		C=Chemical		M=Mechanical		S=Surface		T=Tests		C=Certification	
Item No.	Number of Pieces	Product Dimensions (mm)	Cast/Heat No.	Pipe No.	C01 C02 Yield Stress Re (N/mm2)	C11 Yield Strength Rm (N/mm2)	C12 Tensile Strength Rm (N/mm2)	C13 Elong A5.65 (%)	D02 Hydro Pressure (Bar)	D01 Charpy Test Type	C03 Charpy Temp (°C)	C04 Charpy Values (J)	C05 Charpy Ave	C06 Charpy Ave	C07 Charpy Ave	C08 Charpy Ave	C09 Charpy Ave	C10 Charpy Ave	C11 Charpy Ave	C12 Charpy Ave	C13 Charpy Ave	C14 Charpy Ave	C15 Charpy Ave
211	1	250.0MM X 150.0MM X 12.50MM	4E61892	R110676	B L	361	507	33		L B	KV	10	-20225	233									
	1	8.500- 8.650 60.42M	4E61890	R112200	B L	419	539	32		L B	KV	10	-20239	244									

C01-C09	Analysis %	C	SI	MN	P	S	CR	MO	NI	AL	B	CU	N	CA	NB	SN	TI	V	CEV
4E61890 LADLE	.14	.19	.21	.39	.014	.005	.028	.001	.009	.031	.0000	.008	.0067	.000	.036	.001	.003	.002	.38
4E61892 LADLE	.14	.19	.41	.39	.012	.006	.024	.006	.010	.043	.0002	.010	.0064	.000	.040	.002	.002	.001	.38

Code Numbers in accordance with EN 10168 (see overleaf).
Alterations to this document or its use for other products shall be regarded as falsification of documents and be subject to criminal sanction.

CE & UKCA marking under the CPR 305/2011 and CPR EU Exit 2020 No 1359 respectively shown on Hartlepool 20" mill inspection documents are affixed according to harmonized standards EN 10210-1 & EN 10219-1 only.
Hartlepool 20" mill Quality System complies with Annex 1 para 4.3 of 2014/68/EU (PED) and S.I. 2016 No. 1105.

CHRIS STEVENS
Section Manager Technical
This document has been prepared by a

Figure 40 – Sample 3.1 Inspection Certificates for S355J2/NH material hot rolled SHS. Provided by National Tube Stockholders Limited.



Tata Steel UK Limited
Corby Works
PO Box 101
Corby Northants UK NN17 5UA

Telephone: + 44 (0) 1536 402121
Email: tubesalesUK@tatasteel.co.uk

INSPECTION CERTIFICATE

EN10204 Type 3.1

Date: 17/03/26
Cert No: 120/2612/0383
Del. Note: CM /0865

Customer: NATIONAL TUBE STOCKHOLDERS LTD
DALTON INDUSTRIAL ESTATE
DALTON, THIRSK
NORTH YORKSHIRE
YO7 3HE



0343 0038
24 21

Quality Management System
Approved to ISO 9001

Tata Steel Ref No: BEB 459355
Sales: 15 9563
Customer Order No: P/O: 53343
Works: OF 21/NOV/25

Product Description: CELSIUS HOT FINISHED WELDED STEEL RECTANGULAR HOLLOW SECTIONS TO EN 10210 : 2006 GRADE S355NH AND DUAL CERTIFIED WITH S355J2H INCLUDING OPTIONS 1.2, 1.4 + 1.5. SILICON CONTENT 0.15-0.25%, CARBON EQUIVALENT 0.43 MAX. CORNER RADIUS 2T MAX. SURFACE CONDITION IN ACCORDANCE WITH EN10163-3 CLASS D SUB-CLASS 3.

FPC CERTIFICATE NUMBER 0343/CPR/LRQ0840080/A/B

FPC CERTIFICATE NUMBER 0038/CPR/LRQ0840080/A/B

NORMALISED ROLLED

Item No.	Number of Places	Product Dimensions (mm)	Cast Heat No.	Bundle no.	Tensile Test				F/Full Section	Kynch/V			Hardness Tests			Steel Making Process
					C10 Yield Stress (N/mm ²)	C11 Tensile Strength (N/mm ²)	C12 Elong A 5.65 (%)	C13 Elong (mm)		C20 Charpy Temp (J/cm ²)	C21 Charpy Temp (J/cm ²)	C22 Charpy Temp (J/cm ²)	C23 Charpy Temp (J/cm ²)			
168	2	150.00 x 100.00 x 10.00 x 14.600M = 29.20M	7824363	24403666	397	515	33		B	KV	7.5	-20	184	191	WELD SEAM 100% EDDY CURRENT TESTED TO EN 10246-3 & EN ISO 10893-2 FLATTENING TEST FOR SEAM WELD QUALITY CONTROL, SATISFACTORY COUNTRY OF MELT AND POUR: CAST PREFIX 7S = IN TUBE MANUFACTURED IN UK	
END OF ITEM																
END OF CERT																

C71-C99 Analysis %		C	Si	Mn	P	S	Cr	Mo	Ni	Al	B	Cu	N	Nb	Sn	Ti	V	CEV
Cast No.	7824363	0.13	0.18	1.41	0.018	0.0049	0.021	0.002	0.024	0.055	0.002	0.011	0.003	0.037	0.001	0.002	0.001	0.37

Code Numbers in accordance with EN 10168 (see overleaf).
 Alterations to this document or its use for other products shall be regarded as falsification of documents and be subject to criminal jurisdiction.

CE & UKCA marking under the CPR 305/2011 and CPR EU Ext 2020 No 1359 respectively shown on Corby Works
 Inspection documents are affixed according to harmonized standards EN 10210-1, EN 10219-1 & EN 10225 only.
 Corby Works Quality System complies with Annex I para 4.3 of 2014/68/EU (PED) and S.I. 2016 No. 1105.
 Note - only tubes in accordance with EN 10217 (T181 grades excluded) have a presumption of conformity with the PED.

Z01 The products covered by this inspection document are certified by Tata Steel UK Limited and comply with the requirements of the Order Acknowledgement.

N Mistry
Quality Control Manager
This document has been prepared by a computer system and is valid without signature.
18 Grosvenor Place London SW1X 7HS This Company Reg No.52280000.

Figure 41 – Sample 3.1 Inspection Certificates for S355J2/NH material hot rolled RHS. Provided by National Tube Stockholders Limited.

15 Appendix H – Weld Procedure Specifications (WPS)

LEDWOOD Engineering, Fabrication, Coating, Construction		WELDING PROCEDURE SPECIFICATION (WPS) in accordance with EN ISO 15609-1 WPS No: GE-W02 Revision No: 03 Date: 15/07/2024		CODES / STANDARDS WPS: EN ISO 15609-1 WPQR: EN ISO 15614-1 Others: N/A Page 1 of 2	
		Manufacturer: Ledwood Mechanical Engineering WPQR No: WPQR/TUV/18271/JC/PF/PD			
Welding process(es): Metal Active Gas with Flux cored electrode (MAG): 136		Parent material: Carbon steels with ≤ 355 MPa yield strength			
Welding technique: Part-mechanised (Semi-automatic)		Group No(s): 1.1, 1.2, 11.1			
Joint type(s) and weld: Fillet weld (FW) Multi-pass		Material thickness, t (mm): ≥ 5.0 mm			
		Welding position(s): All positions (except vertical down PJ, PG & J-L045)			
		Outside diameter, D (mm): ≥ 500 mm (≥ 150 mm PA or PC rotated)			
Joint preparation method		Pre/Inter-pass cleaning method		Second side back gouging method	
Thermal cut and grind &/or Machine ✓		Grinding, chipping & wire brushing ✓		None ✓	
Wire brush and degrease ✓		Other: Ensure wire is clean from grease		If necessary, grind back to sound metal	
Other: Mechanical saw, hacksaw, cutting disc and grinding.		before use.		DPI / MPI check (as per contract)	
				Other:	
Joint design			Welding sequences		
SEE PAGE. 2			SEE PAGE. 2		
Welding details					
Welding process:		MAG: 136			
Weld metal deposited thickness, s (mm):		No restriction on leg length			
Filler material designation:		EN ISO 17632-A: T 46 3 P M 1 H5			
Filler material group(s):		FM1			
Filler material make & trade name:		Lincoln/Oerlikon Fluxofil 14 HD or equivalent			
Filler material size (mm):		Ø1.2			
Filler material, any special baking or drying:		Store in a dry atmosphere at a minimum temperature of 18°C and a maximum relative humidity of 60% RH. Continued page.2			
Shielding gas/flux:		Argon - ISO 14175: Class M26 78% Ar, 20% CO ₂ , 2%O ₂ or similar			
Shielding gas flow rate (ltrs/min):		15 - 20			
Backing gas/flux:		N/A			
Backing gas flow rate (ltrs/min):		N/A			
Minimum preheat temperature (°C):		*t = ≤ 20 mm – min 10°C *t = > 20 mm – min 125°C			
Heating method/control:		Method: Heating by propane gas torch / electric resistance coils. Control: Tempilstiks or contact pyrometer.			
Maximum inter-pass temperature (°C):		300			
Post heating (hydrogen release)		Without Post heating			
Post weld heat treatment (PWHT):		Without PWHT			
Weaving (maximum width of run):		Stringer / Slight weave within specified parameters below			
No. of electrodes:		Single wire			
Oscillation:		N/A			

Figure 42 – LMEL Weld Procedure Specification for MAG process for a fillet welds on structural steel S355 and below, number GE-W02.

LEDWOOD Engineering, Fabrication, Coating, Construction	WELDING PROCEDURE SPECIFICATION (WPS) in accordance with EN ISO 15609-1 WPS No: GE-W05 Revision No: 03 Date: 15/07/2024		CODES / STANDARDS WPS: EN ISO 15609-1 WPQR: EN ISO 15614-1 Others: N/A Page 1 of 2	
	Manufacturer: Ledwood Mechanical Engineering		WPQR No: WPQR/17071/TUV/JC2/JC3	
Welding process(es): Metal Active Gas with Flux cored electrode (MAG): 136		Parent material: Carbon steels with ≤ 360 MPa yield strength		
Welding technique: Part-mechanised (Semi-automatic)		Group No(s): 1.1, 1.2, 11.1		
Joint type(s) and weld: Butt weld (BW): Single-V, Single-sided (ss), Metal / Material Backed (mb), Double-V, Both-sides (bs) multi-pass (ml)		Material thickness, t (mm): BW: 14.0 - 25.0 mm		
		Outside diameter, D (mm): ≥ 500 mm (≥ 150 mm PA or PC rotated)		
		Welding position(s): All positions (except vertical down PJ, PG & J-L045)		
Joint preparation method		Pre/Inter-pass cleaning method		Second side back gouging method
Thermal cut and grind &/or Machine ☒		Grinding, chipping & wire brushing ☒		None
Wire brush and degrease ☒		Other: Ensure wire is clean from grease		If necessary, grind back to sound metal ☒
Other: Mechanical saw, hacksaw, cutting disc and grinding.		before use.		DPI / MPI check (as per contract)
				Other:
Joint design		Welding sequences		
SEE PAGE. 2		SEE PAGE. 2		
Welding details				
Welding process:		MAG: 136 (FCAW)		
Weld metal deposited thickness, s (mm):		BW: (s) 14.0 - 25.0		
Filler material designation:		EN ISO 17632-A: T 46 3 P M1 H5		
Filler material group(s):		FM1		
Filler material make & trade name:		Lincoln/Oerlikon Fluxofil 14 HD or equivalent		
Filler material size (mm):		Ø1.2		
Filler material, any special baking or drying:		Store in a dry atmosphere at a minimum temperature of 18°C and a maximum relative humidity of 60% RH. Continued page.2		
Shielding gas/flux:		Argon - ISO 14175: Class M26 78% Ar, 20% CO ₂ , 2%O ₂ or similar		
Shielding gas flow rate (ltrs/min):		15 - 20		
Backing gas/flux:		N/A		
Backing gas flow rate (ltrs/min):		N/A		
Minimum preheat temperature (°C):		*t ≤ 20 mm – min 10°C *t > 20 mm – min 125°C		
Heating method/control:		Method: Heating by propane gas torch / electric resistance coils. Control: Tempilstiks or contact pyrometer.		
Maximum inter-pass temperature (°C):		250		
Post heating (hydrogen release)		Without Post heating		
Post weld heat treatment (PWHT):		Without PWHT		
Weaving (maximum width of run):		Stringer / Slight weave within specified parameters below		
No. of electrodes:		Single wire		
Oscillation:		N/A		

Figure 43 – LMEL Weld Procedure Specification for MAG process for groove welds on structural steel S355 and below, number GE-W05.

LEDWOOD Engineering, Fabrication, Coating, Construction		WELDING PROCEDURE SPECIFICATION (WPS) in accordance with EN ISO 15609-1 WPS No: GE-W11 Revision No: 02 Date: 16/07/2024		CODES / STANDARDS WPS: EN ISO 15609-1 WPQR: EN ISO 15614-1 Others: N/A Page 1 of 2	
		Manufacturer: Ledwood Mechanical Engineering WPQR No: WPQR/TUV/20429/CW7/PF/PD			
Welding process(es): Manual Metal Arc (MMA): 111		Parent material: Carbon steel with ≤ 355 MPa yield strength			
Welding technique: Manual		Group No(s): 1.1, 1.2, 11.1			
Joint type(s) and weld: Fillet weld (FW), Single-pass (sl)		Material thickness, t (mm): ≥ 5.0			
		Outside diameter, D (mm): ≥ 500 (≥ 150 PA or PC rotated)			
		Welding position(s): All positions (except vertical down PJ, PG & J-L045)			
Joint preparation method		Pre/Inter-pass cleaning method		Second side back gouging method	
Thermal cut and grind &/or Machine ☒		Grinding, chipping & wire brushing ☒		None ☒	
Wire brush and degrease ☒		Other: Ensure wire is clean from grease		If necessary, grind back to sound metal	
Other: Mechanical saw, hacksaw, cutting disc and grinding.		before use.		DPI / MPI check (as per contract)	
				Other:	
Joint design			Welding sequences		
See Page. 2			See Page. 2		
Welding details					
Welding process:		MMA: 111			
Weld metal deposited thickness, s (mm):		2.7 - 6.3 throat thickness (a) / 4.0 - 9.0 leg length (z)			
Filler material designation:		EN ISO 2560-A: E 42 5 B 3 2 H5			
Filler material group(s):		FM1			
Filler material make & trade name:		Lincoln/Oerlikon Supercito 7018S or equivalent			
Filler material size (mm):		Ø3.2			
Filler material, any special baking or drying:		Store in a dry atmosphere at a minimum temperature of 18°C and a maximum relative humidity of 60% RH. Continued page.2			
Shielding gas/flux:		N/A			
Shielding gas flow rate (ltrs/min):		N/A			
Backing gas/flux:		N/A			
Backing gas flow rate (ltrs/min):		N/A			
Tungsten electrode type/size(mm):		N/A			
Minimum preheat temperature (°C):		*t = ≤ 20 mm – min 10°C *t = > 20 mm – min 125°C			
Heating method/control:		Method: Heating by propane gas torch / electric resistance coils. Control: Tempilstik or contact pyrometer.			
Maximum inter-pass temperature (°C):		N/A - Single-pass			
Post heating (hydrogen release)		With or Without Post heating			
Post weld heat treatment (PWHT):		Without PWHT			
Weaving (maximum width of run):		Stringer / Slight weave (2.5x electrode dia max. within specified parameters below)			
Oscillation:		N/A			

Figure 44 – LMEL Weld Procedure Specification for MMA process for a fillet welds on structural steel S355 and below, number GE-W11.

LEDWOOD Engineering. Fabrication. Coating. Construction		WELDING PROCEDURE SPECIFICATION (WPS) in accordance with EN ISO 15609-1 WPS No: GE-W13 Revision No: 01 Date: 16/07/2024		CODES / STANDARDS WPS: EN ISO 15609-1 WPQR: EN ISO 15614-1 Others: N/A Page 1 of 2	
Manufacturer: Ledwood Mechanical Engineering			WPQR No: WPQR/TUV/17266/DC1		
Welding process(es): Manual Metal Arc (MMA): 111			Parent material: Carbon steels with ≤ 355 MPa yield strength		
Welding technique: Manual			Group No(s): 1.1, 1.2, 11.1		
Joint type(s) and weld: Butt weld (BW): Single-V Single-sided (ss), Metal/Material Backed (mb), Double-V, Both-sides (bs) multi-pass (ml)			Material thickness, t (mm): BW: 14.0 - 25.0mm		
			Outside diameter, D (mm): ≥ 500 (≥ 150 PA or PC rotated)		
			Welding position(s): All positions (except vertical down PJ, PG & J-L045)		
Joint preparation method		Pre/Inter-pass cleaning method		Second side back gouging method	
Thermal cut and grind &/or Machine ☒		Grinding, chipping & wire brushing ☒		None	
Wire brush and degrease ☒		Other: Ensure wire is clean from grease before use.		If necessary, grind back to sound metal ☒	
Other: Mechanical saw, hacksaw, cutting disc and grinding.				DPI / MPI check (as per contract)	
				Other:	
Joint design			Welding sequences		
See Page. 2			See Page. 2		
Welding details					
Welding process:		MMA: 111			
Weld metal deposited thickness, s (mm):		BW: (s) 14.0 - 25.0			
Filler material designation:		EN ISO 2560-A: E 42 5 B 3 2 H5			
Filler material group(s):		FM1			
Filler material make & trade name:		Lincoln/Oerlikon Supercito 7018S or equivalent			
Filler material size (mm):		$\varnothing 2.5$ / $\varnothing 3.2$ / $\varnothing 4.0$			
Filler material, any special baking or drying:		Store in a dry atmosphere at a minimum temperature of 18°C and a maximum relative humidity of 60% RH. Continued page.2			
Shielding gas/flux:		N/A			
Shielding gas flow rate (ltrs/min):		N/A			
Backing gas/flux:		N/A			
Backing gas flow rate (ltrs/min):		N/A			
Tungsten electrode type/size(mm):		N/A			
Minimum preheat temperature (°C):		*t = ≤ 20 mm – min 10°C *t = > 20 mm – min 125°C			
Heating method/control:		Method: Heating by propane gas torch (if required) / Control: tempilstik or contact pyrometer.			
Maximum inter-pass temperature (°C):		250			
Post heating (hydrogen release)		Without Post heating			
Post weld heat treatment (PWHT):		Without PWHT			
Weaving (maximum width of run):		Stringer / Slight weave (2.5x electrode dia max. within specified parameters below)			
Oscillation:		N/A			

Figure 45 – LMEL Weld Procedure Specification for MMA process for a Groove weld on structural steel S355 and below, number GE-W13.

16 Appendix I – Padeye Calculations to ASME BTH-1

16.1 Material Properties

Steel Youngs Modulus

$$E \approx 210 \text{ GPa}$$

Note: both base and lug will be of S355 therefore:

Lug and Base Ultimate Tensile Strength

$$F_u = 470 \text{ MPa}$$

Lug and Base Yield Strength

$$F_y = 355 \text{ MPa}$$

B30.20-2018 to ASME BTH-1 Design Category B, this means that a Nominal Design Factor $N_d = 3.0$ is used.

16.2 Lug Geometry

As defined in the main body of the text a hole size of 74mm will be used to fit the 55Te shackle and plate thickness of 55mm is used as the initial size.

Lug thickness

$$t_{lug} = 55 \text{ mm}$$

Hole Diameter

$$D_h = 74 \text{ mm}$$

Pin Diameter

$$D_{pin} = 70 \text{ mm}$$

Material Above Pin

$$a = 74 \text{ mm}$$

Side material each side

$$b_e = 74 \text{ mm}$$

Total lug width

$$B_{tot} = D_h + 2b_e = 222 \text{ mm}$$

Top radius

$$r = \frac{D_h}{2} + a = 111 \text{ mm}$$

Material below hole

$$h = 95 \text{ mm}$$

Total lug height

$$H = h + D_h + a = 243 \text{ mm}$$

16.3 Calculate Allowable Stresses for Static Design

16.3.1 Gross Tension Yield

$$F_{t_{gross}} = \frac{F_{y_{lug}}}{N_d} = 118.34 \text{ MPa} \text{ ASME BTH-1 Eq. 3-1}$$

$$F_{t_{effect}} = \frac{F_{u_{lug}}}{1.2 \times N_d} = 98.62 \text{ MPa} \text{ ASME BTH-1 Eq. 3-2}$$

$$\Omega_{tension_{gross}} = \frac{F_{y_{lug}}}{F_{t_{gross}}} = 3.599$$

16.3.2 Minor Axis Bending – Solid Bars

$$F_b = \frac{1.25 \times F_{y_{lug}}}{N_d} = 147.92 \text{ MPa} \text{ ASME BTH-1 Eq. 3-25}$$

$$\Omega_{\text{bending}} = \frac{F_{y_{lug}}}{F_b} = 2.399 \text{ ASME BTH-1 Eq. 3-25}$$

16.3.3 Bearing Stress

$$F_p = \frac{1.25 \times F_{y_{lug}}}{N_d} = 147.92 \text{ MPa} \text{ ASME BTH-1 Eq. 3-53}$$

$$\Omega_{\text{bearing}} = \frac{F_{y_{lug}}}{F_b} = 2.399$$

16.3.4 Shear on Bars, Pins & Plates

$$F_v = i f \frac{a}{t_{lug}} \leq 2.45 \times \sqrt{\frac{E}{F_{y_{lug}}}} = 32.71 \text{ MPa} \text{ ASME B30 Sec. 3-2.3.6}$$

$$\Omega_{\text{shear}} = \frac{F_{y_{lug}}}{F_v} = 10.85$$

16.4 Failure Mode 1 – Yielding of Gross Section

$$P_{\text{allow}_{\text{gross}_{\text{yield}}}} = \frac{t_{lug} \times B_{\text{tot}} \times F_{y_{lug}}}{\Omega_{\text{tension}_{\text{gross}}}} = 1204.38 \text{ kN} \text{ Adapted from ASME BTH-1 Eq. 3.2}$$

This is significantly above the required lifting load, indicating that yielding of the full lug cross-section is not expected to govern the design.

16.5 Failure Mode 2 – Tension Rupture on Effective Area

$$\Omega_{\text{tens}_{\text{rupt}}} = 5 \text{ Taken from BS EN 131414-1.}$$

$$P_{\text{allow}_{\text{tens}_{\text{rupt}}}} = \frac{F_{u_{lug}} \times (2 \times t_{lug} \times b_e)}{\Omega_{\text{tens}_{\text{rupt}}}} = 577.94 \text{ kN} \text{ Adapted from AISC 360-16}$$

This is lower than the gross yielding resistance, showing that the reduced effective area around the pin hole is more critical than the full section capacity.

16.6 Failure Mode 3 – Shear Rupture on Planes Parallel

Shear Plane Loading Angle $\Phi_{\text{loading}_{\text{angle}}} = 15^\circ$ this angle has been adopted as a conservative design assumption to account for potential load misalignment, rigging tolerances, sling movement and out-of-plane effects during lifting operations. This assumption introduces secondary bending and eccentric loading effects within the lifting lug, resulting in a more onerous stress condition than idealised in-plane loading.

Shear Plane Reduction Due to Curve:

$$z_{\text{prime}} = r - \sqrt{r^2 - \left(\frac{D_{\text{pin}}}{2} \times \left(\sin(\Phi_{\text{loading}_{\text{angle}}}) \right) \right)^2} = 0.37 \text{ mm} \text{ ASME B30 Eq. C-2}$$

$$A_{sf} = 2 \times \left(a - z_{\text{prime}} + \frac{D_{\text{pin}}}{2} \right) \times t_{lug} = 11949.3 \text{ mm}^2 \text{ Adapted from AISC 360-16}$$

$$P_{\text{allow}_{\text{shear}}} = \frac{0.6 \times F_{u_{lug}} \times A_{sf}}{\Omega_{\text{shear}}} = 310.56 \text{ kN} \text{ Adapted from AISC 360-16}$$

This is the lowest calculated resistance and therefore represents the governing failure mode for the current lug geometry. The result indicates that the effective shear area around the pin connection is insufficient compared with the other failure modes, and this should be the primary focus for design refinement and further analysed section 4.12.4.

16.7 Failure Mode 4 – Bearing Failure @ Pin

$$P_{allow\ bearing} = \frac{1.8 \times F_{y\ lug} \times t_{lug} \times D_{pin}}{\Omega_{bearing}} = 1025.49kN \quad \text{Adapted from AISC 360-16}$$

This is considerably higher than the shear rupture resistance, indicating that local bearing failure between the pin and lug hole is not governing for the current pin diameter and lug thickness.

16.8 Failure Mode 5 – Out of Plane Bending – Weak Axis

$$Z_{plastic} = \frac{1}{4} \times B_{tot} \times t_{lug}^2 = 167887.5mm^3$$

$$P_{allow\ flexure} = \frac{F_{y\ lug} \times Z_{plastic}}{\Omega_{bending} \times \sin(\Delta_{out\ plane}) \times \left(h + \frac{D_h}{2}\right)} = 1135.96kN \quad \text{Adapted from AISC 360-16}$$

This indicates that the lug has adequate weak-axis bending capacity under the assumed out-of-plane loading angle. However, this failure mode remains important because lifting operations are rarely perfectly aligned.

17 Appendix J – Material Costs

A full Bill of Materials was generated using the frames lengths, sizes and classifications and quotes from dyfed steels the total Material costs are £11866 to supply all listed materials.

From: Josh Axon <Josh.Axon@ledwood.co.uk>
Sent: 03 April 2026 16:00
To: Kadi Garstang <Kadi.Garstang@ledwood.co.uk>; Ceri Worcester <Ceri.Worcester@ledwood.co.uk>
Subject: Dissertation Help

Hi Both,

For my dissertation I'm designing a structural lifting frame would you be able to get indicative prices and potentially some material certificates for the below when you get the chance:

ID	Length	Member Designation	Weight	Grade
BS 4 - 356x171x67 - 6560	6560.000	356x171x67	440.239	S355
	mm		kg	
BS 4 - 356x171x67 - 6560	6560.000	356x171x67	440.239	S355
	mm		kg	
BS 4 - 356x171x67 - 6560	6560.000	356x171x67	440.239	S355
	mm		kg	
BS 4 - 356x171x67 - 6560	6560.000	356x171x67	440.239	S355
	mm		kg	
BS 4848 - 250 x 250 x 12.5 HFRHS - 3000	3000.000	250 x 250 x 12.5	270.180	S355
	mm		kg	
BS 4848 - 250 x 250 x 12.5 HFRHS - 3000	3000.000	250 x 250 x 12.5	270.180	S355
	mm		kg	

£1900
y.l.m

£1800
y.l.m

1

Figure 46 – Page 1 of Email sent from document owner to procurement department of Ledwood to provide quotations for supply of materials for FTM.

BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1500	1500.000 mm	250 x 150 x 12.5	105.653 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 250 x 150 x 12.5 HFRHS - 1260	1260.000 mm	250 x 150 x 12.5	88.748 kg	S355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	S355

5900
4.4M

Figure 47 – Page 2 of Email sent from document owner to procurement department of Ledwood to provide quotations for supply of materials for FTM

BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-2832.64	2832.644 mm	60x60x10	24.710 kg	\$355
BS 4848 - 60x60x10-879.49	879.490 mm	60x60x10	7.672 kg	\$355
BS 4848 - 60x60x10-879.49	879.490 mm	60x60x10	7.672 kg	\$355
BS 4848 - 60x60x10-879.49	879.490 mm	60x60x10	7.672 kg	\$355
BS 4848 - 60x60x10-879.49	879.490 mm	60x60x10	7.672 kg	\$355
BS 4848 - 60x60x10-879.49	879.490 mm	60x60x10	7.672 kg	\$355
BS 4848 - 60x60x10-879.49	879.490 mm	60x60x10	7.672 kg	\$355
BS 4848 - 60x60x10-879.49	879.490 mm	60x60x10	7.672 kg	\$355
BS 4848 - 60x60x10-879.49	879.490 mm	60x60x10	7.672 kg	\$355
BS 4848 - 150 x 100 x 10 HFRHS - 200	200.000 mm	150 x 100 x 10	6.818 kg	\$355
BS 4848 - 150 x 100 x 10 HFRHS - 200	200.000 mm	150 x 100 x 10	6.818 kg	\$355
BS 4848 - 150 x 100 x 10 HFRHS - 200	200.000 mm	150 x 100 x 10	6.818 kg	\$355
BS 4848 - 150 x 100 x 10 HFRHS - 200	200.000 mm	150 x 100 x 10	6.818 kg	\$355

41800

4200

4466

4200

Figure 48 – Page 3 of Email sent from document owner to procurement department of Ledwood to provide quotations for supply of materials for FTM.

BS 4848 - 150 x 100 x 10 HFRHS - 200	200.000 mm	150 x 100 x 10	6.818 kg	S355
BS 4848 - 150 x 100 x 10 HFRHS - 200	200.000 mm	150 x 100 x 10	6.818 kg	S355
BS 4848 - 60x60x10-530	530.000 mm	60x60x10	4.623 kg	S355
BS 4848 - 60x60x10-530	530.000 mm	60x60x10	4.623 kg	S355
BS 4848 - 60x60x10-530	530.000 mm	60x60x10	4.623 kg	S355
BS 4848 - 60x60x10-530	530.000 mm	60x60x10	4.623 kg	S355
BS 4848 - 60x60x10-530	530.000 mm	60x60x10	4.623 kg	S355
BS 4848 - 60x60x10-530	530.000 mm	60x60x10	4.623 kg	S355
BS 4848 - 60x60x10-530	530.000 mm	60x60x10	4.623 kg	S355
BS 4848 - 60x60x10-530	530.000 mm	60x60x10	4.623 kg	S355

Kind Regards,

Figure 49 – Page 4 of Email sent from document owner to procurement department of Ledwood to provide quotations for supply of materials for FTM.