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Integral Steel Bridges:

Design of a Single-Span Bridge - Worked Example

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FOREWORD

The concept of designing bridges as 'integral bridges', i.e. without any movement joints, is being encouraged by the Highways Agency, in the desire to improve their durability.

This document is one of three SCI publications dealing with the design of Integral Steel Bridges. It comprises a worked example for a single span fully integral bridge and is a companion document to *Integral steel bridges: Design guidance* (SCI-P163). The third publication in this series will be *Integral steel bridges: Design of a multi-span bridge - Worked example* (SCI-P189).

The document was written by J A Way and E Yandzio, with the assistance of A R Biddle and D C Iles, all of The Steel Construction Institute. Advice and contributions from the following experienced bridge designers and foundation engineers are gratefully acknowledged:

D Rowbottom	British Steel Piling
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A Low	Ove Arup and Partners
J L Vincett	Tony Gee and Partners
R E Craig	WS Atkins

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CONTENTS

SUMMARY

This publication provides a Worked Example for the design of a single-span fully integral bridge, that utilises High Modulus Pile abutments and a composite plate girder deck. It has been based on the findings of studies undertaken on steel integral bridges and steel substructures by The Steel Construction Institute since 1993.

Calculations are provided for each design stage, together with a detailed commentary explaining the background to the methods employed and the parameters chosen. Computer-based numerical techniques have been used to enable full soil-structure interaction to be considered in the analysis.

A design is proposed for a reinforced concrete capping beam to provide a full moment connection between the High Modulus Piles and the deck.

Ponts en acier de type intégral: Exemple d'application pour un pont à simple portée

Résumé

Cette publication donne un exemple de dimensionnement d'un pont en acier, à simple portée, utilisant le concept de "pont intégral", ce qui signifie que les piles et culées sont intégrées au tablier, réalisé de manière composite acier-béton. Ce concept est basé sur des recherches effectuées par le Steel Construction Institute depuis 1993.

Les calculs sont établis pour chaque stade de construction et sont accompagnés de commentaires détaillés expliquant les bases des méthodes utilisées et le choix des paramètres. Des techniques numériques ont été utilisées pour prendre en compte l'interaction entre le sol et la structure dans le dimensionnement.

Une méthode de dimensionnement est proposée pour le calcul de la poutre chapeau en béton armé afin que cette dernière assure une transmission totale des moments de flexion à la liaison entre le tablier et les piles.

Rahmenbrücken aus Stahl: Berechnung einer einfeldrigen Brücke - Berechnungsbeispiel

Zusammenfassung

Diese Veröffentlichung enthält ein Berechnungsbeispiel für eine einfeldrige Rahmenbrücke aus Stahl, bestehend aus Widerlagem (steife Stahlpfähle) und geschweißten Verbundträgern für den Brückenbalken. Als Grundlage dienen die Ergebnisse von Studien über Rahmenbrücken aus Stahl und Stahlunterbauten, die vom Steel Construction Institute seit 1993 angestellt wurden.

Berechnungen sind für jede Phase aufgestellt, detaillierte Kommentare erläutern den Hintergrund der gewählten Methoden und Parameter. Computer-unterstützte numerische Methoden wurden verwendet um die volle

Interaktion zwischen Boden und Tragwerk bei der Berechnung zu berücksichtigen.

Ein Berechnungsverfahren wird für den bewehrten Pfahlkopfträger vorgeschlagen, der eine biegesteife Verbindung zwischen den Stahlpfählen und dem Brückenbalken herstellt.

Ponti integrali in acciaio: esempio applicativo per un ponte a una campata

Sommario

Questa pubblicazione propone un esempio applicativo di progettazione di un ponte integrale in acciaio a una campata, il quale utilizza spalle di tipo High Modulus Pile e un impalcato con profili composti a parete piena. L'esempio riportato è basato sui risultati di studi sviluppati, a partire dal 1993, dallo Steel Construction Institute, su ponti integrali in acciaio e su sottostrutture metalliche.

I calcoli sono riportati per ogni fase progettuale, unitamente a una spiegazione dettagliata relativa alle nozioni retrospettive relative ai metodi impiegati e ai parametri scelti. Adeguate tecniche numeriche, basate sull'uso dell'elaboratore elettronico, sono state usate per simulare nell'analisi progettuale l'interazione tra suolo e struttura.

L'esempio progettazione riportato prevede l'utilizzo di una elemento di trave di copertura in conglomerato cementizio armato per garantire un collegamento a completo ripristino tra le spalle High Modulus Piles e l'impalcato.

Puentes integrales de acero: Ejemplo desarrollado para un puente de un vano

Resumen

Este informe contiene un ejemplo desarrollado del proyecto de un puente íntegramente de acero de un vano que utiliza estribos formados por pilas de gran módulo y un tablero compuesto de vigas y chapa. Se basa en los estudios llevados a cabo por el Steel Construction Institute sobre puentes integrales de acero con subestructuras de acero también, desde 1993.

Para cada etapa de proyecto se suministran los cálculos así como comentarios detallados explicando los fundamentos de los métodos empleados y de los parámetros que se han escogido.

Se han utilizado técnicas numéricas mediante computador para permitir la posibilidad de tener en cuenta en el cálculo los efectos de interacción terreno-estructura. También se propone un proyecto para un cabezal de hormigón armado que suministre una unión rígida entre las pilas de gran módulo y el tablero.

Stålbroar: Beräkningsexempel för en ändskärmsbro

Sammanfattning

Denna publikation innehåller ett genomräknat exempel av en ändskärmsbro med stålbalkar, stålspont, stålpålar och samverkande betongfarbana. Stålpålarna,

som är valsade I-balkar, har här svetsats till stålsponten, för att kunna föra över gynnsamma moment av jordtrycket till samverkanstvärsnittet.

Beräkningar utförs i olika dimensioneringsskeden, tillsammans med detaljerade kommentarer som förklarar det valda dimensioneringsförfarandet.

Datorberäkningar har gjort att det statiska samspelet mellan jorden och bron har kunnat utnyttjats till fullo. En dimensioneringsmetod för tvärbalken över pålarna, som medger full momentöverföring mellan pålarna och samverkanstvårsnittet, föreslås.

1 INTRODUCTION

To comply with the Highways Agency's Standard BD 57, *Design for durability*, the majority of bridges in the UK with spans less than 60 m should be designed as integral bridges. An integral bridge is a bridge built without movement joints at the abutments or between supports. Currently, the simplest form of integral bridge in terms of design, constructability and ease of maintenance, is a fully integral single span bridge - essentially a portal frame, with full structural continuity between the deck beams and the supporting elements.

Whilst the design of a traditional simply supported bridge can be straightforwardly split into deck design and substructure design, this is not possible for an integral bridge. In an integral bridge, consideration must be given to the interaction between the deck and the abutment, and between the abutment and the retained ground. The soil behind the abutment is not simply a load, it is part of the supporting structure. An analysis which incorporates the response of the retained soil will be more economic than one which does not.

Structural behaviour is well understood by bridge designers, but soil behaviour is less well understood by them. Integral bridge design requires a merging of the specialist fields of soil mechanics and structural mechanics. However, the modelling of soil behaviour has historically been based on simplified stability based criteria. Such techniques are insufficiently sophisticated to analyse the interaction between soil, abutment and deck. The worked example in this publication illustrates some of the methods of assessing the interaction between substructure and superstructure.

Integral bridges with abutments formed from embedded high modulus steel piles offer a particularly economic solution for integral bridges of moderate length. High Modulus Piles facilitate rapid construction by eliminating the need for temporary sheet pile walls. Their well defined stiffness also reduces uncertainties in the design process. High modulus piles also form a flexible, 'compliant' foundation, which deflects to accommodate temperature-induced strains in the deck, thus reducing the values of 'locked in' stresses.

To date, sheet pile walls have often been designed using permissible stress techniques, in contrast to the partial safety factor approach adopted by the majority of modern structural codes. In the Worked Example the design of embedded High Modulus Piles for the abutment is carried out to limit state principles in accordance with BS 5400: Part 3.

The connection between the deck and the abutment is an area of particular concern for the integral bridge designer. The Worked Example proposes a design method for such a connection.

This publication provides an illustrative Worked Example. For further guidance on integral bridge design the reader is referred to the SCI publication *Integral steel bridges: Design guidance* (SCI-P163).

1.1 Reference documents

Within this publication, reference has been made to the following documents.

British Standards Institution

BS 5400: Steel, concrete and composite bridges
Part 1: 1988: General statement
Part 2: 1978: Specification for loads
Part 3: 1982: Code of practice for design of steel bridges
Part 4: 1990: Code of practice for design of concrete bridges
Part 5: 1979: Code of practice for design of composite bridges

BS 8002: 1994: Code of practice for earth retaining structures
BSI, 1994

BS EN 10 025: Hot rolled products of non-alloy structural steels - Technical delivery conditions
BSI, 1993

Draft for Development DD ENV 1997-1: 1995
Eurocode 7: Geotechnical design
Part 1: General rules (includes the United Kingdom National Application Document)
BSI, 1995

European Committee for Standardisation (CEN)

Draft prENV 1993-5
Eurocode 3: Design of steel structures
Part 5: Piling
BSI, 1996

Highways Agency

Design manual for roads and bridges:

Volume 1, Section 3

BD 13/90	Design of steel bridges. Use of BS 5400: Part 3: 1982
BD 16/82	Design of composite bridges. Use of BS 5400: Part 5: 1979
BD 24/92	Design of concrete bridges. Use of BS 5400: Part 4: 1990
BD 42/96	The design of integral bridges

Volume 2, Section 1

BD 42/94	Design of embedded retaining walls and bridge abutments (unpropped or propped at the top)

All Highways Agency documents are published by The Stationery Office.

Software

FREW Flexible Retaining Wall Analysis
OASYS GEO program suite
OASYS Ltd., 1991 (Tel: 0171 580 1531)

ReWaRD Version 1.5, Advanced Retaining Wall Design and Analysis
Geotechnical Consulting Group Ltd., 1992 (Tel: 01709 402166)
Available through British Steel Piling (Tel: 01724 280280)

WALLAP Version 4.0, Anchored and Cantilevered Retaining Wall Analysis Program
Geosolve 1996 (Tel: 0181 674 7251)

Soil analysis

Akroyd, T.N.W.
Earth-retaining structures: introduction to the Code of Practice BS 8002
The Structural Engineer, Vol. 74, No. 21, 5 November 1996

Borin, D. L.
WALLAP anchored and cantilevered retaining wall analysis program
User's manual (Version 4)
Geosolve, London, 1988

Caquot, A. and Kerisel, J.
Tables for calculation of passive, active pressure and bearing capacity foundations; translated from French by M A Bec, London
Gauthier-Villars, Paris, 1948

Institution of Structural Engineers
Soil structure interaction - the real behaviour of structures
ISE, 1989

Padfield, C. J. and Mair, R. J.
Report 104: Design of retaining walls embedded in stiff clays
Construction Industry Research and Information Association (CIRIA), 1984

Pappin, J. W., Simpson, B., Felton, P. J. and Raison, C.
Numerical analysis of flexible retaining walls
Proc. Symp. Computer Applications in Geot. Engng.
Midland Geot. Soc. Birmingham Univ. pp. 195-212
1986

Springman, S.M., Norrish, A.R.M., Ng, C.W.W.
TRL Report 146: Cyclic loading of sand behind integral bridge abutments
Transport Research Laboratory, 1996

Steel piling

British Steel Sections, Plates & Commercial Steels
Piling Handbook, Seventh Edition, 1997

Federation of Piling Specialists
Specification for steel sheet piling
FPS, 1991

McShane, G.
Steel sheet piling used in the combined role of bearing piles and earth retaining members
Proc. 4th Int. Conf. Piling and Deep Foundations, Stresa, Italy, 7-12 April 1991
TESPA, (Technical European Sheet Piling Association), 1991

Structural analysis

Coates, R. C., Coutie, M. G. and Kong, F. K.
Structural Analysis
Van Nostran Reinhold (UK) Co. Ltd, 1987

Geoguide 1 - Guide to retaining wall design (2nd Edition)
Geotechnical Engineering Office, Civil Engineering Dept., Hong Kong, 1993

Low, A.
Concepts in the design of the abutment in integral bridges
TTU Technical Paper BD/TP/158/92
Transport Research Laboratory, 1992

The Steel Construction Institute

Biddle, A. R., Iles, D.C. and Yandzio, E.
Integral steel bridges: Design guidance (SCI-P163)
The Steel Construction Institute, 1997

Yandzio, E.
Design guide for steel sheet piled bridge abutments (SCI-P187)
The Steel Construction Institute (to be published)

Steel Designers' Manual - 5th Edition
The Steel Construction Institute and Blackwell Science, 1994

1.2 Highways Agency document BA 42/96

The methods proposed in the recently published Highways Agency advisory document BA 42/96 *The design of integral bridges* for the derivation of earth pressures have not been used in this Worked Example.

That document, which gives guidance on the assessment and analysis of soil effects due to cyclic deck thermal movements, recommends the use of simplified passive earth pressure distributions for the maximum loadings on rigid retaining wall bridge abutments. These simplified distributions are based on the use of research on abutments backfilled with sand to predict simplified passive pressures for clay soils and this would appear to be somewhat questionable.

The Worked Example is largely concerned with the effects of the full moment connection, and since this can only be modelled effectively using numerical soil analysis programs that are based on a strain related model, the simplified pressure distributions given in BA 42 cannot be used.

In addition, the work of Springman *et al* that forms the research basis for BA 42 indicates that for the small amplitudes of wall rotation such as those that occur in this Example, the effect of cyclic wall rotation on passive lateral earth pressures does not appear to be significant.

2 CALCULATION PROCEDURES

The calculation procedure followed in the Worked Example is illustrated by the flow chart shown in Figure 2.1. The design sequence is essentially no different from that of a simply supported bridge, except that the structural analysis stage must take account of the interaction between the substructure and the deck.

Once the initial conditions (basic structure, loading and soil parameters) were established, a stability analysis was performed to determine the required depth of embedment of the high modulus pile wall. Following this, a computer analysis was carried out to evaluate the interaction between the deck and the abutment/soil. The structural forces obtained were then used to design the abutment wall, deck and the capping beam connection.

For simplicity, only a beam analysis of the deck was carried out. Where a grillage analysis is required, the deck and substucture could be analysed separately, with interaction taken into account by the use of appropriate boundary conditions.

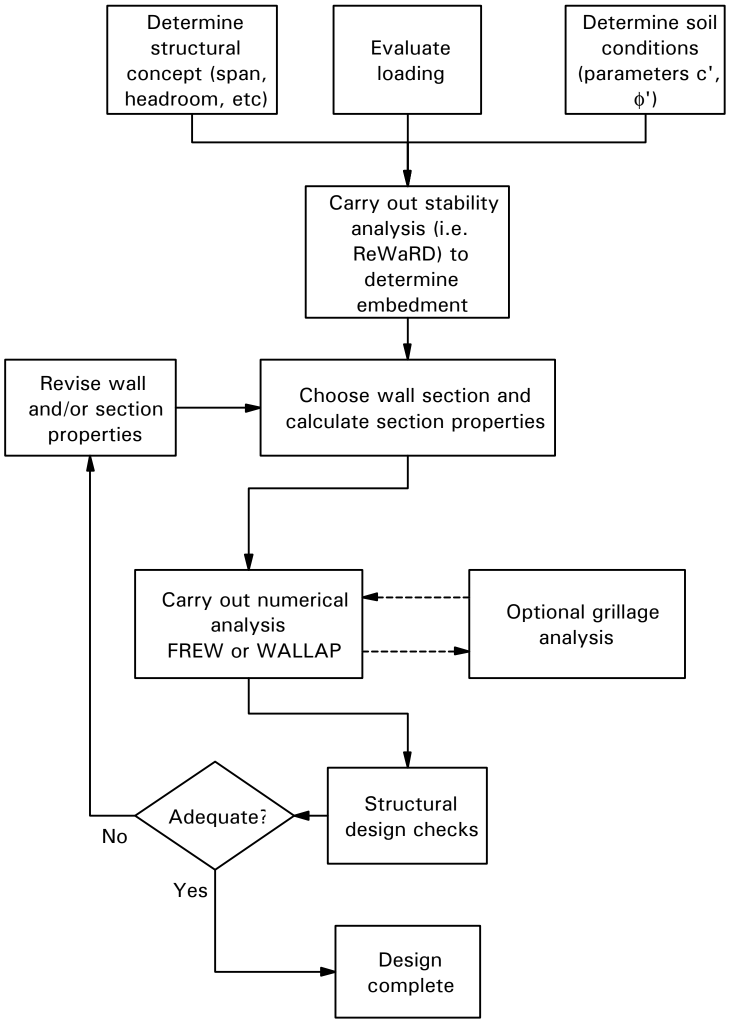


Figure 2.1 *Design procedure adopted in Worked Example*

3 THE WORKED EXAMPLE

3.1 Design basis

The design basis for the various parts of the integral bridge are generally in accordance with the Standards given in Table 3.1.

Table 3.1 *Design basis*

Structural element	Design Standard	Loading Specification
Deck	BS 5400: Part 3 BS 5400: Part 5	BD 37/88
High Modulus Pile Wall	BS 5400: Part 3	BD 37/88
Capping beam	BS 5400: Part 4 BS 5400: Part 5	BD 37/88
Wall - stability against overturning	(Limit equilibrium method) Factor of Safety on Strength method.	Eurocode 7 Table 2.1

3.1.1 Loading

Design loads for dead and imposed loads on the bridge are generally in accordance with BS 5400: Part 2, as implemented and modified by BD 37/88. For simplicity, only HA loading is considered (see Section 3.4). Load factors are applied in accordance with BD 37/88.

The soil behind the retaining wall acts as both a load and a resistance. Additional information is given below.

3.1.2 ULS load effects due to the pressure of retained earth on structural elements

For embedded retaining walls in integral bridges, the Highways Agency requirements and recommendations appear to be contradictory.

BD 42/94 *Design of embedded retaining walls and bridge abutments (unpropped or propped at the top)* requires that ULS earth pressure effects are "calculated using worst credible strength parameters multiplied by a partial load factor (γ_{fL}) of 1.0". All other loading is applied in accordance with BS 5400: Part 2 as implemented by BD 37.

BA 42/96 *The design of integral bridges* recommends that "earth pressure forces on abutments should be subject to load factors γ_{fL} of 1.5 at ULS and 1.0 at SLS" i.e. in accordance with BD 37, and using earth pressure coefficients in accordance with BS 8002 multiplied by γ_m of 1.0 when considering disadvantageous forces and γ_m of 0.5 when considering advantageous forces resisting secondary loads.

For the structural elements in the Worked Example, ULS load effects resulting from earth pressure have been derived in accordance with BD 37, that is, for adverse load effects due to soil pressure $\gamma_{fL} = 1.5$, and for beneficial load effects $\gamma_{fL} = 1.0$. The use of worst credible soil parameters in conjunction with $\gamma_{fL} > 1.0$ is considered to be excessively conservative, so 'characteristic values' as defined by Eurocode 7 have therefore been adopted.

The choice of safety factors for the stability analysis is more complex. For a discussion of the factors used in the Worked Example, refer to Section 3.6.

3.1.3 Design resistance

Design resistances are determined in accordance with BS 5400: Parts 3, 4, and 5, for the steel, concrete and composite elements respectively. The integral bridge deck and High Modulus Piles are designed to BS 5400: Parts 3 and 5. The capping beam is designed to BS 5400: Parts 4 and 5.

In the calculation of the friction resistance of the soil against the pile under vertical loading, the ULS resistance has been reduced by applying a factor of γ_S = 1.3, in accordance with Eurocode 7.

3.1.4 The application of γ_{f3}

BS 5400: Part 4 applies the γ_{f3} factor on the opposite side of the effect:resistance equation to that presumed in Part 3. However, for consistency thoughout the Worked Example the format adopted by Part 3 (which applies γ_{f3} on the resistance side of the equation) has been used.

3.2 General arrangement

The span and headroom clearance in the Worked Example have been chosen to correspond to an overbridge for a dual 2-lane all-purpose road (see Figure 3.1).

Composite plate girders have been chosen for the deck in order to illustrate the application of this form of deck construction to integral bridges (see Figure 3.2).

Bridge dimensions:	Span	=	33 m
	Clearance	=	5.7 m

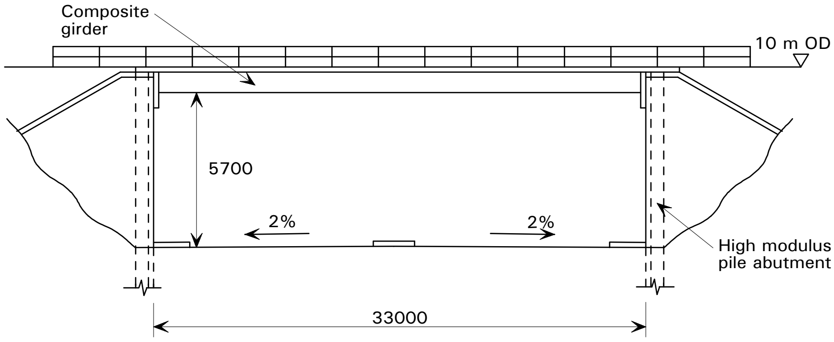


Figure 3.1 *Integral bridge elevation*

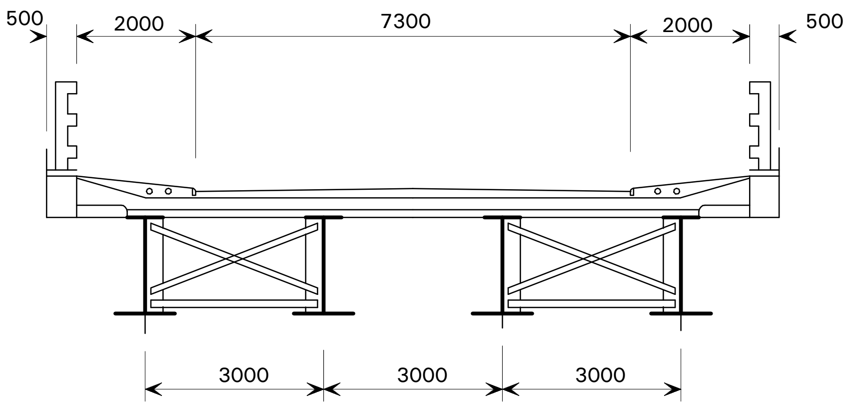


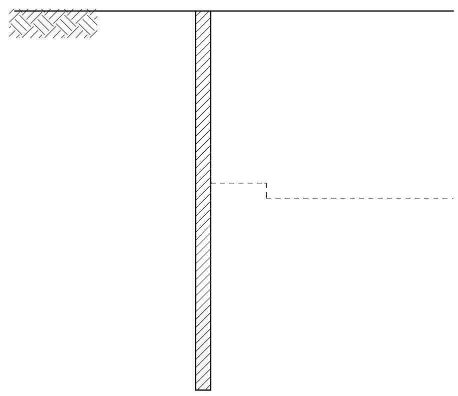
Figure 3.2 *Bridge deck section*

3.3 Construction sequence

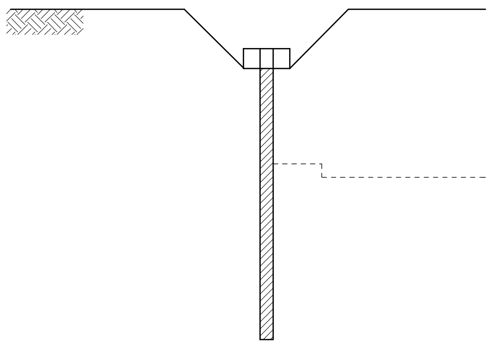
It has been assumed that the new motorway will be constructed in cutting, with the bridge deck surface at the current ground level. The opportunity to construct the deck before the cutting is excavated provides a prop that will reduce wall deflection and bending moments. The chosen construction sequence is therefore as follows:

1) Drive piles (Figure 3.3)
2) Cast stage 1 pile cap (Figure 3.3)
3) Place steel deck beams/cast the concrete deck of the composite section (Figure 3.3)
4) Cast stage 2 pile cap (Figure 3.4)
5) Excavate to motorway formation level (Figure 3.4).

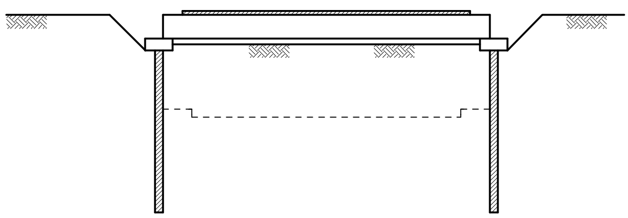
Although bridges in the UK on ‘greenfield’ sites such as this may become a rarity, the same method of embedded walls with deck propping is applicable to replacement motorway and trunkroad bridges where the excavation has been completed before placement of the deck. This construction sequence illustrates a greater utilisation of soil/structure interaction to provide economies in wall construction than the more common sequence of deck construction after soil excavation. The alternative construction sequences are discussed throughout the document where appropriate.



Stage 1: Drive High Modulus Piles

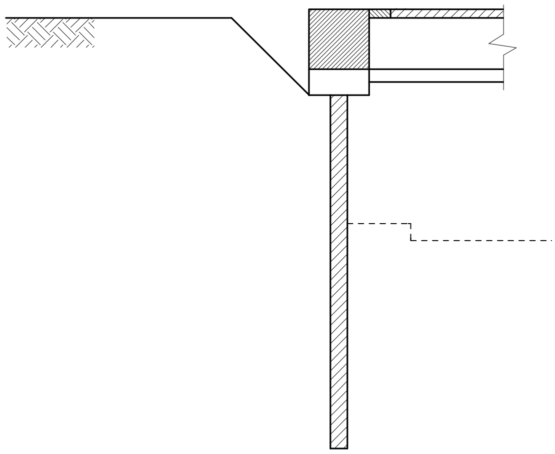


Stage 2: Cast lower part of pile cap

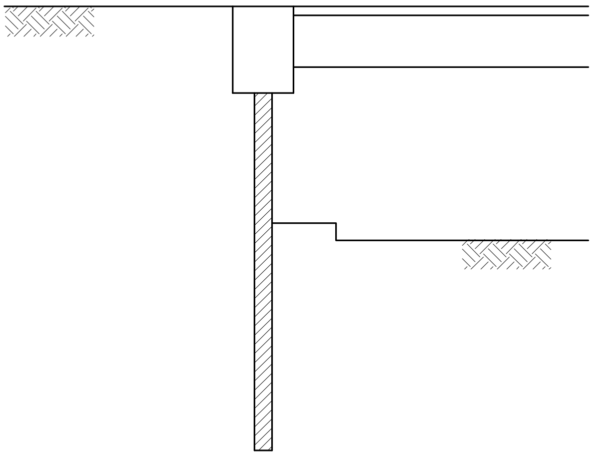


Stage 3: Place deck beams and cast deck

Figure 3.3 *Construction sequence, stages 1 to 3*



Stage 4: Cast upper part of pile cap



Stage 5: Excavate to formation level

Figure 3.4 *Construction sequence: stages 4 to 5*

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CALCULATION SHEET

Job No:	OSC397	Page 1 of 50	Rev A
Job Title	Design of a single-span integral steel bridge		
Subject	General arrangement		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

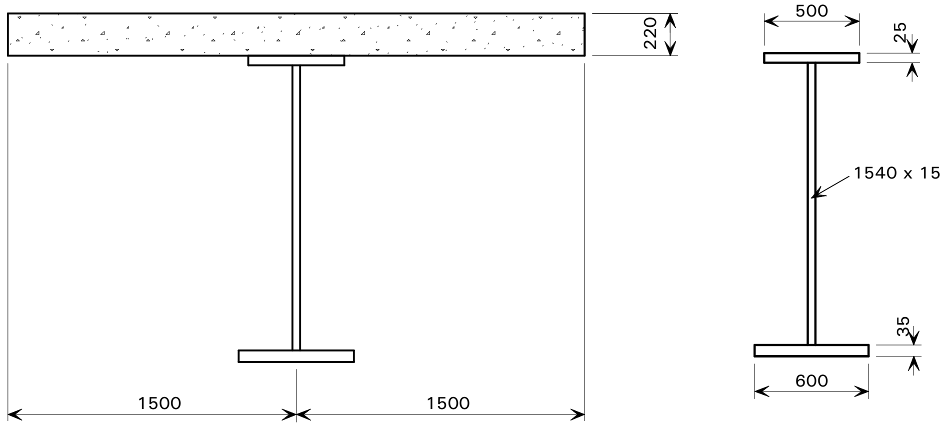
RETAINED HEIGHT (Ref fig 3.1)

Retained height = clearance + depth to beam centroid
= 5.7 + 1.2

= 6.9 m

For simplicity assume = 7.0 m

It is assumed that the position for the prop is at the centroid of the composite beam



Deck girder dimensions

Girder dimensions - Overall depth approximately span/20, plate sizes similar to those featured in the SCI publication 'Design Guide for Simply Supported Composite Bridges'

SECTION PROPERTIES - INNER COMPOSITE GIRDER

All properties are in 'steel units'

Assume short term modular ratio = 6.61
For cracked section properties assume two layers of T32 at 150 centres

Section	A (cm^2)	$\bar{y}$ (cm)	I_{xx} (cm^4)	Z_{bf} (cm^3)	Class
Short-term	1564	133.9	6274000	46840	Compact
Long-term	1065	116.5	5247000	45010	Compact
Cracked	880.6	105	4580000	43630	Non-compact

From spreadsheet (not included)

3.4 Loading

The calculations present a simplifed assessment of the deck loading for the purposes of this Worked Example. Therefore all loads have been calculated as a UDL per linear metre of deck girder to suit a 2-dimensional model using the FREW program.

For simplicity only HA loading (to BD 37/88) has been applied, since the FREW method is particularly suitable for symmetrical load cases. Asymmetrical load cases can be analysed, but this requires additional modelling.

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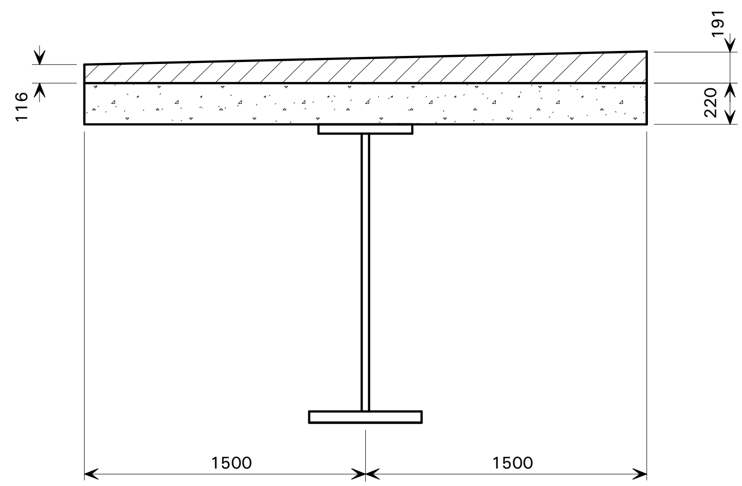
CALCULATION SHEET

Job No: OSC397	Page 2 of 50	Rev A
Job Title	Design of a single-span integral steel bridge	
Subject	Loading	
Client	Made by JAW	Date Dec 1996
	Checked by EDY	Date Dec 1996

DECK DEAD LOAD - consider inner girder only

Inner girder



Section through deck (inner girder)

1. Girder: $(500 \times 25 + 1540 \times 15 + 600 \times 35) \times 78.5 \times 10^{-6}$ $= 4.4$ kN/m

2. Slab: $3 \times 0.22 \times 25$ $= 16.5$ kN/m

3. Surfacing: $3 \times (0.116 + 0.191)/2 \times 24$ $= 11.1$ kN/m

4. Permanent formwork: $(3 - 0.4) \times 0.5$ $= 1.3$ kN/m

Total unfactored load $=$ *33.3 kN/m*

ULS factored load $= 4.4 \times 1.05 + 16.5 \times 1.2$

$+ 11.1 \times 1.75 + 1.3 \times 1.2$

$=$ *45.4 kN/m*

The Steel Construction Institute	Job No: OSC397	Page 3 of 50	Rev A
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944	Job Title: Design of a single-span integral steel bridge		
	Subject: Loading		
CALCULATION SHEET	Client	Made by JAW	Date Dec 1996
		Checked by EDY	Date Dec 1996

LIVE LOADING

HA UDL

Loaded length = *33 m*

$$w = 336 \left(\frac{1}{33}\right)^{0.67}$$

BD 37/88 Clause 6.2.1

$$= 32.3 \text{ kN/m of notional lane}$$

As UDL/girder $= 32.3 \times \dfrac{3}{3.65} = \underline{26.5 \text{ kN/m}}$

HA KEL

KEL = *120 kN per notional lane*

BD 37/88 Clause 6.2.2

$$= 120 \times \frac{3}{3.65} = 99 \text{ kN per girder}$$

LONGITUDINAL BRAKING FORCE

BD 37/88 Clause 6.10.1

Nominal braking force for type HA loading:

8 kN/m of loaded length + 250 kN

$= 33 \times 8 + 250 = 514$ kN *per notional lane*

As a load per m of wall $= \dfrac{514}{3.65} = \underline{141 \text{ kN/m}}$

Spreading the load over 3.65 m is conservative - it could be spread over a greater width of wall

LOADING DUE TO WEIGHT OF SOIL

Vertical and horizontal pressures depend on the weight of soil. Unit weights are given on sheet 4.

PARTIAL LOAD FACTORS

Unfactored loading has been calculated at this stage for inclusion in the FREW model (see Section 3.9.3). Structural effects from FREW have subsequently been factored by the appropriate values of γ_{fL} from BD 37/88.

3.5 Soil parameters

The importance of reliable soil parameters is well established. For fully integral bridges this information is even more important, since the soil parameters used in the analysis will directly affect the forces for which the structure is designed.

The parameters used in this worked example are assumed to have been established by appropriate laboratory and *in situ* testing. The values actually used have been taken from the best reference sources currently available and should be similar to those found by testing. Establishing reliable values for the Young's modulus and Poisson's ratio parameters which are required in soil analysis programs such as FREW and WALLAP, and assessing the effects on structural behaviour of their variation, will provide a challenge to both geotechnical and structural engineers. For the purposes of this Worked Example it is assumed that these parameters are accurately and reliably known.

The soil profile assumed in the Worked Example is illustrated in Figure 3.5. It is predominantly composed of overconsolidated London clay, with the first 5 m made up of terrace gravel.

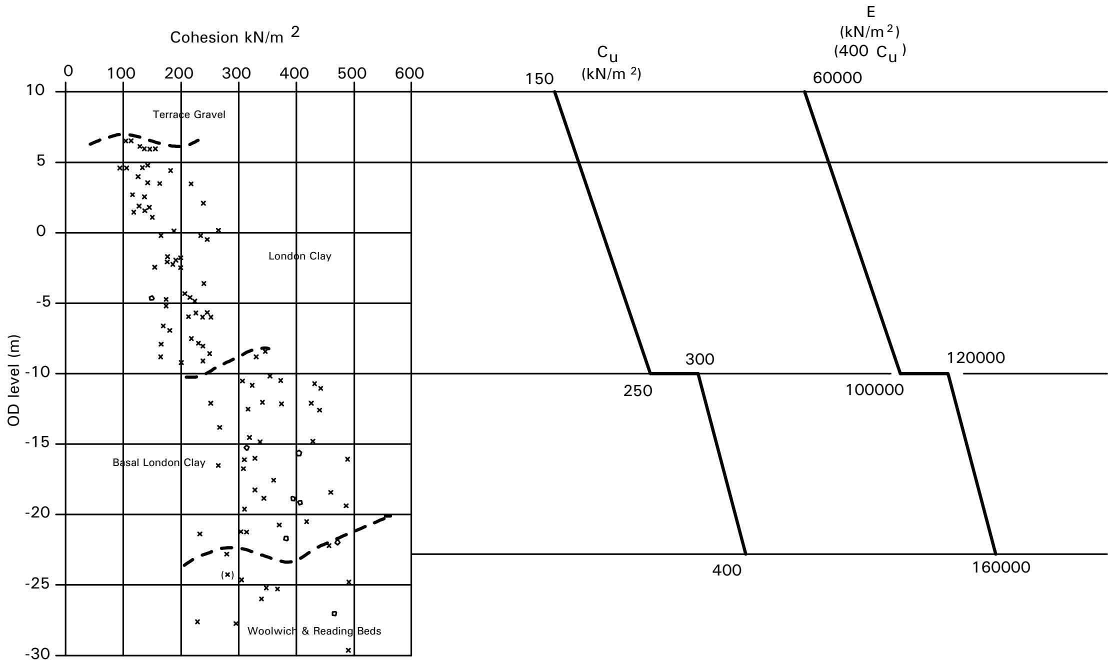


Figure 3.5 *Soil profile: Undrained shear strength and elastic modulus*

The Steel Construction Institute	Job No: *OSC397*	Page *4* of *50*	Rev *A*
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944	Job Title *Design of a single-span integral steel bridge*		
	Subject *Soil profile*		
CALCULATION SHEET	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*

SOIL PARAMETERS

Layer	*Bulk* γ_w	c'	ϕ	k_a , k_p	k_{ac} , k_{pc}
Terrace gravel	*21 kN/m*^{3}	*0*	*30°*	*Caquot & Kerisel*	-
London clay	*19 kN/m*^{3}	*20 kN/m*^{2}	*20°*		$2\sqrt{k_a}$ (k_p)

Characteristic values (see Section 3.1.2)

Young's modulus:

- *Terrace gravel, i.e. normally consolidated/cohesionless: varies linearly with depth, for a medium/dense material typically 4950 kN/m*^{2}*/m (WALLAP manual, written by D.L. Borin, published by Geosolve).*
- *London Clay: Related to* c_u *- Young's modulus* $= 400 \times c_u$.

Poisson's ratio (drained analysis):

- *Terrace gravel - typical value in the range 0.2-0.3, selected 0.25*
- *London clay - typical value in the range 0.1-0.2, selected 0.15*

IN SITU EARTH PRESSURE

Jaky's formula

1. *Terrace gravel* - *Normally consolidated*

   - $k_o = 1 - \sin\phi$

   $= 0.5$

2. *London clay* - *Over-consolidated*

   k_o *varies* - *@ 5 m O.D.* $k_o = 3$

   @ 0 m O.D. $k_o = 1.5$

   @ −15 m O.D. $k_o = 1.0$

WALL FRICTION

1. *Retained (active) side:* *Assume zero wall friction, i.e.* $\delta = 0°$

2. *Excavated (passive) side:* *Assume* $\delta = \frac{1}{2}\phi$

Since the wall will displace downwards under deck loading, it is considered unconservative to allow for wall friction on the active face

3.6 Stability analysis

The purpose of the stability analysis is to determine a wall configuration that will:

- Prevent an overall failure mechanism in the soil mass.
- Limit in-service displacements.

3.6.1 Method of analysis

Many different methods have been developed for calculating factor of safety against overturning of cantilever and single propped walls, each utilising limit equilibrium techniques. The forthcoming SCI publication *Design guide for steel sheet piled bridge abutments* provides additional information.

For the Worked Example, the computer program ReWaRD was used to carry out the stability analysis. ReWaRD is available through British Steel. (See Section 1.1).

ReWaRD is a limit equilibrium-based retaining wall analysis program that, to quote British Steel, "draws on the results of comprehensive research studies on retaining walls at Imperial College, and combines these with a considerable body of practical design experience". It is a user friendly program that works within the Windows environment.

ReWaRD provides analysis according to four methods:

- Gross Pressure Method.
- Net Pressure Method.
- Burland/Potts Method.
- Factor-on-Strength Method.

The aim in selecting the most appropriate method was to ensure that a consistent approach was adopted throughout the Worked Example. Since the structural design would be carried out to limit state principles, it was decided that a similar approach should be adopted when considering overall ground stability. For this reason the Factor-on-Strength method was chosen.

3.6.2 Factor of safety on strength

In the context of a limit equilibrium analysis, the factor of safety on strength has two functions:

- To make allowance for uncertainties in the evaluation of the soil parameter.
- To ensure that deflections in service are not excessive.

The former can be allowed for either by using a *worst credible* parameter with a factor of unity, or by using a *moderately conservative* parameter with a suitable factor of safety (>1.0).

Since weaker soils produce larger structural displacements at the point of limit equilibrium, service displacements can be limited by the application of a further factor. This additional factor would be applied to either the worst credible parameter or to the moderately conservative parameter.

In practice both functions are grouped together in one *lumped factor* although this practice is at odds with the trend towards discrete *partial factors* in structural engineering (see discussion in the SCI publication *Integral steel bridges: Design guidance*).

The partial factor approach has been adopted in the Worked Example.

3.6.3 Choice of factor for the Worked Example

CIRIA Report 104 states that, based on moderately conservative soil parameters, factors of safety in the range 1.2-1.5 should be chosen but "...usually 1.5 except for $\phi' > 30°$ when lower value may be used..". The Hong Kong Geoguide recommends 1.2 for drained shear strength parameters.

Many engineers are of the opinion that BS 8002 can be implied to recommend a factor of 1.2 (called a *mobilisation factor*), based on worst credible soil parameters. However, in his paper *Earth-retaining structures: Introduction to the Code of Practice*, T.N.W. Akroyd, Chaiman of the BS 8002 drafting committee, emphasises that the purpose of the mobilisation factor is solely to limit in-service deflections.

Eurocode 7 (issued by BSI as DD ENV 1997-1: 1995) makes provision for the design of both the soil stability and the structural behaviour of the retaining wall/soil interaction, using a partial safety factor method. It was therefore considered worthwhile to adopt the approach recommended by that document in the context of the Worked Example. The Eurocode approach was considered to be consistent with other limit state codes (i.e. BS 5400).

Table 2.1 of Eurocode 7 gives partial factors of 1.25 for tan ϕ' and 1.6 for c' (where ϕ' and c' are characteristic values).

The ReWaRD program allows individual factors-of-safety-on-strength (F_S) values to be assigned to each parameter. One value is selected as the variable whilst the other is kept constant. In the Example, the summary of the ReWaRD analysis shows how the required embedded length varies according to the factor chosen for c', when a factor of 1.6 is applied to ϕ'. For F_S = 1.25 the wall length required for stability is 18.2 m. An overall wall length of 19 m was adopted.

3.6.4 Additional considerations

Strictly, all load effects acting on the wall should be taken into account when checking overall stability. Integral bridges differ from simply supported bridges in that forces and moments are also induced at the top of the wall due to expansion, contraction and moment continuity. ReWaRD does not have provision for these effects to be taken into account when assessing stability. However, subsequent investigations using the program WALLAP (for availability see Section 1.1) indicated that deck expansion had a negligible effect on overall stability, and that the effect of moment continuity at the top of the wall actually improved overall rotational stability.

In the Worked Example the deck is placed prior to making the excavation to motorway formation level, and thus acts as a prop to the top of the wall. By propping the wall prior to excavation, the embedment depth required for stability is reduced, which was a significant advantage for the large retained height considered in the Example. A separate analysis indicated that without the prop at this level the embedded length required for stability would be more than double that for the propped case.

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Silwood Park, Ascot, Berks SL5 7QN
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Fax: (01344) 22944

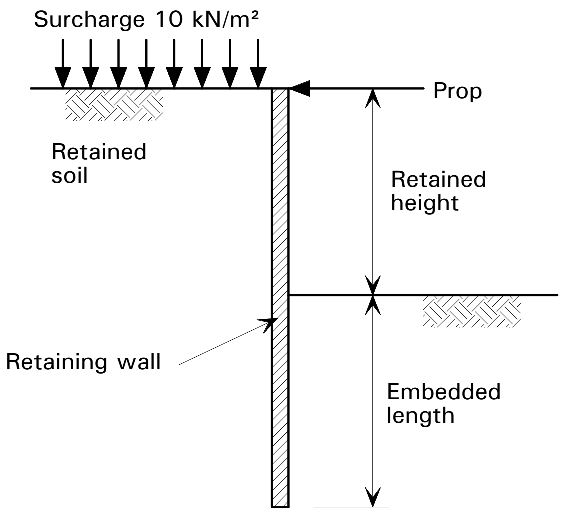
CALCULATION SHEET

Job No: *OSC397*	Page *5* of *50*		Rev *A*
Job Title	*Design of a single-span integral steel bridge*		
Subject	*ReWaRD stability analysis*		
Client	Made by *JAW*	Date *Dec 1996*	
	Checked by *EDY*	Date *Dec 1996*	

RESULTS OF REWARD ANALYSIS

Factor of safety on strength method Clause 5.8.2.1(1)

- *Retained height from calculation sheet 1*
- *Soil properties from calculation sheet 4*
- *Surcharge loading from BD 37/88 - 10 kN/m²*
- *It is assumed that the deck is connected to the wall before excavation and thus acts as a prop for the wall*



Factor of safety on strength criteria

Assume partial factor on c′ of 1.6

As EC7 Table 2.1

ReWaRD analysis gives the following required lengths, for different values of tanϕ′

Factor of safety (tanϕ')	*Wall length (m)*
1.0	*15.4*
1.1	*16.4*
1.2	*17.6*
1.3	*18.8*
1.4	*20.1*

Required factor on tanϕ′ = *1.25*

∴ *Take design length* = *18.8, say 19 m*

Retained height = *7 m*

∴ *Embedded length* = *12 m*

3.7 Wall and capping beam stiffness

3.7.1 Retaining wall

The retaining wall can be considered to exist in two conditions, uncorroded and corroded. It is not immediately obvious which condition will be critical for the structural design, since the reduced stiffness of the corroded wall will in turn reduce the soil pressures acting upon it (and therefore the induced moments and shears). It is safe but unduly onerous to check the capacity of a wall with corroded section properties using the results obtained from the analysis of a wall with uncorroded section properties. Consequently, both sets of section properties were calculated.

Uncorroded section properties

Properties of uncorroded High Modulus Piles are available from a number of sources including British Steel's *Piling Handbook* and the *Steel Designers' Manual*.

Corroded section properties

Because of the geometrical complexity of the Frodingham sheet pile, the section is difficult to model accurately. This is exacerbated by the lack of full dimensioning of the sheet pile in British Steel published literature. However, for companies with AutoCad, British Steel make available a disk containing definitive dimensional information for the range of Frodingham sections. This can then be modified accurately to produce section properties for the corroded section. Alternatively, British Steel will calculate corroded section properties on request.

For the Worked Example, a spreadsheet was used to calculate corroded section properties based on a Frodingham section idealised as flat plates. The dimensions of the plates were based on additional information supplied by British Steel. The stiffness values calculated by the spreadsheet were then compared with corroded section properties calculated by British Steel in-house. This was considered to be sufficiently accurate for use in the Worked Example.

3.7.2 The capping beam

The size of the capping beam was estimated, and its section properties calculated by assuming that the High Modulus Pile and the concrete surround acted fully compositely.

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Telephone: (01344) 23345
Fax: (01344) 22944

CALCULATION SHEET

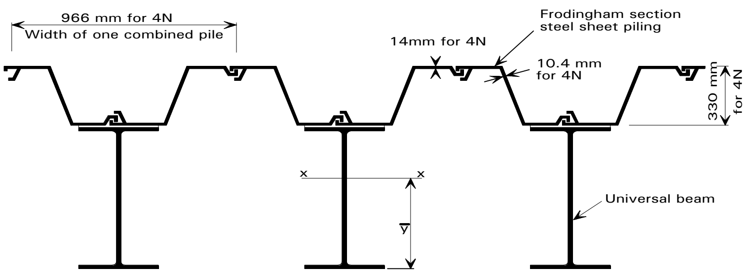
Job No: *OSC397*	Page *6* of *50*	Rev *A*
Job Title: *Design of a single-span integral steel bridge*		
Subject: *Wall and capping beam stiffness*		
Client	Made by *JAW*	Date *Dec 1996*
	Checked by *EDY*	Date *Dec 1996*

SECTION PROPERTIES - HIGH MODULUS PILE

UNCORRODED SECTION PROPERTIES

1. ***High modulus pile selected (based on preliminary design using Limit Equilibrium analysis- not shown) - 4N - 914 × 419 × 388 kg***

Various sources: British Steel Piling Handbook, Steel Designers' Manual



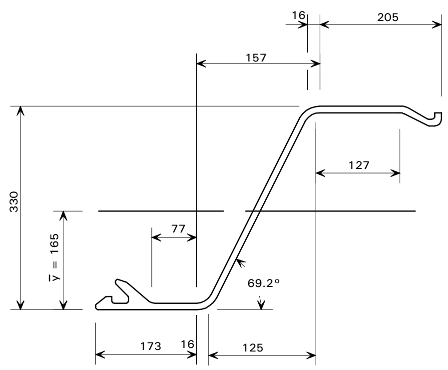
Type 4N High Modulus Pile

A (cm^2)	$\bar{y}$ (cm)	I_{xx} (cm^4/m wall)	Z_{flange} (cm^3/m wall)
665	*63.13*	*1353000*	*21440*

CORRODED SECTION PROPERTIES — ***BD 42/94 Section 5***

Embedded faces	-	*2 mm corrosion*	*Clause 5.3*
Exposed faces	-	*4 mm corrosion*	*Clause 5.8*

N.B. It is assumed that the sheet pile wall has a non-structural cladding facia and therefore the steel is subjected only to atmospheric corrosion on the exposed face.



Dimensions of type 4N sheet pile

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CALCULATION SHEET

Job No: OSC397	Page 7 of 50	Rev A	
Job Title	Design of a single-span integral steel bridge		
Subject	Wall and capping beam stiffness		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

SECTION DIMENSIONS AFTER CORROSION ALLOWANCE

Universal beam

Top flange		Bottom flange		Web	
B (mm)	*T* (mm)	*B* (mm)	*T* (mm)	*d* (mm)	*t* (mm)
306	34.6	416.5	32.6	851.8	17.5

Frodingham 4N

Outer flange thickness (mm)	Inner (beam) flange thickness (mm)	Web thickness (mm)
8	10	4.4

SECTION PROPERTIES

A (cm^2)	$\bar{y}$ (cm)	I_{xx} (cm^4/UB)	$Z_{flange)}$ (cm^3/m wall)
490.9	55.7	908000	16300

From spreadsheet

Properties are for the gross section after corrosion loss. Properties for the effective section where Frodingham is in compression are slightly less (because of allowance for slender sections).

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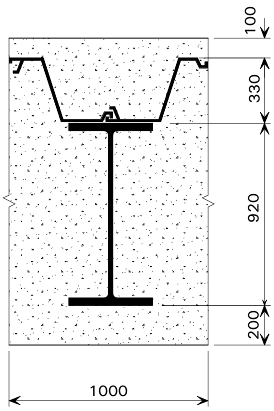
Silwood Park, Ascot, Berks SL5 7QN
Telephone: (01344) 23345
Fax: (01344) 22944

CALCULATION SHEET

Job No: *OSC397*	Page *8* of *50*	Rev *A*	
Job Title	*Design of a single-span integral steel bridge*		
Subject	*Wall and capping beam stiffness*		
Client	Made by *JAW*	Date *Dec 1996*	
	Checked by *EDY*	Date *Dec 1996*	

SECTION PROPERTIES - PILE ENCASED IN CAPPING BEAM



Section through high modulus pile at capping beam level

Second moments of area:

Concrete only:

$$I = \frac{BD^3}{12} = \frac{1000\ ´\ 1550^3}{12} = 3.10 \times 10^{11}\ mm^4$$

High modulus pile only:

British Steel Piling Handbook

$$I = 1.353 \times 10^{10}\ mm^4$$

$\therefore$ ***For concrete section only:***

$I = 3.10 \times 10^{11} - 1.353 \times 10^{10} = 2.965 \times 10^{11}\ mm^4$

E *(high modulus pile)* $= 205 \times 10^6\ kN/m^2$

E *(concrete - short term)* $= 30 \times 10^6\ kN/m2$

Modular ratio $= \dfrac{205}{30} = 6.83$

$\therefore$ ***For composite section:***

$I = 1.353 \times 10^{10} + 2.965 \times 10^{11} / 6.83 = 5.682 \times 10^{10}\ mm^4$

EI *(composite)* $= 5.694 \times 10^{10} \times 205 \times 10^6/1 \times 10^{12}$

$= 1.165 \times 10^7\ kNm^2/m$

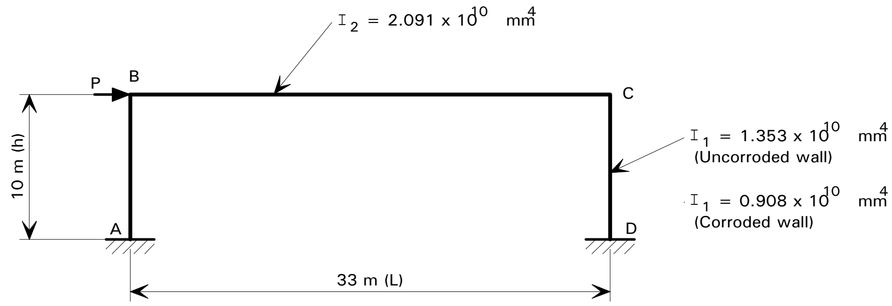
3.8 Analysis of longitudinal loading

The formulae for rigid frames developed by Professor Kleinlogel (see *Steel Designers' Manual*) have been used in the Worked Example in order to assess the effects of longitudinal load due to braking and traction. The soil/structure interaction has been simplified to a portal frame with a fixed base. The level of the fixed base has been taken as 3 m below formation level (after McShane, see 'Steel piling' - Section 1.1).

The Example calculates the moments at the wall/deck junction acting as a portal frame without restraint from the soil and makes the conservative assumption that the load is carried by only 3.65 m width of wall (it could be argued that the full width of wall is mobilised). This overestimates the moments. Further calculations (not shown) indicated that, under this loading, the sway deformation would be of the order of 4 mm. The restraint offered by the soil would reduce the value of moments and displacement by around 40%.

The Steel Construction Institute Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944 **CALCULATION SHEET**	Job No: *OSC397*	Page *9* of *50*	Rev *A*
	Job Title *Design of a single-span integral steel bridge*		
	Subject *Analysis of longitudinal loading*		
	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*

Analyse longitudinal braking load using Kleinlogel formulae: — *Steel Designers' Manual*



$$k = \frac{I_2}{I_1} \times \frac{h}{L} = \frac{2.091}{1.353} , \frac{10}{33} = 0.468 \quad \text{- Uncorroded wall}$$

$$\text{and} = \frac{2.091}{0.908} , \frac{10}{33} = 0.697 \quad \text{- Corroded wall}$$

$$N_2 = 6k + 1 = 3.808 \quad \text{- Uncorroded}$$

$$\text{and} = 5.182 \quad \text{- Corroded}$$

From Kleinlogel graph,

hogging moment at capping beam $= M_c = \dfrac{Ph}{2} , \dfrac{3k}{N_2}$

$$\frac{3k}{N_2} = 0.368 \quad \text{- uncorroded}$$

$$\text{and} = 0.404 \quad \text{- corroded}$$

$$P = 141 \text{ kN}$$ — *Calculation sheet 3*

$$\frac{Ph}{2} = \frac{141 \times 10}{2} = 705 \text{ kNm}$$

$$\therefore \text{ For uncorroded wall, } M_c = 705 \times 0.368 = 259 \text{ kNm/m}$$

$$\text{For corroded wall, } M_c = 705 \times 0.404 = 285 \text{ kNm/m}$$

3.9 Numerical analysis

In a fully integral bridge there is interaction between the bending of the deck and the abutment wall, and displacement of the retained earth. The retained earth acts to stiffen the retaining wall, picking up load and producing greater fixed-end moments at the deck/abutment connection; correspondingly it reduces the sagging moments at mid-span. A realistic distribution of forces within the integral bridge therefore requires a more sophisticated analysis than the limit equilibrium methods used by programs such as ReWaRD. Modern retaining wall analysis programs, which use numerical analysis techniques, are able to model the interaction between structure and soil. Two widely available programs of this type are FREW and WALLAP. The use of these programs to analyse integral bridges is described in the following sections.

3.9.1 Two-dimensional modelling

Since a full 3-D model (taking account of the transverse distribution of the deck and wall, in addition to longitudinal distribution) would require powerful (and expensive) software, two-dimensional modelling is employed in this Worked Example. Such modelling is adequate where each of the main girders carries similar loading (i.e. the loading is roughly uniform across the bridge). Some account could be taken of 3D effects by use of grillage models of the deck. This is explained in Section 3.9.7.

3.9.2 The FREW model

After examining the functionality of the available embedded retaining wall analysis software, it was decided that the program that most clearly illustrates the soil structure interaction of integral bridges is FREW (**F**lexible **Re**taining **W**all).

General

FREW has the advantage over alternative programs (e.g. WALLAP) that stiffness values can be varied between nodes. The level and separation of the nodes is set by the user, and the user is able to extend the model above ground level. Stiffness of individual elements can be varied between nodes. Thus wall, deck and capping beam can be modelled individually.

In the FREW model, the soil structure interaction is modelled by rotating half of the deck span into the vertical and restraining its top (i.e. midspan) with a moment fixity. Moment fixity is modelled using eccentrically applied props with infinite stiffness. This restraint condition implies both a symmetric structure and symmetric loading. Asymmetric loading could be modelled by replacing the moment restraint with a fixed pin. The propping effect of the deck is modelled by applying a prop at *deck level* (i.e. the top of the retaining wall). Deck loading is also applied using props (see Section 3.9.4).

The unit width of wall modelled by FREW is 1 m.

The use of FREW in the analysis of an integral bridge (the Stockley Park Canal Bridge, near Heathrow) has been referred to in a number of papers by A. Low.

Soil model

FREW is able to model soil behaviour using one of three methods:

- Subgrade reaction model - the soil is represented as a set of non-interactive springs, one at each node position.
- SAFE flexibility method - the soil is represented as an elastic solid with the soil stiffness matrices being developed from pre-stored stiffness matrices calculated using the 'SAFE' finite element program. This method is ideally suited to a soil with linearly increasing stiffness with depth, but empirical modifications are used for other cases.
- Mindlin method - the soil is represented as an elastic solid with the soil stiffness based on integrated forms of the Mindlin Equations. This method can model a wall of limited length in plan but is ideally suited to a soil with constant stiffness with depth, but again empirical modifications are used for other cases.

Of these three methods the SAFE flexibility method was considered to be the most appropriate, since it models interaction between soil layers, whilst permitting the soil model to be created with relative ease.

It is necessary to model the variation of E and K_0 with depth. A limitation of FREW is that provision has only been made for a linear variation of these parameters between the surface and the bottom node. Layers with differing E values cannot be modelled in this way. Variation of E and K_0 between layers can be approximated by dividing soil strata into sub-layers with defined E and K_0 values. The program will then generate a best fit elastic profile. It should be noted that if soil layers differ markedly from the best fit elastic profile the analysis may be inaccurate. Soil layer interfaces must occur midway between nodes, and this must be taken into account when planning the arrangement of the model.

Poisson's ratio is entered in the form of K_R defined as: $\nu/(1-\nu)$. The Poisson's ratio used in the design is that for the drained condition. Basic parameters cannot be varied for different stages of the analysis, so a choice must be made whether drained or undrained conditions will be analysed. For the retaining wall the design effects are dominated by the excavation load-case, which is critical in the long term.

3.9.3 Modelling construction sequence and subsequent loading

In modelling the integral structure, it must be recognised that some loadings are applicable to a model with short-term deck properties (i.e. the live load), some to a model with long-term deck properties (dead and superimposed loads after making the moment conection) and possibly some to a model with no moment connection (but with the deck acting as a strut). Additionally, asymmetric loadcases applied to a 'half-bridge' model must be analysed as symmetric and anti-symmetric components, with the two components applied to models with different boundary conditions.

However, since it is impossible to separate the strength of the soil from its self-weight, any loadcase must include the soil in its excavated condition. To determine load effects due to a particular loading, a model with appropriate properties and boundary conditions must therefore be analysed for both the excavation case alone and with the particular loading. The difference between the two sets of load effects is then the effects of the loading that is to be considered. Total load effects on the integral structure are then calculated as the summation of all the (factored) loadings for the combination being considered. This effectively requires that the principle of superposition is applied. That principle is valid when behaviour is linear, but will have some degree of inaccuracy when behaviour is non-linear. To test the sensitivity to non-linear behaviour of the retained soil, three alternative approaches were examined for one model:

- Unfactored loadcases were applied seperately and the effects due to each were factored before adding them together.
- Separate factored loadcases were applied and the effects combined.
- Combinations of factored loadings were applied.

For the model in this Example, each method produced almost identical results, which indicates that the soil model behaved linearly for the range of loading examined. Clearly, the first approach is the easiest to deal with in design and, in general, is recommended provided the linear response of the soil is verified.

3.9.4 FREW model loading in the Worked Example

Excavation

Earth pressure loading resulting from excavation to formation level is the simplest load to model, since this is the type of loading that the program is specifically designed for. Once the structure has been defined, earth pressures and resulting structural effects are automatically calculated by defining the excavation level. The excavation loadcase has been modelled using long-term properties for the deck stiffness.

If excavation had been assumed to take place before the deck is in place (or before it is connected to the top of the wall) this could have been modelled by omitting the prop at deck level for this stage. This would of course have led to higher moments in the wall.

Also, the deck could be assumed to be a pinned prop, or to be a full moment connection, prior to excavation. These options can be modelled by either removing or inserting respectively the rolling moment fixity (eccentric prop) at *midspan* i.e. the top node of the model (see calculation sheet 11).

FREW results are presented on calculation sheets 17, 18 and 19 for the case of full moment connection. In the calculations relating to the deck, load effects for both options are given (see calculation sheet 32).

Deck dead loads

Since the deck beams will be placed, and the deck slab cast, prior to casting the moment connection at the pile cap, deck dead loads are not applied to the FREW model. They have therefore been modelled separately, as a simply supported load-case. In this Worked Example, because of the simplicity of the arrangement, a hand calculation method was used. Moments and shears were established at FREW node positions, in order to assist post-analysis.

Discuss me ...

Live load

Deck loads have been modelled using props with zero stiffness, and prestress set to the load value. The props can only be applied at node positions, thus UDLs have to be simplified as a set of point loads. The number of nodes could be increased to refine the analysis. Live load has been modelled using short-term properties for the deck stiffness.

Surcharge

Surcharge can be specified automatically in the program. A UDL of intensity 10 kN/m^2 was applied, corresponding to the HA loading (BD 37/88). The surcharge can be modelled as a separate load-case (without the prestressed props representing the live load on the deck). Surcharge has been modelled using short-term properties for the deck stiffness.

Deck expansion and contraction

As a result of the integral abutment connection, deck expansion induces, in addition to axial loads, uniform hogging in the deck and contraction induces uniform sagging moments. A load combination involving these actions may therefore result in a worst case for design.

In the Worked Example a 'unit' displacement of 10 mm was applied at deck level, both as an expansion and a contraction. The thermal strain given in BA 42 (0.0005) corresponds to a movement at each abutment of 8.5 mm. The actual movement experienced by the integral structure depends on interaction with the retained soil (see Figure 3.6), but generally is unlikely to be much constrained by the soil; for simplicity, the Example includes, in the total design moments, an allowance for the effects of displacements of 7 mm. An example of the determination of the displacement and force at equilibrium is included in the calculations relating to the use of the program WALLAP (see Section 3.9.6).

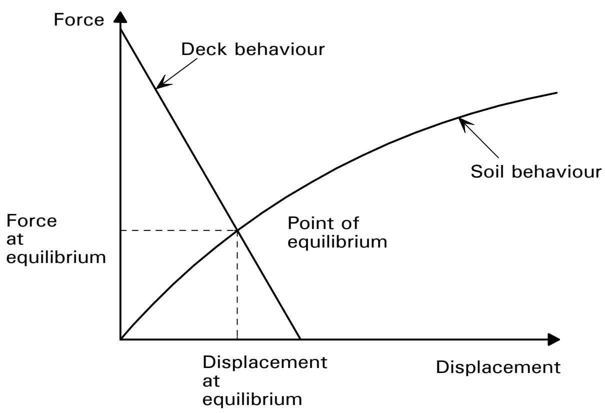


Figure 3.6 *Equilibrium condition between deck and soil*

Displacements are applied directly to the FREW model by inserting a prop with appropriately proportioned values of stiffness and prestress. The stiffness of the prop is measured as load per metre change in prop length per metre width of wall. Since an expansion/contraction of 10 mm is required, it follows that the prestress should be 1/100th of the strut stiffness per metre. Since the deck still acts as a stiff strut after expansion, the stiffness of the prop should be as high as possible. The highest prestress capable of being applied to a prop in FREW is 9.9×10^6 kN/m and this value was used in the analysis.

Differential temperature

Differential temperature cannot be modelled directly using FREW, although secondary moments due to the specified differential temperature loading could be applied as a loading at the top of the wall.

3.9.5 FREW analysis and results

Two FREW models were created, one with uncorroded section properties for the retaining wall, and another with corroded section properties. By applying the midspan moment restraint at the appropriate stage, the effect of creating the full moment connection between the deck and the abutment both before and after soil excavation was examined. The capping beam was modelled by placing its section properties between nodes 9 and 11, effectively 2 m into the deck and 2 m into the retaining wall (see calculation sheet 10). Since the actual stiffness of the capping beam was uncertain, the FREW model was also run without the capping beam, in order to provide a worst case condition for sagging in both the wall and the span.

Worst case design moment for hogging at the capping beam level was found to be produced when the retaining wall was in its corroded condition. The design moment was produced by the following load combination:

excavation + dead load + live load + surcharge + expansion

Deck expansion is a Combination 3 load-case in BD 37/88; the HA loading is subject to a reduced load factor in this combination.

The ULS load due to deck expansion resulted in larger hogging moments at the capping beam than did longitudinal loading; load combination 4 was therefore not examined further.

3.9.6 The WALLAP model

The WALLAP software analyses a uniform wall beam-element subject to earth pressures and to imposed forces and moments along its length. It is more able than FREW to model soil properties that vary with depth, but it cannot give a direct model of the interaction between wall and deck nor can it allow for a different stiffness over the depth of the capping beam. For interaction a separate deck analysis is needed, to determine the effective spring stiffnesses and fixed-ended moments. The idealisation and the two models are shown diagrammatically in Figure 3.7.

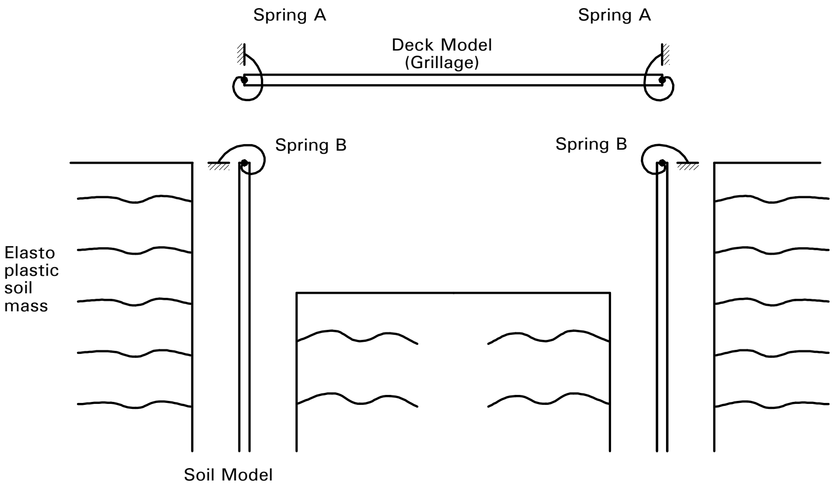


Figure 3.7 *Idealisation of integral bridge when using separate deck and wall models*

In the Worked Example, the WALLAP analysis determines an effective rotational stiffness to moments applied at the top of the wall. A separate analysis of a line beam, representing a deck girder, calculates the rotational stiffness of the deck (the familiar $2EI/L$ for the symmetric condition of equal moments applied at both ends), together with fixed-ended moments. Assuming that both stiffnesses are linear, the interaction is calculated by applying the opposite of the fixed-ended reactions to the two springs, as in the moment distribution method. Clearly, the result depends on the linearity of the soil response, and this is checked in the Worked Example.

Similarly, the interaction between the horizontal displacement of the wall and the axial strain of the deck is determined by calculating the two stiffnesses and establishing an equilibrium condition. The response of the two elements is shown graphically in Figure 3.6. In the Worked Example, it is shown that the soil has little restraining effect on the expansion of the deck and that the soil response in that range of displacement is effectively linear.

The force-displacement line for the deck is straightforward to evaluate; two points are calculated, for the case of a 20 mm expansion of the deck (i.e. 10 mm at each end). These are:

1. Displacement = 10 mm, Force = 0 kN

2. Displacement = 0 mm, Force = εAE kN
   where ε = strain in deck corresponding to 10 mm expansion over the half length.

The force-displacement line for the soil mass is potentially non-linear and is determined as a series of points, using the WALLAP model.

WALLAP does not permit more than one prop to be created at any one level. In addition, the removal of one fixed prop and its subsequent replacement with a prestressed prop must each form a separate construction stage, thus the retaining wall will either behave as a cantilever, or the prestress will be carried by the existing strut.

The most effective way of modelling deck expansion is to apply a horizontal force at the deck centroid position. The following sequence has been followed in the Worked Example:

- Run excavation stage with a stiff prop at deck level - WALLAP calculates the strut force.
- Re-run the WALLAP model, replacing the stiff prop with the horizontal force. The strut force is the horizontal force which, applied at deck level, will produce zero displacement.
- Increase the horizontal force in increments, and determine the corresponding displacements at deck level. The difference between the strut force and the original horizontal force is the reaction of the soil to deck expansion.
- Plot soil reaction against soil displacement and determine the intersection of this curve with the deck force/displacement line. This gives the force and displacement in the integral bridge at equilibrium for the assumed thermal strain.

The horizontal force could be applied in increments up to the fully restrained force in the deck but in practice this value is unlikely to be achieved.

3.9.7 Grillage-based analysis

Whilst the 2-dimensional FREW analysis with deck elements is suitable for illustrative and preliminary design purposes, the majority of deck designs will require a grillage analysis to establish a more detailed distribution of forces. As yet there are no programs that allow a three dimensional soil-structure model to be produced easily. Consequently, the behaviour of the abutments and bridge deck will have to be modelled separately.

This can be achieved in a similar manner to that for the two-dimensional analysis using WALLAP, as outlined in Section 3.9.6. However, instead of determining fixed end moments and stiffnesses for a line-beam, stiffnesses and moments will need to be determined at the end of each main girder. Additionally, the wall stiffness will have to be that for a width of wall equal to the spacing of the main girders.

Account is taken of interaction of the grillage model and the numerical soil model by establishing appropriate boundary conditions for the grillage and wall models. Boundary conditions are provided at the connection between the two models. Both FREW and WALLAP are capable of being used in such an analysis.

The rotational stiffness of the combined wall and soil can be calculated using the FREW or WALLAP models, by applying moments directly to the top of the wall and measuring the resulting rotation. It is suggested that the full fixed end moment is applied as an upper bound, in order to first establish whether the soil response is linear or not within the expected range of moments.

The rotational stiffness of decks with varying section properties can be established in a similar manner by applying a moment and determining the resulting rotation at the end of a grillage or frame model. Appropriate boundary conditions for both symmetric loading (i.e. excavation) and asymmetric loading (i.e. longitudinal deck loading) can be established in this way.

During the analysis, the values of the respective boundary forces are transferred between models. This process is illustrated by the flowchart in Figure 3.8.

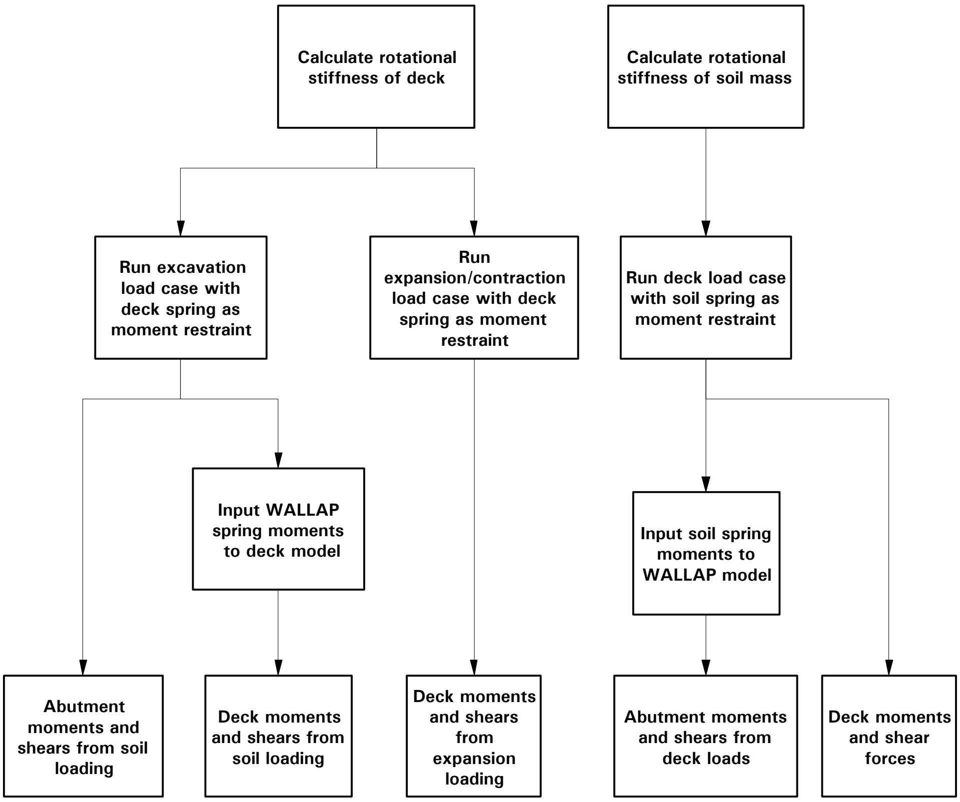


Figure 3.8 *Suggested analysis sequence when using a grillage analysis*

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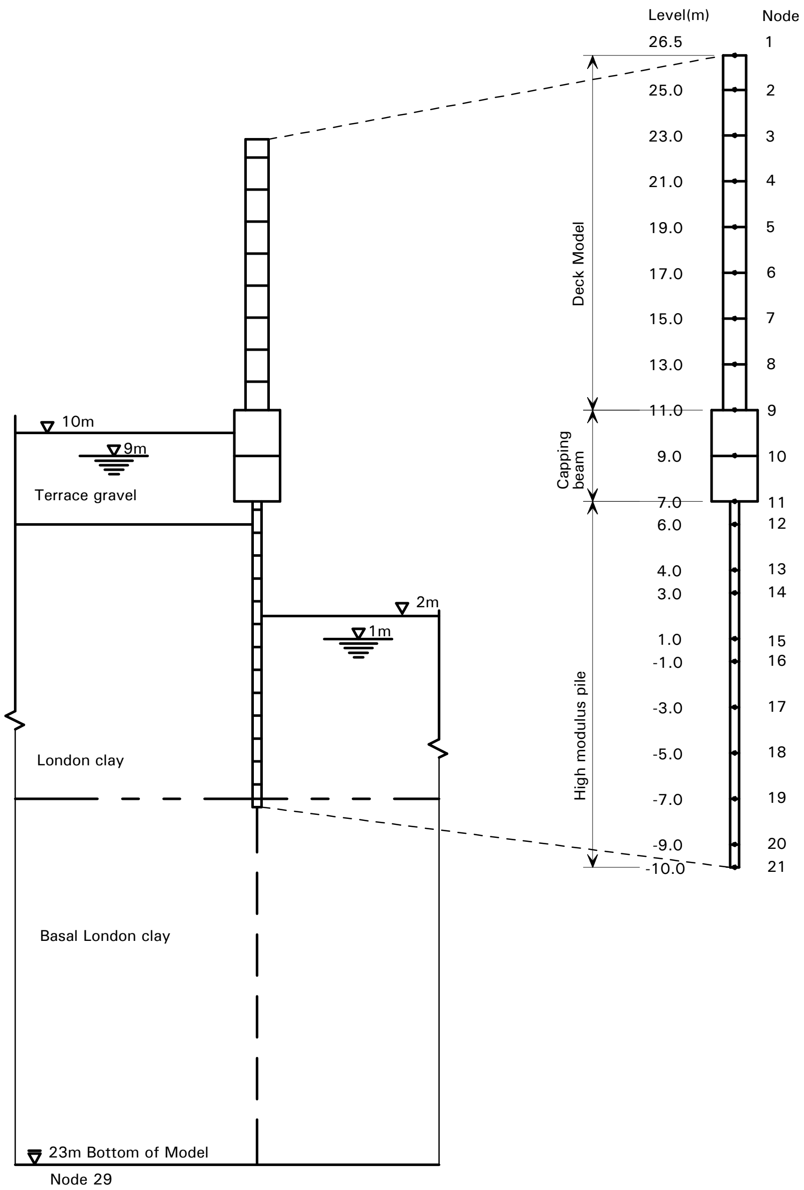
CALCULATION SHEET

Job No: *OSC397*		Page *10* of *50*	Rev *A*
Job Title	*Design of a single-span integral steel bridge*		
Subject	*FREW model*		
Client	Made by *JAW*		Date *Dec 1996*
	Checked by *EDY*		Date *Dec 1996*



FREW model (representing 1 m width of wall)

- *Soil properties as given on Sheet 4*
- *Wall and capping beam stiffnesses as on Sheets 6, 7, 8*
- *Girder stiffness as on Sheet 1, divided by beam spacing of 3 m.*

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CALCULATION SHEET

Job No:	*OSC397*	Page	*11* of *50*	Rev	*A*
Job Title	*Design of a single-span integral steel bridge*				
Subject	*FREW model*				
Client		Made by	*JAW*	Date	*Dec 1996*
		Checked by	*EDY*	Date	*Dec 1996*

APPLYING LOADS AND MOMENT RESTRAINTS USING PROPS

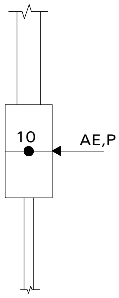
All values are per m width of wall

1. Positional fixity at deck level

Use a single 'stiff' prop

$AE/L = 1 \times 10^8$ *kN/m (v.stiff prop)*

$P = 0$ *kN/m*



The axial stiffness of the deck may be used here if it is considered that this will significantly influence the result.

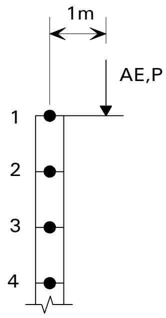
2. Rotational (moment) fixity at midspan of deck

i.e. moment restraint from deck symmetry and symmetric loading

- *Stiff and eccentric prop*

$AE/L = 1 \times 10^8$ *kN/m*

$P = 0$ *kN/m*

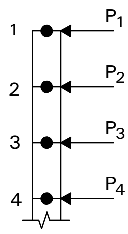


3. Point loads on deck

i.e. HA UDL, KEL, HB45, etc.

Use props with zero stiffness, $AE = 0$

Apply prestress = Applied load, P



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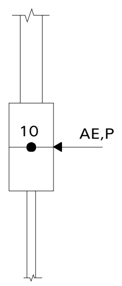
CALCULATION SHEET

Job No: OSC397	Page 12 of 50	Rev A
Job Title	Design of a single-span integral steel bridge	
Subject	FREW model	
Client	Made by JAW	Date Dec 1996
	Checked by EDY	Date Dec 1996

4. *Applied displacement at top of wall (deck expansion or contraction)*

- *Appropriate ratio of stiffness to prestress applied at 'deck' level*
- *Stiffness should be high in order to produce zero displacement under earth pressures*
- *See Section 3.9.4 for the basis of these values*

i.e. 10 mm expansion:



$AE = 9.9 \times 10^8$ kN/m/m

$P = 9.9 \times 10^6$ kN/m

10 mm contraction:

$AE = 9.9 \times 10^8$ kN/m/m

$P = - 9.9 \times 10^6$ kN/m

DECK LOADING

Deck loads can only be applied at node positions. Live loading is calculated on sheet 3. The load per girder is divided by the girder spacing, to correspond to the 1 m unit width of the FREW model (it is assumed that the capping beam distributes deck loads uniformly into the High Modulus Pile). The HA UDL is then apportioned to each FREW node, in accordance with the node spacing.

HA UDL = 26.5 kN/m of girder (unfactored) *Sheet 10*

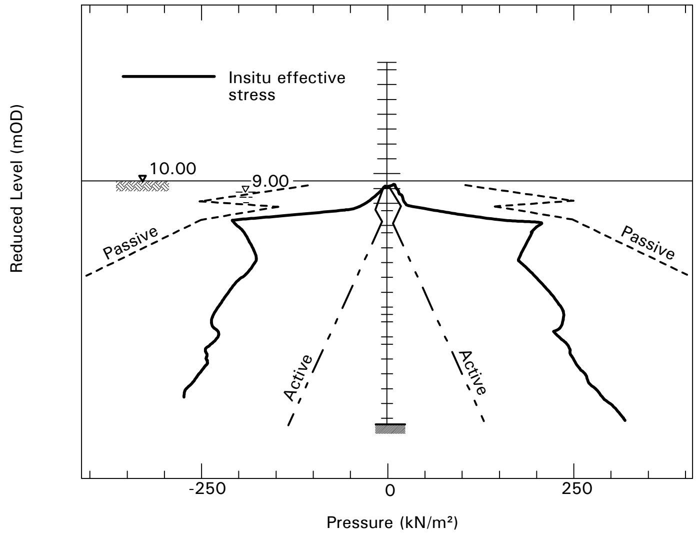
HA KEL = 98.6 kN per girder (unfactored)

Frew Node	1	2	3	4	5	6	7	8	9
HA UDL per girder	20	46	53	53	53	53	53	53	53
HA UDL per m wall	7	16	18	18	18	18	18	18	18
HA KEL per girder	50	0	0	0	0	0	0	0	0
HA KEL per m wall	17	-	-	-	-	-	-	-	-
Total HA Load (kN)	24	16	18	18	18	18	18	18	18

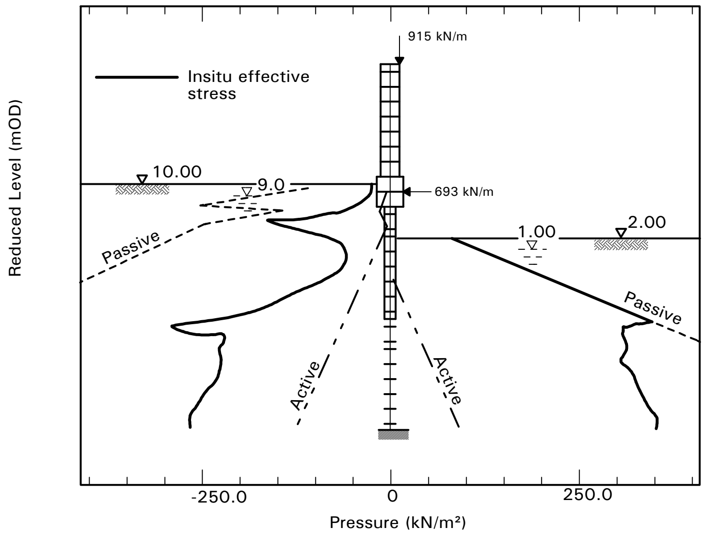
Discuss me ...

The Steel Construction Institute	Job No: *OSC397*	Page *13* of *50*	Rev *A*
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944	Job Title *Design of a single-span integral steel bridge*		
	Subject *FREW results and post analysis*		
CALCULATION SHEET	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*

The figures shown on this page and calculation sheets 14 and 15 present sample output from a FREW model with corroded section properties for the retaining wall. Unfactored soil parameters have been used (as described in the design basis). The figures on this page show the in situ earth pressures both before and after excavation (denoted 'effective'). Limiting active and passive pressures are also shown. Note the reduction in effective pressure on the retaining face of the retaining wall at midspan, and the increase at the toe, following excavation.



In situ earth pressure diagram (long-term deck stiffness)



Earth pressure diagram after excavation (long-term deck stiffness)

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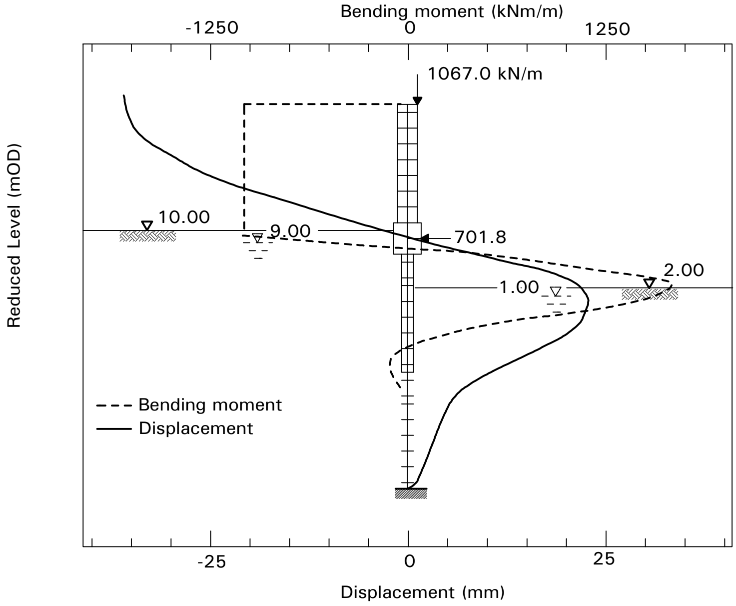
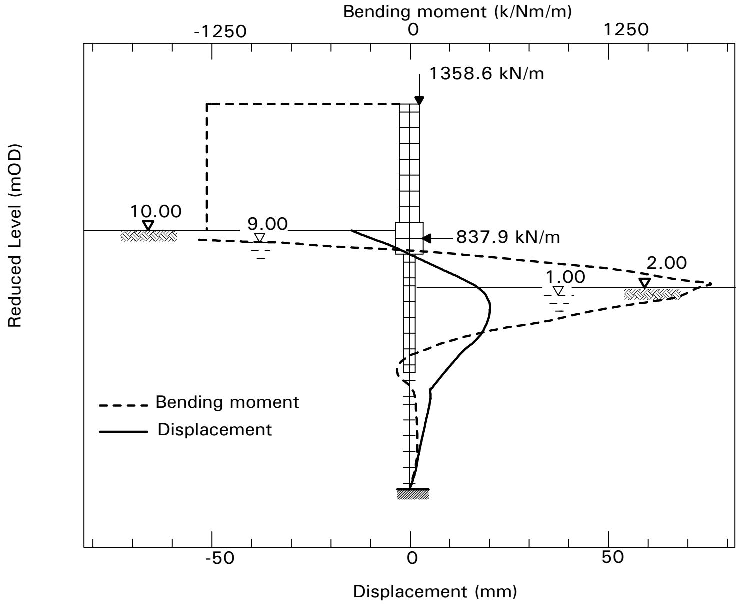
	Job Title *Design of a single-span integral steel bridge*		
	Subject *FREW results and post analysis*		
	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*

The following three figures present moment and displacement for each loading type, according to the FREW analysis.



Bending moment/displacement - 8 m excavation (short-term deck stiffness)



Bending moment/displacement - excavation plus deck expansion of 10 mm

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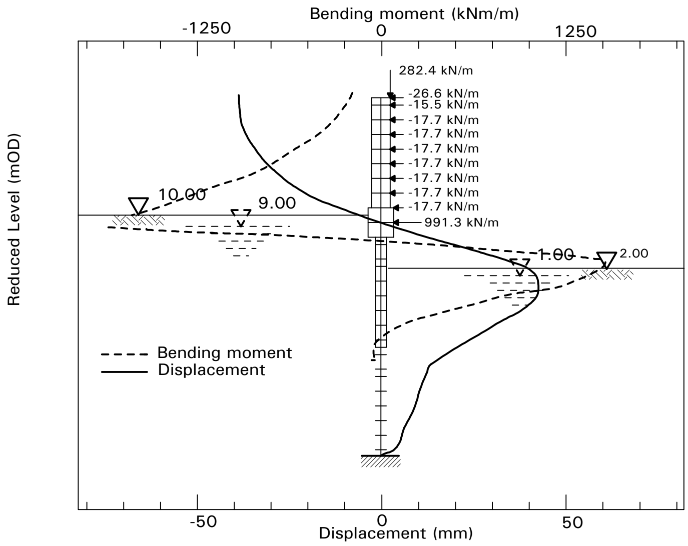
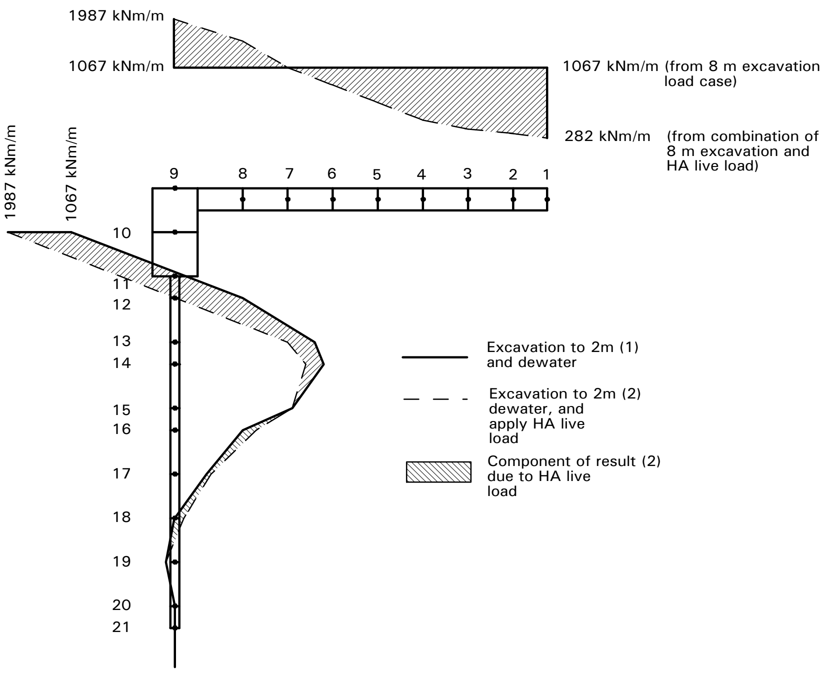
	Job Title *Design of a single-span integral steel bridge*		
	Subject *FREW results and post analysis*		
	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*



Bending moment/displacement - excavation plus unfactored HA live load on deck (9 kN/m of wall)

The results for excavation and live load may be presented as a combined diagram



Evaluation of bending moment due to HA live load only

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CALCULATION SHEET

Job No: *OSC397*	Page *16* of *50*	Rev *A*	
Job Title	*Design of a single-span integral steel bridge*		
Subject	*FREW results and post analysis*		
Client	Made by *JAW*	Date *Dec 1996*	
	Checked by *EDY*	Date *Dec 1996*	

Partial safety factor γ_{fL} is introduced at this point in the calculations to evaluate design load combinations at the ultimate and servicability limit states. In order to calculate the design forces for the structure, consider the following load combinations (the calculations of the bending moments in Table 1, see calculation sheet 17, are fully illustrated):

1) For deck midspan sagging moment:

a) Dead + live + beneficial earth pressure (BD 37/88 combination 1)
b) Dead + live + beneficial earth pressure + deck contraction (BD 37/88 combination 3)

ULS dead load = 45.3 kN/m per girder
= 15.1 kN/m per m of wall

ULS Midspan moment due to dead load $= \dfrac{15.1 \times 33^2}{8}$
$= 2055$ *kNm/m*

Unfactored midspan moment due to excavation (long-term)
$= -915$ *kNm/m (hogging)*
Unfactored midspan moment due to excavation (short-term)
$= -1067$ *kNm/m (hogging)*
Unfactored midspan moment due to HA load
$= (1067 - 282)$
$= 785$ *kNm/m (sagging)*
Unfactored midspan moment due to deck contraction
$= 173$ *kNm/m (sagging)*

For 1(a) ULS midspan moment
$= 2055 + 785 \times 1.5 - 915 \times 1.0 = 2318$ *kNm/m*

For 1(b) ULS midspan moment
$= 2055 + 785 \times 1.25 - 915 \times 1.0 + 173 \times 1.3$
$= 2346$ *kNm/m*

2) For capping beam hogging moment:

(a) Dead + live + earth pressure + surcharge (BD 37/88 combination 1)
(b) Dead + live + earth pressure + surcharge + deck expansion (BD 37/88 combination 3)

Girders placed and deck cast before creating moment connection between deck and wall
$\therefore$ *dead load moment = 0 kNm/m*

Unfactored moment due to excavation (long-term)
$= -915$ *kNm/m (hogging)*

Unfactored hogging moment due to excavation (short-term)
= −1067 kNm/m
Unfactored hogging moment due to HA load = (1987 − 1067)
= −920 kNm/m

Unfactored hogging moment due to surcharge = −17 kNm/m
Unfactored hogging moment due to deck expansion = −204 kNm/m

For 2(a) ULS capping beam moment
$= 0 + 920 \times 1.5 + 915 \times 1.5 + 17 \times 1.5$
$= 2778$ kNm/m

For 2(b) ULS capping beam moment
$= 0 + 920 \times 1.25 + 915 \times 1.5 + 17 \times 1.5 + 204 \times 1.3$
$= 2813$ kNm/m

3) For retaining wall sagging moment:
(a) Dead + earth pressure + surcharge + deck expansion (BD 37/88 combination 3)

Dead load moment = 0 kNm/m

Unfactored moment due to excavation = 1676 kNm/m

Unfactored moment due to surcharge = 19 kNm/m

Unfactored moment due to deck expansion = 169 kNm/m

∴ 3(a) ULS capping beam moment
$= 0 + 1676 \times 1.5 + 19 \times 1.5 + 169 \times 1.3$
$= 2762$ kNm/m

These combinations have been calculated for both corroded and uncorroded High Modulus Piles. The following tables summarise the most onerous results of the combinations of factored load effects:

Table 1: ULS design forces - High Modulus Pile corroded section

Criterion	Moment (kNm/m)	Co-existant shear (kN/m)	Loadcase
Deck midspan sagging	2346	34	1(b)
Capping beam hogging	−2813	514 (deck) 1268 (wall)	2(b)
Retaining wall sagging	2762	88	3(a)

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CALCULATION SHEET

Job No:	OSC397	Page	18 of 50	Rev	A
Job Title	Design of a single-span integral steel bridge				
Subject	FREW results and post analysis				
Client		Made by	JAW	Date	Dec 1996
		Checked by	EDY	Date	Dec 1996

Similarly, moments and shears were also calculated at SLS for the corroded wall section and at ULS for the uncorroded wall section. The results are shown below.

SLS forces will be needed for checks on shear connections

Table 2: SLS design forces - High Modulus Pile corroded section

Criterion	Moment (kNm/m)	Co-existant shear (kN/m)	Loadcase
Deck midspan sagging	1514	27	1(b)
Capping beam hogging	2208	401 (deck) 896 (wall)	2(b)
Retaining wall sagging	1821	46	3(a)

Table 3: ULS design forces - High Modulus Pile uncorroded section

Criterion	Moment (kNm/m)	Co-existant shear (kN/m)	Loadcase
Deck midspan sagging	2395	34	1(b)
Capping beam hogging	−2697	545 (deck) 1327 (wall)	2(b)
Retaining wall sagging	3344	54	3(a)

Table 4: ULS axial load (from net strut force in strut No. 2 of FREW model - High Modulus Pile corroded section)

Load case	Unfactored strut force (kN)	γ_{fl} factor		ULS strut force (kN/m)	
		Comb 1	Comb 3	Comb 1	Comb 3
1. Excavation	693	1.5	1.5	1040	1040
2. Live load (HA)	286	1.5	1.25	429	358
3. Surcharge	20	1.5	1.5	30	30
4. Expansion (7 mm)	95		1.3		124
5. Contraction	−114		1.3		−163
	Total ULS load			1499	1552

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CALCULATION SHEET

Job No: OSC397	Page 19 of 50	Rev A	
Job Title	Design of a single-span integral steel bridge		
Subject	FREW results and post analysis		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

Table 5: SLS axial load (from net strut force in strut No. 2 of FREW model - High Modulus Pile corroded section)

Load case	Unfactored strut force (kN)	γ_{fl} factor		SLS strut force (kN/m)	
		Comb 1	Comb 3	Comb 1	Comb 3
1. Excavation	693	1.0	1.0	693	693
2. Live load (HA)	286	1.2	1.0	343	286
3. Surcharge	20	1.0	1.0	20	20
4. Expansion (7 mm)	95		1.0		95
5. Contraction	−114		1.0		−114
	Total SLS load			1056	1094

SLS forces will be needed for checks on shear connections

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	Job Title *Design of a single-span integral steel bridge*		
	Subject *WALLAP model (illustrative)*		
	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*

The calculation sheets which follow are illustrative of the use of WALLAP to analyse an integral bridge and are based on the general arrangement described in Section 3.2. The results are not used in the subsequent design checks, which are based on the FREW model.

WALLAP ANALYSIS - Response to rotation at capping beam
A full description of the WALLAP software and its use to analyse integral bridges is given in Section 3.9.6.

Check linearity of soil response up to value of fixed-end moment

Fully encastré moment assuming UDL $= \dfrac{WL^2}{12}$

Since one purpose of this calculation is to check soil linearity over the range of applied moments, apply ULS ($\gamma_{fL} \times \gamma_{f3}$) moments to the soil model (unfactored loads from sheets 2 and 3):

ULS dead load $= (4.4 \times 1.05 + 16.5 \times 1.15 + 11.1 \times 1.75 + 1.3 \times 1.2) \times 1.1$
$= 48$ *kN/m per girder*

ULS HA UDL $= 26.5 \times 1.5 \times 1.1 = 43$ *kN/m per girder*

Dead + HA UDL $= 48 + 43 = 92$ *kN/m per girder*

$\therefore$ *Fully encastré moment from ULS UDL* $= \dfrac{92 ´ 33^2}{12}$

$= 8349$ *kNm per girder*

Fully encastré moment from KEL (198 kN @ ULS) $= \dfrac{PL}{8}$

$= \dfrac{198 ´ 33}{8}$

$= 817$ *kNm per girder*

Total encastré moment $= 8349 + 817 = 9166$ *kNm per girder*

Fixed-end moment per m of wall $= 9166/3 = 3055$ *kNm/m*

Results for four values of applied moment up to 3055 kNm/m (no deck spring in model). These results were produced by a model with corroded wall section properties.

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CALCULATION SHEET

Job No: OSC397	Page 21 of 50	Rev A	
Job Title	Design of a single-span integral steel bridge		
Subject	WALLAP model (illustrative)		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

Load No.	Moment (kNm/m)	Wall rotation (mRad)
1	310	0.31
2	1240	1.23
3	2170	2.15
4	3100	3.08

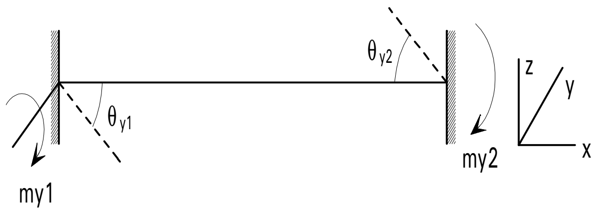
By inpection, rotation is essentially linear for this soil model within ULS range

Stiffness for deck model = *1240/1.23*
= *1008 kNm/mRad per m of wall*

CALCULATION OF DECK STIFFNESS

For a single span deck of constant stiffness

Member stiffness/flexibility equation (from Coates, Coutie, Kong)



Sign convention

$$M_{y1} = \frac{-6EI_y}{L^2} d_{z1} + \frac{4EI_y}{L} q_{y1} + \frac{6EI_y}{L^2} d_{z2} + \frac{2EI_y}{L} q_{y2} \quad (1)$$

Under excavation loadcase, rotation is symmetrical

$$\therefore \theta_{y2} = -\theta_{y1}$$

Also $d_{Z1} = d_{Z2} = 0$

$$\therefore M_{y1} = \frac{4EI_y}{L} q_{y1} - \frac{2EIq_{y1}}{L} = \frac{2EIq_{y1}}{L}$$

$$\therefore \frac{M_{y1}}{q_{y1}} = \frac{2EI}{L}$$

= *Stiffness of spring B for symmetric load-cases*

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CALCULATION SHEET

Job No: OSC397	Page 22 of 50	Rev A	
Job Title	Design of a single-span integral steel bridge		
Subject	WALLAP model (illustrative)		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

For this worked example:

Use short-term, uncracked section properties.

Calculation sheet 1

$I_{xx} = 6274000 \text{ cm}^4$

$E = 205 \times 10^6 \text{ kN/m}^2$

$$\frac{M_{y1}}{q_{y1}} = 2 \times 205 \times \frac{10^6}{33} \times 6.274 \times 10^{-2}$$

$$= 780000 \text{ kNm/radian}$$

Beam separation = *3.0 m*

∴ *Assuming stiffness is distributed evenly to abutment model*

$$\frac{M_{y1}}{q_{y1}} = \frac{780000}{3} = 260000 \text{ kNm per radian per m width}$$

$$= 260 \text{ kNm per m Rad per m width}$$

The rotational stiffness of the deck (260 kNm/mRad) and the rotational stiffness of the abutment (1008 kNm/mRad) should now be incorporated into the soil model and the deck model, following the analysis sequence illustrated by Figure 3.8. The remaining analysis using this method has not been carried out in the Worked Example.

WALLAP analysis response to displacement

A WALLAP model was created with a moment restraint and prop to represent interaction with the deck. Unfactored soil parameters were used. Excavation to formation level was analysed using the Finite Element method . This resulted in a force in the 'deck' prop of 560 kN/m.

The deck prop was then replaced with a horizontal force applied at this level. The initial force applied was 560 kN/m. This was increased in increments to 2500 kN/m. The table below gives the results of this modelling:

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CALCULATION SHEET

Job No: OSC397		Page 23 of 50	Rev A
Job Title	Design of a single-span integral steel bridge		
Subject	WALLAP model (illustrative)		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

Horizontal force (kN/m)	*Soil reaction to displacement (kN/m)*	*Displacement (mm)*
560	*0*	*0*
610	*50*	*2*
660	*100*	*5*
710	*150*	*9*
1000	*440*	*28*
1500	*940*	*67*
2000	*1440*	*113*
2500	*1940*	*191*

Deck response to axial strain

Force to fully restrain deck $= \varepsilon AE$

$A = 1564\ cm^2$ *(from sheet 1)* $= 0.1564\ m^2$ *per girder*

$E = 205 \times 10^6\ kN/m^2$

For 10 mm strain at each end (i.e. temperature load effect)

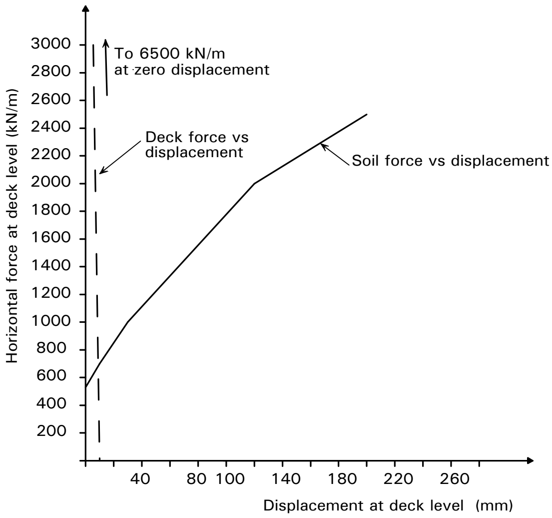
$\varepsilon = 10/16500 = 6.06 \times 10^{-4}$

$F = 6.06 \times 10^{-4} \times 0.1564 \times 205 \times 10^6$

$= 19429.6$ *kN per girder*

$= 6500$ *kN per m of wall*

The Steel Construction Institute Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944 CALCULATION SHEET	Job No: *OSC397*	Page *24* of *50*	Rev *A*
	Job Title *Design of a single-span integral steel bridge*		
	Subject *WALLAP model (illustrative)*		
	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*



Force-displacement graph at deck level (from WALLAP analysis)

The equilibrium condition occurs at a displacement of approximately 9 mm and a soil reaction/deck load of 700 kN.

Conclusion: ***This soil mass has negligible restraining effect on deck***

Comparison of the results of FREW and WALLAP

1. Excavation loadcase
For this loadcase the FREW model produces a deck moment of 1067 kNm/m (see top Figure on calculation sheet 14) whereas the WALLAP model produces a deck moment of 655 kNm/m. The primary difference between the models is that the WALLAP model does not model the capping beam, which will increase the amount of 'hogging' moment attracted to the top of the wall during excavation.

For deck axial load (prop force), FREW produces an axial force of 693 kN/m, whereas WALLAP produces 560 kN/m. Further modelling using WALLAP indicates that the prop force is increased in proportion to an increasing moment restraint. Thus, modelling the capping beam is likely to bring the results of FREW and WALLAP in closer agreement.

2. Response to displacement
For a displacement of 10 mm, FREW predicts a soil reaction of 137 kN, and WALLAP predicts a soil reaction of 166 kN.

Overall it is considered that the results of FREW and WALLAP are sufficiently close for the numerical method to be considered valid for the evaluation of structural effects, particularly considering the differences in the modelling methods employed by each program.

3.10 Structural checks - retaining wall

3.10.1 Definition of terms

In the calculations the term 'sagging' is used to denote bending which induces tension in the Frodingham section (the exposed face). Similarly 'hogging' induces compression in the Frodingham section.

3.10.2 Load effects

The retaining wall of a non-integral bridge is subject to the following load effects:

- Sagging moment from pressure of retained earth (assuming top of wall anchored during excavation).
- Shear from earth pressure.
- Axial load from the deck.
- Moment and shear due to eccentricity of the axial load.

The retaining wall of a fully integral bridge is subject to these effects and in addition to the following:

- Hogging moment from the connection to the deck.
- Additional shear due to the deck moment.
- Moment and shear due to deck expansion and contraction.
- Hogging moment from earth pressure (where moment fixity is constructed prior to excavation).

The structural checks which follow use the ULS load effects derived from the FREW analysis presented earlier. A summary of the most onerous ULS moments and co-existent shears is given on calculation sheets 17 and 18.

Vertical loads from the deck cause additional moments in the wall due to inevitable out-of-verticality of the pile wall. A deviation of 75 mm has been assumed in the Worked Example, corresponding to the upper level of accepted reasonable practice. For the sagging condition, an additional eccentricity has been considered, corresponding to the displacement of the pile due to earth pressure at the position of maximum moment.

3.10.3 Calculations

The calculations are arranged as follows:

- Design forces are presented (taken from the FREW analysis - Section 3.9.5).
- Section resistances are calculated.
- The moment resistances for both the uncorroded and corroded sections are calculated per m of retaining wall (see Figure 3.9). The shear resistance calculation has only been presented for the corroded section, since it is subsequently shown that this condition is critical for the design of the High Modulus Pile.

- Load resistance checks are then carried out for pure bending, pure shear, combined bending and shear, and combined bending and axial load.

3.10.4 Conclusions

It was found that fixing the deck prior to excavation maximised the efficiency of the structural behaviour of the portal frame, and that variations in the stiffness of the High Modulus Pile had a significant effect on the distibution of moments within it. A detailed description of the results of varying the structural model are given below:

Capping beam

Modelling the capping beam results in a significant amount of additional moment being attracted to this location, particularly for the corroded wall, to a level in excess of the capacity of the high modulus pile. However, the capping beam has greater moment capacity than the High Modulus Pile. At the underside of the capping beam, where the pile section acts alone, the bending moments are significantly less, and allowance has been made for this in calculating the design moment for the High Modulus Pile (see calculation sheet 25).

Fixing the deck before or after the excavation

Fixing the deck before excavation has the effect of slightly reducing the sagging moments in the middle of the wall by mobilising some of the stiffness of the deck to resist earth pressures. This has the effect of reducing the sagging moment to within the capacity of the corroded section (see calculation sheet 28). Naturally, hogging moments at the top of the wall were increased, but these were found to be still within the capacity of the section (see check on calculation sheet 29) but fixing the deck after excavation would lessen the design moments in the capping beam.

Corroded or uncorroded section properties

Figure 3.9 illustrates the procedure for the design using both corroded and uncorroded section properties. By inspection of the design moments on calculation sheet 26 it can be seen that the use of corroded section properties significantly reduces sagging moments in the central region of the retaining wall. Although the capacity of the High Modulus Pile was also reduced the section had sufficient capacity to carry the lower forces (see calculation sheet 30).

3.10.5 Vertical load resistance of High Modulus Pile

A calculation sheet has been included in this section, assessing the vertical load carrying capacity of the High Modulus Pile. In the past, allowance was made for vertical loads by the provision of a length of sheet pile in addition to that required for stability. This is considered to be over-conservative. The approach adopted in the Worked Example is to consider that only the soil on the passive side contributes to vertical resistance. This is because under active failure conditions the soil moves downwards, the same direction as the wall under vertical loads, thus negating the frictional effects. The soil on the passive face moves in the opposite direction, and frictional resistance is possible. Partial factors for resistance/materials have been taken from DD ENV 1997-1: 1995, Table 3 (see Section 3.1.3).

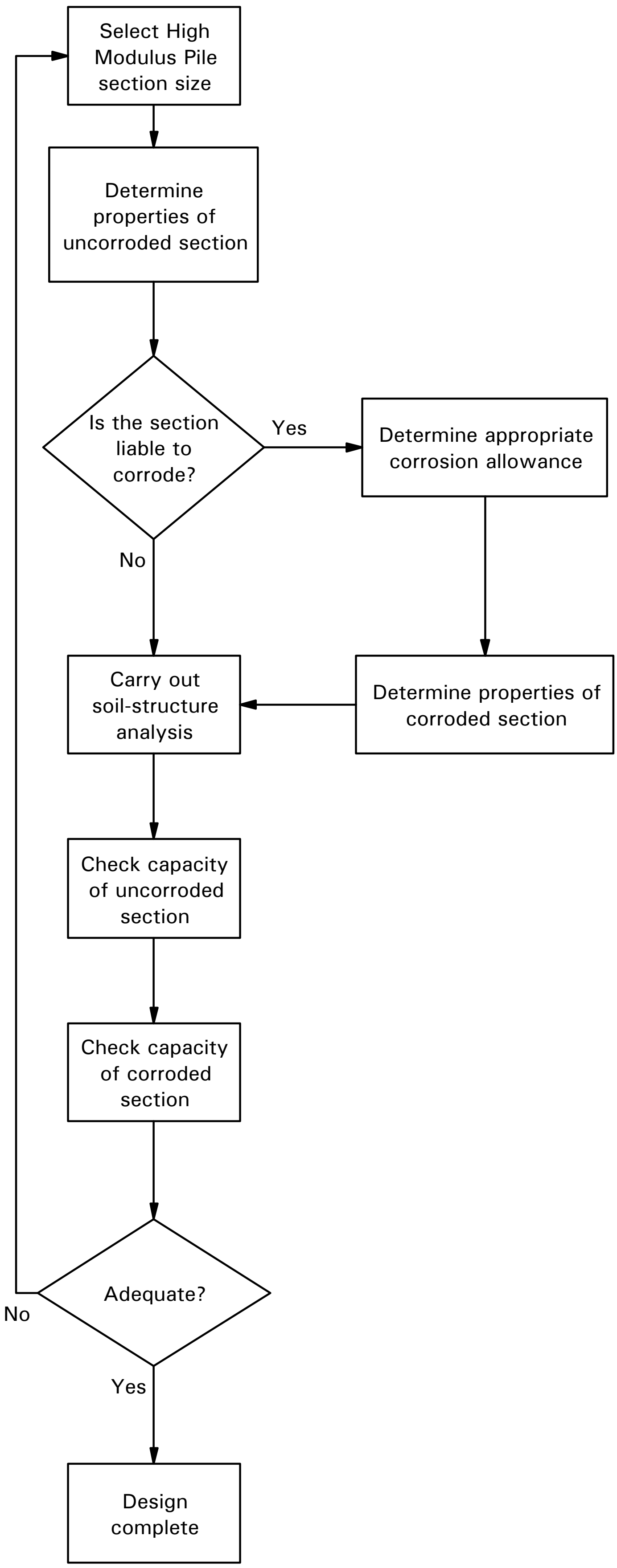


Figure 3.9 *Design using corroded and uncorroded section properties*

Discuss me ...

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CALCULATION SHEET

Job No: OSC397	Page 25 of 50	Rev A
Job Title	Design of a single-span integral steel bridge	
Subject	Structural checks - retaining wall	
Client	Made by JAW	Date Dec 1996
	Checked by EDY	Date Dec 1996

RETAINING WALL - DETAILED CHECKS TO BS 5400: PART 3

Unless otherwise stated, all of the following references are to BS 5400: Part 5 as implemented by BD 16/82

Load effects to be considered:

(1) Moment due to:

(a) Deck continuity
(b) Deck axial load multiplied by:

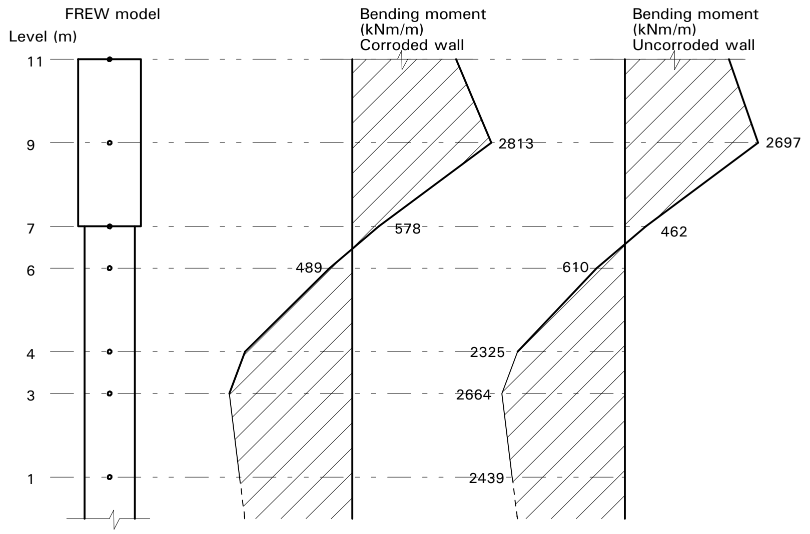
(i) Eccentricity (displacement) in wall
(ii) Eccentricity of deck

(c) Earth pressure

(2) Shear

(3) Axial load

Load cases 2(b) and 2(c) : Capping beam hogging combination (see Sheet 18)



ULS Moment distribution in retaining wall

By inspection of the above Figure, it can be seen that the moment in the capping beam reduces rapidly from the peak value. It would seem reasonable to design the High Modulus Pile for half the peak value, but the capping beam must be designed for the full moment.

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CALCULATION SHEET

Job No: OSC397		Page 26 of 50		Rev A	
Job Title	Design of a single-span integral steel bridge				
Subject	Structural checks - retaining wall				
Client		Made by	JAW	Date	Dec 1996
		Checked by	EDY	Date	Dec 1996

The following are values of load effects per m width of wall from the FREW analysis, for the construction condition of full moment fixity between the deck and the retaining wall before and after excavation:

Values for "fixed before" from calculation sheet 17

Values for "fixed after" from other analyses (not presented)

	Load Case		Fixed before excavation		Fixed after excavation
			Moment (kNm)	Shear (kN)	Moment (kNm)
Design hogging moment	2(b)	Corroded section	−1407	1268	−1177
		Uncorroded section	−1348	1327	−1275
Design sagging moment	3(a)	Corroded section	2762	88	2875
		Uncorroded section	3344	54	3522

Sagging moments give tension in the Frodingham section.

Axial load:
From FREW model and separate calculation of deck dead load

ULS axial load = 514 kN

Table 1, loadcase 2(b)

MOMENT RESISTANCE

Type 4N High Modulus Pile grade S275

Since the UB flange is fully restrained by the surrounding soil, the limiting compressive stress will not be controlled by lateral torsional buckling. Hence $\sigma_{lc} = \sigma_y$ *, Clause 9.8.3 does not apply and* $\gamma_M = 1.05$ *for compression resistance. (This is not covered adequately by Part 3, but is taken to be a reasonable interpretation).*

Value of γ_M *is as allowed in composite construction when the compression flange is connected to the slab*

1. Uncorroded Section

$I = 1.353 \times 10^{10}$ mm^4 (Sheet 6)

$\bar{y}_{flange} = 631.3$ mm

$\bar{y}_{Frod} = (920 + 330) - 631.3 = 618.7$

$Z_{flange} = 1.353 \times 10^{10}/631.3 = 2.144 \times 10^7$ mm^3

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CALCULATION SHEET

Job No: *OSC397*	Page *27* of *50*		Rev *A*
Job Title	*Design of a single-span integral steel bridge*		
Subject	*Structural checks - retaining wall*		
Client	Made by *JAW*	Date *Dec 1996*	
	Checked by *EDY*	Date *Dec 1996*	

$Z_{Frod} = 1.353 \times 10^{10}/618.7 = 2.187 \times 10^{7}$ mm^3

M_D = The lesser of: *Clause 9.9.1.3*

$Z_{xc}\ \sigma_{lc}/\gamma_m\ \gamma_{f3}$ (1)

or $Z_{xt}\ \sigma_{yt}/\gamma_m\ \gamma_{f3}$ (2)

The UB flange governs in both cases

$\therefore$ Moment resistance $= \dfrac{Z_{flange}\ \sigma_y}{\gamma_m\ \gamma_{f3}}$

$\sigma_y = 265$ N/mm^2 *BD 13/90, Clause 6.2*

$$= \frac{2.144 \times 10^{7} \times 265 \times 10^{-6}}{1.05 \times 1.1}$$

= 4916 kNm per HMP

Moment resistance per metre of wall $= 4916 \times \dfrac{1000}{966} = 5089$ kNm/m

2. Corroded Section *sheet 5*

From calculated properties (Sheet 7)

$Z_{flange} = 1.630 \times 10^{7}$ mm^3

$Z_{Frod} = 1.322 \times 10^{7}$ mm^3

Moment Resistance

$$M_D = \frac{Z_{Frod}\ \sigma_y}{\gamma_m\ \gamma_{f3}}$$

$$= \frac{1.322 \times 10^{7} \times 265 \times 10^{-6}}{1.05 \times 1.1} = 3033 \text{ per HMP}$$

Resistance per metre of wall $= 3033 \times \dfrac{1000}{966} = 3139$ kNm/m

$M_R = F_f\ d_f/(\gamma_m\ \gamma_{f3})$

$A_{fe} = 416.5 \times 32.6 = 13578$ mm^2

$F_f = 13578 \times 265 \times 10^{-3} = 3598$ kN

$M_R = 3598 \times 0.884/(1.2 \times 1.1) = 2409$ kNm/m

Based on UB only for simplicity Clause 9.9.3.1(d)

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CALCULATION SHEET

Job No: OSC397	Page 28 of 50		Rev A
Job Title	Design of a single-span integral steel bridge		
Subject	Structural checks - retaining wall		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

SHEAR RESISTANCE (calculated for the corroded section)

BS 5400: Part 3 Clause 9.9.2.2

Shear resistance: $V_D = \left[\frac{t_w(d_w - h_h)}{\gamma_{f3}\gamma_m}\right]\tau_l$

$t_w = 17.5$

$d_w = 920.5$

$h_h = 0$

It is assumed that shear is only carried by beam web

τ_l depends on the panel slenderness, given by:

$$\lambda = \frac{799.1}{17.5}\sqrt{\left(\frac{265}{355}\right)} = 39$$

$$\lambda < 55, \text{ hence } \tau_l = \tau_y = \frac{265}{\sqrt{3}} = 152 \text{ N/mm}^2$$

$$V_D = \frac{17.5 \times 920.5 \times 152}{1.1 \times 1.05}$$

$$= 2119 \text{ kN}$$

and $V_R = V_D$

Check pure bending

Based on the load effects on Sheet 26 and the resistances calculated on Sheets 27 and 28, the usage factors (U.F.) for moment are given below:

		Fixed before excavation U.F.	Fixed after excavation U.F.
Design hogging moment	Corroded section	0.45	0.37
	Uncorroded section	0.26	0.25
Design sagging moment	Corroded section	0.88	0.92
	Uncorroded section	0.66	0.69

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CALCULATION SHEET

Job No: OSC397	Page 29 of 50	Rev A	
Job Title	Design of a single-span integral steel bridge		
Subject	Structural checks - retaining wall		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

By inspection of the above, creating moment continuity after excavation results in a larger governing sagging moment in the High Modulus Pile, therefore it is beneficial for the moment continuity between deck and wall to be established prior to excavation. It can also be seen that the corroded section results in governing usage factors for both hogging and sagging moments. Further checks will therefore be made on the corroded section.

Check pure shear

U.F.: 1268/2119 = 0.60 OK

Check combined hogging bending and shear — *BS 5400: Part 3 Clause 9.9.3.1*

Moment due to eccentricity of bearing from centroid of sheet pile (this calculation was omitted from the check on pure bending for simplicity but has been included in the detailed check of the corroded section):

Assume $\pm$ 75 mm tolerance on position

$\therefore$ Moment = 514×0.075 = 39 kNm/m — *Loads from sheet 26*

$\therefore$ Total bending moment = 1407 + 39 = 1446 kNm/m

$M = 1446$ kNm, $V = 1295$ kN $\therefore M < M_R$ and $V < V_R$

$\therefore$ no need to check interaction

Check combined hogging bending and axial load (stress check) — *Clause 9.9.4*

$$\frac{P}{A_c} + \frac{M}{Z_x} \leq \frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

Bending stress in Frodingham = $\dfrac{1446 \times 10^6}{1.322 \times 10^7}$ = 109 N/mm^2 — *Using gross section modulus (after corrosion)*

Axial stress = $514 \times 10^3/49090$ = 10 N/mm^2 — *Area of corroded section from Sheet 7*

$$109 + 10 < \frac{265}{1.1 \times 1.05} = 229 \text{ N/mm}^2 \therefore \text{OK}$$

Value of stress using effective section (after corrosion) is about 20% greater, but still OK

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CALCULATION SHEET

Job No: OSC397		Page 30 of 50	Rev A
Job Title	Design of a single-span integral steel bridge		
Subject	Structural checks - retaining wall		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

Check retaining wall 'midspan' sagging bending

M = *2762 kNm* — *Table 1, Sheet 17*

There is no shear at position of maximum sagging moment.

Displacement at maximum moment = *22 mm* — *Displacement from FREW analysis*

Out-of-verticality = *75 mm*

Additional moment = $514 \times (75+22) \times 10^{-3}$

= *50 kNm/m*

Total applied moment = *2762 + 50* = *2812 kNm/m*

From previous calculations M_D = *3139 kNm/m* — *Sheet 26*

M_R = *2409 kNm/m*

Beam is OK for M = 2762 kNm — *Clause 9.9.3.2*

and V = 0

Combined sagging bending and axial load: — *Clause 9.9.4*

Bending stress in UB flange = $\dfrac{2812 \times 10^6}{1.630 \times 10^7}$ = $172\ \mathrm{N/mm^2}$ — Z_{flange} *from Sheet 7*

Axial stress = $10\ \mathrm{N/mm^2}$ — *From Sheet 29*

$172 + 10 = 182 < 229\ \mathrm{N/mm^2}$ *- OK*

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	Job Title *Design of a single-span integral steel bridge*		
	Subject *Structural checks - retaining wall*		
	Client	Made by *JAW* / Checked by *EDY*	Date *Dec 1996* / Date *Dec 1996*

VERTICAL RESISTANCE

Frictional resistance is assumed to act on the passive side only. Consider the surface area of the side of Frodingham 4N in contact with the passive soil face:

Total surface area of one 4N section = $1.61\ m^2$

Width of one section = $0.483\ m$

British Steel Piling Handbook

Only one side in contact with soil face

∴ *Surface area of one side per metre width of wall* $= 1.61 \times \frac{1}{0.483}$ ´ $\frac{1}{2}$

$= 1.66\ m^2$

Friction coefficient, (long-term) $\mu = \alpha\ c_u$

$\mu = 0.5 \times 150$

$\mu = 75\ kN/m^2$

$\alpha = 0.5$ *from Steel Bearing Piles Guide,* $c_u = 150$ *from soil profile Figure 3.5*

ULS resistance = surface area $\times\ \mu / \gamma_s$

where γ_s *is a factor of safety, which may be taken from Eurocode 7, Table 3, as 1.30*

See Section 3.1.3

Total ULS load = $742\ kN/m$

Axial load + pile self weight

∴ *Minimum required embedded length* $= \frac{742 \times g_s}{1.66 \times 75} = \frac{742 \times 1.3}{1.66 \times 75}$

$= 7.45\ m$

Actual embedded length $= 12\ m > 7.45m$ *O.K.*

∴ <u>*Sufficient length for vertical load resistance*</u>

3.11 Structural checks - deck

3.11.1 Structural design of deck

Design of the deck of an integral bridge differs from conventional bridge decks only in the additional axial loads that should be taken into account. In the case of the Worked Example it can be seen that excavation, live load, surcharge and expansion all contribute towards increased axial loads. Deck contraction acts to relieve earth pressures. The axial load is assumed to act uniformly over the area of the composite deck girder.

By inspection, the critical position for combined moment and axial load is the bottom flange in the hogging zones at the ends. A comparison of total longitudinal stress in the extreme fibre with the limiting compressive stress, indicates that the girder has sufficient capacity.

3.11.2 Effect of construction options

With reference to calculation sheet 32, achieving moment continuity between wall and the deck before excavation has the effect of increasing the hogging moment and decreasing the sagging moment in approximately equal proportions. The pressure of the retained earth also induces additional axial load in the deck. If the moment connection is made before excavation, the sagging moments in the deck are significantly reduced, which could be taken advantage of in the design of the midspan region. However, increased hogging moments are usually more difficult to deal with in a normal composite beam (with the slab at the top) because the connection detail is more complex and hogging moment resistance is less than sagging moment resistance.

The Steel Construction Institute Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944 **CALCULATION SHEET**	Job No: *OSC397*	Page *32* of *50*	Rev *A*
	Job Title *Design of a single-span integral steel bridge*		
	Subject *Structural checks - deck*		
	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*

DECK: DETAILED CHECKS TO BS 5400: PART 3

Applied loading

These checks are based on the results obtained from a FREW model with corroded retaining wall section properties, configured for the condition of full moment fixity between the deck and the retaining wall prior to excavation.

The numerical analysis has calculated, for a variety of loadcases, the three components
- *Moment*
- *Shear*
- *Axial load*

The FREW results (sheets 17 and 18) have been multiplied by the girder spacing of 3 m (it is assumed that loads are distibuted uniformly by the capping beam).

All design loads from combination 3 loadcase

		Deck fixed after excavation	Deck fixed before excavation
		ULS Moment	ULS Moment
Design hogging moment	Wall corroded	−3532	−8439
	Wall uncorroded	−3826	−8091
Design sagging moment	Wall corroded	11042	7038
	Wall uncorroded	10735	7185

Shear (for corroded wall)
From FREW results on sheet 17 ULS shear force (= ULS axial force in wall) per m of wall = 514 kN
$\therefore$ *shear force per girder* $= 3 \times 514 = 1542$ *kN*

Axial load (for corroded wall):
From FREW results on sheet 18 ULS axial load (= ULS shear force in wall) per m of wall = 1552 kN
$\therefore$ *axial load per girder* $= 3 \times 1552 = 4656$ *kN*

Since the girder is compact at midspan, all the moment there may be assumed to be carried on the plastic composite section.

The Steel Construction Institute	Job No: OSC397	Page 33 of 50	Rev A
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944	Job Title: Design of a single-span integral steel bridge		
	Subject: Structural checks - deck		
CALCULATION SHEET	Client	Made by JAW	Date Dec 1996
		Checked by EDY	Date Dec 1996

MOMENT RESISTANCE (composite deck girder)

(see section defined in calculation sheet 1)

Short-term section moduli for this section are:

Composite uncracked section, bottom flange, $Z_x = 4.684 \times 10^7$ mm^3

Composite cracked section, bottom flange, $Z_x = 4.363 \times 10^7$ mm^3

Composite cracked section, tension reinforcement, $Z_x = 6.358 \times 10^7$

Yield stress $\sigma_y = 355$ N/mm^2 *(Grade S355 steel to BS EN 10025: 1993)*

Bending resistance in sagging: — *Clause 9.9.1.3*

$$M_D = \frac{Z_{xt}\,\sigma_{yt}}{\gamma_m \gamma_{f3}} = \frac{4.684 \times 10^7 \times 355}{1.1 \times 1.05} \times 10^{-6} = \underline{14400 \text{ kNm}}$$

Bending resistance in hogging:

(a) Based on compression flange

$$M_D = \frac{Z_{xc}\,\sigma_{lc}}{\gamma_m \gamma_{f3}}$$

The value of σ_{lc} *is given by Clause 9.8.3, depending on the value of the limiting compressive stress,* σ_{li} *and* $D/2_{yt}$.

The bottom flange has been provided with adequate lateral restraint to prevent lateral torsional buckling

Assume that $\lambda_{LT} < 45$ *and* $D/2_{yt} = 0.96$

Hence $\sigma_{li} = 355$ N/mm^2 *and thus* $\sigma_{lc} = 0.96 \times 355 = 339$ N/mm^2

$M_D = 4.363 \times 10^7 \times 339/(1.1\times1.2) \times 10^{-6} = 11200$ kNm

(b) Based on deck reinforcement in tension:

$M_D = 0.87 f_{ry} \times Z_{xt} / \gamma_{f3}$

$= 6.358 \times 10^7 \times 0.87 \times 460/1.1 \times 10^{-6} = 23100$ kNm

$\therefore$ *Compression flange governs and* $M_D = \underline{11200 \text{ kNm}}$

(c) Reduced moment resistance M_R *(for use in shear/moment interaction)*

$$M_R = \frac{F_f\, d_f}{\gamma_m \gamma_{f3}}$$

F_f *(top flange)* $=$ *(Flange area + reinf. area)* $\times$ *yield stress*

$= (3500 + 32169 \times 200/205) \times 355$

$= 13.8 \times 10^6$ N

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CALCULATION SHEET

Job No: OSC397	Page 34 of 50	Rev A	
Job Title	Design of a single-span integral steel bridge		
Subject	Structural checks - deck		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

F_f *(bottom flange)* $= 35 \times 600 \times 355$

$= 7.455 \times 10^6$ N

df = 1668 mm *(to centroid of flange + reinforcement)*

By inspection, bottom flange governs

$$M_R = \frac{7.455 \times 10^6 \times 1668}{1.1 \times 1.2} \times 10^{-6} = 9420\,kNm$$

SHEAR RESISTANCE — *BS 5400: Part 3 Clause 9.9.2*

$$V_D = \left[t_w \frac{(d_w - h_h)}{g_{f3} g_m} \right] t_l$$

$t_w = 15$ mm, $d_w = 1540$ mm, $h_h = 0$

Parameters to determine τ_l — *BS 5400: Part 3 Figure 11*

$d_{we} = 1540$ mm $\quad \therefore \quad l = \dfrac{1540}{15} = 102$

Assume intermediate stiffeners at 2.4 m centres

$$\phi = \frac{2400}{1540} = 1.56$$

b_{fe} *from lesser of* (a) $10\,t_f = 250$ mm
(b) 250 mm

$m_{fw} = 250 \times 25^2/(2 \times 1540^2 \times 15) = 0.002$

For $m_{fw} = 0 \quad \tau_l/\tau_y = 0.70 \quad \tau_l = 143$ N/mm²

For $m_{fw} = 0.005 \quad \tau_l/\tau_y = 0.80$

$\therefore$ for $m_{fw} = 0.002 \quad \tau_l/\tau_y = 0.74 \quad \tau_l = 152$ N/mm²

Hence $$V_R = \frac{15 \times 1540 \times 143}{1.1 \times 1.05} \times 10^{-3} = 2860kN$$

$$V_D = \frac{15 \times 1540 \times 152}{1.1 \times 1.05} \times 10^{-3} = 3040kN$$

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CALCULATION SHEET

Job No:	OSC397	Page 35 of 50	Rev A
Job Title	Design of a single-span integral steel bridge		
Subject	Structural checks - deck		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

AXIAL RESISTANCE

Bottom flange: Unstiffened outstand: $b_0 / t_0 = 300/35 = 8.5 < 12$ *OK* — *Clause 10.3.1*

Assume bottom flange is braced at 5 m centres

Take $l_e = 0.85 \times 5000 = 4250$ *mm and* $r_y = 600/\sqrt{12} = 173$ *mm* — *Clauses 10.4.1 & 12.4.1*

Hence $l_e/r_y = 4250/173 = 25$

and thus $\sigma_c = 0.935 \quad \sigma_y = 0.935 \times 355 = 332$ N/mm^2 — *Reduced for full depth of web in compression, using* k_c *from Clause 9.4.2.4*

Gross area of cracked section $= 88060$ mm^2

A_e *= total area of section* $= 68940$ mm^2

$$P_D = A_e\sigma_c / \gamma_m\gamma_{f3}$$

$$= 72050 \times 332 / (1.1 \times 1.05) \times 10^{-3} = 20710 \text{ kN}$$

Clause 9.9.3

CHECK COMBINED BENDING AND SHEAR

Hogging moment (corroded wall) $M = 8439$ *(sheet 32)*
Shear (corroded wall) $V = 1542$ *(sheet 32)*
$M < M_R$ *and* $V < V_R$*, so beam is ok*

M_R is the smaller of the top and bottom flange values

__Check combined bending and axial load__

Clause 9.9.4.1 will be satisfied automatically

Check maximum load effects against Clause 9.9.4.2

$$\frac{P}{P_D} + \frac{M}{M_D} = \frac{4656}{20710} + \frac{8439}{11200} = 0.98 \quad \therefore \text{ OK}$$

Note that there is no explicit check in BS 5400: Part 3 for combined bending, axial load and shear. However, it would seem reasonable to combine the two requirements into one expression, thus:

$$\frac{P}{P_D} + \frac{M}{M_D} + \left(1 - \frac{M_R}{M_D}\right)\left(\frac{2V}{V_R} - 1\right),$$ *which has a value of 0.99 for this Example*

The full expression is evaluated here, even though $M < M_R$ *and* $V < V_R$

A check in the midspan (sagging) region has also been carried out, and the section was found to be adequate.

Summary of usage factors

The following table summarises the usage factors for moment for the four construction options modelled by FREW:

		Deck fixed after excavation	*Deck fixed before excavation*
Design hogging moment	*Wall corroded*	*0.53*	*0.98*
	Wall uncorroded	*0.57*	*0.95*
Design sagging moment	*Wall corroded*	*0.87*	*0.64*
	Wall uncorroded	*0.87*	*0.66*

3.12 Capping beam design

The capping beam has two purposes:

- To distribute the forces between the girders and the High Modulus Piles.
- To accommodate construction tolerance in level and line.

The capping beam is designed and constructed in two sections. An illustrative view of the capping beam is shown in Figure 3.10.

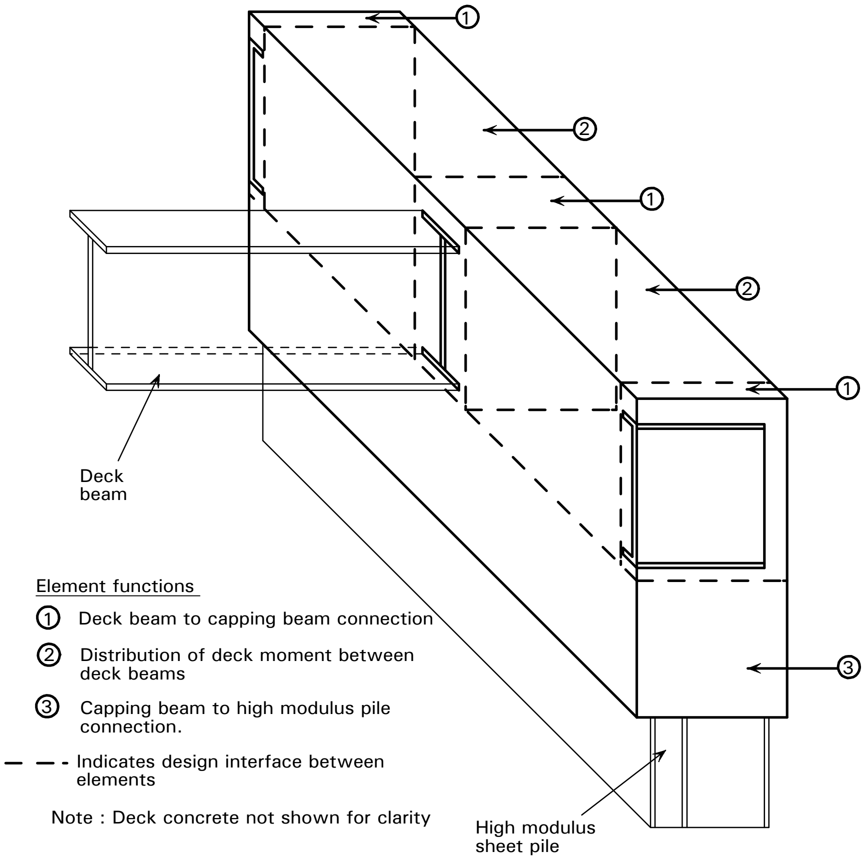

Figure 3.10 *Capping beam design elements*

3.12.1 Loading

The capping beam is designed primarily to transfer the worst-case hogging moment between the deck and the retaining wall. This condition has been shown to be produced by modelling the High Modulus Pile with corroded section properties. The capping beam is also designed to transfer shear from the retaining wall into axial load in the deck girders, and vice-versa. The capping beam is checked for both ULS and SLS loading, evaluated in accordance with BD 37/88. Design values for these loads are given on calculation sheets 17, 18 and 19.

3.12.2 Commentary on the detailed design of each element

Lower capping beam (element 3)

The purpose of this element is to transfer the forces from the High Modulus Pile into a reinforced concrete block, and to provide an initial landing platform for the deck beams during construction. Shear studs combined with transverse reinforcement are provided to achieve this transfer, the design method being based on the design for longitudinal shear flow in composite girders (BS 5400: Part 5 as implemented by BD 16/82). The maximum applied moment is assumed to be carried at flange and Frodingham levels, and the resulting forces assumed to be distributed uniformly between the shear studs provided.

Since the actual length of the sheet piles will only be known after driving, the shear studs must be welded to the sheet pile *in situ*, i.e. in its vertical position. This operation is possible, but requires more care than welding in the horizontal position. Stud welding specialists advise that the maximum stud size that can currently be welded to a vertical member is 19 mm diameter. Alternatively, if the shear is high, bar or channel, connections may be used.

Reinforcement also has to be provided to carry the tensile force into the upper section. In the worked example T40's at 150 crs are required for this purpose. The anchorage length required to fully transfer the tensile force into the T40's defines the depth of the lower section.

End-of-girder cap (element 1)

The purpose of this element is to transfer the forces from the deck girder into the concrete block. The design is controlled by the large forces generated by the moments and axial forces in the girder. Forces in each flange have been calculated based on the stress in the extreme fibre for both ultimate and servicability limit state. Shear connectors must always be checked at the servicability limit state because the value of γ_m for this condition (1.85) is higher than that for the ultimate limit state (1.4). The moment and axial force carried by the web has also been converted into equivalent forces at flange level. In order to carry the force generated in the bottom flange, hoop-type shear connectors are used, which have significantly higher transfer capacities than stud connectors. Positioning the hoops on the inside face of the botttom flange enables the flange to be placed directly onto the lower capping beam. For the top flange, the force is much less and stud connectors have sufficient capacity.

In addition to shear connectors, seven No. T32s and eight No. T40s are required transversely to transfer the top and bottom flange forces respectively across the assumed failure planes, into the body of the upper capping beam. Holes will need to be drilled in the web of the girder in order to accommodate the bottom flange transverse bars.

Upper capping beam - internal force distribution (element 2)

In order to simplify the design of the capping beam, a conservative set of internal forces has been assumed. In terms of load path from girder to High Modulus Pile, the following has been assumed:

- A deck girder introduces a moment, a vertical force and a horizontal force into the capping beam.
- The moment is seen by the capping beam as an applied torsion which is resisted by uniform restraining torsion on either side, reducing the torsion in the beam to zero midway between the adjacent beams. The uniform restraint is provided by the bending resistance of the pile wall and lower element of the capping beam.
- The horizontal load is resisted by horizontal bending of the capping beam, restrained by a uniform horizontal shear from the lower element (i.e. the top of the wall).
- The vertical load is distributed to the piles without any bending action in the capping beam.

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The Steel Construction Institute	Job No: OSC397	Page 37 of 50	Rev A
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944	Job Title: Design of a single-span integral steel bridge		
	Subject: Capping beam design		
CALCULATION SHEET	Client	Made by JAW	Date Dec 1996
		Checked by EDY	Date Dec 1996

Capping beam Design - Overview

With reference to Figure 3.10 the capping beam must ensure that:

1. *Each element has sufficient strength to fulfil its function.*

2. *The interfaces have sufficient strength to transfer the design forces between elements.*

The design has been split into the following stages:

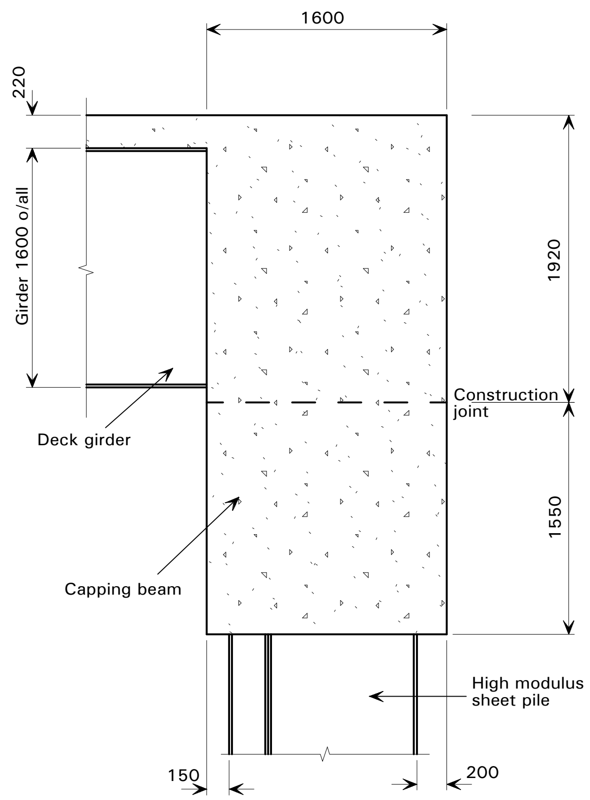
1. Connection of High Modulus Pile to lower capping beam (element 3).

2. Connection between element (3) and elements (2) and (1).

3. Connection of deck beam to end-of-girder cap (element 1).

4. Design of upper capping beam (element 2) for torsion.

Unless otherwise stated, all references in this section are to BS 5400: Part 5 as implemented by BD 16/82



General arrangement of capping beam

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CALCULATION SHEET

Job No: *OSC397*	Page *38* of *50*	Rev *A*	
Job Title	*Design of a single-span integral steel bridge*		
Subject	*Capping beam design*		
Client	Made by *JAW*	Date *Dec 1996*	
	Checked by *EDY*	Date *Dec 1996*	

(1) CONNECTION OF HIGH MODULUS PILE TO CAPPING BEAM

See calculation sheet 6 for HMP centres

Summary of design forces:

All design loads from combination 3 loadcases

At SLS: Moment = 2208 kNm/m × 0.966 = 2132 kNm per HMP
Axial load = 401 kNm/m × 0.966 = 387 kNm per HMP

Calculation sheet 17

At ULS: Moment = 2813 kNm/m × 0.966 = 2717 kNm per HMP
Axial load = 514 kNm/m × 0.966 = 496 kNm per HMP

Convert design moment into equivalent couple at flange levels. Cross section areas of Frodingham and UB flange are approximately equal. For simplicity, assume that the lever arm for the couple is to the mid depth of the High Modulus Pile and that two thirds of the axial load is carried by the Frodingham.

$$\text{Lever arm} = 0.917 + \frac{0.326}{2} = 1.080\,\text{m}$$

Since the axial load and the couple force are in the same sense on the Frodingham face but in the opposite sense on the outstand flange, the Frodingham will govern the design for shear connection.

Design this connection and make the same provision on UB flange for simplicity.

$$\text{SLS design moment} = 2132 + 387 \times 0.075 = 2161\ \text{kNm/m}$$

$$\text{SLS force in studs} = 2161 / 1.08 + \frac{2 \times 387}{3} = 2259\,\text{kN}$$

$$\text{ULS design moment} = 2717 + 496 \times 0.075 = 2754$$

$$\text{ULS force in studs} = 2754 / 1.08 + \frac{2 \times 496}{3} = 2880\,\text{kN}$$

Shear studs to be connected in the vertical position, therefore the largest practical shear stud to use is 19 mm ϕ.

Nominal static strength 109 kN per connector

BS 5400: Part 5 Table 7

$$\therefore \text{No. of studs required @ SLS} = \frac{2259 \times 1.85}{109} = 38,$$

$$\text{No. of studs required @ ULS} = \frac{2880 \times 1.4 \times 1.1}{109} = 41$$

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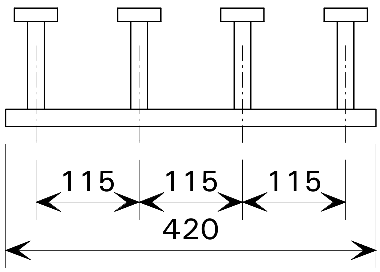
	Job Title *Design of a single-span integral steel bridge*		
	Subject *Capping beam design*		
	Client	Made by *JAW*	Date *Dec 1996*
		Checked by *EDY*	Date *Dec 1996*



Provide 11 rows of 4 studs

Shear studs - High Modulus Pile UB flange

If shear studs are placed at 125 crs vertically, overall length occupied by studs $= 125 \times 10 = 1250$ mm

Allow 100 mm cover to end of High Modulus Pile and 200 mm cover to lowest shear stud

Depth of element $= D_p = 1250 + 300 = 1550$

Longitudinal shear around studs — *Clause 6.3*

ULS longitudinal shear = 2880 kN per HMP

Design shear per unit length must be less than or equal to: — *Clause 6.3.3.2*

(a) $k_1 f_{cu} L_s$

(b) $v_1 L_s + 0.7 A_e f_{ry}$

L_s = *length of shear plane* $= 300 + 2 \times 100 = 500$ mm
or $= 2 \times 200 = \underline{400 \text{ mm}}$

As noted in Section 3.1.4, γ_m is applied on the resistance side, to be consistent

(a) $(0.15 \times 400 \times 40)/1.1 = 2182$ kN/m

over 1.5 m $= 2182 \times 1.5 = 3273 > 2880$ *OK*

(b) *Transverse reinforcement: Try 12 No. T32* ($f_{ry} = 460$ N/mm^2)

$$2A_b = A_e = 2 \times \frac{\pi}{4} \times 32^2 \times 12 = 19301 \text{ mm}^2/\text{m}$$

$\therefore$ *capacity* $= (0.9 \times 400 + 0.7 \times 19301 \times 460)/1.1 = 5650$ kN

$5650 \text{ kN} > 2880 \text{ kN}$ $\therefore$ ***12 No. T32 OK***

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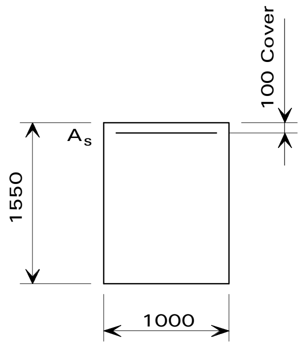
CALCULATION SHEET

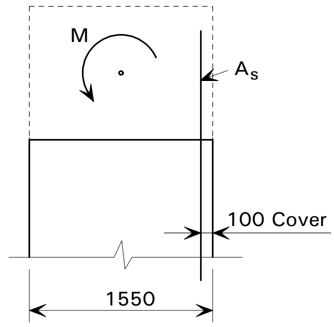
Job No: *OSC397*	Page *40* of *50*	Rev *A*	
Job Title	*Design of a single-span integral steel bridge*		
Subject	*Capping beam design*		
Client	Made by *JAW*	Date *Dec 1996*	
	Checked by *EDY*	Date *Dec 1996*	

2) CONNECTION OF LOWER CAPPING BEAM (ELEMENT 3) TO UPPER CAPPING BEAM (ELEMENTS 1 AND 2)

Transfer of moment across construction joint



Horizontal section at construction joint (1 m width)



Vertical section at construction joint

Maximum design moment = 2813 kNm/m — *Sheet 18*

Try tensile reinforcement = T40s @ 150 crs — *(100 mm cover assumed)*

Clause 5.3.2.3, Equation 5

$$z = \left(1 - \frac{1.1 f_y A_s}{f_{cu} b_d}\right) d$$

$$z = \left(1 - \frac{1.1 \times 460 \times 8378}{40 \times 1500 \times 1000}\right) \times 1450 = 1348\ mm^3$$

Equation 1

$$M_{us} = 0.87\, f_y\, z/\gamma_{f3}$$
$$= 0.87 \times 460 \times 8377 \times 1348 \times 10^{-6}/1.1$$
$$= 4108\ kNm/m$$

Equation 2

$$M_{uc} = 0.15\, f_{cu}\, bd^2\ /\gamma f3$$
$$= 0.15 \times 40 \times 1000 \times 1500^2 \times 10^{-6}/1.1$$
$$= 12270\ kNm/m$$

2813 < 4108 ∴ T40's @ 150 crs OK

A crack control check should also be carried out
Check anchorage in lower capping beam
Length of T40 required to achieve ultimate stress in bar — *Part 4 Clause 5.8.6.3*
Length of perimeter = πd = 125 mm
$l_{req} = 3107 \times 1.1 \times 10^3/(7 \times 125 \times 3.3) = 1184$ mm

∴ OK within depth of element 3.

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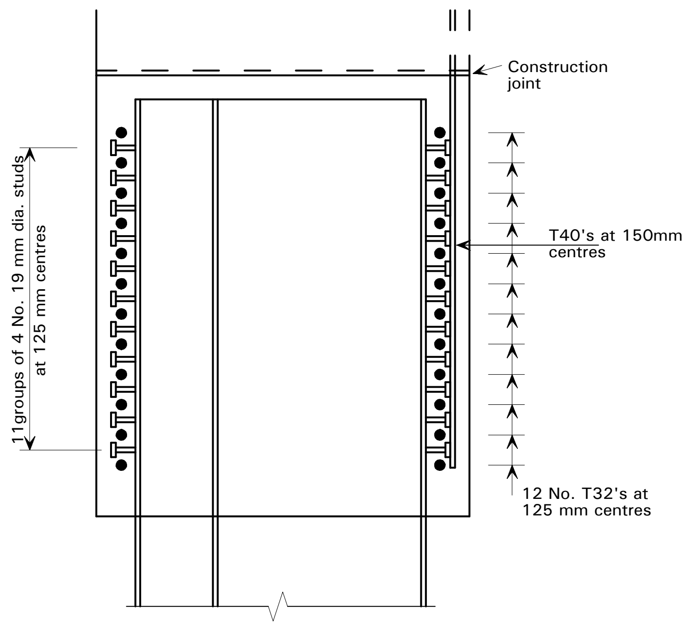
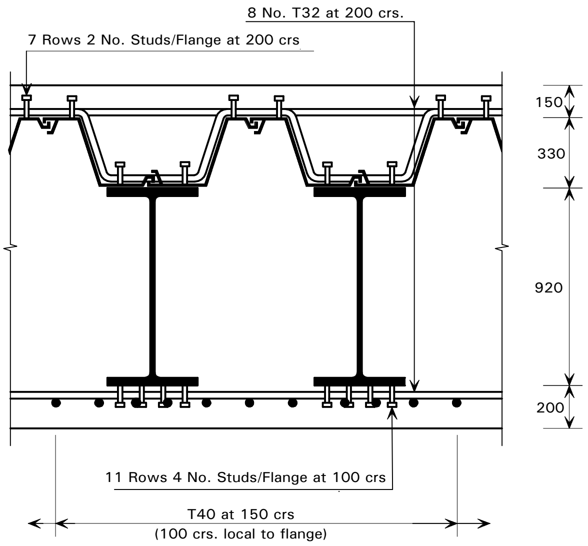
The Steel Construction Institute	Job No: OSC397	Page 41 of 50	Rev A
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 23345 Fax: (01344) 22944	Job Title: Design of a single-span integral steel bridge		
	Subject: Capping beam design		
CALCULATION SHEET	Client	Made by JAW	Date Dec 1996
		Checked by EDY	Date Dec 1996



Vertical section through lower capping beam



Horizontal section through lower capping beam

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	Job Title: Design of a single-span integral steel bridge		
	Subject: Capping beam design		
	Client	Made by JAW	Date Dec 1996
		Checked by EDY	Date Dec 1996

3) CONNECTION OF DECK GIRDER TO CAPPING BEAM (ELEMENT 1)

Design the connection on the basis that the deck beam moment and axial load from retaining wall is transferred to the capping beam via shear connectors at top and bottom flange levels.

SUMMARY OF DESIGN FORCES

From Sheet 18

At SLS: Moment = 2208 × 3 = 6624 kNm/girder
Axial load = 1094 × 3 = 3282 kNm/girder

At ULS: Moment = 2813 × 3 = 8439 kNm/girder
Axial load = 1552 × 3 = 4656 kNm/girder

Calculate the equivalent flange forces resulting from this loading.

1. Flange forces due to moment

Cracked section properties

Refer to sheet 1

$A = 88060 \text{ mm}^2$

$\bar{y}_c = 1050 \text{ mm}$; $\bar{y}_t = 1600 - 1050 = 550 \text{ mm}$

$I = 4.580 \times 10^{10}$

At ULS: Stress in extreme fibre:

$$\text{Top flange} = \frac{8439 \times 550 \times 10^6}{4.580 \times 10^{10}} = 101 \text{ N/mm}^2$$

$$\text{Bottom flange} = \frac{8439 \times 1050 \times 10^6}{4.580 \times 10^{10}} = 193 \text{ N/mm}$$

Bottom flange force = 193 × 600 × 35 = 4053 kN

Top flange force = 101 × 500 × 25 = 1263 kN

Stress at web/flange connection:

$$\text{Top flange} = 101 \times \frac{525}{550} = 96 \text{ N/mm}^2$$

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CALCULATION SHEET

Job No: *OSC397*		Page *43* of *50*	Rev *A*
Job Title *Design of a single-span integral steel bridge*			
Subject *Capping beam design*			
Client	Made by *JAW*		Date *Dec 1996*
	Checked by *EDY*		Date *Dec 1996*

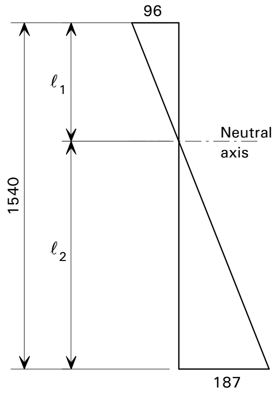
Bottom flange $= 193 \times \frac{1015}{1050} = 187\ N/mm^2$

$\frac{96}{\ell_1} = \frac{187}{\ell_2}$ $\qquad (\ell_1 + \ell_2) = 1540$

$\frac{96}{\ell_1} = \frac{187}{\left(1540 - \ell_1\right)}$

$\ell_1 = 522\ mm$

$\therefore \ell_2 = 1540 - 522$
$= 1018\ mm$



Compressive force in web $= 1018 \times 15 \times \frac{187}{2} \times 10^{-3}$

$= 1427\ kN$

Equivalent force at bottom flange level $= 1427 \times ⅔ = 951\ kN$

Tensile force in web $= 522 \times 15 \times \frac{96}{2} \times 10^{-3} = 376\ kN$

Equivalent force at top flange level $= ⅔ \times 376 = 250\ kN$

Total force, bottom flange $= 4053 + 951 = 5004\ kN$

Total force, top flange $= 1263 + 250 = 1513\ kN$

At SLS

Top flange stress = 101 ´ $\frac{6573}{8439}$ = 7 9 N/mm^2

Bottom flange stress $= 193 \times \frac{6573}{8439} = 150\ N/mm^2$

Top flange force $= 79 \times 500 \times 25 = 988\ kN$

Bottom flange force $= 150 \times 600 \times 35 = 3150\ kN$

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CALCULATION SHEET

Job No: OSC397	Page 44 of 50	Rev A	
Job Title	Design of a single-span integral steel bridge		
Subject	Capping beam design		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

Top flange force due to moment in web

$= 250 \times 6573/8439 = 195$

Bottom flange force due to moment in web

$= 951 \times 6573/8439 = 741$

The web forces at ULS have been reduced by the ratio of SLS/ULS moments.

Total force top flange $= 988 + 195 = 1183$ kN

Total force bottom flange $= 3150 + 741 = 3891$ kN

2. Flange forces due to axial load (all axial load transferred through flanges)

Distribute axial load in proportion to total cross-section area.

			A_f/A_T
Area of top flange	=	12500 mm^2	0.19
Area of reinforcement	=	31384 mm^2	0.48
Area of bottom flange	=	21000 mm^2	0.32
Total area (A_r)	=	64884 mm^2	1.0

Distribute SLS force:

Top flange $= 3282 \times 0.19 = 623$ kN

Bottom flange $= 3282 \times 0.32 = 1050$ kN

Distribute ULS force:

Top flange $= 4656 \times 0.19 = 885$ kN
Bottom flange $= 4656 \times 0.32 = 1490$ kN

Combine forces due to axial load and forces due to moment. Note - axial load relieves tension in top flange.

At SLS

Design force top flange $= 1183 - 623 = 560$ kN
Design force bottom flange $= 3891 + 1050 = 4941$ kN

At ULS

Design force top flange $= 1513 - 885 = 628$ kN

Design force bottom flange $= 5004 + 1490 = 6494$ kN

BOTTOM FLANGE CONNECTION

Bottom flange shear connection

Try 50 mm × 40 mm × 200 mm bar for shear connectors

In grade 40 concrete, nominal static strength = 963 kN — BS 5400: Part 5 Table 7

No. of connectors required at ULS $= \dfrac{6494 \times 1.4 \times 1.1}{963} = 11$

No. of connectors required at SLS $= \dfrac{4941\,´\,1.85}{963} = 10$

$\therefore$ *Use 12 connectors to suit transverse reinforcement layout*

Longitudinal shear

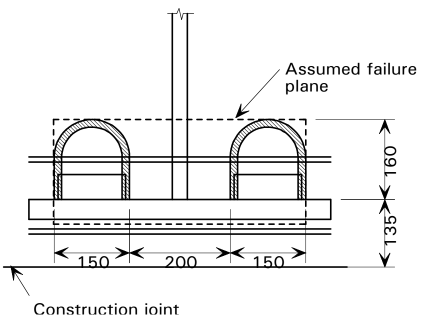
Clause 6.3.3.2

The longitudinal shear force per unit length q_p on any shear plane through the concrete should not exceed the lesser of the following:

(a) $(k_1, f_{cu}\ L_s)/\gamma_{f3}$

(b) $(v_1, L_s + 0.7\ A_e f_{ry})/\gamma_{f3}$

Assume shear failure plane around connectors and along underside of bottom flange.



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CALCULATION SHEET

Job No: *OSC397*	Page *46* of *50*	Rev *A*	
Job Title	*Design of a single-span integral steel bridge*		
Subject	*Capping beam design*		
Client	Made by *JAW*	Date *Dec 1996*	
	Checked by *EDY*	Date *Dec 1996*	

Length of plane $= 2 \times 160 + (500 - 15) = 805$ *mm*

Longitudinal shear force per unit length, $q_p = \dfrac{F}{d}$

where: F = *ULS force*
d = *depth of embedment*

F = *6494 kN (ULS)*

From (a) $d \geq \dfrac{6494 \times 10^3 \times 1.1}{(0.15 \times 40 \times 805)} = 1495\ \text{mm} < 1.6\ \text{m} \therefore \text{OK}$

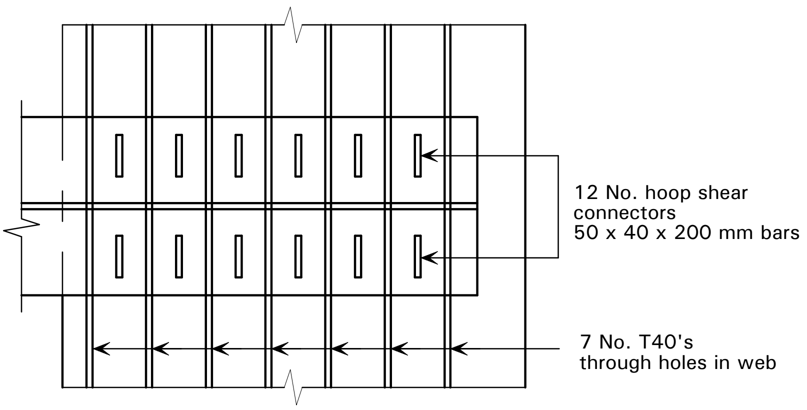
From (b) $\left(\dfrac{6494}{1.60}\right) \times 1.1 \leq 0.9 \times 805 + 0.7 \times A_e \times 460$

$A_e \geq \dfrac{(4465 - 725)}{322} = 11.6\ \text{mm}^2/\text{mm}$

Over depth of embedment, area required $= 11.6 \times 1494 = 17330\ \text{mm}^2$

Area provided by T40 $= 2A_b = \dfrac{2\pi \times 40^2}{4} = 2573\ \text{mm}^2$

$\therefore$ *Requires 17330/2573* = *6.7, i.e. 7 bars*



Bottom flange transverse reinforcement

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CALCULATION SHEET

Job No: OSC397	Page 47 of 50	Rev A	
Job Title	Design of a single-span integral steel bridge		
Subject	Capping beam design		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

TOP FLANGE SHEAR CONNECTION

Since axial load opposes the force due to moment, design the connection using the forces due to moment only.

ULS top flange design force = *1513 kN*

SLS top flange design force = *1183 kN*

Try 25 mm diameter, 100 mm high shear studs — *Table 7*

Nominal capacity = *168 kN*

No. of studs required is the greater of:

$$\frac{1513 \times 1.4 \times 1.1}{168} = 15 \text{ (ULS)}$$

$$\frac{1183 \times 1.85}{168} = 13 \text{ (SLS)}$$

Provide 8 rows of 3 studs @ 150 crs

Longitudinal shear

Studs on top flange of girder: $L_s = 2 \times 220 =$ *440 mm* — *Clause 6.3.3.2*

From (a) $$d \geq \frac{1513 \times 10^3 \times 1.1}{(0.15 \times 40 \times 440)} = 630 \text{ mm - OK}$$

For embedment depth, d, = 8×150 = *1200 mm*

From (b) $$\left(\frac{1513 \times 10^3}{1200}\right) \times 1.1 \leq 0.9 \times 440 + 0.7 \times A_e \times 460$$

$$\therefore A_e \geq \left(\frac{1387 - 396}{322}\right)$$

$= 3.1 \text{ mm}^2/\text{mm}$

Over embedded length = 3.1×1200 = 3720 mm^2

Try T20s: *For each bar,* $A_e = 2 \times \pi \times 20^2/4 = 628 \text{ mm}^2$

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CALCULATION SHEET

Job No: *OSC397*	Page *48* of *50*	Rev *A*	
Job Title	*Design of a single-span integral steel bridge*		
Subject	*Capping beam design*		
Client	Made by *JAW*	Date *Dec 1996*	
	Checked by *EDY*	Date *Dec 1996*	

$\therefore$ *No. of bars required* $= \dfrac{3720}{628} =$ *5.9, i.e. 6 bars*

DECK TENSILE REINFORCEMENT

Design hogging moment = 8439 kNm — *sheet 32*

$Z_{rt} = 4.580 \times 10^{10}/(1600 + 220 - 40 - 1050)$ — *sheet 1*

$= 6.274 \times 10^{7}\ mm^{3}$

$\therefore$ *ULS stress in reinforcement* $= \dfrac{8439 \times 10^{6}}{6.274 \times 10^{7}}$

$= 134\ N/mm^{2}$

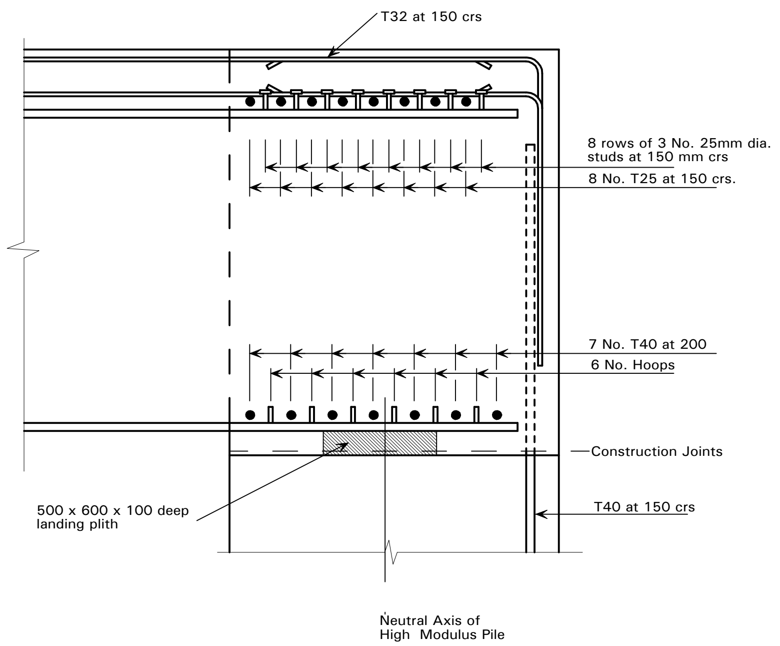
Anchorage length for T32 @ ULS: — *Part 4 Clause 5.8.6.3*

$$\frac{\pi \times 32^{2}}{4} \times 134 \times \left(\frac{1}{\pi \times 32 \times 3.3}\right) = 325\ mm$$

$\therefore$ *no problem anchoring deck steel to pile-cap.*

To provide continuity of reinforcement around the 90° bend, provide T32 'L' bars at 150 crs, lapped with T32s from deck and T40s from lower capping beam.

__Note:__ The reinforcement in the region of the deck girder is densely distributed. A more efficient design may result from reducing deck reinforcement and increasing plate girder flange sizes.



Vertical section through upper capping beam at deck girder

Discuss me ...

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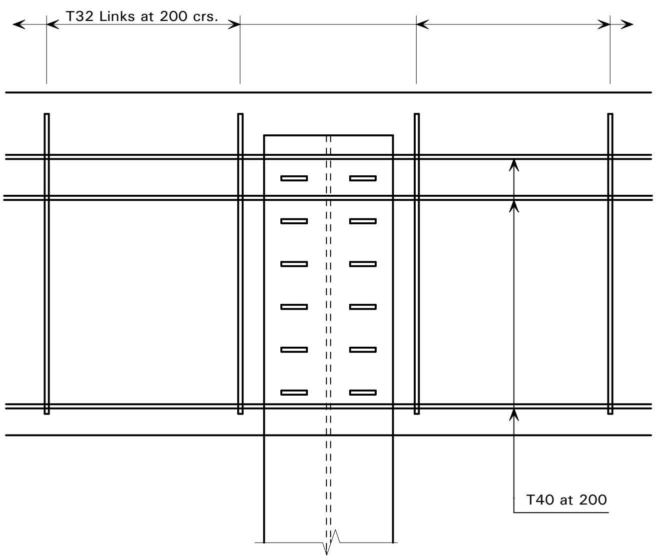
CALCULATION SHEET

Job No: OSC397		Page 49 of 50	Rev A
Job Title	Design of a single-span integral steel bridge		
Subject	Capping beam design		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	



Horizontal section showing bottom flange connection

4. CAPPING BEAM TORSION (ELEMENT 1)

ULS hogging moment from deck $= 8439 \times 1.1$ $=$ 9238 kNm — *Sheet 32*

$\therefore$ *Torsion at section either side of girder* $=$ 4641 kNm

For simplicity multiply load by γ_{f3} at this stage in accordance with Part 4 format.

(This could be reduced by about 20% to allow for moment transferred to wall over width to girder)

Assume that section resisting torsion has section dimensions 1550 × 1820

$$v_t = \frac{2 \times 4641 \times 10^6}{1550^2\,(1820 - 1550/3)} = 2.96\ N/mm^2$$

BS 5400: Part 4 Clause 5.3.4.4 Equation 9(a)

Assuming 100 mm cover to tension reinforcement:

Shear stress, $\dfrac{V}{b\,d} = \dfrac{514 \times 1.5 \times 10^3}{1550 \times 1720}$

$$= 0.28\ N/mm^2$$

Total ULS shear stress $= 2.96 + 0.28 = 3.24\ N/mm^2$

$v_{tmin} = 0.42\ N/mm^2$ $\therefore$ *torsion links are required*

Part 4: (Table 10 for 40 N/mm concrete)

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CALCULATION SHEET

Job No: OSC397		Page 50 of 50	Rev A
Job Title	Design of a single-span integral steel bridge		
Subject	Capping beam design		
Client	Made by JAW	Date Dec 1996	
	Checked by EDY	Date Dec 1996	

Calculate torsion links required: — *Part 4 Equation 10(a)*

$$\frac{A_{st}}{S_v} = \frac{T}{1.6\,X_1\,Y_1\,(0.87\,f_{yv})}$$

say $X_1 = 1400$ mm and $Y_1 = 1600$ mm

$$\frac{A_{st}}{S_v} = \frac{4641\times10^6}{1.6\times1400\times1600\times(0.87\times460)}$$

$$= 3.23\,mm$$

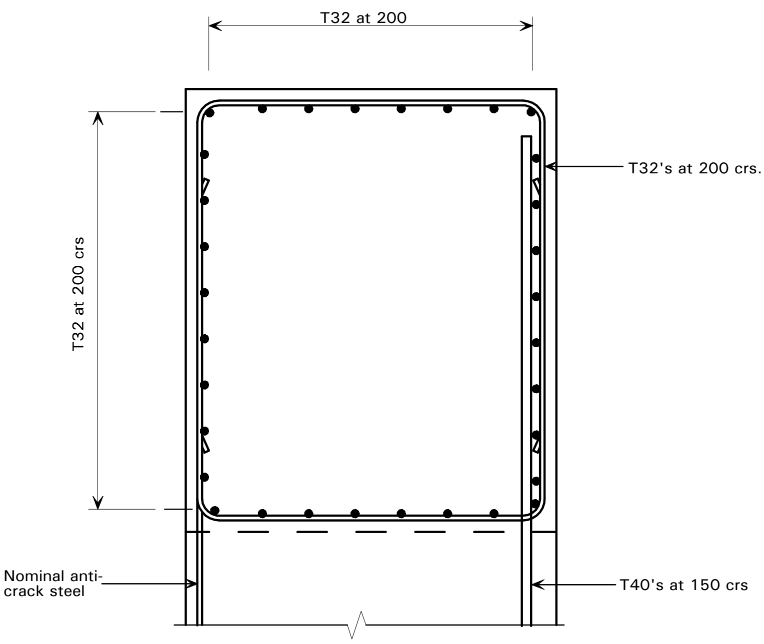
If bar spacing $=$ *200 crs*

$A_{st} = 3.23 \times 200 = 646\ mm^2$

Implies a bar diameter of $\left(\frac{4\times646}{p}\right)^{1/2} = 28\,mm$

If only 80% of torsion used (see Sheet 49) T25 would just be ok

∴ *T32 @ 200 crs OK* *Provide same reinforcement for longitudinal steel*



Vertical section through upper capping beam between deck girders

The upper capping beam will also be subject to bending and shear due to the horizontal force between deck girders. These effects are likely to be small and have not been checked in this Example. It is suggested that the shear force across the construction joint could be resisted by the provision of reinforcement dowels (extending from the lower capping beam after construction stage 2 - see page 9) but Part 4 will require link rebars.