









Composite Highway Bridge Design: Worked Examples



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Composite highway bridge design: Worked Examples

In accordance with Eurocodes and the UK National Annexes

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FOREWORD

This publication is the second of two SCI bridge design guides that reflect the rules in the Eurocodes. It gives two worked examples, one for a multi-girder bridge and one for a ladder deck bridge. It is a companion to a publication giving general guidance on composite highway bridge design.

The guidance in this publication has been developed from earlier well-established guidance in a number of SCI bridge design guides. The previous guides referred to BS 5400 for the basis of design.

The publication was prepared by David Iles, of The Steel Construction Institute. A technical review of the examples, to confirm compliance with the Eurocode rules, was carried out by Atkins. Thanks are expressed to Chris Hendy, Rachel Jones and Jessica Sandberg, all of Atkins, for their comments.

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[†] This publication includes references to Corus, which is a former name of Tata Steel in Europe

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SUMMARY

This publication presents worked examples of the detailed design of two composite highway bridges. Each bridge is formed by steel girders acting compositely with a reinforced concrete deck slab. The first example is of multi-girder form, the second is of ladder-deck form. The examples cover the principal steps in the verification of the designs in accordance with the Eurocodes, as implemented by the UK National Annexes.

The publication is complementary to SCI publication P356, *Composite highway bridge design*, which describes both forms of construction and presents general guidance and an introduction to the relevant detailed requirements of the Eurocodes.

INTRODUCTION

This publication presents two worked examples of the design of composite highway bridges using beam and slab construction. The evaluations of design values of actions (loads), action effects (bending moments, shears, etc.) resistances (of cross sections and of members in buckling) and limiting SLS criteria are carried out in accordance with the Eurocodes, as implemented by the UK National Annexes. Reference is made to selected documents providing non-contradictory complementary information.

References are made in the right-hand margins of the sheets to relevant clauses of the Eurocode Part, National Annex or other document. For brevity, the Eurocodes are designated as, for example, '3-1-5', meaning BS EN 1993-1-5 and its National Annex. National Annex clause numbers are all prefixed 'NA'.

The two examples are:

- 1. A two-span integral bridge, each span 28 m, carrying a two-lane roadway. The reinforced concrete deck acts compositely with four main girders of constant depth. The example shows the calculation of action effects (from the results of a computer global analysis) and the verification of the main girders in bending and shear. The adequacy of a bolted splice in the main girders is verified. Fatigue assessment is carried out for certain key details.
- 2. A three-span ladder deck bridge, spans 24.5 m, 42 m , 24 5 m, also carrying a two-lane roadway. The reinforced concrete deck acts compositely with a ladder-deck configuration of two main girders, at 11.7 m centres, and cross girders at 3.5 m centres. The main girders are of variable depth. The example shows the calculation of action effects (from the results of a computer global analysis) and the verification of the main girders and cross girders in bending and shear. The adequacy of the bolted connection between main girders and cross girders is verified. Fatigue assessment is carried out for certain key details.

The detailed design of the deck slab, for local loading, is not covered in either example.

WORKED EXAMPLE 1: Multi-girder two-span bridge with integral abutments

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example Section 2	1: Multi-gi 2: Design ba	rder two-sp asis	oan bric	lge				
CALCULATION SHEET	Client			Made by	DCI	Dat	е	July 2	2009	
	SCI			Checked by	JMS	Dat	е	Sep 2	.009	
2 Design basis										
The bridge is to be designed in a National Annexes.	accordan	ce with th	e Eurocode	s, as modif	ied by	the U	JK			
The basis of design set out in E	N 1990 i	s verificat	ion by the p	partial facto	or metho	od.				
In this example the ultimate limit design situations, using the com	it state S bination	TR/GEO	is verified for given by (6	or persister 5.10):	nt/transi	ent				
$E\left(\sum \gamma_{\mathrm{G},j}G_{\mathrm{k},j} + \gamma_{\mathrm{P}}P + \gamma_{\mathrm{Q},1}Q_{\mathrm{k},1} + \right)$	$-\sum_{i>1}\gamma_{\mathbf{Q},i}\psi$	$\left(\boldsymbol{Q}_{\mathrm{k},i} \right)$						EN 1 (6.10	1990))	
The fatigue limit state is verified of the simplified fatigue load mo	l for the odel (see	reference below).	stress range	e due to the	e applic	atior	1	Secti	on 3.	3
Stresses in the structural steel, c serviceability limit state for the	oncrete a	and reinfo ristic com	rcement are bination of a	e verified at actions give	the the by (6	5.14t))			
$E\left(\sum G_{\mathbf{k},j} + P + Q_{\mathbf{k},1} + \sum_{i>1} \psi_{0,i}Q_{\mathbf{k}}\right)$	(x,i)							EN 1 (6.14	1990 Ib)	
Crack widths in the deck slab ar permanent combination of action	e verifie 18 given	d at the so by (6.16b	erviceability)	limit state	for the	qua	si-			
$E\left(\sum G_{\mathbf{k},j} + P + \sum_{i\geq 1} \psi_{2,1} Q_{\mathbf{k},i}\right)$								EN 1 (6.16	1990 5b)	
2.1 Partial factors on action	าร									
For persistent design situations,	partial fa	actors on	actions at U	LS are give	en by tl	ne N	A:			
Permanent actions	Unf a	avourable actions	_					BS E NA.	EN 19 A2.40	90 B)
Concrete self weight	γG	1.35	Each of the	ese actions i	s repres	ente	d			
Steel self weight	γG	1.20	factors for	favourable a	actions v	; voul	d			
Super-Imposed dead	γG 15	1.20	only be need	eded if the t	otal effe	ct of	:	1-1-	1/5.1	(1)
	/G	1.20	the action	were favour	able.					
Weight of soil	γG	1.35								
Variable actions	₽G	1.55								
Road traffic actions	νo	1.35								
Pedestrian actions (gr3, gr4)	γο	1.35								
Wind actions	γα	1.70								
Thermal actions	γα	1.55								
No values are given for transien assumed that the above factors f	t situatio or perma	ons (such a anent actio	as during coons may be	nstruction) used.	but it i	S				
The partial factor on shrinkage 2 EN 1992-1-1.	_{7Sh} is set	at unity fo	or both ULS	S and SLS	by			2-1-	1/2.4	.2.1

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SCI	Job Title Co	omposite	e highway	bridges: W	orked o	example	es		
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2.2 Easters for combination				I					
Factors for combination values	of actions ar	e given	by the NA	A to BS 199	00 as:				
	Wo	W1	W2				BS F	EN 19	990
LM1 - TS	0.75	0.75	<i>₽</i> - 0				NA.	A2.1	//0
LM1 - UDL	0.75	0.75	0						
Footway loads with LM1	0.4	0.4	0						
LM2 Single Axle	0	0.75	0						
Horizontal Forces	0	0	0						
gr5 vertical forces from SV	0	0	0						
vehicles									
Wind persistent design	0.5	0.2	0						
situations	0.5	0.2	0						
Wind during execution	0.8	-	0						
Wind during execution (Fw*)	1	-	0						
Thermal actions	0.6	0.6	0.5						
2.3 Factors on strength									
The values of the various $\frac{1}{2}$, parti	ial factors		ULS	SLS			3-2/	61	
are given by the NA to BS EN 19	993–2 as:	2 M0	1.00	525			NA.	2.17	
		γ	1.10						
		γ M2	1.25						
		<i>у</i> М3	1.25	1.10					
The values of the partial factors f given by the NA to BS EN 1992-	for strength or $\gamma_{\rm C} = 1-1$ as $\gamma_{\rm C} = 1-1$	of concre 1.5 and	te and reir 1 $\gamma_{\rm S} = 1.1$	nforcement a 5.	at ULS	are	2-1- Tabl	1/2.4 e NA	.2.4, 1
2.4 Structural material prop	perties								
It is assumed that the following	structural m	aterial g	grades will	be used:					
Structural steel: S355 to EN	10025-2								
Concrete: C40/50 to E Reinforcement: B500 to EN	N 206-1 10080 and 1	BS 4449	1						
For structural steel, the value of	$f_{\rm v}$ depends	on the p	oroduct sta	ndard.			NA	to 3-1	1-1
(Use 355 N/mm ² for $t < 16$ mm	• 345 N/mm	1^{2} for 16	mm > t	< 40 mm·	and		2-1-	1,	
$335 \text{ N/mm}^2 \text{ for } t > 40 \text{ mm}$, 545 10/1111	1 101 10	11111 <i>× i</i>	<u>→</u> +0 mm,	and		Tabl	e 3.1	
For concrete, $f_{ck} = 40$ MPa									
For reinforcement $f_{yk} = N/mm^2$									
The modulus of elasticity of bot 210 GPa (as permitted by EN 19	h structural 994-2).	steel and	d reinforci	ing steel is	taken a	S	4-2/3	3.2	

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CALCULATION SHEET	Client SCI		Made by	DCI	Date	July	2009			
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The modulus of elasticity of the	concrete	e is given by EN 1992	2-1-1 as:			2-1-2	1/			
$E_{\rm cm} = 35$ GPa. This 28-day value of fects and resistances and the n	ue will b nodular r	e used for determinat atio is thus	ion of all s	hort-ter	m	Tabl	e 3.1			
$n_0 = 210/35 = 6.0$										
For long-term effects, the modu	lar ratio	is given by 4-2/5.4.2	.2 as:			4-2/	5.4.2.	2		
$n_{\rm L} = n_0 (1 + \psi_{\rm L} \varphi_{\rm t})$										
For the evaluation of the creep of the first loading is applied at an humidity is 70%.	coefficier average	ant φ_t (= $\varphi(t,t_0)$ in 2-1 age of t_0 = 21 days	-1/B.1) it i and that the	s assum e relativ	ed tha e	t				
For $t \to \infty \varphi(t,t_0) = \varphi_0$										
Where $\varphi_0 = \varphi_{\rm RH} \beta(f_{\rm cm}) \beta(t_0)$						2-1-1	l/B.1			
For $f_{\rm cm} > 35$ MPa (here $f_{\rm cm} = 48$	8 MPa, f	rom 2-1-1/Table 3.1)				2-1-				
$\varphi_{\rm RH} = \left[1 + \frac{1 - RH / 100}{0.1 \sqrt[3]{h_0}} \alpha\right]$	$\left[\alpha_{2}\right]$					1/1a 2-1-3	ble 3. $1/(B.3)$. 1 3a)		
$\alpha_1 = \left[\frac{35}{f_{\rm cm}}\right]^{0.7} = \left[\frac{35}{48}\right]^{0.7} =$	0.802					2-1-3	l/(B.8	Bc)		
$\alpha_2 = \left[\frac{35}{f_{\rm cm}}\right]^{0.2} = \left[\frac{35}{48}\right]^{0.2} =$	= 0.939									
For a 250 thick slab $(h_0 = 250)$										
$\varphi_{\rm RH} = \left[1 + \frac{1 - 70 / 100}{0.1 \sqrt[3]{250}} 0.8\right]$	0.93	39 = 1.298								
$\beta(f_{\rm cm}) = \frac{16.8}{\sqrt{f_{\rm cm}}} = \frac{16.8}{\sqrt{48}} = 2.4$	2									
$\beta(t_0) = \frac{1}{\left(0.1 + t_0^{0.20}\right)} = \frac{1}{\left(0.1 + t_0^{0.20}\right)} = \frac{1}{\left(0.1 + t_0^{0.20}\right)}$	$\frac{1}{21^{0.20}}$	= 0.516				2-1-3	1/(B.5	5)		
Hence										
$\varphi_0 = 1.298 \times 2.42 \times 0.516 =$	1.621									
For permanent loads, $\psi_{\rm L} = 1.1$	and thus	3:				4-2/	5.4.2.	2(2)		
$n_{\rm L} = 6.0(1+1.621 \times 1.1) = 6$	$.0 \times 2.7$	9 = 16.7								
Long term modulus = $210/16.7$	= 12.6	GPa								

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CALCULATION SHEET	Client SCI		Made by	DCI	Dat	е	July 2	2009	
	ber		Checked by	JMS	Dat	е	Sep 2	2009	
If the design effects need to be a will need to be modified to refle average age at which the perman opening to traffic is 56 days. The $\beta_{\rm c}(t, t_0)$ and in this case, with	determine ect the shiftenent action the creep of $t-t_0 = 2$	ed at the time of oper ort duration of loadin ons are imposed is 21 coefficient is modified 35 days, $\beta_c = 0.418$	ning, the cr ng. In this e days and t d by the pa	eep coe example the age rameter	effici e, the at	ent e	2-1-3	l/ (B.	.7)
Thus, for permanent loads at op (2)	ening:								
$n_{\rm L} = 6.0(1 + 0.418 \times 1.621 \times 1.621)$	(1.1) = 6	$0.0 \times 1.75 = 10.5$							
And modulus = $210/10.5 = 20$.0 GPa								
Shrinkage The shrinkage strain on the cond by EN 1992-2. The values dependent are considered - at bridge openin and at the end of the design life. For shrinkage, the age at loadin	crete decl nd on the ng, for w , for whi g (i.e. at	k and the appropriate e age since casting; ir which an average age ch it is assumed that age $t_s = 1$, the begin	modular rational modular modular rational modular modular modular modular modular modular modular modular rational modular rational modular rational modular rational modular rational modular modu	atios ar ple two is assu	e giv age med, inka	yen s	4-2/5	5.4.2	.2(4)
in $2-1-1/3.1.4$) is assumed to be	one day			8		5-			
The autogenous shrinkage strain	at $t = c$	∞ is:					2-1-2	1/3.1.	.4
$\varepsilon_{\rm ca}(\infty) = 2.5(f_{\rm ck} - 10) \times 10^{-6}$	= 2.5(4	$(0-10) \times 10^{-6} = 7.5$	5×10^{-5}						
At $t = 56$ days, the strain is given by	en by:								
$\varepsilon_{\rm ca}(t) = \beta_{\rm as}(t)\varepsilon_{\rm ca}(\infty)$									
Where $\beta_{as}(t) = 1 - \exp(-0.2t^0)$.5) = 1 -	$e^{-1.5} = 0.777$							
Thus $\varepsilon_{ca}(56) = 0.777 \times 7.5 \times 10^{-10}$	$10^{-5} = 5$	5.8×10^{-5}							
The drying shrinkage depends o expression (B.11) in B.2 (or by	n the nor interpola	ninal unrestrained dry	ying shrink EN 1992-1-	age, gi -1).	ven l	у	2-1-2	l/B.2	
$\varepsilon_{\rm cd,0} = 0.85 \times [220 + 110 \times \alpha_{\rm ds}]$	$_1 \times \exp(-$	$-\alpha_{\rm ds2} f_{\rm cm}/f_{\rm cmo}) \times \mu$	$(\beta_{\rm RH}] \times 10^{-6}$	5			2-1-2	l/(B.1	11)
For 70% relative humidity, f_{ck} =	= 40 MP	a and class N cement	:						
$f_{\rm cmo}$ = 10, $\alpha_{\rm ds1}$ = 4 and $\alpha_{\rm ds1}$ =	= 0.12						2-1-2	l/(B.1	12)
$\beta_{\rm RH} = 1.55 \left[1 - (RH/100)^3 \right] = 1.33$	55[1-0.7]	7^3 = 1.018							
$\mathcal{E}_{cd,0} = 0.85 \times [220 + 110 \times 4 \times 100]$	exp(- 0	$.12 \times 40/10) \times 1.018$	$[8] \times 10^{-6} =$	32 × 1	0^{-5}				
The drying shrinkage at time t is	s given b	by:							
$\varepsilon_{\rm cd}(t) = \beta_{\rm ds}(t, t_{\rm s})k_{\rm h}\varepsilon_{\rm cd,0}$									
Where $k_{\rm h} = 0.80$ (from Table 3	.3, with	$h_0 = 250$) and $\beta_{\rm ds}(t)$	$(t_s) = \frac{1}{t-t}$	$\frac{t-t_{\rm s}}{t_{\rm s}+0.0}$)4√ <i>I</i>	$\overline{\overline{n_0^3}}$	2-1-2	l/(3.1	.0)

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For $t = 56$ and $t_s = 1$ (see 4-2/	5.4.2.2(4	4)), $\beta_{\rm ds} = 0.258$						
For $t = \infty$, $\beta_{\rm ds} = 1$,							
Thus the drying shrinkage is:								
At $t = 56$ days $\varepsilon_{cd} = 0.258 \times$	0.80×3	$32 \times 10^{-5} = 6.60 \times 10^{-5}$	0 ⁻⁵					
At $t = \infty \varepsilon_{cd} = 0.80 \times 32 \times 10^{-10}$	$0^{-5} = 25$	5.6×10^{-5}						
The total shrinkage is thus:								
At $t = 56$ days $\varepsilon_{cd} = 5.8 \times 10$	$^{-5} + 6.6$	$10 \times 10^{-5} = 12.4 \times 1$	0^{-5}					
At $t = \infty \varepsilon_{\rm cd} = 7.5 \times 10^{-5} + 2$	25.6×10^{10}	$0^{-5} = 33.1 \times 10^{-5}$						
For the modular ratio, the creep age at first loading is assumed to	o factor i o be 1 da	s calculated as for logay. Thus:	ng term loa	ding bu	t the	4-2/	5.4.2.	.2(4)
$\beta(t_0) = \frac{1}{\left(0.1 + t_0^{0.20}\right)} = \frac{1}{\left(0.1 + t_0^{0.20}\right)} = \frac{1}{\left(0.1 + t_0^{0.20}\right)}$	$\left(\frac{1}{1^{0.20}}\right) =$	- 0.91				2-1-2	l/(B.5	5)
The final creep coefficient is cal $\beta(t_0) = 0.91$, and thus $\varphi_0 = 1$	culated a $1.298 \times$	as above for long term $2.42 \times 0.91 = 2.86$	m effects bu	it with				
For shrinkage, $\psi_{\rm L} = 0.55$ and t	hus:					4-2/3	5.4.2.	.2(2)
$n_L = 6.0(1 + 2.86 \times 0.55) = 6$	5.0×2.5	57 = 15.4						
At opening to traffic ($t = 56$ da $\beta_{\rm c}(t,t_0)$ and in this case $\beta_{\rm c} = 0$	ys) the c).475 and	reep coefficient is m d $n_{\rm L} = 10.5$.	odified by t	he para	meter	2-1-2	l/(B.7	7)
In this example, the shrinkage e values where they are unfavoura at 56 days could be considered l	ffects wi able. Wh but it is o	ll be taken into accou ere the effects are fa conservative to negle	unt at their vourable, th ct shrinkage	long ter ne lesser e in that	m value case.	es		

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 3: Actions o	rder two-sp n the bridg	pan brid ge	lge						
CALCULATION SHEET	Client		Made by	DCI	Dat	te	July 2	2009			
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CALCULATION SHEET 3 Actions on the bridge 3.1 Permanent actions Self weight of structural elem The 'density' of steel is taken as taken as 25 kN/m ³ . The self weight Self weight of surfacing The total nominal thickness of th Assume that the 'density' is 23 L The self weight generally produce based on nominal thickness + 55 $g_k = 1.55 \times 0.13 \times 23 = 4$. Self weight of footway conse The nominal thickness of the foot 200 mm and a uniform density of the nominal dimensions and thus $g_k = 1.0 \times 0.2 \times 24 = 4.80$ Self weight of parapets A nominal value of 2 kN/m is a Self weight of soil The density of the granular fill the 3.2 Construction loads Construction loads are classed at For global analysis, a uniform c during casting and the weight of $Q_{cc} = 0.50 \text{ kN/m}^2$. Additionally 1 kN/m ³ greater than that of har adds $Q_{cf} = 0.25 \text{ kN/m}^2$ The total construction load is the 3.3 Traffic loads Road traffic Normal traffic is represented by	Client SCI SCI nents 77 kN/r sights are he surfactor kN/m^3 for ces adver 63 kN/m truction 5%. Thus 63 kN/m ² truction 5% kN/m ² ssumed f behind th s variable onstruction tempora , wet con dened co 13 : $Q_c =$	m ³ and the density of based on nominal di ing, including waterp r the whole thickness rese effects and for that 2 omprising concrete fil /m ³ is assumed. The second for each parapet. e integral abutments is e loads. on load of $Q_{ca} = 0.72$ ry formwork is assum- ncrete is assumed to H oncrete; for a slab thic 0.75 + 0.50 + 0.25 odel 1 (LM1).	Made by Checked by Checked by reinforced mensions. roofing lay t case the s l and a thin self weight is taken as 5 kN/m^2 is ned to be have a dens ckness of 2 5 = 1.5 kN	DCI JMS concret er is 13 self wei n surfac is base 21 kN/n assume sity of 50 mm	Dat Dat Dat Dat d at d at d at d at d at d at d at d	is is	July 2 Sep 2 1-1-1 A.1 1-1-1 NA.1 1-1-6 1-1-6 Table	2009 009 //Tab //Tab	le		
For the road carried by this brid traffic be represented by special	ge, the h vehicle s	highway authority spe SV100, as defined in	cifies that a the UK Na	abnorma ational A	al Anne	ex.	1-2/ NA.2	2.16.	1.2		

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SCI	Job Title	Job Title Composite highway bridges: Worked example								
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 3: Actions of	rder two-spon the bridg	pan bric je	lge					
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009			
	501		Checked by	JMS	Date	Sep	2009			
Pedestrian traffic Pedestrian traffic is represented BS EN 1991-2, Table NA.3 and $(= 0.6 \times 5.0 = 3 \text{ kN/m}^2)$. The Fatigue loads For fatigue assessment, Fatigue as recommended by 3-2/9.2.2	by the relicion for the second	educed value given by NA.2.36. Thus $0.6q_{fk}$ n for longer loaded le odel 3 (FLM3), defin	the NA to is applied ngths is no ed in 1-2/4	, t made. .6.4, is	used	1-2 NA	/Table 3	2		
2.4 Thermal actions										
Shade temperatures Maximum and minimum shade a period are defined in EN 1991-1 Maximum 33°C Minimum -17°C	air tempe 5 NA.2	eratures for the UK, f 2.20. For this bridge l	for a 50-yea	ar return e values	n s are:	1-1 NA	-5/ 2.20			
Thermal range of effective b	ridge te	mperature (for dete	ermination	of soil						
pressures) For the purposes of determining temperature for a 50 year return initial restraint position.	soil pres period i	ssure, the total range s relevant, not the ra	of effective nge from a	e bridge n assum	ened	PD 6694-1 ¹³ clause 7.4.2 (draft)				
The values of maximum/ minim EN 1991-1-5, 6.1.3.1; these are	um unifo referred	form bridge temperature $T_{e,min}$ and $T_{e,max}$	res are give	en by						
For Type 2 deck (concrete slab	on steel	girders)								
$T_{e,max} = T_{max} + 4$ (EN 1991-1- $T_{e,min} = T_{min} + 5$	5, Figur	e 6.1)								
Hence the total range = $(33 + $	4) - (-	$17 + 5) = 49^{\circ}C$								
(The adjustments for surfacing the adjustments for surfacing the asymptotic structure of the structure of th	hickness nd have	over 100 mm given b been neglected.)	y the NA w	ould re	sult i	n 1-1 NA	-5/ 2.4			
Thermal range (for determination of the maximum design life are relevant but according to the plying $\gamma_Q = 1.55$ to character	ution of um move rding to istic valu	extreme value of th ement at ULS, the value of the valu	nermal mo lues for a 1 re determin rn period.	20 yean 20 yean aed by	it)	Tał NA No	ole A2.4 te 6	·(B)		
For change of length in compositions is 12×10^{-6} per °C.	ite sectio	ns, the coefficient of	linear ther	nal exp	ansio	n 4-2	/5.4.2	.5		
Vertical temperature different The vertical temperature different temperature difference will be con- temperature change, as recommends surfacing thickness other than 10	ce nce giver onsidered ended in 00 mm, i	n in EN 1991-1-5, Ta d to act simultaneousl NA.2.12, if that is m interpolate in 1-1-5/Ta	ble 6.2b with unif y with unif nore onerou able B.2, a	ill be us form is. For s follov	sed an vs:	nd				

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SCI		Job Title	Composite highway bridges: Worked examples										
Silwood Park, Ascot, I Telephone: (01344) 6 Fax: (01344) 636570	Berks SL5 7QN 36525	Subject	Example 1: Multi-g Section 3: Actions of	irder two-sj on the bridg	pan brid ge	lge							
CALCULATION S	HEET	Client		Made by	DCI	Date	July	2009					
		SCI		Checked by	JMS	Date	Sep 2	Sep 2009					
Surfacing thickness (mm)	Surfacing ΔT_1 for slab thickness (mm) thickness (mm) 200 300												
100 150	13 10.5	1	16 2.5										
Interpolating for sl $\Delta T_1 = 12.7^{\circ}$.	ab thickness 2	250 mm,	surfacing thickness	130 mm, gi	ves								
(The 55% increase ignored.)	over nomina	l thicknes	ss, where surfacing la	oad is adve	rse, is								
For temperature di is 10×10^{-6} per °C	fference in co	omposite	sections, the coeffici	ent of thern	nal expa	ansion	4-2/:	5.4.2	.5				
3.5 Geotechnic	al actions												
Design values of At the ULS that de thermal actions ma movement (from m	thermal mo esigns the sup by be consider nean position)	vements erstructur ed as an is:	s giving rise to geo re, traffic loads are to accompanying action	technical he leading a h. In that ca	actions action as use the	nd							
$(1.2 \times 10^{-5} \times 2800)$	$00 \times 49/2) \times 2$	$\psi_{\rm Q} \times \psi_0 =$	$= 8.23 \times 1.55 \times 0.60$	= 7.65 m	m								
For maximum axia the leading action a	Il force due to and the move	o restrain ment (fro	t of temperature, the om mean position) we	thermal action thermal action the second sec	tion wo	uld be							
$8.23 \times \gamma_{\rm Q} = 8.23$	\times 1.55 = 12	.8 mm.											
Accompanying LM (since $\psi_1 = 0.75$ f	11 traffic load for gr1a). (LN	s would 13 is not	be at 75% of their va considered as an acc	alue as a lea ompanying	ading ac action.	tion)	Shee	et 3					
Soil pressure coe	efficients												
Soil pressure coeff To determine the n the value of the tot point and the K_0 va	ficient K* naximum soil cal thermal mo alue for the so	pressure ovement bil.	on the endscreen war range, the height of t	all, PD 669 the wall abo	4-1 require the	iires pivot	PD ((dra:	5694- ft)	·1 ^[3]				
The PD gives:													
$d = \alpha L_{\rm x}(T_{\rm e,max})$	$-T_{\rm e,min}$)												
NOTE - The PD does not refer to characteristic values or to frequent values, it simply ignores the design basis; it could be argued that it is the frequent value as a leading action that determines the pressure coefficient, in which case the partial factor γ is unity and the factor ψ_1 should be applied ($\psi_1 = 0.6$ according to the UK NA)													
Here, following the	e PD												
$d = 1.2 \times 10^{-1}$	$5 \times 28000 \times 4$	49 = 16.	5 mm										
(i.e. \pm 8.25 about the mean position)													

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SCI	Job Title	Composite highway	bridges: W	orked	examp	amples								
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 3: Actions of	irder two-spont the bridg	pan bri ge	dge									
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009							
	301		Checked by	JMS	Date	Sep 2	2009							
For the present configuration, we the movement is essentially transby sliding is more applicable.	For the present configuration, where the piles are much more flexible than the deck, the movement is essentially translational and thus the expression for K^* for movement by sliding is more applicable.													
Thus the multiplier on K_p is (40)	× 16.5/22	$(250)^{0.4} = 0.612$				PD	6994-	-1,						
And thus $K^* = K_0 + 0.612K_p$						7.4.	5							
Assume for a granular fill that $K_0 = 0.5$ and $K_p = 4.6$ (K_p from PD) and apply modelling factor $\gamma_{\text{Sd;K}}$ to K_p .														
$K^* = 0.5 + 0.612 \times (4.6 \times 1.2)$	2) = 3.88	(not more than $K_{\rm p}$	- OK)											
Movement - pressure coeffic	ient diag	ram												
At any time, the soil pressure co	pefficient	for characteristic val	lues of action	ons lies	withi	n								
the envelope shown diagrammat	ically belo)W.												
		K*												
ant K														
afficie		Ļ												
ssure														
bre		K ₀												
│														
position at $\theta_{\min,k}$ mean position	tion p	osition at $\theta_{\max,k}$												
The most unfavourable pressure	, in terms ent K^* T	of the greatest horiz	zontal force	e in the	deck,	of								
thermal expansion from the mea	in position	1. At the ULS design	1 value of ϵ	expansion	on, the	$\stackrel{n}{=} PD$	6694- ft)	-1						
displacement is greater but the v	alue of K	* may still be used	(see PD 66	94-1).		(ura	11)							
When thermal expansion is an a	ccompany	ring action at ULS, t	he moveme	ent from	n the									
characteristic. The value of K^*	= 3.88 w	ill be used here for	both leadin	g and										
accompanying thermal actions.														
Vertical soil pressures				.1 77										
pressures (see PD 6694-1, claus	tot need to $(7.5.1)$.	b de considered in co	onjunction	with K^{2}	P									
	ץG Or γΩ	Characteristic valu kN/m ²	e Desig	n value kN/m²	(ULS)									
Road surfacing	1.2	4.63		5.6										
Total at top of wall	1.35	2.25 × 21		აკ. 5.6										
Total at bottom of wall				69.4										

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SCI	Job Title	Composite highway	v bridges: W	/orked	examp	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi- Section 3: Actions	girder two-s on the bridg	pan brio ge	lge			
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009	
	SCI		Checked by	JMS	Date	Sep 2	2009	
Horizontal soil pressures The design value of horizontal p leading or an accompanying acti	oressure a on is:	at ULS when therma	l actions is	either a				
	Press	ure						
At top of slab $5.6 \times$ At bottom of wall $69.4 \times$	3.88 = × 3.88 =	= 22 kN/m ² =269 kN/m ²						
The pressure is applied to the er	nd diaphi	ragms as a hydrostat	ic pressure.	icient is	· ·			
$K_0 + (K^* - K_0) \times 1.6/2.0 = 0.5$	5 + 3 38	x = 3.21 (= 8)	3% of K^*	i viviti 18				
H () + (H H () × H () Z (0) U (Z)	1 5.50		5 /0 OF IX)					

					1			1	
		Job No.	BCR113		Sheet	12 of	64	Rev	Α
SCI		Job Title	Composite highway	bridges: W	orked e	exampl	es		
Silwood Park, Ascot	t, Berks SL5 7QN	Subject	Example 1: Multi-g	irder two-sj	pan brid	lge			
Telephone: (01344) Fax: (01344) 63657	636525 70		Section 4: Girder m	ake-up and	slab re	inforce	ment		
CALCULATION	SHEET	Client		Made by	DCI	Date	July	2009	
		SCI		Checked by	JMS	Date	Sep 2	2009	
4 Girder m	ake-up and s	slab rein	nforcement 28000						
	21.7 m span	girder	12.6 m pier girder	21.7 m sj	oan girde	er			
Top flange	500 × 40)	500 × 40	500	× 40				
Web	12	10	14	10	12				
Bottom flange	500 × 40)	600 × 60	500	× 40				
Top rebars	B16 @ 150 m	m crs	B25 @ 150 mm	B16 @ 15	i0 mm c	rs			
Bottom rebars	B16 @ 150 m	m crs	B25 @ 150 mm	B16 @ 15	60 mm c	rs			
Bracing arrange	m bars is 60 mm (4 lso 2-2/4.2 and the ements	0 mm + 20 respective	mm); this is appropriate to NAs.)	XC4. (See 2-1	-1/4.4.1.2	for			
700 700	810 590	590	810 700	700					
The above bracin design. The mod support coincide nodes; the bracin positions might a	ng arrangements el nodes and in with nodes in t gs are 400 mm lso be adjusted	s are non termedia he FE m closer t during f	ninal and might be ad te bracings other than odel. The splice posi o the supports than th inal design.	ljusted durin n either side tions coinci ne splice po	ng detai e of the de with sitions;	led centra model these			



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SCI	Job Tit	le Coi	mposite highw	vay	bridges: V	Vorked	example	es		
Silwood Park, Ascot, Berks SL5 7QN	Subject	t Exa	ample 1: Mult	i-gi	rder two-s	pan bri	dge			
Telephone: (01344) 636525 Fax: (01344) 636570		Sec	tion 5: Beam	cro	ss sections	5	U			
CALCULATION SHEET	Client				Made by	DCI	Date	July 2	2009	
	SCI				Checked by	JMS	Date	Sep 2	:009	
Bare steel cross sections										
			Span girder	P	ier girder]			
Area		Α	50200		70000	(mm²)				
Height of NA			550		436	(mm)		Prop	erties	In .
Second moment of area		/v	1.212E+10	1.	562E + 10	(mm ⁴)		calcu	latea Idshai	Dy of
Elastic modulus, centroid top flang	е	W _{tf,y}	2.287E+07	2.	425E+07	(mm ³)		sprea	usnet	-1
Elastic modulus, centroid bottom fl	lange	W _{bf.v}	2.287E+07	3.	847E+07	(mm ³)				
Section class		,,	4*	3	(hogging)	(,				
Plastic bending resistance		Mol	8237	•	9882	(kNm)				
* The section is only marginally Class 4. F	or stress	s build-u	p during construc	tion	the bare stee	l section n	」 nay be			
section at the wet concrete stage.	ne secu		iss 3 of Deller. Se	e sn	eet 31 101 mo	iaun or the	enective			
Note: As an example, the classifi	ication	for th	e span girder	is i	as follows:					
Flange outstand $c = (500 - 10)$	/2 = 2	245 m	m (welds negl	ect	ed)					
and thus $c/t = 245/40 = 6.12$								3-1-1	[/	
Outstand limit for class 1 is c/t^2	≥9ε =	$9 \times \sqrt{2}$	35/345 = 7.5,	so	flange is c	class 1		1 201	3.2	(2)
Depth of web c = $1100 - 2 \times 4$	40 = 1	l020 n	nm and thus c	/t =	= 1020/10	= 102		3-1-1	1/	
Limit for class 3 internal part is	$c/t \ge 1$	24 <i>ɛ</i> =	$124 \times \sqrt{235/35}$	55 =	=100.2, so	web is	class 4	Tabl	e 5.2	(1)
Composite cross sections (sh	ort te	rm) -	sagging (n.	_	6 0)					
			Span girder			[7			
Area		Δ		-	iei giluei	(mm^2)				
Alea		~	209800		1016	(mm)				
Second moment of area		1.	1098	Б	040E ± 10	(mm^4)		Value	of N	1
Second moment of area		14	3.288E + 10	5	.0402 + 10	(11111) (mama ³)		calcu	lated	- pi
Elastic modulus, top of slab		146	0.334E + 00			(11111) (mama ³)		using	f_v/γ_M	10
Elastic modulus, centroid top hang	e 	ννττ,γ	2 0505 + 07			(11111)		value	s for	
Elastic modulus, centroid bottom fi	lange	VVbt,y	3.050E+07			(mm²)		steel,	0.85	f_{ck}/γ_C
The cross section of the span girder is a connectors within the spacing limits in 4	lass 1, 1-2/6.6.	provide 5.5 (in 1	d that the top fla this case, max sp	nge bacii	is restrained ng 730 mm,	(KINM) by shear max edge	 9	for c	oncrei	te
distance 299 mm).	re need	ed for ca	loulation of shear	flow	1					
			. ,	non						
Composite cross sections (lo	ng ter	'm) - s	agging (<i>n</i> _L =	= 1	6.7)	T	٦			
			Span girder			1 2	-			
Area		A	107500			(mm²)				
Height of NA			934			(mm)				
Second moment of area		Iу	2.634E+10			(mm ⁴)				
Elastic modulus, top of slab		Wc	9.439E+08			(mm ³)				
Elastic modulus, centroid top flang	е	<i>W</i> tf,y	1.804E+08			(mm³)				
Elastic modulus, centroid bottom f	lange	<i>W</i> bf,y	2.882E+07			(mm³)				

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SCI	Job Tit	ile Co	mposite hi	ighway	bridges: W	/orked e	exampl	es	<u>.</u>	
Silwood Park, Ascot, Berks SL5 7QN	Subjec	t Ex	ample 1: 1	Multi-g	irder two-s	pan bric	lge			
Telephone: (01344) 636525 Fax: (01344) 636570		See	ction 5: Be	eam cro	oss sections					
CALCULATION SHEET	Client				Made by	DCI	Date	July	2009	
	SCI				Checked by	JMS	Date	Sep 2	2009	
Composite cross sections (ong ter	m sh	rinkage) -	· saggi	ng (<i>n</i> _L =	15.4)	1			
			Span gir	der		(2)	-			
Area		A	11240	0		(mm²)	-			
Height of NA			948			(mm)	_			
Second moment of area		Iу	2.684E+	10		(mm ⁴)				
Elastic modulus, top of slab		Wc	9.145E+	08		(mm ³)				
Elastic modulus, centroid top flar	ige	<i>W</i> tf,y	2.033E+	-08		(mm ³)				
Elastic modulus, centroid bottom	flange	<i>W</i> bf,y	2.892E+	07		(mm ³)				
			1				1			
Cracked composite sections	s - hogg	ging (cracked)			1	7			
			Span gir	der I	Pier girder					
Area		Α	74450)	94250	(mm²)				
Height of NA			788		653	(mm)				
Second moment of area		Iу	2.092E+	10 2	.845E+10	(mm ⁴)		Valu	e of N	I_{pl}
Elastic modulus, top rebars		W	3.806E+	07 4	.184E+07	(mm ³)	calculated			
Elastic modulus, centroid top flan	ige	W _{tf,y}	7.164E+	07 6	.663E+07	(mm ³)	using f_y/γ_M			10
Elastic modulus, centroid bottom	flange	W _{bf,y}	2.724E+	07 4	.567E+07	(mm ³)		value	s jor	for
Section class	-		3		3			reha	, Jyk/ /S r	5 101
Plastic bending resistance		Mpl			16990	(kNm)		rebu	,	
5.2 Primary effects of tem Temperature difference For calculation of primary effect $E_{cm} = 35$ GPa (For steel, E	peratucts, use $T = 210$	re dif the sh GPa)	ference 8	k shrin nodulus	kage s for concre	ete:		Shee	et 3	
Note: For each element of section then determine force and centre For a fully restrained section, t	on, call e of forc he restr	<i>culate</i> <i>ce for</i> raint fo	stress as s that area.	s <i>train ×</i> noment	<i>modulus c</i> in the spar	of elastic	<i>city,</i> , inner			
beam, due to the characteristic	values	of tem	perature d	lifferen	ce noted or	Sheet	8 are:			
	Av strair	n	Force (kN)	Cent Below top	re of force / Above NA	Mor e (kN	ment Nm)			
Top part of slab	0.0008	34	1632	62	240	3	92			
Bottom part of slab	0.00003	6	466	198	104	4	8			
Haunch	0.00003	80	34	274	28		1			
Top flange	0.00002	6	109	320	_18	_	2			
Web (to 400 below slab)	00001	2	5	410	_108		1			
		-	2246		-100	4:	38			
		-								

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SCI	Job Title	Composite highway	bridges: W	orked	exar	nple	s		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-g Section 5: Beam cro							
CALCULATION SHEET	Client		Made by	DCI	Da	te	July 2	2009	
	501		Checked by	JMS	Da	te	Sep 2	.009	
The strains and forces are illust	rated diag	grammatically below:							
	U U U	Strain 12.7 × 10 ⁻⁵							
	Û	4.0 × 10-5							





Hence the primary effects (stresses) are given by:

	W (steel	Restraint	Release o	f restraint	Total
	units)	nestiaint	Bending	Axial	Total
Top of slab	1.09E+08	-4.4	0.7	1.8	-1.9
0.6 into slab	2.16E+08	-1.4	0.3	1.8	0.7
bottom of slab	6.32E+08	-1.1	0.1	1.8	0.8
bottom of haunch	1.64E+10	-1.0	0.0	1.8	0.8
Top of top flange	1.64E+10	-5.9	0.0	10.7	4.8
400 below slab	-9.45E+07	0.0	-4.6	10.7	6.1
bottom flange	-2.99E+07	0.0	-14.6	10.7	-3.9

Diagrammatically:



The release of the restraint moments is applied along the span, in the uncracked regions, as a separate loadcase, to determine the secondary effects of vertical temperature difference.

Note that the omission of restraint moments in cracked regions is not mentioned in EN 1994-2 but the view has been taken that the omission permitted for shrinkage (see EN 1994-2, 5.4.2.2(8)) may be used for the calculation of secondary effects of temperature difference.

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SCI		Job Title (Composite	highway	v bridges	s: Wo	rked e	examp	les		
Silwood Park, Ascot, Be Telephone: (01344) 636 Fax: (01344) 636570	rks SL5 7QN 525	Subject I	Example 1: Section 5: 1	Multi-g Beam cr	girder tw oss secti	vo-spa ions	n brid	lge			
CALCULATION SHE	ET	Client			Made by	y I	DCI	Date	July	2009	
		SCI			Checked	dby J	MS	Date	Sep	2009	
Shrinkage For complete verification to traffic and at the optimary and secondaria values are greater that advantageous, they at The characteristic variant the modular ratio effects generally and will be used for both For a fully restrained	ation, shring end of the ry effects an those and the neglection of shring of shring of shring of shring for determined a section	nkage effect service life are calculate t opening) a ed. inkage strair 15.4. This i nining the set	s should b and the me ed only for nd where t n is given o s very clos econdary e	e calcula ore oner the lon he total on Sheet se to the ffects, t	ated at the ous value g-term s effects of 6 as ε_{cd} value for the long-time time the sector $1 + 1 + 1 + 1 + 1 + 1 + 1 + 1 + 1 + 1 $	the time is used ituation of shrift $= 33$ or lon term p	e of o ed. He on (the inkage 3.1 × 1 g-term proper	openin ere, e are 10^{-5} n ties	g		
For a fully restrained beam, due to the cha	a section, a	values of sl	force and hrinkage st	moment rain are	given by	span g y:	girder,	innei			
				Cen	tre of for	се					
		Strain	Force	Belov	w At	ove	mon	nent Im)			
Slab		-0.000331	-4164	125	3	327	-13	362			
Haunch		-0.000331	-146	275 17		77	-26				
Hence the primary e	ffects are: W (stee	el Bootro	Re	elease of	restraint		Tota				
Ton of slab	5 93F ± (17 4 5	Ber	nding	Axial		0.5				
bottom of slab	1.33E+0	08 4.5	_	0.7	-2.5		1.3	3			
bottom of haunch	1.76E+0	08 4.5	_	0.5	-2.5		1.5	5			
Top of top flange	1.76E+0	0.0	-	7.9	-38.4		-46.	.3			
bottom flange	-2.83E+	07 0.0	4	9.0	-38.4	·	10.0	6			
Diagrammatically:			1								
1600 1400 1200 1000 800 600 400 200 0 -60.0 -40.0	-20.0 (0.0 20.0									
The release of the rest separate loadcase, to c	traint mom letermine t	ents is applie he secondary	d along the effects of	span, ir shrinkag	the uncr e.	racked	l regio	ns, as	a 4-2/ 5.4.	2.2(8)

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SCI	Job Title	Composite highway	bridges: W	orked e	example	es					
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 6: Global an	lge								
CALCULATION SHEET	Client		Made by	DCI	Date	July	July 2009				
	SCI		Checked by	JMS	Date	Sep 2					
 6 Global analysis 6.1 3D FE model A 3D model of the structure wa the girder webs, and with beam beams. 	s created elements	, with FE elements for the girder flange	or the deck and for the deck	slab an he RC e	d for edge						
Concrete diaphragms were provi pressures were applied as equal	ided at b and oppo	oth abutments, with vosite hydrostatic press	vertical supportions supported by support of the second se	port on two en	ly (soil ds).						
For the cracked regions over the elements were given anisotropic stiffness transversely).	e interme properti	diate support (15% o es (cracked stiffness l	f each span longitudinal	i), the s lly, unc	lab racked	4-2/:	5.4.2	.3			
The use of FE elements means t for, since shear lag effects are ta	hat shear aken into	account in the analysis	be explicit sis.	tly allow	wed						
The results of the analysis, in te the software into equivalent force (each comprising a steel girder a without the application of extern composite beams include axial for 3D behaviour and the verification axial forces.	rms of s ses and n and a wid oal horize orces as on of the	tresses in all the elemnoments on longitudin dth of slab). In generation of slab, in generation of the effective of the effective of the effective of the effective of the end of the elements. The composite beams muticipate of the element of the e	ents, are contained composition of the composition of the second	onverted te bean ns that, idividua equence ount of	d by ns even l e of the these						

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SCI		Job Ti	tle Compos	site highway	bridges: W	orked e	exampl	es		
Silwood Park, As Telephone: (0134 Fax: (01344) 636	cot, Berks SL5 4) 636525 570	7QN Subjec	t Exampl Section	e 1: Multi-gi 6: Global ar	irder two-sp nalysis	oan bric	lge			
CALCULATIO	N SHEET	Client			Made by	DCI	Date	July	2009	
		501			Checked by	JMS	Date	Sep 2	2009	
6.2 Constru It is presumed	ction stage	es c will be co	ncreted in	two stages -	the whole of	of span	1,			
Separate analyt	ical models	are therefor	re provided	for:	creted after	span 2	2.			
Stage 1 All st Stage 2 Comp Stage 3 Comp Stage 4 Comp	eelwork, we posite structu posite structu posite structu	t concrete i are in span are in both s are (short te	n span 1 1 (long-terr spans (long rm propert	n properties) -term proper ies)	, wet concr ties)	ete in s	span 2			
(For simplicity, includes the lor model. The difj design of the m	the weight ng-term prop ference betw pain beams.)	of the edge perties of th een the two	beams is a e edge bear approache	pplied to the ms, rather th s is negligib	e stage 3 ma can introduc le, in relatio	odel, wi e anoth on to th	hich her he			
A further mode determine the r	el, a modific otational sti	ation of Sta ffness of the	ige 1, witho e beams at	but the wet s that stage.	lab, was an	alysed	to			
6.3 Analysis	s results									
All the following appropriate particular	ng results ar tial factors o	e for design	n values of ristic values	actions, i.e. s of actions.	after applic	ation o	f			
For construction construction sta traffic and pede at a girder splic support) and at support).	n loading, r ages. For tra estrian loadin ce (the same a 'mid-span'	esults are g affic loading ng for wors position as (taken to be	iven for the g the results t bending e g the first b e at the bra	e total effects are given for effects at three racing adjace cing position	s at each of for the comb be locations ent to the in a, 12.4 m fr	the thr ination - at the termed om the	ee of e pier, iate end			
Stage 1										
Self weight of	steelwork	spap 1								
Construction lo	ads on span	1								
Distance from		ULS			SLS					
pier (m)	<i>M</i> _y (kNm)	<i>F</i> _× (kN)	<i>F</i> _z (kN)	<i>M</i> _y (kNm)	<i>F</i> _x (kN)	<i>F</i> _z (k	N)			
6.3	-2573 1024	0 -2	689 415	-1958 752	-1	312	2			
15.6	3132	-3	43	2343	-2	33				
28	25	1	-521	19	0	-39	1			
Note: F _x is axial for	rce, <i>F</i> z is vertica	l shear								
Stage 2										
Self weight of	concrete on	span 2								
Removal of con	nstruction lo	ads on spar	n 1							
		×								

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sci	Job Title Composite highway bridges: Worked examp							les	
Silwood Park, As Telephone: (0134 Fax: (01344) 630	cot, Berks SL5 14) 636525 6570	7QN Subjee	ct Example Section	e 1: Multi-g 6: Global ai	irder two-sp nalysis	oan brid	lge		
CAI CUI ATIO	N SHFFT	Client			Made by	DCI	Date	July 2	2009
CALCOLATIO		SCI			Checked by	JMS	Date	Sep 2	2009
Distance from	$\Lambda A_{\rm c}$ (kNm)			M (kNm)			NIV		
	7//γ (KINIII) 2/100	7× (KIN)	<i>F</i> z (KIN)		<i>F</i> × (KIN)	<i>F</i> z (K)			
63	-2499	-22	-13	-1001	-10	- 10	,		
15.6	-2354	-5	96	-17 44 -1270	_3	71			
28	-5	3	182	-4	2	135	5		
Self weight of Self weight of Self weight of Self weight of Removal of co	concrete ed parapets carriageway footway con nstruction lo	ge beams v surfacing ostruction bads on span	n 2						
Distance from		ULS			SLS				
nier (m)	$M_{\rm V}$ (kNm)	<i>F</i> _× (kN)	Fz (kN)	<i>M</i> y (kNm)	<i>F</i> _× (kN)	<i>F</i> z (k	N)		
pier (iii)									
0.0	-1705	-230	308	-1444	-168	260)		
0.0 6.3	-1705 89	-230 -87	308 193	-1444 51	-168 -63	260 163	3		
0.0 6.3 15.6	-1705 89 1265	-230 -87 114	308 193 46	-1444 51 1033	-168 -63 84	260 163 38) 3		
0.0 6.3 15.6 28.0	-1705 89 1265 165	-230 -87 114 -17	308 193 46 –198	-1444 51 1033 117	-168 -63 84 -12	260 163 38 –16) 3 6		
0.0 6.3 15.6 28.0 Long term shu	-1705 89 1265 165 rinkage (res t both ULS	-230 -87 114 -17 straint mon and SLS sin	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$	-1444 51 1033 117 ed in uncrao	-168 -63 84 -12 cked region	260 163 38 –16 s)	6	Shee	t 2
0.0 6.3 15.6 28.0 Long term shu Values apply a Distance from	-1705 89 1265 165 rinkage (res t both ULS	-230 -87 114 -17 straint mon and SLS sin characteristic	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$	-1444 51 1033 117 ed in uncrae	-168 -63 84 -12 cked region	260 163 38 –16 s)	6	Shee	t 2
0.0 6.3 15.6 28.0 Long term shu Values apply a Distance from pier (m)	-1705 89 1265 165 rinkage (res t both ULS <i>M</i> _y (kNm)	-230 -87 114 -17 straint mon and SLS sin characteristic F_{x} (kN)	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. c F_z (kN)	-1444 51 1033 117 ed in uncrao	-168 -63 84 -12	260 163 38 –16 s)	6	Shee	t 2
0.0 6.3 15.6 28.0 Long term shi Values apply a Distance from pier (m) 0.0	-1705 89 1265 165 rinkage (res t both ULS <i>M</i> _y (kNm) -1552	-230 -87 114 -17 straint mon and SLS sin characteristic <i>F_x</i> (kN) -43	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. c $F_z (kN)$ 45 52	-1444 51 1033 117 ed in uncrae	-168 -63 84 -12	260 163 38 –16 s)	6	Shee	t 2
0.0 6.3 15.6 28.0 Long term shi Values apply a Distance from pier (m) 0.0 6.3	-1705 89 1265 165 rinkage (res t both ULS <i>M</i> _y (kNm) -1552 -1189	-230 -87 114 -17 Straint mon and SLS sin characteristic <i>F_x</i> (kN) -43 16 2	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. c $F_z (kN)$ 45 53 54	-1444 51 1033 117 ed in uncrao	-168 -63 84 -12	260 163 38 –16 s)	6	Shee	t 2
0.0 6.3 15.6 28.0 Long term sh Values apply a Distance from pier (m) 0.0 6.3 15.6 28.0	-1705 89 1265 165 rinkage (res t both ULS $M_{\rm y}$ (kNm) -1552 -1189 -695 -3	-230 -87 114 -17 straint mon and SLS sin characteristic F_x (kN) -43 16 2 33	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. c F_{z} (kN) 45 53 54 50	-1444 51 1033 117 ed in uncrae	-168 -63 84 -12	260 163 38 –16 s)	6	Shee	t 2
0.0 6.3 15.6 28.0 Long term shi Values apply a Distance from pier (m) 0.0 6.3 15.6 28.0 Stage 4 – tra Traffic loads f (The effects du gr1a, including	-1705 89 1265 165 rinkage (res t both ULS <i>M</i> _y (kNm) -1552 -1189 -695 -3 msient acti for worst have <i>e</i> to gr5 load <i>g</i> footway lo	-230 -87 114 -17 straint mon and SLS sin characteristic F_{\times} (kN) -43 16 2 33 fons ogging at in ds without j ading.)	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. c F_z (kN) 45 53 54 50 metermediate footway load	-1444 51 1033 117 ed in uncrad 0 .0	-168 -63 84 -12 cked region cked region	260 163 38 -16 s)	6 2 2 <i>to</i>	Shee	t 2
0.0 6.3 15.6 28.0 Long term shi Values apply a Distance from pier (m) 0.0 6.3 15.6 28.0 Stage 4 – tra Traffic loads i (The effects du gr1a, including Distance from	-1705 89 1265 165 rinkage (res t both ULS $M_{\rm y}$ (kNm) -1552 -1189 -695 -3 msient acti for worst h e to gr5 loa g footway lo	-230 -87 114 -17 straint mon and SLS sin characteristic F_x (kN) -43 16 2 33 fons ogging at in ds without j ading.) ULS	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. c F_z (kN) 45 53 54 50 metermediate footway load	-1444 51 1033 117 ed in uncrae 0	168 63 84 12 cked region cked region f cked region ster than th	260 163 38 -16 s)	6 2 to	Shee	t 2
0.0 6.3 15.6 28.0 Long term shu Values apply a Distance from pier (m) 0.0 6.3 15.6 28.0 Stage 4 – tra Traffic loads i (The effects du gr1a, including Distance from pier (m)	-1705 89 1265 165 rinkage (rest t both ULS M_y (kNm) -1552 -1189 -695 -3 msient acti for worst h <i>e</i> to gr5 load <i>g</i> footway lo	-230 -87 114 -17 straint mon and SLS sin characteristic F_{\times} (kN) -43 16 2 33 fons ogging at in ds without j ading.) ULS F_{\times} (kN)	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. $F_z (kN)$ 45 53 54 50 metermediate <i>footway load</i> $F_z (kN)$	1444 51 1033 117 ed in uncrad .0 .0 support (g ding are grea	168 63 84 12 cked region cked region ster than th SLS F_x (kN)	260 163 38 -16 s) s) <i>s</i>)	0 3 6 2 <i>to</i>	Shee	t 2
0.0 6.3 15.6 28.0 Long term shi Values apply a Distance from pier (m) 0.0 6.3 15.6 28.0 Stage 4 – tra Traffic loads (The effects du gr1a, including Distance from pier (m) 0	-1705 89 1265 165 rinkage (res t both ULS M_y (kNm) -1552 -1189 -695 -3 msient acti for worst h e to gr5 load g footway load M_y (kNm) -3621	-230 -87 114 -17 straint mon and SLS sin characteristic F_{\times} (kN) -43 16 2 33 fons ogging at in ds without j ading.) ULS F_{\times} (kN) 622	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. c F_z (kN) 45 53 54 50 metermediate footway load F_z (kN) 499	-1444 51 1033 117 ed in uncrad 0 .0 support (g ding are grea <u>M_y (kNm)</u> -2682	168 63 84 12 cked region cked region ster than th SLS <i>F</i> _× (kN) 461	260 163 38 -16 s) s) F_{z} (kl 370) 3 6	Shee	t 2
0.0 6.3 15.6 28.0 Long term shi Values apply a Distance from pier (m) 0.0 6.3 15.6 28.0 Stage 4 – tra Traffic loads f (<i>The effects du</i> gr1a, including Distance from pier (m) 0 6.3	-1705 89 1265 165 rinkage (res t both ULS <i>M</i> _y (kNm) -1552 -1189 -695 -3 msient acti for worst h <i>e to gr5 loa</i> <i>g footway lo</i> <i>M</i> _y (kNm) -3621 -1139	-230 -87 114 -17 estraint mon and SLS sin characteristic <i>F</i> _x (kN) -43 16 2 33 fons ogging at in <i>ds without j</i> <i>ading.</i>) ULS <i>F</i> _x (kN) 622 432	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. C F_z (kN) 45 53 54 50 mentermediate footway load F_z (kN) 499 348	-1444 51 1033 117 ed in uncrae 0 .0 support (g ling are grea <u>My</u> (kNm) -2682 -844	168 63 84 12 cked region cked region ster than th SLS <i>F</i> _x (kN) 461 320	260 163 38 -16 s) <i>s</i>) <i>s</i>) <i>s</i>) <i>s</i>) <i>s</i>)) 3 6	Shee	t 2
0.0 6.3 15.6 28.0 Long term shi Values apply a Distance from pier (m) 0.0 6.3 15.6 28.0 Stage 4 – tra Traffic loads is (The effects du gr1a, including Distance from pier (m) 0 6.3 15.6 28.0	-1705 89 1265 165 rinkage (rest t both ULS M_y (kNm) -1552 -1189 -695 -3 ensient acti for worst have to gr5 load g footway load M_y (kNm) -3621 -1139 781	-230 -87 114 -17 straint mon and SLS sin characteristic F_{\times} (kN) -43 16 2 33 fons ogging at in ds without j ading.) ULS F_{\times} (kN) 622 432 66	308 193 46 -198 ments applie nce $\gamma_{Sh} = 1$. c F_z (kN) 45 53 54 50 metermediate footway load F_z (kN) 499 348 83	-1444 51 1033 117 ed in uncrad 0 .0 support (g ding are grea $M_{\rm v}$ (kNm) -2682 -844 579	168 63 84 12 cked region cked region scked region SLS <i>F</i> _* (kN) 461 320 49	260 163 38 -16 s) s) F_{z} (kl 370 258 61) 3 6 2 <i>to</i> N) 2 3	Shee	t 2

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SCI	Job Title	ob Title Composite highway bridges: Worked examples							
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	^{ubject} Example 1: Multi-girder two-span bridge Section 6: Global analysis							
CALCULATION SHEET	Client		Made by	DCI	Da	te	July 2	2009	
	SCI	SCI Checked by JMS Date Sep 2009							
Traffic loads for worst hoggin (The effects due to gr5 loads with	ng at splice position (gr5 loads) ithout footway loading are greater than those due to								

gr1a, including footway loading.)

Distance from		ULS			SLS	
pier (m)	<i>M</i> y (kNm)	F _x (kN)	Fz (kN)	<i>М</i> у (kNm)	F _x (kN)	Fz (kN)
0	-2895	479	184	-2145	355	136
6.3	-1980	479	183	-1467	355	135
15.6	-528	207	117	-391	153	86
28	-82	-7	-78	-61	-5	-57

Traffic loads for worst sagging at splice position (gr5 loads)

(The effects due to gr5 loads without footway loading are greater than those due to gr1a, including footway loading.)

Distance from		ULS			SLS	
pier (m)	<i>M</i> y (kNm)	F _x (kN)	Fz (kN)	<i>M</i> y (kNm)	<i>F</i> _x (kN)	Fz (kN)
0	-2457	264	1268	-1820	196	939
6.3	2839	-160	303	2103	-118	225
15.6	2058	-296	-151	1524	-219	-112
28	-370	53	-198	-274	39	-147

Traffic loads for worst sagging at 'mid-span' (12.4 m from abutment) (gr5 loads) (*The effects due to gr5 loads without footway loading are greater than those due to*

gr1a, including footway loading.)

Distance from		ULS		SLS					
pier (m)	<i>M</i> y (kNm)	<i>F</i> _x (kN)	Fz (kN)	<i>M</i> y (kNm)	<i>F</i> _x (kN)	Fz (kN)			
0	-2425	381	661	-1796	282	490			
6.3	1040	12	525	771	9	389			
15.6	4952	-692	156	3668	-513	116			
28	-1180	145	-691	-874	107	-512			

gr5 traffic loads for maximum shear forces

		ULS		SLS					
	<i>М</i> у (kNm)	<i>F</i> _× (kN)	Fz (kN)	<i>М</i> у (kNm)	<i>F</i> _× (kN)	<i>F</i> _z (kN)			
Pier	-3206	387	1482	-2375	287	1098			
Splice +	2367	-11	669	1754	-8	496			
Splice-	733	-101	-24	543	-75	-18			
Span-	2805	-502	-396	2078	-372	-293			
Abut-	-1347	119	-1354	-998	88	-1003			

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SCI		Job Ti	Job Title Composite highway bridges: Worked examples									
Silwood Park, Asco Telephone: (01344 Fax: (01344) 6365	ON Subjec	Subject Example 1: Multi-girder two-span bridge Section 6: Global analysis										
CALCULATION	Client			Made by	DCI	Date	• July 2009					
		501			Checked by	JMS	Date	Sep 2	2009			
Effects of ther	mal actions	5										
	Vertical ter (restraint uncr	mperature moments a acked regi	difference applied in ons)	Soil pressures due to characteristic value of thermal expansion								
Distance from	cł	naracteristi	с									
pier (m)	My (kNm)	Fx (kN)	Fz (kN)	My (kNm)	Fx (kN)	Fz (k	N)					
0	423	3	-10	1058	1211	-50)					
6.3	340	-12	-14	-25	1187	-55						
15.6	215	27	-15	-537	1253	-58						
28	17	-9	-14	-1265	1204	-54	1					
					_	•						

Note that the effect of the soil pressure (due to restraint of thermal expansion) introduces hogging moments at the abutments and sagging moments at the intermediate support, as well as axial force. The total effect at the pier is therefore favourable, in terms of stresses in the bottom flange (and, in the rebars, the moment and axial force both reduce tension). Similarly, there is a hogging moment at the 'midspan' position and again the total effect is favourable, both in the bottom flange and the slab.

Range of effects due to passage of fatigue vehicle Worst bending effects

	Pi	er	Sp	lice	Span		
	<i>M</i> y (kNm)	<i>F</i> _× (kN)	My (kNm)	F _x (kN)	$M_{\rm y}$ (kNm)	<i>F</i> _x (kN)	
Lane 1 pos	0	0	407	-29	653	60	
Lane 1 neg	-428	76	-267	85	-135	15	
range	428	-76	674	-114	788	45	
Lane 2 pos	0	0	387	-17	626	49	
Lane 2 neg	-401	89	-253	98	-125	15	
	401	-89	640	-115	751	34	

Worst shear effects

	Pier	Splice	Span	Abut
	Fz (kN)	Fz (kN)	Fz (kN)	Fz (kN)
Lane 1 pos	265	93	51	14
Lane 1 neg	-6	-6	-25	-269
range	271	99	76	283
Lane 2 pos	235	88	45	13
Lane 2 neg	0	-2	-23	-247
	235	90	68	260

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				Job Title	Job Title Composite highway bridges: Worked examples									
Silwood Par	rk, Ascot,	Berks S	SL5 7QN	Subject	Exar	nple 1: 1	 Multi-	girder t	wo-s	pan brid	ge			
Telephone: Fax: (01344	(01344) 4) 63657(636525 C			Secti	on 7: D	esign	values of	of the	e effects	of con	nbined	acti	ons
CALCULA	ATION S	HEET		Client				Made	by	DCI	Date	July 2	2009	
				SCI				Check	ed by	JMS	Date	Sep 2	.009	
7 Dec			of the	offooto	of o	ombine	d oo	tiona						
Design va	lues of a	affects		en below	for c	ertain de	su au	situation	s fo	r the dec	ion of			
the inner l	beams.]	In prac	tice, fur	rther situa	ations	for othe	r par	ts of the	s, io	cture wo	ould			
also need	to be co	nsider	ed.				-							
7.1 Effe	7.1 Effects of construction loads (ULS)													
Generally.	the eff	ects of	constru	iction loa	ds api	olv to di	fferen	t cross	sectio	on prope	rties.	Elast	ic sec	ction
although f	or span	1, the	cross se	ections fo	r the	inner be	am ai	e the sa	ime a	it stages	2	modi	ıli are	?
and 3. The	e follow	ing tal	bulations	s summar	ize th	e forces	and 1	noment	s at e	each stag	e and	tabul Secti	ated i on 5	n 1
the stresse	es due to	b those	effects,	, for selec	cted c	ross sect	ions.					Seen	<i>m o</i>	L
Stresses	at pier													
				Bottom f	lange	Top fla	inge	Top re	bar	Axia	ıl			
	11	r	г	<i>W</i>	_	<i>W</i>	_	<i>W</i>	_	A	_			
Stage 1	νι _γ -2573	r× ∩	Fz 689	(10° mm°)	σ -67	(10° mm°)	σ 106	(10° mm²)	σ	(10° mm²)	σ	Using	g stee	l and
Stage 2	-2373	_22	_13	45 67	-07	66 63	38	41 84	60	94.3	0	crack	xed se	ction
Stage 2	-2499	-22	308	45.07	-33	66 63	26	41.04 //1.8/	00 ⊿1	94.3 94.3	2	prop	erties	
Shrinkage	-1552	-230	45	45.67	-34	66 63	20	41.84	37	94.3	0			
$(\gamma_{sh} = 1)$	-1332	-40	-5	+3.07	-0+	00.00	20	+1.0+	57	54.5	0			
-	-8329	-295	1029		-193	-	193		138		2			
-						_								
Stress at	splice	(pier	girder, o	cracked	secti	on)		_						
				Bottom f	ange	Top fla	nge	Top re	bar	Axia	al			
	Мv	Fx	F ₇	$(10^6 \mathrm{mm^3})$	σ	$(10^6 \mathrm{mm^3})$	σ	$(10^6 \mathrm{mm^3})$	σ	$A_{(10^3 \mathrm{mm}^2)}$	σ			
Stage 1	1024	-2	415	38.47	27	24.25	-42	(,		70.0	0	Using	z stee	l and
Stage 2	-2354	31	45	45.67	-52	66.63	35	41.84	56	94.3	0	nron	ea se erties	ction
Stage 3	89	-87	193	45.67	2	66.63	-1	41.84	-2	94.3	1	prop	. Tues	
Shrinkage $(\gamma_{sh} = 1)$	-1189	16	53	45.67	-26	66.63	18	41.84	28	94.3	0			
(join 1)	-2430	-42	706		-49		10		82		1			
Stress at	mid-s	pan (s	span dir	rder)										
				Bottom f	ange	Top fla	inge	Top of	slab	Axia	al			
				W		W		W (10 ⁶ mm ³		А				
	Mу	Fx	Fz	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ)	σ	(10 ³ mm ²)	σ	Using	g stee	l and
Stage 1	3132	-3	43	22.87	137	22.87	-137			50.2	0	sections-	ierm m	
Stage 2	-1714	-5	96	28.82	-59	180.4	10	943.9	1.8	107.5	0	prop	erties	
Stage 3	1265	114	46	28.82	44	180.4	-7	943.9	-1.3	107.5	-1	1 1		
Shrinkage $(\gamma_{sh} = 1)$	(no	t adver	se)											
-	2683	106	185		122	· · ·	-134	_	0.5	- -	-1			
-										-				

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SCI					Job Title Composite highway bridges: Worked examples										
Silwood Park, Ascot, Berks SL5 7QN			7QN	Subject Example 1: Multi-girder two-span bridge											
Fax: (0134	(01344) 4) 63657	6365. 0	25			Section	on 7: D	esign v	values o	of the	effects	s of co	mbine	1 acti	ons
CALCUL	ATION S	SHEE	ΞT		Client SCI				Made	by	DCI	Date	July	2009	
									Check	ed by	JMS	Date	Sep 2	2009	
Stress at	abutm	ent	(abu	utmer	nt girde	r)									
				I	Bottom fl	ange	Top fla	nge	Top of	slab	Axi	ial	Usin	o stee	el and
	Mу	Fx	F	z	<i>W</i> (10 ⁶ mm ³)	σ	<i>W</i> (10 ⁶ mm ³)	σ (<i>W</i> 10 ⁶ mm ³)	σ	A (10 ³ mm ²)) σ	long	s siee -term	i unu
Stage 1	25	1	-5	21	22.87	1	22.87	-1			50.2	0	secti	on anti aa	
Stage 2	-5	3	18	32	28.82	0	180.4	0	943.9	0.0	107.5	0	prop	ernes	
Stage 3	165	13	/ _ 2 5	98	28.82	<u> </u>	180.4	-1	943.9	-0.2	107.5				
-	105	-10	, -5	57				-2	-	-0.2	-				
7.2 Effe	ects of	traf	fic lo	ads p	olus cor	nstruc	tion lo	ads (U	JLS)						
Loading 1	for max	kimu	m ho	oggin	g at pie	r									
The worst	t effects	are	due to	5 gr5	traffic lo	oads. 1	Effects of	due to	temper	ature	differe	ence ar	e		
not advers	se.														
Effects a	t pier p	osit	ion												
					Bottom	flange	Top fla	ange	Top re	bar	Axial (steel)	Usin	g crae	cked
	N	1√	F×	Fz	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ	10 ⁶ mm ³)	σ	A (10 ³ mm ²	σ	prop	erties	for
Constructi	on -83	329	-295	1029		-193		193	,	138		2	varia	is oj ihle	
Gr 5 traffic	c <u>-36</u>	621	622	499	45.67	-79	66.63	54	41.84	87	94.3	-7	actic	ons	
	-11	950	327	1528	-	-272	<u>.</u>	247	-	225		-5			
The effects o	of soil pres	sure a	are an a	axial for	ce plus a s	sagging	moment a	and the t	otal effec	ts are	not advei	rse.			
Coexiste	nt effe	cts a	at spl	ice p	osition										
					Bottom	flange	Top fla	ange	Top re	bar	Axial(s	steel)	Usin	g crao erties	cked for
	Л	Л _v	Fx	Fz	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ(10 ⁶ mm ³)	σ	A (10 ³ mm ²) σ	effec	ts of	<i>J</i> 07
Constructi	on –24	430	-42	706	(,	-49	(,	10	,	82		1	varia	ıble	
Gr 5 traffic	c <u>-1</u>	139	432	348	45.67	-25	66.63	17	41.84	27	94.3	-5	actic	ns	
	-3	569	390	1054	-	-74	_	27	-	109	<u>.</u>	-4			
Looding	formo	dimu		agin	a handi	20									
The maxin		agina	iii sa	yyını nents	on the c	omno	site hear	n occi	ir at an	nrov	imately				
midsnan.	the resu	lts fo	or the	node	nositior	12.4	m from	the al	n at ap butmen	t oive	e the				
maximum	values	from	the g	global	analysis	5, for	construc	ction lo	oads an	d for	traffic	loads.			
			-		Bottom	flange	Top fl	ange	Top of	slab	Axial (steel)			
		_	_	_	W	0	Ŵ	U	W		A				
o , , , ,	A OO	<i>1</i> γ	F _x	F _z	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³	³) σ	(10 ³ mm ²) σ			
Constructi	on 26 ⊿0	52	-692	185	30 50	162	1827	-134 3	653 /	0.5	209.8	- I 2	Usin	g sho	rt
Traffic gr5	+3)	52	-032	150	30.50	102	1027	5	055.4	7.6	209.0	5	term	comp	osite
Temp	20	00	25	-14	30.50	7	1827	0	653.4	_	209.8	0	prop	erties	for
difference	*				_		-		-	0.3			varia	ible	
	78	35	-561	327		291		-131		_ 7.4		2	actic	ns	
* γ _Q = 1.55 aι	nd $\psi_0 = 0.0$	6 appl	ied to c	characte	- eristic valu	es	-		_						
The effects o	of soil pres	sure o	the stru	hermal	expansion	are no	t adverse a	at bottor	n flange l	evel a	nd have c	only a			
	,			Job No. BCR113					Sheet	25 of	64	Rev	Α		
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SCI				Job Title	Com	posite hi	ghw	ay bridg	es: V	Vorked e	exampl	es	•		
Silwood Park, A	scot, Ber	ks SL5	7QN	Subject	Exam	ple 1: N	Aulti	-girder t	wo-s	pan brid	lge				
Telephone: (013 Fax: (01344) 63	344) 6369 36570	525			Sectio	on 7: De	esign	values	of th	e effects	of cor	nbineo	l acti	ons	
CALCULATIO	ON SHE	ET	-	Client				Made	by	DCI	Date	July	2009		
				SCI				Check	ked by	JMS	Date	Sep 2	2009		
Loading for The maximum maximum sag	maxim n hoggir gging mo	um ho ng mor oment,	o ggin nent so th	g at spl at the sp nis will g	ice lice po overn	osition is the des	s mu ign c	ch great of the sp	er th lice.	an the					
Pier side of	splice														
				Bottom	flange	Top fla	nge	Top re	ebar	Axial(s	teel)				
	11	E	F	<i>W</i>	-	(10 ⁶ mm ³	_	<i>W</i>	-	A	-	Usin	g cra	cked	
Construction	2420	rx AD	706	(10° mm°)	0)	0 10	(10° mm°)	0 00	(10° mm²)	0	prop	erties	jor	
	-2430	-4Z	100	45.67	-49	66.00	20	41 04	02	04.2	ו ד	lvari	is Oj		
Gr 5 traffic	-1980	4/9	183	45.67	-43	66.63	30	41.84	47	94.3	-5 12	actic	ns		
nress)	-25	1107	55	45.67	-1	00.03	0	41.84	I	94.3	-13	uciic	113		
p ,	-4435	1624	944		-93		40	_	130	_	-17				
No results are tak	ulated for	the brac	cing po	sition, 0.4 r	n close	r to the ce	ntre si	upport, but	value	s may be					
interpolated linea	rly with su	fficient a	ccurac	y. In praction	ce, mod	lel nodes r	night	be position	ed at	bracing loc	ations.				
Span side of	f splice														
-	-			Bottom	flange	Top fla	nge	Top re	bar	Axial(s	teel)				
				W	σ	Ŵ	σ	Ŵ	σ	A	σ				
	Mу	Fx	Fz	(10 ⁶ mm ³)		(10 ⁶ mm ³)		(10 ⁶ mm ³)		(10 ³ mm ²)		Usin	g cra	cked	
Construction	-2430	-42	706		-82		4		91		1	prop	erties	for	
Gr 5 traffic	-1980	479	183	27.24	-73	71.64	28	38.06	52	74.5	-6	effec	ts of		
Temp (soil press)	-25	1187	55	27.24	0	71.64	0	38.06	0	74.5	-16	varia	ible a	ction	
p1000,	-4435	1624	944	_	-155	-	32	-	143	_	-21				
Loading for	maxim	um sh	ear												
Maximum sl	near at	pier p	ositi	on											
The value of	the max	imum	shear	is neede	ed to v	verify th	e sh	ear resis	tance	e of the v	web	Usin	g cra	cked	
and to determ	ine the	longitı	ıdinal	shear of	n the s	stud con	necto	ors.				prop	erties	for	
				Bottom	flange	Top fla	nge	Top re	bar	Axial (s	steel)	effec	ts of		
		_	_	W	Ū	Ŵ	0	Ŵ		A		vario	ible		
- ·	Mу	Fx	Fz	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ	(10 ³ mm ²)	σ	aciic	115		
Construction	-8329	-295	1029)	-93		203		138		2				
Gr 5 traffic	-3206	387	1482	45.67	-70	66.63	48	41.84	//	94.3	-4				
-	-11535	92	2511	_											
		onling													
	iear at	spiice	e pos	ition											
The value of	the max	1mum	shear	is neede	ed to c	letermin	e the	e maxim	um l	ongitudi	nai				
shear on the s	stud con	nector	s.												
	Mу	Fx	Fz												
Construction	-2430	-42	706												
Gr 5 traffic	2367	-11	669												
	-63	-53	1375	<u> </u>											
				_											

	7			Job No.	BCR113				Sheet	26 of	64	Rev	Α	
SCI				Job Title	Comp	posite hi	ghwa	y bridge	es: W	/orked e	xampl	es	•	
Silwood Park, A	scot, Berl	s SL5		Subject	Exam	ple 1: N	Aulti-	girder t	WO-S	pan brid	ge			
Telephone: (01) Fax: (01344) 6	344) 6365 36570	25			Sectio	on 7: De	esign	values of	of the	e effects	of cor	nbinec	1 acti	ons
CALCULATI	ON SHEI	ET		Client				Made	by	DCI	Date	July	2009	
				SCI				Check	ed by	JMS	Date	Sep 2	2009	
Maximum s The value of shear on the	hear at the maxistud com	midsp imum nectors	shear s.	osition is neede	ed to c	letermir	e the	maxim	um lo	ongitudi	nal			
Construction	<i>Ν</i> Ιγ 2683	<i>F</i> ×	185											
Gr 5 traffic	2003	-502	-30	, 6										
GI 5 trainc	5/88	206	-39	1										
	5466	-390	-21	<u> </u>										
Maximum s	hear at	abutn	nent											
	Mу	Fx	Fz											
Construction	185	-13	-53	7										
Gr 5 traffic	-1347	119	-135	4										
	-1162	106	-189	1										
							. ,							
7.3 Effects	s of trai		ads p	olus con	struc	tion loa	ads (SLS)						
The values of	f effects	at SLS	S are	needed t	o veri	fy crack	cont	rol in th	ne sla	b at the	pier			
and to verify	the sup	Tesista		i ule spi	ice.									
Effects at p	ier posit	tion												
				Bottom	flange	Top fla	nge	Top re	bar	Axial (s	steel)			
		~	~	W		W		W		A				
Store 1	<i>ΙVI</i> γ 1064	Fx AA	Fz 404	(10° mm°)	σ 51	(10° mm°)	σ 01	(10° mm°)	σ	(10° mm²)	σ			
Stage 7	1904	44 20	494 2	30.47 45.67	-51	24.25	28	11 81	11	0/ 3	0			
Stage 2	-1451	_178	290	45.67	-32	66 63	20	41.84	35	94.3	2			
Shrinkage	-1546	59	47	45.67	-34	66 63	23	41 84	37	132.2	0			
Gr 5 traffic	-2695	719	483	45.67	-59	66.63	40	41.84	64	94.3	-7			
	-9513	673	1312	_	-217		194	-	180	-	-5			
The effects of se	oil pressur	es due t	to ther	nal expan	sion are	e not adve	erse at	this posit	ion.	_				
		.,.			,									
Effects at s	plice po	sition	(WOI	rst snea	r)									
T let slue				Bottom	flange	Top fla	nae	Top re	bar	Axial(s	teel)			
				W	U	Ŵ	0	Ŵ		A				
	Mу	Fx	F _z	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ	(10^3mm^2)	σ			
Stage 1	740	-5	359	38.47	20	24.25	-31			70.0	0			
Stage 2	-1/45	30	39	45.67	-38	66.63	26	41.84	42	94.3	0			
Stage 3	39	-98 F4	189	45.67	1	00.03	- I 10	41.84	- 1 20	94.3				
Shrinkage	-1210	54 200	53 141	45.67	-26	00.03	18 22	41.84 41.04	28 2⊑	132.2	0			
	-14/4	390	701	49.07	-32	00.03	22	41.04	30	94.3 -	-4			
	-3050	3/1	101	-	-15	-	34		104	-	-3			

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SCI	Job Title Composite highway bridges:	Worked	exampl	es		
Silwood Park, Ascot, Berks SL5 7QN	Subject Example 1: Multi-girder two	-span brid	lge			
Telephone: (01344) 636525 Fax: (01344) 636570	Section 7: Design values of	the effects	of cor	nbined	actio	ons
CALCULATION SHEET	Client Made by	DCI	Date	July 2	2009	
	Checked	by JMS	Date	Sep 2	009	
Snan sida						
Span suc	Bottom flange Top flange Top reba	Axial(steel)			
<i>M</i> ., E., E.	W = W = W	A	σ			
Stage 1 740 -5 359	9 22.87 33 22.87 -33	50.2	0			
Stage 2 -1745 30 39) 27.24 -64 71.64 24 38.06 4	6 74.5	0			
Stage 3 39 -98 189	9 27.24 2 71.64 -1 38.06 -	1 74.5	1			
Shrinkage -1210 54 53	3 27.24 -44 71.64 17 38.06 3	1 74.5	0			
Gr 5 traffic -1474 390 14	1 27.24 -54 71.64 20 38.06 3	9 74.5	-5			
-3650 371 78	$\frac{1}{1}$ $\frac{-127}{-127}$ $\frac{27}{27}$ $\frac{1}{1}$	5	4			
7.4 Effects due to fatigue	vehicle					
The range of handing offects du	to the passage of the fatigue vehicle	in aaah la	no io			
determined at the three location	s already considered for static loading	in each ia	lie is			
determined at the time locations	s arready considered for static loading.					
At pier						
	Bottom flange Top flange Top reba	- Axial(steel)			
		A	01001,			
	W W W	(10 ³ mm	2			
$M_{\rm Y}$ $F_{\rm X}$ $F_{\rm Z}$	$(10^{6} \text{ mm}^{3}) \sigma$ $(10^{6} \text{ mm}^{3}) \sigma$ $(10^{6} \text{ mm}^{3}) \sigma$		σ			
Range, lane 1 428 –76	45.67 9.4 66.63 -6.4 41.84 -1	J.Z 94.3	0.8			
Range, lane 2 401 –89	45.67 8.8 66.63 -6.0 41.84 -8	.6 94.3	0.9			
Ratio lane 2/lane 1 moments = 0.937						
At splice (span side, uncrack	ked)					
	Bottom flange Top flange Top reba	Axial(s	steel)			
	W W W	A				
My F _x F _z	$(10^6 \mathrm{mm^3})$ σ $(10^6 \mathrm{mm^3})$ σ $(10^6 \mathrm{mm^3})$ σ	7 (10 ³ mm ²)	σ			
Range, lane 1 674 –114	30.50 22.1 -1,827 0.4 653.4 -1	.0 209.8	0.5			
Range, lane 2 640 -115	30.50 21.0 -1,827 0.4 653.4 -1	.0 209.8	0.5			
Ratio lane 2/lane 1 moments = 0.950						
Δt midsnan						
At muspun	Pottom flange Ton flange Ton robe	Avial	stool)			
			sleel)			
My Fx Fz	$(10^6 \mathrm{mm^3})$ σ $(10^6 \mathrm{mm^3})$ σ $(10^6 \mathrm{mm^3})$	ла (10 ³ mm ²)) σ			
Range, lane 1 788 45	30.50 25.8 -1,827 0.4 653.4 -1	.2 209.8	-0.2			
Range, lane 2 751 34	30.50 24.6 -1,827 0.4 653.4 -1	.1 209.8	-0.2			
Ratio lane 2/lane 1 moments = 0.953						

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SCI	Job Title Composite highwa	y bridges: Worke	d exampl	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Example 1: Multi- Section 8: Verifica	girder two-span b ation of bare steel	ridge girder d	uring		
CALCULATION SHEET	Client	Made by DC	Date	July 2	2009	
		Checked by JMS	b Date	Sep 2	:009	
8 Verification of bare s	teel girder during const	ruction				
The paired beams are susceptibl wet concrete (i.e. before it hard	e to lateral torsional bucklin ens and provides restraint to	g under the weigh the top flanges).	t of the			
The beams are partially restraine each pair at three points in the s restraint to the beams.	ed against buckling by the b span. This connection provid	racing frames between the set of	ween al			
Use the expressions in Appendix slenderness and thus the bucklin verification of buckling resistant	x C of P356 to determine the g resistance. Initially, use g ce, even where the section is	e non-dimensional ross section prope s Class 4.	rties for	P356	; ^[4]	
8.1 Torsional flexibility of p	paired beams					
In the global model, horizontal a levels at the three bracing locati	forces of 10 kN were applie ons, on each beam.	d at top and botto	n flange			
In the model, the forces at the to	op flange are applied at the	⊯ model nodes, whi	ch are			
above the level of the steel flang thus $1400 - 250/2 - 40/2 = 1$	ge, at the mid-thickness of the 255 mm	he slab. The lever	arm is			
The total torque applied is thus:						
$3 \times 2 \times 10 \times 1.255 = 37.65$ kN	Im					
The horizontal deflections at eac	ch beam given by the analys	is were:				
(bracings modelled at nodes rath At 6.3 m from pierAt 6.3 m from pier $+0.523$ At 15.6 m from pier $+0.858$ At 21.8 m from pier $+0.622$	her than at positions indicate 5 mm, -0.243 mm 8 mm, -0.420 mm 2 mm, -0.308 mm	d in Section 4)				
The bracings are not equally spa sensitive to the spacing and they application of the expressions in	aced but the torsional restrai 7 may be considered as equa 1 Appendix C of P356	nt that they provid lly spaced, for the	le is not			

	Job No.	BCR113		Sheet	29 of	64	Rev	Α
SCI	Job Title	Composite highway	bridges: W	orked e	exampl	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-g Section 8: Verificati	irder two-sj ion of bare	pan brid steel gi	lge rder du	iring		
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009	
	SCI		Checked by	JMS	Date	Sep 2	.009	
The rotations are thus:								
At 6.3 m from pier(0.525At 15.6 m from pier(0.858At 21.8 m from pier(0.622	6 + 0.242 8 + 0.420 2 + 0.300	$\begin{array}{l} 3) / 1255 = 6.12 \times 1 \\ 0) / 1255 = 10.18 \times 1 \\ 8) / 1255 = 7.41 \times 10 \end{array}$	0^{-4} rad 10^{-4} rad 0^{-4} rad					
Thus, use:								
$\theta_{\rm R} = 10.18 \times 10^{-4} / (37.65 \times 10^{-4})$	$0^{6}) = 2.7$	$705 \times 10^{-11} \text{ rad/Nmm}$						
8.2 Evaluation of non-dime	nsional	slenderness						
$L_{\rm ev} = 28000$ mm and for the	span gir	der cross section, the	section pro	operties	are:			
$I_{zc} = I_{zt} = 4.167 \times 10^8 \mathrm{mm}$	4		F	· · · · · · ·				
$I_{\rm T} = 2.167 \times 10^7 {\rm mm}^4$								
$i_z = 128.8 \text{ mm}$								
h = 1100 mm								
$t_{\rm f}$ = 40 mm								
$d_{\rm f}$ = 1060 mm								
$\lambda_{\rm F} = \frac{L_{\rm w}}{i_z} \cdot \frac{t_{\rm f}}{h} = \frac{28000}{128.8} \cdot \frac{4}{11}$	$\frac{40}{00} = 7.$	91				P356 and	5/C.4 C.3.2	.3 2
$a = \frac{I_{z,c}}{I_{z,c} + I_{z,t}} = 0.5$ (equa	l flanges))						
$\psi_a = 0.8(2a-1) = 0.8 \times (2 \times 10^{-1})$	0.5 - 1	1) = 0						
To determine V_{eq} , the following	are need	led:						
$\tau = 4a(1-a) + \psi_a^2 = 4$	$\times 0.5 \times 0$	(1-0.5) + 0 = 1.0						
$\omega = \frac{\pi^2 d_{\rm f}^2 E I_{\rm z}}{G I_{\rm T} L_{\rm w}^2} = \frac{\pi^2 \times 1}{2}$	$\frac{1060^2 \times 1000}{167 \times 100}$	$\frac{8.33 \times 10^8 \times 2.6}{0^7 \times 28000^2} =$	1.414 (usin	g <i>E/G</i> =	= 2.6)	P356	6/C.4	.5
Thus:								
$V_{\rm eq} = \left[\frac{2a\omega}{\left[\sqrt{4+\tau\omega}+\psi_a\sqrt{\omega}\right]^2}\right]$	0.25 =	$\left[\frac{2 \times 0.5 \times 1.414}{\left[\sqrt{4+1.414}+0\right]^2}\right]^{0.2}$	= 0.715					
Thus the restraint parameter $V_{\rm eq}$	${}^{4}L_{\rm w}{}^{3}/[EI_{\rm z}]$	$_{\rm c}\theta_{\rm R}d_{\rm f}^{2}(1-a)] = 4316$						

	Job No.	BCR113	3		Sheet	30 of	64	Rev A		
SCI	Job Title	Compos	ite highway	bridges: W	orked e	exampl	es			
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example Section	e 1: Multi-gi 8: Verificati	irder two-sj ion of bare	oan bric steel gi	lge rder dı	ıring			
CALCULATION SHEET	Client			Made by	DCI	Date	July	2009		
	501			Checked by	JMS	Date	Sep 2	2009		
And using the expression $k =$	- 1 + - -	$V_{\rm ec}^2$ $r^4 EI_{\rm z,c} d$	$\frac{1}{f^2} \frac{L_w^3}{\theta_{\rm R} (1-a)}$](-0,25)						
The value of $k = 0.385$										
The limiting (minimum) value o	f k is (1	$.7 - 0.7V_{e}$	$_{ m q})L_{ m r}/L_{ m w}$							
Taking $L_{\rm r} = 8200$ (the longest u	inbraced	length -	this is conse	rvative, the	limit is	5:				
$(1.7 - 0.7 \times 0.715) \times 8.2/28.0$	= 0.351	, so use k	x = 0.385				Das			
Assume $1/\sqrt{C_1} = 1.0$ (uniform	n momer	it - consei	vative assur	nption)			P350 P350	5/C.4.2 5/C.4.3		
U = 1.0 (welded section)	10	,	0.5							
$V = \left\{ \left 4a(1-a) + 0.05\lambda_{\rm F}^2 \right \right\}$	$+\psi_a^2 \right]^0$	$+\psi_a$								
$= \left\{ \left[4 \times 0.5(1 - 0.5) + 0.6 \right] \right\}$	05×7.9	$[1^2 + 0]^{0}$	$\left. {}^{5} + 0 \right\}^{-0.5} =$	0.702						
Take D 1.2 (destabilising lo	ads)						P356/C.1			
$\lambda_{\rm z} \qquad = \frac{kL_{\rm w}}{i_{\rm z}} \qquad = \frac{0.385 \times 2k}{128.8}$	8000 =	83.7								
$\lambda_1 \qquad = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{21000}{345}}$	$\frac{0}{2} = 77$.5								
$\beta_{\rm w} = \frac{W_{\rm y}}{W_{\rm pl,y}} = 2.287 \times 10^7$	7 / (8237	× 10 ⁶ /345	5) = 0.958				W _y a from	and $M_{\rm pl}$ a Sheet 14		
Thus:										
$\overline{\lambda}_{\rm LT} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_{\rm w}}$	= 1×1	× 0.702	$\times 1.2 \times \frac{83.7}{77.3}$	$\frac{7}{5}\sqrt{0.958}$	= 0.89		P350	5/C.1		
Slenderness determined from buckling analysis Alternatively, and less conservatively, slenderness could be derived from an elastic buckling analysis of the structure at the bare steel girder stage and then the value of $\overline{\lambda}_{LT}$ would be given by $\overline{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$ where M_{cr} is given by the analysis. A buckling analysis was not available for this example.										

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SCI	Job Title	Composite highway	bridges: W	orked o	example	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-g Section 8: Verificat	irder two-sj ion of bare	pan bric steel gi	lge rder du	ring		
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009	
	SCI		Checked by	JMS	Date	Sep 2	2009	
8.3 Reduction factor Since $h/b > 2$, use buckling curves $\phi_{LT} = 0.5 \Big[1 + \alpha_{LT} (\overline{\lambda}_{LT} - 0.2) + \overline{\lambda}_{LT} \Big]$ $= 0.5 \Big[1 + 0.76 (0.89 - 0.2) + \overline{\lambda}_{LT} \Big]$	rve d, $\alpha_{\rm f}$ $\bar{\ell}_{\rm LT}^2$ 0.89^2	$_{T} = 0.76$ = 1.16				3-2/ 3-1- 3-1- 1/NA	6.3.2 -1/6.3 - A.2.1	.2 5.2.2 6
$\chi_{\rm LT} = 1 / \left(\phi_{\rm LT} + \sqrt{\phi_{\rm LT}^2 - \overline{\lambda}_{\rm LT}^2} \right) =$								
8.4 Verification								
$M_{\rm b,Rd} = \frac{\chi W_{\rm el} f_{\rm y}}{\gamma_{\rm M1}} = \frac{0.525 \times 10^{-10}}{10^{-10}}$	$\frac{2.287 \times 1.1}{1.1}$	$\frac{10^7 \times 345}{10^{-6}} \times 10^{-6} =$	3766 kNm			3-1-	-1/6.3	9.2.1
$M_{\rm Ed}$ = 3132 kNm (Sheet 23)	$< M_{\rm b,Rd}$	= 3766 kNm - OK						
The above calculations assume t Class 4, as noted on Sheet 14. T section for this particular cross parameters:	hat the c The deter section (cross section is Class mination of the proper neglecting fillet welds	3. In fact i erties of the s) gives the	t is mar e effecti followi	rginally ive ng			
$k_{\alpha} = 23.9$ $\bar{\lambda}_{p} = 0.907$ $\rho = 0.069$						3-1-	-5/4.4	Ļ
$b_{\rm eff} = 494.2$, which means that centroid $b_{\rm eff}/2$ below the undersi	there is d de of the	a 'hole' in the web 1. e top flange.	5.8 mm ver	tically v	vith its			
The section moduli are then:								
$2.281 \times 10^7 \mathrm{mm^4}$ at the mid-thick	kness of	the top flange and						
$2.288 \times 10^7 \mathrm{mm^4}$ at the mid-thick	kness of	the bottom flange						
The modulus for the effective set $M_{b,Rd}$. Here the difference is neg	ction sho ligible.	uld be used in the ex	pressions fo	or β_w and	nd			

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SCI	Job Title	Composite high	way	bridges: W	orked e	example	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Mul Section 9: Verif	lti-gi ficati	rder two-sp on of comp	oan bric oosite g	lge irder			
CALCULATION SHEET	Client			Made by	DCI	Date	July	2009	
	SCI			Checked by	JMS	Date	Sep 2	2009	
9 Verification of composed 9.1 In hogging bending with The composite cross section is O The elastic design bending resists design effects at the stages. From Sections 7.1 and 7.2, the the total moment is 11950 kNm. (cracked) section is 9377 kNm.	Sep 2	2009							
steel con (be	nposite Inding) e do not	composite (axial) need to be includ	led.				4-2/	6.2.1	.5(5)
For verification of cross section stresses f_{vd} and f_{sd} .	resistance	ce, the stresses sh	nould	not exceed	d the lin	niting		0.2.1	(.)
For this verification:									
$f_{yd} = f_y / \gamma_{M0} = 335/1.0 = 335 \text{ N}$ $f_{sd} = f_{yk} / \gamma_s = 500/1.15 = 435 \text{ N}$	N/mm ² fo N/mm ² fo	or the 60 mm bott for the reinforceme	tom f ent	flange					
By inspection, the stresses in bo	th are O	K							
The member is subject to combi a linear interaction will be assur	ned benc ned (con	ling and axial for servative).	rce a	nd for men	iber res	sistance			
For verification of buckling resi section (on which $M_{b,Rd}$ is based	stance in) has to 1	bending, the des	sign i ing:	resistance of	of the c	ross			
$M_{\rm el,Rd} = M_{\rm a} + k M_{\rm c,Ed}$							4-2/	6.4.2	
Where k is a factor such that a s	stress lim	nit is reached due	to b	ending alor	ne.				
In this case the bottom flange w	ill reach	its limit first and	l the	limit is:					
$f_{\rm yd} = f_{\rm y}/\gamma_{\rm M1} = 335/1.1 = 305 \text{ N}$	J/mm ²								
Thus $M_{\rm el,Rd} = 2573 + \frac{(305 - 205)}{205}$	<u>67)</u> × 93	877 = 13460 kN	Im						

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SCI	Job Title C	omposite highway	bridges: W	orked e	example	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject E	xample 1: Multi-gi ection 9: Verificati	rder two-sj on of comp	pan brid posite gi	lge irder			
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009	
	5C1		Checked by	JMS	Date	Sep 2	:009	
To evaluate $M_{b,Rd}$, determine the	e slendernes	S						
The slenderness of the length of at 5.9 m into the span could be e the effective width of slab and th conservative to use the simplified	beam betwe evaluated co le steel girde d method of	en the intermediate nsidering the LTB er but it is much sin EN 1993–2, as all	e support ar of a sectior npler and a owed by El	nd the bin comprise a little le N 1994-	racing ising ess -2.	4-2/0	5.4.3	.2
Consider an effective Tee section depth of the part of the web in or under total effects, including ax	n comprisin compression ial force.	ng the bottom flang I. Take the depth in	e and one n compress	third of ion as t	the hat	3-2/0	5.3.4	.2
Flange area is $600 \times 60 \text{ mm}$								
Height to zero stress:		Top flapge	Stress	Mid-h	eight			
$(277/532) \times 1050 + 30 = 5901$ Height of web in compression	mm = 530 mm	Bottom flange	255	30	50 D			
Area of Tag = $600 \times 60 \pm (52)$	-350 mm $0 \times 14)/3 =$	- 38470 mm ²			-			
Lateral 2^{nd} moment of area 600^3	$(\times 60 / 12) =$	$= 1080 \times 10^6 \mathrm{mm}^4$						
Radius of gyration = $\sqrt{1080 \times 1000}$	$\frac{10^{6}}{38470}$	$= 168 \mathrm{mm}$						
For a buckling length of 5900 n	m (support	to first bracing).						
$N_{\rm E} = \pi^2 \frac{EI}{L^2} = \pi^2 \frac{210000 \times 10000}{590} \times 100000 \times 100000 \times 100000000$	$\frac{1080 \times 10^6}{00^2}$	$\times 10^{-3} = 64300$	kN			3-2/	(6.12))
Initially, take <i>m</i> conservatively a	as 1.0							
Then $N_{\rm crit} = N_{\rm E} = 64300 \text{ kN}$								
$\overline{\lambda}_{\rm LT} = \sqrt{\frac{A_{\rm eff} f_{\rm y}}{N_{\rm crit}}} = \sqrt{\frac{38470 \times 3}{64300 \times 3}}$	$\frac{\overline{335}}{10^3} = 0.44$	18				3-2/	(6.10))
Since $h/b > 2$, use buckling cur	ve d ($\alpha = 0$	0.76)				3-2/	6.3.2	.2
$\phi_{\rm LT} = 0.5 \left[1 + \alpha_{\rm LT} \left(\overline{\lambda}_{\rm LT} - 0.2 \right) \right]$	$+ \overline{\lambda}_{LT}^2 \Big]$					3-1- 3-1- 1/N/	1/6.3	6.2.2
= 0.5 [1 + 0.76(0.448 - 0.5)]	2) + 0.448	2 = 0.695				1/11/2	1.2.1	0
Hence		-						
$\chi_{\rm LT} = 1 / \left(\phi_{\rm LT} + \sqrt{\phi_{\rm LT}^2 - \overline{\lambda}_{\rm LT}^2} \right)$) = 1/(0.6)	$95 + \sqrt{0.695^2 - 0}$	$(.448^2) =$	0.815				
$M_{\rm b,Rd} = \chi M_{\rm el,Rd} = 0.815 \times 13$	3460 = 109	070 kNm				3-1-	1/6.3	.2.1

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SCI	Job Title	Composite highway	bridges: W	orked e	example	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-g Section 9: Verificati	irder two-s on of com	pan brid posite gi	lge irder			
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009	
	SCI		Checked by	JMS	Date	Sep 2	2009	
For verifying the contribution of the same Tee section (and thus the No effective section for axial force effective Tee that would buckle la slenderness with this amount of w $N_{b,Rd} = \chi A_{Tee} f_{yd} = 0.815 \times 3$ $N_{Ed} = A_{Tee} \times stress = 3841$	f axial re- the same terally shoeb is ver $38470 \times 0 \times 5 = 3$	esistance in the interact slenderness and redu a in EN 1993–2 but it of could comprise half the y little different from the 305 = 9560 kN	ction criteri ction factor could be arg e area of the hat derived	on, con c). gued tha e web: th above b	sider at the he ending.			
Considering first the interaction linear interaction, since the buck for variation over the buckling l	for valu cling mo ength:	es of $M_{\rm Ed}$ and $N_{\rm Ed}$ at the same for both	the support th) and with	(using h no allo	a owance			
$\frac{M_{\rm Ed}}{M_{\rm b,Rd}} + \frac{N_{\rm Ed}}{N_{\rm b,Rd}} = \frac{11950}{10970} + \frac{1}{950}$	$\frac{92}{560} = 1$.11						
This is inadequate, so consider t	the varia	tion of moment and le	ocation for	$M_{\rm Ed}$.				
Allowance for varying moment	over buc	kling length					() (2
Evaluate the m parameter in 6.3	.4.2(6)					3-2/	6.3.4	.2
Coexisting total moment at the s	splice =	3569 kNm and shear	= 1054 k	N				
Assume values at the brace posi	tion of <i>M</i>	M = 4000 kNm and W	V = 1080 k	N				
(If the model had been given not section, actual values could hav for the small distance involved i	des at th e been u n this ex	e bracing position as sed but the result woi ample.)	well as at ıld be negli	the char gibly di	ige of fferent			
Using the Note to 6.3.4.2(7) and restraint provided by the web (v	d ignorir vhich wi	ng any contribution from the second sec	om the con take $\gamma = 0$	tinuous))				
$M_2/M_1 = 4000/11950 = 0.335$ $\mu = V_2/V_1 = 1080/1528 -$	0 707							
$\Phi = 2(1 - M_2/M_1)/(1 + \mu)$	$= 2(1 - 0)^{-1}$	(0.335)(1+0.707) = 0.7	78					
$m = 1 + 0.44(1 + \mu)\phi^{1.5} = 1$	+ 0.44 >	$((1+0.707) \times 0.78^{1.5})$	=1.52					
Hence								
$N_{\rm crit} = 1.52 \times N_{\rm E} = 1.52 \times 643$	00 = 97	740 kNm						
And								
$\overline{\lambda}_{\rm LT} = \sqrt{\frac{A_{\rm eff} f_{\rm y}}{N_{\rm crit}}} = \sqrt{\frac{38470 \times 10^{-10}}{97740}}$	$\frac{10^6 \times 33}{0 \times 10^3}$	$\frac{15}{10} = 0.363$						
$\phi_{\rm LT} = 0.5 [1 + 0.76 (0.363 - 0.5)]$	2)+0.3	$[63^2] = 0.628$				3-1-	1/6.3	3.2.2
$\chi_{\rm LT} = 1 / \left(0.628 + \sqrt{0.628^2} - 1 \right)$	0.363^2	= 0.877						

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SCI	Job Title	Composite highway	bridges: W	orked	example	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-g Section 9: Verificat	irder two-sj ion of comp	pan brio posite g	dge irder			
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009	
	501		Checked by	JMS	Date	Sep 2	2009	
$M_{\rm b,Rd} = \chi M_{\rm el,Rd} = 0.877 \times 13$	3460 =	11800 kNm				3-1-	1/6.3	.2.1
Consider the moment at a distant	ice 0.251	L_k from the support, v	where $L_{\rm K} =$	L/\sqrt{n}	n	3-2/	5.3.4.	.2(7)
Distance = $0.25 \times 5900/\sqrt{1.52}$	= 1196	mm from the support	t					
$M_{Ed} = 11950 - (11950 - 400)$ interpolation)	0)×119	06/5900 = 10340 kN	m (conserva	ative				
The axial force and $N_{b,Rd}$ could a that adjustment is not made here	also be r e (the dif	reduced and the resistant ference in the result is	ance N _{b,Rd} e is negligible	enhance e).	d but			
The utilisation is now:								
$\frac{M_{\rm Ed}}{M_{\rm b,Rd}} + \frac{N_{\rm Ed}}{N_{\rm b,Rd}} = \frac{10340}{11800} + \frac{1}{95}$	$\frac{92}{560} = 0$.90 Acceptable						
The results of the cross section that some economy could be ach	indicate	2						
Interaction with shear must also	be cons	idered.						
9.2 Maximum shear at sup	port							
The maximum shear in the girde	er at the	intermediate support	= 2511 kN	1		Shee	et 25	
Assume that first transverse web (i.e. divide the length to the first	o stiffene t bracing	er is provided at 1967 g into three panels).	mm from	the sup	port			
For the web panel adjacent to the	e suppo	rt						
$a_{w} = 1967 \text{ mm}$ $h_{w} = 1000 \text{ mm}$ $t = 14 \text{ mm}$ $f = 355 \text{ N/mm}^{2}$								
The factor $n = 1.0$ according to	the NA					3-1-	5/NA	.2.4
From Equation (5.6):		_				3-1-	5/5.3	
$\overline{\lambda}_{\rm w} = \frac{h_{\rm w}}{37.4t\varepsilon\sqrt{k_{\rm t}}}$ where $\varepsilon = 1$	$\sqrt{235/f_{\rm s}}$	$\sqrt{235/355} = 0.8$	81					
Since $a_{\rm w} > h_{\rm w}$ and there are no	longituc	linal stiffeners:				3-1-	5/A.3	1
$k_{\rm t} = 5.34 + 4.0(h_{\rm w}/a)^2 = 5.32$	34 + 4.0	$0(1000/1967)^2 = 6.3$	57					
$\overline{\lambda}_{\rm w} = \frac{1000}{37.4 \times 14 \times 0.81 \sqrt{6.37}}$	= 0.934	Ļ						
Since the girder is continuous, co	onsider a	s a 'rigid endpost' cas	se; thus, fro	om Tabl	e 5.1:	3-1-	5/	
$\chi_w = 0.83/\overline{\lambda}_w = 0.83/0.934 =$	= 0.889	-				Tabl	e 5.1	

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SCI	Job Title	Composite highway	bridges: W	orked e	example	es				
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-g Section 9: Verificat	irder two-sj ion of comp	oan brid oosite gi	lge irder					
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009			
	SCI		Checked by	JMS	Date	Sep 2	2009			
$V_{\rm bw,Rd} = \frac{\chi_{\rm w} f_{\rm yw} h_{\rm w} t}{\sqrt{3} \gamma_{\rm M1}} = \frac{0.889 \times 10^{-10}}{10^{-10}}$	$\times 355 \times 1$ $\sqrt{3} \times 1$.	$\frac{1000 \times 14}{1} \times 10^{-3} = 2$	2319 kN			3-1-:	5/5.2			
This resistance is adequate for the contribution from the flanges for	This resistance is adequate for the maximum moment situation but requires a contribution from the flanges for the maximum shear situation.									
Maximum contribution from flam	nges is g	iven by:								
$V_{\rm bf,Rd} = \frac{b_{\rm f} t_{\rm f}^2 f_{\rm yf}}{c \gamma_{\rm M1}} \left(1 - \left(\frac{M_{\rm Ed}}{M_{\rm f,Rd}} \right) \right)$	$\left \begin{array}{c} 2 \\ \end{array} \right $					3-1-:	5/5.4			
$c = a \left(0.25 + \frac{1.6b_{\rm f} t_{\rm f}^2 f_{\rm yf}}{t h_{\rm w}^2 f_{\rm yw}} \right) = 1$	967 0.2	$25 + \frac{1.6 \times 600 \times 60}{14 \times 1000^2}$	$\left(\frac{2\times 335}{355}\right) =$	950 m	m					
$M_{\rm f,Rd}$ is the resistance of the flam	iges alon	e (no web).								
The axial resistance of the top b	ars and	top flange is:								
$(2 \times 12, 108) \times (500/1.15) + 20$										
And of the bottom flange is $36000 \times (335/1.0) = 12060$ kN										
Take the lever arm between top	and bott	om as 1159 mm and	thus:							
$M_{\rm f,Rd} = 12060 \times 1159 \times 10^{-3} =$	13980 k	iNm								
For the design situation for max a small tensile force; no reduction	imum sh on is nee	hear, the net axial for ded to $M_{\rm f,Rd}$ for this	ce on the cr axial force.	oss sec	tion is					
For the maximum shear situation	n, $M_{\rm Ed}$ =	= 11535 kNm				Shee	t 25			
$M_{\rm Ed} / M_{\rm f,Rd} = 11535/13980 = 0$	0.83									
$V_{\rm bf,Rd} = \frac{600 \times 60^2 \times 335}{950 \times 1.1} \left(1 - \frac{1}{1000}\right)$	0.83^2	= 692 × (1 – 0.69) =	= 215 kN							
The total shear resistance is thus	8:									
$V_{\rm b,Rd} = 2319 + 215 = 2534 \rm kM$	$\eta_3 = 1$	2487/2534 = 0.99) S	Satisfactory							
9.3 Combined bending she When $M_{\rm Ed} > M_{\rm f,Rd}$ and when $V_{\rm H}$ axial force must be reduced for	ar and a $E_{Ed} > 0.5$ the coex	Exial force $V_{\rm bw,Rd}$ the design resi isting shear force.	stance to be	ending a	ind	3-1-	-5/7.1	l		
Maximum shear with coexist As noted above, $M_{\rm Ed} / M_{\rm f,Rd} = 0$ be reduced for shear (instead, the coexisting moment).	ing mo 0.83, the he shear	ment refore the bending re resistance has already	esistance doo been reduc	es not n ced for	eed to					
Maximum moment with coex $V_{\rm Ed} = 1528 \text{ kN}$ $M_{\rm Ed} = 11950 \text{ k}$	kisting s xNm F _{x,E}	shear $_{\rm bd} = 327 \text{ kN} \text{ (axial compared})$	ompression)			Shee	t 24			

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SCI	Job Title	Job Title Composite highway bridges: Worked examples									
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Subject Example 1: Multi-girder two-span bridge Section 9: Verification of composite girder									
CALCULATION SHEET	Client		Made by	DCI	Dat	te	July 2	2009			
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The value of $M_{f,Rd}$ is reduced for axial force in accordance with 3-1-5/5.4(2) by applying the factor:

$$\left(1 - \frac{N_{Ed}}{\left(N_{bf,Rd} + N_{tf,Rd}\right)}\right) = \left(1 - \frac{327}{12060 + 17430}\right) = 0.99$$

And hence $M_{f,Rd} = 13700 \text{ kNm}$

Hence, since $M_{\rm Ed} < M_{\rm f,Rd}$ (11950 < 13700), bending resistance does not need to be reduced for shear.

Note: PD 6696–2 and Hendy and Johnson^[5] suggest that, for use in 3-1-5/7.1, M_{Ed} should be determined as the product of the accumulated stress and the section modulus for the relevant fibre of the cross section. However, a proposed revision of EN 1994–2 would modify the wording of 4-2/6.2.2.4 to confirm that it is the total moment that should be used. For this example, in both cases the value is less than $M_{f,Rd}$.

Although interaction does not need to be evaluated, the limiting combinations of M and V given by 3-1-5/5.4 and 3-1-5/7.1 are plotted below, for information. The values of maximum moment with coexisting shear and maximum shear with coexisting moment are shown on the plot. (For the different design situations $M_{pl,Rd}$ and $M_{f,Rd}$ are slightly different but the differences are very small.)



9.4 In sagging bending

The composite cross section is Class 1 (pna in the top flange) so the plastic resistance can be utilised.

The plastic bending resistance of the short term composite section is 13070 kNm and the total design value of bending effects is 7835 kNm, with a very small axial tensile force, so the section is satisfactory by inspection.

It can also be seen that the stresses calculated elastically, taking account of construction in stages are also satisfactory, as follows:

From Sections 7.1 and 7.2, the design value of stresses are as shown below.



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Silwood Park, Ascot, Telephone: (01344) (Fax: (01344) 63657(Berks SL5 7QN 636525 0	Subject	Example 1 Section 10	: Multi-gi : Longituc	rder two-sp linal shear	oan bric	lge			
CALCULATION S	HEET	Client			Made by	DCI	Date	July	2009	
		SCI			Checked by	JMS	Date	Sep 2	2009	
10 Longitudi The resistance to connectors and the	n al shear longitudinal s e transverse i	shear is ve reinforcem	rified for th ent at the p	ne web/flan ier, at the	nge weld, t splice and	he shea at mid-	r -span			
10.1 Shear forc	es									
ULS values Shear on steel sect Shear on long-term Shear on short-term effects)	ion (stage 1) composite se n composite s	ection ection (wor	Pier 689 340 st 148	Splic 415 291 2669	e Span 43 142 –396	Abuti –5: –1 –13	ment 21 6 354	Shee	xt 25	
SLS values										
PierSpliceSpanAbutmentShear on steel section (stage 1)51931233-391Shear on long-term composite section295196109-31Shear on short-term composite section (worst1098496-293-1003effects)										
10.2 Section pr	operties									
To determine shea For composite sec	ar flows the petions, uncraces	parameter cked unrein	\overline{Az}/I_y is n	eeded for	each sectio	n and s	tage. 1 be	4-2/	2 1(2))
	Pior ai	rdor	Snan	airdar	Abute	oont air	lor	0.0.	2.1(2)	,
	Web/top fl	Top fl/slab	Web/top fl	Top fl/slat	Web/top	fl Top	fl/slab			
	Az/I_{V} (m ⁻¹) A	$\frac{1}{4z/I_v}$ (m ⁻¹)	Az/I_{v} (m ⁻¹)	$A\overline{z}/I_{v}$ (m ⁻	¹) Az/I_{v} (m ⁻	$^{-1}) A_z/I_v$	(m ⁻¹)			
Steel section	0.825	1 3	0.875	/ 3	0.861	/ y				
Long term section	0.836	0.706	0.843	0.732	0.843	0.	732			
Short term section	0.831	0.805	0.826	0.836	0.836	0.8	844			
At the pier, the values at the bottom flange/web junction are 0.936, 0.739 and 0.704 for steel, long-term and short-term cross sections respectively. In addition to the shear flows determined from the vertical shear, the inequality of forces and moments on the four girders for any particular design situation leads to different axial forces on the composite beam sections that are verified to the design rules. These axial forces vary longitudinally and the variation is associated with a shear flow transferred between one composite section and the adjacent section. In this example, detailed interrogation of the analysis results identifies, for example, a negative shear flow of 34 kN/m at the pier due to permanent actions and a positive shear flow of 35 kN/m for the traffic loading for the worst shear case (both at the edge of the section, not at the steel/concrete interface). There is therefore no significant overall contribution. Values for other situations give only very small shear flows.							2			

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Example Section 10	1: Multi-gi): Longitud	rder two-sp dinal shear	pan brid	lge				
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009		
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10.3 Shear flow at ULS									
Force at web/top flange junc	tion								
At pier 689 × 0	.825 +340 × 0.836	+ 1482 × (0.831 = 20	84 kN/n	n				
At splice (span girder) 415 ×	0.875 +291 × 0.843	3 + 669 ×	0.826 = 11	61 kN/n	n				
At mid-span 43 ×	0.875 +142 × 0.84	3 – 396 ×	0.826 = -1	69 kN/n	n				
At abutment -521	× 0.861 –16 × 0.843	– 1354 ×	0.836 = 15	94 kN/n	n				
Force at flange/slab junction									
At nier	340 × 0 706	+ 1482 × (0.805 = 14	.33 kN/n	n				
At splice (span girder)	291 × 0 732	2 + 669 × 1	0.836 = 77	2 kN/m					
At mid-span	142 × 0.73		0.843 = -2	30 kN/n	n				
At abutment	-16 × 0.732	$-16 \times 0.732 - 1354 \times 0.844 = -1155 $ kN/m							
At the web/bettern flange in	lange junction								
At the web/bottom hange ju		1400	0 704 10	20 1/11/2	-				
At pier $689 \times 0.$	936 + 340 × 0.739	+ 1462 × 1	0.704 = 19	39 KIN/II					
10.4 Shear flow at SLS									
Force at flange/slab junction									
At pier	295 × 0.706	+ 1098 ×	0.805 = 10)92 kN/r	n				
At splice (span girder)	196 × 0.73	2 + 496 ×	0.836 = 55	8 kN/m					
At mid-span	109 × 0.73	34 –293 ×	0.843 = -1	67 kN/r	n				
At abutment	-31 × 0.73	2 –1003 ×	0.844 = -8	870 kN/r	n				
The shear flow at SLS is require	ed for verification o	f the shear	connectors	8					
10.5 Web/flange welds									
Design weld resistance given by	the simplified meth	nod of EN	1993-1-8,	4.5.3.3	is:	3-1-8	8/4.5	.3.3	
$F_{\rm w,Rd} = f_{\rm vw.d}a$ where $f_{\rm vw.d} =$	$\frac{f_{\rm u}}{\sqrt{3}}$								
For 6 mm throat fillet weld (8.4	mm leg length) a	= 6 mm							
For web and flange grade S355	in thickness range	3 - 100 mn	n, $f_{\rm u} = 4$	70 N/m	nm ²				
From Table 3–1–8/4.1 $\beta = 0.9$, j u						
Thus $F_{w,Rd} = \frac{6 \times 470/\sqrt{3}}{0.9 \times 1.25} = 1447 \text{ N/mm (kN/m)}$									
Resistance of 2 welds = 2890 k flange - OK	N/m > 2084 kN/m	shear flow	w in pier gi	irder at	top				
By inspection, 5 mm throat weld regions.	ls would be satisfac	would be satisfactory at the splices and in the span							

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CALCULATION SHEET	Client		Made by	DCI	Date	July	2009			
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Shear flows at bottom flange are interaction with vertical effects a Section 13.3)	e slightly at the bea	less and are OK by i aring stiffener need to	be checke	out the ed (see		EN	10025	5-2		
10.6 Shear connectors										
Stud shear connectors 19 mm diameter 150 mm long (type SD1 to EN ISO 13918) are assumed, with $f_u = 450 \text{ N/mm}^2$										
The resistance of a single stud is given by $4-2/6.6.3.1$ as the lesser of:										
$P_{\rm Rd} = \frac{0.8 \times f_{\rm u} \times \pi \times d^2 / 4}{\gamma_{\rm V}}$						6.6.1 Eq (3.1(1) 6.18))		
$P_{\rm Rd} = \frac{0.29 \times \alpha \times d^2 \sqrt{f_{\rm ck} \times E_{\rm cl}}}{\gamma}$	em					Eq (6.19)			
$\alpha = 1.0 \text{ as } \frac{h_{sc}}{d} = \frac{150}{10} > 4$										
$P_{\rm Rd} = \frac{0.8 \times 450 \times \pi \times (19^2 / 4)}{1.25} \times 10^{-3} = 81.7 \text{ kN}$										
$P_{\rm Rd} = \frac{0.29 \times 1.0 \times 19^2 \times \sqrt{40}}{1.25}$	× 35 × 1	$\frac{\overline{0^3}}{10^{-3}} \times 10^{-3} = 99.1 \text{ kN}$				Eq (6.19)				
Therefore the design resistance	of a sing	le headed shear conne	ector is							
$P_{\rm Rd}$ = 81.7 kN										
If studs are grouped and spaced reinforcement), then a row of 3	at 150 n studs ha	nm spacing along the s a design resistance of	beam (to s of:	uit trans	sverse					
$F_{\rm Rd} = 81.7 \times 3 / 0.150 = 1630$	kN/m									
This is adequate at the pier (F_R)	_d =1630	$> F_{\rm Ed} = 1433 \ {\rm kN/m}$)							
Rows of 2 studs would be adequed The change from 3 studs per row the splice (where the shear is a 2 6.6.5.5 to consider groups of co 2 studs would not quite be adequed	ate at th w to 2 st little high onnectors uate at th	e splice position (F_{Rd} uds per row can be m her), taking advantage but that option is not he abutment.	= 1090 > nade on the e of the per t explored b	$F_{Ed} =$ pier sie mission here. Re	772). de of in ows of					
The shear flow calculated above is based on elastic section properties and in this example the elastic bending resistance in the span is adequate, even though the composite section is class 2. If plastic bending resistance were utilised, the shear flow would need to be determined between the position where the elastic resistance is just mobilised and the position where the plastic resistance is developed (based on the difference in slab force over that length).								.2		
Resistance at SLS At SLS the shear connector resid		4–2/ 4–2/	6.8.1 NA.2	.11						
The resistance of 3 studs at 150 resistance with 2 studs per row	mm spa is 815 kl	cing is thus 0.75×16 N.	530 = 1220) kN an	d the					

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CALCULATION SHEET	Client		Made by	DCI	Date	July	2009	
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The SLS shear flows in Section and since the ULS peak shear fl SLS requirement is satisfactory	10.4 are ows are by inspe	all between 72% and all less than the ULS ction.	d 77% of th design resi	ne ULS istances	values , the			
10.7 Transverse reinforceme	ent							
Consider the transverse reinforc the studs at the pier, i.e. 1630 k	ement re N/m.	quired to transfer the	full shear	resistan	ce of			
For a critical shear plane around the studs (type b-b in 4-2/Figure 6.15 and shown dotted above) the shear resistance is provided by twice the area of the bottom bars.								.1
The shear force to be resisted is	given b	y 4-2/(6.21) as 1630/	$\cot \theta$,					
Take $\cot \theta = 1$, hence required	resistanc	e = 1630 kN/m				2-1-1	1/6.2	.4
Assume B20 bars at 150 mm sp	acing:							
Resistance = $A_{\rm sf} f y_{\rm d} / s_{\rm f} = (2 \times 3)$	14) × (50	$00/1.15)/150 \times 10^{-3}$	= 1821 kN	ſ/m		2-1-1	1/6.2	.4
The transverse bars are adequate transverse sagging moment, the combined effects.)	e. (If the resistance	y were also required be would need to be a	to provide dequate for	resistan coexist	ce to ting			
The underside of the heads of the bars. In this case an overall stud haunches are only 50 mm deep.	ne studs i I height o	need to be at least 40 of 175 mm should be	mm above sufficient,	the tran if the	nsverse	4-2/0	5.6.5	.4

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CALCULATION SHEET	Client		Made by	DCI	Date	July	2009			
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11 Fatigue assessment	ral steel	details								
The design value of the stress ra	ange in s	tructural steel is given	n as			3-2/9.4.1,				
$\gamma_{\rm Ff} \Delta \sigma_{\rm E2} = \gamma_{\rm Ff} \lambda \phi_2 \Delta \sigma_{\rm p}$ where $\phi_2 = 1.0$ and $\gamma_{\rm Ff}$ is given by	by the N	A as 1.0				9.5.1 3-2/NA.2.35				
The value of $\lambda = \lambda_1 \ \lambda_2 \ \lambda_3 \ \lambda_4$						3-2/9	9.5.2			
For intermediate supports in spar the length of the critical influence	Figu 3-2/9	re 9.5 9.5.2(5 (2)a							
Thus $\lambda_1 = (2 - 0.3 \times (L - 10))/20$	0) =1.7	3								
For span regions, $\lambda_1 = (2.55 - 0.05)$	$0.7 \times (L -$	(-10)/70) and here I	L = 28 as 1	before		Figu	re 9.5	5		
Thus $\lambda_1 = (2.55 - 0.7 \times (L - 10))$	(70) =	2.37								
The value of λ_2 is given by $\lambda_2 =$										
Where $Q_0 = 480$ kN and $N_0 =$	0.5×10^{-10}	96								
From 3–2/NA.2.39, $Q_{m1} = 260$	kN									
From 1–2/Table NA.4, $N_{\text{Obs}} =$	1×10^{6}									
Hence $\lambda_2 = \left(\frac{260}{480}\right) \times \left(\frac{1.0}{0.5}\right)^{0.2} =$	0.62									
For a 120 year design life the va	alue of λ	₃ given by Table 9.2	is 1.037:			3-2/9).5.2((5)		
The value of of λ_4 depends on the formula of the value of λ_4 depends on the formula of the value of th	he relativ	ve magnitude of the st	tress range	due to	the					
passage of FLM3 in the second	lane and	is given by:				3-2/9	9.5.2((5)		
$\lambda_4 = \left(1 + \frac{\text{effect in lane 2}}{\text{effect in lane 1}}\right)^{0.2}$										
Design stress ranges at pier										
At the pier, the stress range $\Delta \sigma_{\rm p}$, in top a	and bottom flanges (at	their mid	thicknes	ss) is:					
Top flange:7.3 N/mm²Bottom flange:9.7 N/mm²The ratio of lane 2/lane 1 effects	s = 0.94	$\lambda_4 = 1.14$	4							
$\lambda = 1.73 \times 0.62 \times 1.037 \times 1.14 =$	= 1.274									
The design stress ranges are thu	s:									
Top flange: $1.0 \times 1.274 \times$ Bottom flange: $1.0 \times 1.274 \times$	< 7.3 = < 9.7 =	9 N/mm ² 12 N/mm ²								
The partial factor for fatigue stre	ength _M	$_{\rm f} = 1.1$				3-1-9	/ NA.	2.5.3		

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CALCULATION SHEET	Client		Made by	DCI	Date	July	2009				
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The worst detail category that might apply is for a bearing plate welded to the underside of the bottom flange, which, for a flange plate over 50 mm thick, is category 36 (3–1–9/Table 8.5, detail 6). Design value of fatigue strength $\Delta \sigma_c / \gamma_{Mf} = 36/1.1 = 33 \text{ N/mm}^2 \text{ OK}$											
$D_{\rm COS}$ value of fairgue strength $\Delta \sigma_{\rm c}$ / $M_{\rm f}$ = 50/1.1 = 55 W/IIIII OK											
Design stress ranges at brac At the bracing position, there is the bottom flange (on the span g effects = 0.947 and thus $\lambda_4 = 1$	ing posi negligib girder sic 1.14 - 1 747	tion le stress range in the le) is 22.2 N/mm ² . Th	top flange. he ratio of 1	The ra lane 2/1	nge in ane 1						
$\lambda = 2.57 \times 0.02 \times 1.057 \times 1.14$	- 1./ 7/										
Bottom flange: 1.0×1.747	(222) =	38 N/mm^2									
The most operous detail at a hole	ted splic	e would be category	112 (3-1-9	/Table	81						
detail 8, at the bolt holes); the s	tress ran	ge is OK, by inspecti	on.	1 1 1010	0.1,						
For a welded splice, a flange butt weld would be category 80 (with size effect factor of $(25/t)^{0.2} = (25/40)^{0.2} = 0.91$). An open cope hole would introduce category 71 in the flange and a stress concentration factor of 2.4 in the web (for which a cut edge is category 125 or, if a butt weld terminates at the cope hole, category 112).								i			
Thus the fatigue strength is eithe	er:										
Flange butt: $80 \times 0.91/1.1 = 66$ Flange at cope: $71 \times 0.91/1.1$ Web at cope: $125/1.1 = 11$ Web butt at cope: $112/1.1 = 10$	5 N/mm^2 1 = 59 N 14 N/mm^2 12 N/mm^2	> 38 N/mm ² OK M/mm^2 > 38 N/mm ² M^2 > 38 × 2.4 = 91 M M^2 > 38 × 2.4 = 91 M	OK N/mm ² OK N/mm ² OK								
Where a transverse web stiffene would be 80 $(3-1-9)$ Table 8.4,	r is attac detail 7)	hed to the bottom fla and thus the fatigue s	nge, the de strength is	tail cate	egory						
$80/1.1 = 73 \text{ N/mm}^2 > 38 \text{ N/m}$	m ² OK										
Design stress ranges in mid- At mid-span, there is negligible flange is 26.2 N/mm ² . The ratio $\lambda = 2.37 \times 0.62 \times 1.037 \times 1.14 =$ The design stress range is thus: Bottom flange: $1.0 \times 1.75 \times$ The most onerous detail would be strength would be 73 N/mm ² (as	span stress ra o of lane = 1.747 26.2 = be a tran s above)	nge in the top flange. 2/lane 1 effects = 0. 46 N/mm ² sverse web stiffener, and this is OK even f	. The range 942 and the for which t for stiffener	the fatig	bottom 1.14 gue ed to						
the bottom flange.											

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CALCULATION SHEET	Client		Made by	DCI	Date	July	2009	
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11.2 Assessment of reinford The design value of the stress ra of $\gamma_{F,fat}$ is given by Table NA.1	ing stee ange in r as _{%,fat} =	einforcing steel is $\gamma_{\rm F,f}$ = 1.0	$_{ m at} arDelta \sigma_{ m S, equ}$ wh	ere the	value	2-1/	6.8.5	
$\Delta \sigma_{\rm S,equ}$ is referred to in EN 1994	$1-2$ as Δ	$\sigma_{\rm E}$, given by:						
$\Delta \sigma_{\rm E} = \lambda \phi \sigma_{\rm max,f} - \sigma_{\rm min,f} $						4-2/	6.8.6.	1
The value of $\lambda = \lambda_s$						2-1-	1/NN	.2.1
and $\lambda_{s} = \phi_{fat} \lambda_{s,1} \lambda_{s,2} \lambda_{s,3} \lambda_{s,4}$						-	1/111	.2.1
Where ϕ_{fat} is a damage equivale	nt impac	t factor						
The value ϕ effectively duplicate	s ϕ_{fat} but	since $\phi = 1.0$, this is	not signifi	cant		4-2/	5.8.6.	1
The value of the stress range due to FLM3 needs to be increased by a factor of 1.75 (in regions of intermediate supports) in accordance with NN.2.1(101). Stresses also need to be increased for the effect of tension stiffening in accordance with $4-2/7.4.3$								
Based on cracked section properties, the stress in the top rebars due to permanent actions is 118 N/mm^2 (see SLS values in Section 7.3). (Coexisting global plus local effects do not govern.)								ĸВ
The stress range due to the FLM Section 7.4) and this is increase thus, ignoring tension stiffening	13 fatigu d by the , gives c	the vehicle in lane 1 is 1.75 factor, giving a $\sigma_{max,f} = 135 \text{ N/mm}^2$.	0 to 8.6 N range of 1	⁷ /mm ² (s 5 N/mn	see n ² and			
Since FLM3 does not cause sag	ging ben	ding, $\sigma_{\min,f} = 118$ N/	mm^2 .					
To determine the effect of tension	on stiffer	ning, the following pa	rameters an	re neede	ed:	4-2/	7.4.3	
$\begin{array}{rcl} A_{\rm a} &= 70000, \ I_{\rm s} = 1.562 \times 10 \\ A &= 94250, \ I = 2.845 \times 10 \\ \rho_{\rm s} &= A_{\rm s}/A_{\rm ct} = 24250/(3700) \\ \alpha_{\rm st} &= AI/A_{\rm a}I_{\rm s} = (94250 \times 2.8) \\ f_{\rm ctm} &= 3.5 \ {\rm MPa} \ ({\rm for} \ {\rm C40}/{\rm 50} \ {\rm cm}) \end{array}$	0^{10} (for t 0^{10} (for th $\times 250$) = 45×10^{10} concrete)	he bare steel section) he cracked section) = 0.0262 ⁰)/(70000 × 1.562 ×	10^{10}) = 2.4	45		Shee 2-1- Tabl See 4-2/ for (et 13 1/ e 3.1 6.8.5. 0.2 fa	4 ctor
$\Delta \sigma_s = \frac{0.2 f_{\rm ctm}}{\alpha_{\rm st} \rho_{\rm s}} = \frac{0.2 \times 3.5}{2.45 \times 0.0262}$	= 11 N/	mm ²						
Thus, the maximum and minimu	um stress	ses including tension s	stiffening a	re:				
$\sigma_{\rm s,max,f} = \sigma_{\rm max,f} + \Delta \sigma_{\rm s} = 135 +$	11 = 14	16 N/mm ²						
And $\sigma_{s,\min,f} = \sigma_{s,\max,f} \frac{M_{E,d,\min,f}}{M_{E,d,\max,f}}$	- 					4-2/	6.8.5.	4
Using the ratio of stresses, rathe	er than d	irectly using moments	8:					
$\sigma_{\rm s,min,f} = 124 \times \frac{118}{135} = 128 \rm N/mr$	n ²							
For intermediate support region	and span	n of 28 m, $\lambda_{s,1} = 0.9$	7			2-2/ NN.	Figure 1	e

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For $N_{\text{Obs}} = 1 \times 10^6$, medium dis	stance tra	ffic and straight bars	$(k_2 = 9):$	$\overline{Q} = 0.$	94 and	2-2/7 NN.		
$\lambda_{s,2} = \overline{Q} \sqrt[k_2]{\frac{N_{\text{Obs}}}{2.0}} = 0.94 \sqrt[9]{\frac{1.0}{2.0}} =$	0.87							
For 120 year design life:								
$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{\text{Years}}}{100}} = \sqrt[9]{\frac{120}{100}} = 1.020$)							
For 2 slow lanes:								
$\lambda_{s,4} = k_2 \sqrt{\frac{\sum N_{\text{Obs},i}}{N_{\text{Obs},1}}} = 9 \sqrt{\frac{2.0}{1.0}} = 1.$	080							
For road surface of good rough	ness $\phi_{\rm fat}$	= 1.2				1-2/2	Anne	хB
Thus $\lambda = 1.2 \times 0.94 \times 0.87 \times$	1.02×1	.08 = 1.08						
$\Delta \sigma_{\rm E} = 1.08 \times 1.0 \times 146 - 1200000000000000000000000000000000000$	8 = 1.08	$8 \times 18 = 19 \text{ N/mm}^2$						
$\gamma_{\rm F,fat} \Delta \sigma_{\rm S,equ} = 1.0 \times 19 = 19$	N/mm ²							
For straight bars, $\Delta \sigma_{Rsk} = 162.3$	5 MPa					2-1-1 Tabl	N	
$\frac{\Delta \sigma_{\rm Rsk}}{\gamma_{\rm s, fat}} = \frac{162.5}{1.15} = 141 \text{ N/mm}^2 >$	>22 mm ²	ОК				2-1/		
Note: If the bars were bent, as the significantly reduced - see 2-	hey migl 1–1/Tab	nt be at the abutment, le 6.3.N.	, the value	of $\Delta \sigma_{ m Rsk}$	would	1		
11.3 Assessment of shear c	onnectio	on						
The design value of the stress ra	ange in sl	hear studs is given as	$\gamma_{\rm Ff} \Delta \tau_{\rm ,E2}$ wl	nere		4-2/	6.8.7	.2
$\varDelta \tau_{\mathrm{E},2} = \lambda_{v} \varDelta \tau$								
In which $\Delta \tau$ is the range of shea	r stress	in the cross section of	f the stud.			3-2/1	NA.2	.35
EN 1994-2 refers to EN 1993-2	2 for the	value of $\gamma_{\rm Ff}$, which is	given by t	the NA	as 1.0			
The value of $\lambda_{v} = \lambda_{v,1} \lambda_{v,2} \lambda_{v,3}$	$\lambda_{\mathrm{v},4}$					4-2/6	5.8.6	.2
Since the span is less than 100 r	n, $\lambda_{v,1} =$	1.55				6.8.0	5.2(4))
The value of λ_2 , λ_3 and λ_4 are c with an exponent of 1/8 rather t	alculated han 1/5	in the same manner	as for strue	ctural st	teel bu	t 6.8.0	5.2(4))
Hence $\lambda_2 = \left(\frac{260}{480}\right) \times \left(\frac{1.0}{0.5}\right)^{0.125}$	= 0.591							
$\lambda_3 = \left(\frac{120}{100}\right)^{0.125} = 1.023$								

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The value of λ_4 depends on the passage of FLM3 in the second	relative 1 lane and	magnitude of the stres is given by:	ss range du	e to the	2						
$\lambda_4 = \left(1 + \frac{\text{effect in lane 2}}{\text{effect in lane 1}}\right)$											
Shear at pier The range of vertical shear force effects is 0.867.	Shee	t 22									
At the pier, the studs are 19 mn	n diamete	er, in rows of 3 at 15	0 mm space	ing							
Thus the stress range $=$ Range	of vertica	al shear $\times Az/I_y \times 0.1$	$50/(3 \times \pi a)$	$d^{2}/4)$							
$\bar{Az}/l_y = 0.805 \text{ m}^{-1}$						Sheet 40					
Stress range $\Delta \tau = 271 \times 0.805$	× 0.150	$/(3 \times 284) = 38 \text{ N/m}$	mm ²								
$\lambda_4 = (1 + 0.867)^{0.125} = 1.081$											
$\lambda_{\rm v} = 1.55 \times 0.591 \times 1.023 \times 1.$	081 = 1.	013									
$\Delta \tau_{\rm E,2} = 1.013 \times 38 = 39 \mathrm{N/mm}$	n^2										
The reference value of fatigue s	trength f	or a shear stud is $\Delta \tau_{\rm c}$	= 90			4-2/6.8.3					
The partial factor on fatigue stre	ength γ_{Mf}	= 1.1.				3-1-9 NA	€)/ 2.5.3				
The design strength is thus 90/1	.1 = 81	$N/mm^2 > 39 N/mm^2$	OK				0.0				
Additionally, since the flange is steel flange must be verified, us	in tensic ing:	on, the interaction wit	h normal s	tress in	the	4-2/0	5.8.7	.2			
$\frac{\gamma_{\rm Ff} \Delta \sigma_{\rm E,2}}{\Delta \sigma_{\rm c} / \gamma_{\rm Mf}} + \frac{\gamma_{\rm Ff} \Delta \tau_{\rm E,2}}{\Delta \tau_{\rm c} / \gamma_{\rm Mf,s}} \le 1.3$											
With $\Delta \sigma_c = 80$.											
Coexistent stresses should be us onerous values for each of $\Delta \sigma_c$ a	ed but co and $\Delta \tau_{\rm c}$	onservatively one can	consider th	ne most	I						
$\frac{1.0 \times 9}{80/1.1} + \frac{1.0 \times 39}{90/1.1} = 0.60 \text{OI}$											
Shear at splice The range of vertical shear force effects is 0.909											
At the splice, the studs are 19 n											
Stress range = $99 \times 0.836 \times 0$.	150 / (2	\times 284) = 22 N/mm ²									

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Examp Section	le 1: Multi-g 12: Main gi	irder two-sj rder splices	pan bri	dge				
CALCULATION SHEET	Client			Made by	DCI	Dat	te	July	2009	
	SCI			Checked by	JMS	Dat	te	Sep 2	2009	
12 Main girder splices						·				
12.1 Forces and moments a	t splice	positior	า							
Design effects to be conside Worst hogging moment at splice	red: e, at UL	S and SL	.S							
Worst shear at splice, at ULS at	nd SLS									
(The worst sagging moment is n before)	nuch les	s than m	aximum hogg	ging momer	nt, as n	oted		Shee	et 25	
Consider the stresses in the pier girder side of the splice. The stress distribution will be different on the span girder side of the splice but the total moments and forces at the splice position must be the same. Because the bottom flange is smaller, more force will be carried in the web on the span side but since the moment on the bolt group on the pier side is increased by its eccentricity from the centreline of the splice and the moment on the group on the span side is decreased, it can be shown that the total effects on the bolt group are less on the span side. A symmetric arrangement of bolts, designed for the pier side, will thus be satisfactory.										
The in-service design combination	ons of a	ctions co	nsidered are:							
(1) Construction load + traffier expansion	ic load f	or worst	hogging + f	force due to	tempe	eratur	e			
(2) Construction load + traff expansion.	ic load f	or worst	shear + for	ce due to te	mperat	ure				
ULS I	nog S	SLS hog	ULS shear	SLS shea	ar					
Top flange stress	23	21	-38	-23 N/mm	1 ²					
Bottom flange stress -1	10	-88	-10	-14 N/mm	1 ²					
Shear force 8	34	650	1320	1011 N/mm	12					
From the above stresses, the for web are as follows:	ces in e	ach flang	ge, the axial f	force and m	oment	in th	e			
ULS	hog	SLS hog	ULS shear	SLS shea	ar					
Top flange force	460	420	-760	-460 kN						
Bottom flange force -3	960	-3168	-360	-504 kN						
Web force –	644	-497	-336	-259 kN						
Web moment	161	132	-33	−11 kNr	n					
Actions at the construction stage It is noted that the compressive stress in the top flange is higher at construction stage 1, under wet concrete load in span 1. At that stage, the splice must provide continuity of stiffness, without slipping, and because the beams are slender at that stage it is appropriate to amplify the design force to ensure adequate continuity of resistance.										
The maximum stress in the top	flange d	uring con	nstruction is 4	42 N/mm ²				Shee	et 23	

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SCI	Job Title	Composite highway	bridges: W	orked e	exampl	es				
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 12: Main gi	rder two-sp rder splices	oan brid	lge					
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009			
	501		Checked by	JMS	Date	Sep 2	2009			
The slenderness of the beam at	elenderness of the beam at that stage is $\overline{\lambda}_{LT} = 0.89$ which means that									
$M_{\rm c,Rk}/M_{\rm cr} = 0.89^2 = 0.79$	$M_{\rm cr} = 0.89^2 = 0.79$									
The midspan bending moment a	t the bare	e steel stage is $M_{\rm Ed} =$ $m_{\rm e} = 2.287 \times 10^7 {\rm mr}$	3132 kNn m ³ × 345 =	n (Sheet	: 23) «Nm					
Hence $\frac{M_{\rm Ed}}{M_{\rm cr}} = \frac{M_{\rm c,Rk}}{M_{\rm cr}} \times \frac{M_{\rm Ed}}{M_{\rm c,Rk}} =$										
Second order effects can thus be	e determi	ned by multiplying by	$y \frac{1}{(1-0.316)}$	$\overline{(0)} = 1.4$	6					
Thus the design force for the top	p flange	is $840 \times 1.46 = 1226$	5 kN							
(Clearly this is more onerous the similar magnitude but the flange needed.)	an in the is restra	final situation, where ined against buckling	e the stresse and no am	es are o plificat	f ion is					
12.2 Slip resistance of bolts										
Use M24 grade 8.8 preloaded b class A friction surface:	olts in do	ouble shear in normal	clearance	holes w	ith	3-1-8/3.9.1				
$d = 24 \text{ mm} d_0 = 26 \text{ mm} f_{ub} = 800 \text{ N/mm}^2 A_s = 353 \text{ mm}^2 \mu = 0.5 k_s = 1.0$										
Preload force $F_{p,C} = 0.7 f_{ub} A_s =$	0.7×80	$0 \times 353 \times 10^{-3} = 19$	98 kN							
ULS Slip resistance of bolts (do	uble shea	ar)								
$F_{\rm s,Rd} = \frac{k_{\rm s} n \mu}{\gamma_{\rm M3}} F_{\rm p,C} = \frac{1.0 \times 2 \times 0}{1.25}$	$\frac{.5}{} \times 198$	= 158 kN								
For SLS slip resistance, use the	same eq	uation but divide by γ	$Y_{M3,ser} (= 1.$	1)						
Slip resistance in double shear =	= 198/1.	1 = 180 kN								
12.3 Shear resistance of bol	ts									
ULS shear resistance of bolt (as	f bolt (assuming shear through threads):									
$F_{\rm v,RD} = \frac{\alpha_{\rm v} f_{\rm ub} A_{\rm s}}{\gamma_{\rm M2}} = \frac{0.6 \times 800 \times 1.25}{1.25}$	$\frac{353}{2} = 13$	36 kN				Tabl	e 3.4			
Resistance in double shear $= 27$	2 kN	1								

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Example 1: Multi- Section 12: Main	girder two-s girder splices	pan brie S	lge						
CALCULATION SHEET	Client	Made by	DCI	Date	July	2009				
	SCI	Checked by	JMS	Date	Sep 2	2009				
12.4 Bolt spacing and edge Limiting spacings for M24 bolts End and edge distances: $1.2d_0$ = Spacing in direction of force: 2. Spacing perpendicular to force 2. Limiting spacings for M24 bolts End and edge distances: $1.5d$ = Spacing: $2.5d$ = 2.5×24 = 60 (The parameter <i>d</i> is not specifie hole diameter for edge distances) For detailing purposes, use min	distances s, for strength: = $1.2 \times 26 = 31.2 \text{ mm}$ $.2d_0 = 2.2 \times 26 = 57.2 \text{ mm}$ $2.4d_0 = 2.4 \times 26 = 62.4 \text{ m}$ s, for fatigue classification: = $1.5 \times 26 = 39 \text{ mm}$ 0 mm ed in Table 8.1 but GN 5.08 s and bolt diameter for spaci ima of 40 mm, 65 mm and 7	n (P185 ^[6]), su ngs.) 70 mm respe	ggests t ctively	ise of	3-1-5 Tabl 3-1-5 Tabl	8/ e 3.3 9/ e 8.1				
Top flange splice (Dimensions for lower covers) Bolt spacing: In line of force: $e_1 = 50$ mm, p Perpendicular to force: $e_2 = 60$ Overall dimension 470 × 195 m Thickness 10 mm	ponfiguration: $ \begin{array}{c} & & & & & & & \\ & & & & & & \\ & & & &$	$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	tes)							

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CALCULATION SHEET	Client		Made by	DCI	Date	July	2009		
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The length of the cover is suffic be welded to the upper cover (th 800 mm).	iently sh ne maxin	ort that stud shear con num permitted longitu	nnectors do dinal spaci	o not ne ng is	ed to	4-2/0	6.6.5	.5	
Bottom flange splice (Dimensions for upper covers)									
Bolt spacing:									
In line of force: $e_1 = 50 \text{ mm}, p$	$_{1} = 65 $ I	nm							
Perpendicular to force: $e_2 = 60$	mm, p_2	= 75 mm							
Overall dimension 210×195 m	m.								
Thickness 20 mm									
Web splice Bolt spacing:									
In line of force: $e_1 = 50 \text{ mm}$ (e)	only a si	ngle column, so no p_1	value)						
Perpendicular to force: $e_2 = 50$	mm, p_2	= 75 mm							
Overall dimension 730×925 m	m.								
Thickness 10 mm									
The web depth on the support si positioned symmetrically within 87.5 mm above the flange and 6 tightening of the bolt (see GN 2	de of the this dep 67.5 mm .06, P18	e splice is 1000 mm a th, the centreline of th above the cover plate $5^{[6]}$).	nd if the w he lowest b e. This is a	eb splic olt will dequate	the is be for the	2			
12.6 Verification of connect	ion resis	stances							
Top flange splice There are 3 rows of bolts, with	4 bolts p	per row across the flat	nge.						
A category C connection is requ buckling of the beam during cor	ired (the struction	e design situation is fo	or resistance	e agains	st				
Slip resistance at ULS = 12×10^{-10}	58 = 18	396 kN >1226 kN ad	lequate						
Bottom flange splice There are 5 rows of bolts, with	4 bolts p	per row across the flat	nge.						
A category B connection is required compression in the flange in ser	ired (the vice).	e design situation is fo	r resistance	e agains	t				
Resistance at ULS									
ULS slip resistance = 20×158 bearing at ULS	= 3160	kN < 3960 kN so the solution of the second secon	ne splice w	ill slip i	into				
ULS shear resistance of bolt gro	$\sup = 20$	$0 \times 272 = 5440 \text{ kN}$ -	adequate						

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CALCULATION SHEET	Client		Made by	DCI	Date	July	2009					
	501		Checked by	JMS	Date	Sep 2	2009					
ULS bearing resistance per bolt	is given	by:										
$F_{\rm b,Rd} = \frac{k_1 \alpha_{\rm b} f_{\rm u} dt}{\gamma_{\rm M2}} =$												
Bolt spacings, for determination	of facto	rs k_1 and α_b										
In line of force: $e_1 = 50 \text{ mm}, p$	$p_1 = 65 r$	nm	1									
Perpendicular to force: $e_2 = 60$	mm, p_2	= 75 mm	= 75 mm									
Since $f_{ub} > f_u$, $\alpha_b = \alpha_d$ (but ≤ 1	1)											
For end bolts: $\alpha_d = e_1/3d_0 = 5$	0 /(3 × 2	26) = 0.64										
For inner bolts: $\alpha_d = p_1/d_0 - \frac{1}{2}$	4 = 65	$/(3 \times 26) - 0.25 = 0.58$										
For edge bolts k_1 is the smaller	of $2.8e_2/$	$d_0 - 1.7$ and 2.5										
$k_1 = \min(2.8 \times 60/26 - 1.7; 2$.5) = 2.	5										
In the upper cover plates there is for the flange and lower cover, ensure that $k_1 = 2.5$	s no 'inr the mear	her' line of bolts (in the value of p_2 that would be the value of p_2 that would be value of p_2	he directior ild apply is	n of for suffici	rce) and lent to	đ						
The value of f_u is given by the	product	standard for S355 pla	tes as 470	kN/mn	n ²	EN	1002	5-2				
Conservatively, using $\alpha_{\rm b} = 0.5$	8 the res	istance of the bolt in	20 mm cov	ers is:								
$F_{\rm b,Rd} = \frac{2.50 \times 0.58 \times 470 \times 24}{1.25}$	$\frac{\times 20}{2} = 2$	62 kN										
Bearing resistance of group, wit	h double	covers = $20 \times 2 \times 2$	262 = 1048	30 kN								
The ULS bearing resistance is a the shear resistance of the bolts. This would reduce the bearing/s (see $3-1-8/3.6.1(12)$) but the re	dequate Note th shear resistance	and the connection re at, on the span side, stance on the upper s would still be adequa	sistance is 20 mm pac shear plane te.	determ king is by abo	ined by used. out 15%	y 6						
Resistance at SLS												
SLS slip resistance of group =	20×180	0 = 3600 kN > 3312	kN satisfac	ctory								
Web splice												
The splice has a single column	of 12 bol	ts at 75 mm spacing										
For this group the 'modulus' for	r the out	er bolts = $\sum r_i^2 / r_{\rm ma}$										
where r_i is the distance of each distance of the furthest bolt.	bolt fron	the centre of the gro	oup and $r_{\rm ma}$	_x is the	2							
Here, the modulus = 1950 mm												
The extra moment due to the sh centreline of the splice	ear = sh	shear force \times eccentricity of group from the										

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SCI	Job Title	ob Title Composite highway bridges: Worked example							
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example Section	le 1: Multi-g 12: Main gi	irder two-sp rder splices	oan brid	ge			
CAI CUI ATION SHEFT	Client			Made by	DCI	Date	July 2	2009	
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Uanaa tha faraa an tha autar ha	lte et III	C and C	C oro						
Thence the force on the outer bo									
UL: Shear V	Shog SL 834	-S hog U 650	LS shear	SLS shear					
Longitudinal force F_{\perp}	609	469	-336	–259 kN					
Moment	155	127	-33	–11 kNm					
Moment due to e = 55 mm	46	36	73	<u>56</u> kNm					
Total Moment <i>M</i>	201	163	40	<u>45</u> kNm					
Force per bolt due to <i>M</i>	103	84	21	23 kN	(= M/1	950)			
Force per bolt due to <i>F</i> _L	51	39	-28	<u>22</u> kN	$(= F_{L}/1$	2)			
Total horizontal force	154	123	49	<u>45</u> kN					
Vertical force due to V	/0	54	110	<u>84</u> kN	(= V/1)	2)			
Resultant force	169	134	120	<u>95</u> kN	(Vector	sum)			
Bearing resistance for web b Note: The directions of the resu	olts	ces are n	ot parallel to	an edge. T	able 3.4	4			
Note: The directions of the resultant forces are not parallel to an edge. Table 3.4 suggests that in such cases the parallel and normal components could be verified separately but no interaction relationship is suggested. Here the direction of the resultant force being not normal to the long edge of the cover plate edge is considered not to have an adverse effect on the factors, since the edge distance is less than the end distance. The factors for resistance in a horizontal direction are therefore used.									
For end bolts (there is only a sin	ngle row	, transve	rse to the fo	rce):					
$\alpha_{\rm d} = e_1/3d_0 = 50 /(3 \times 26) = 0$	0.64								
For edge bolts $k_1 = \min(2.8 \times 3)$	50/26 -	1.7; 2.5) = 2.5						
For inner bolts $k_1 = \min(1.4 \times$	75/26 -	1.7; 2.5	i) = 2.34						
With two 10 mm covers, the be again on the 10 mm web, althou splice are lower and are not sho	aring stro gh the v wn here)	ess on th alues for	e 14 mm we the design t	b is higher forces on the	(and is at side o	higher of the			
$F_{\rm b,Rd} = \frac{2.50 \times 64 \times 470 \times 24 \times 10^{-10}}{1.25}$	$\frac{14}{2} = 202$	kN (for	end bolts, 1	92 kN for i	nner bo	lts)			
The bearing resistance is less th so bearing resistance governs.	an the re	sistance	of the bolts	in double sh	near (27	2 kN),			
The maximum resultant force at but is less than the resistance in satisfactory. The maximum force there is no slip at SLS. The force inspection.	t ULS (10 bearing the at SLS ces on the	64 kN) e and shea is 134 k e inner b	xceeds the s r (202 kN) t N and the re- polts are less	lip resistanc herefore the esistance is and are sati	e (158 l e bolt gi 180 kN isfactory	kN) coup is so y by			
12.7 Forces in cover plates									
The cover plates are verified as with EN 1993-1-1.	member	s in tensi	ion or compi	ression, in a	ccordan	ice			

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-g Section 12: Main gi	irder two-sj rder splices	pan brie S	dge				
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009		
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Top flange The covers are in tension. Assume The force per cover plate thus =	me half = 1302/2	of the load is carried $4 = 326 \text{ kN}$	in the lowe	r cover	plates				
Area of gross cross section $= 1$	95 × 10	$= 1950 \text{ mm}^2$							
Area of net section = $1950 - 2$	$2 \times 26 \times$	$10 = 1430 \text{ mm}^2$							
This is a Category C slip resista given by:	int conne	ection, therefore the d	lesign tensio	on resis	tance is	5			
$N_{\rm net,Rd} = \frac{A_{\rm net}f_{\rm y}}{\gamma_{\rm M0}} = \frac{1430 \times 355}{1.0} >$	$< 10^{-3} =$	508 kN Satisfactory				3-1-1	/6.2.	3	
The maximum spacing of bolts Table 3.3 as the smaller of $14t$ which is less than 9ε (=7.2) but satisfactory.	is 110 m (= 140 r ckling do	m and the limiting sp mm) and 200 mm. Sin pes not need to be che	= 3.25, 3-1-1/ Table 3.1						
Bottom flange The covers are in compression. plates. The force per cover plate	Assume e thus =	half of the load is ca 3960/4 = 990 kN	rried in the	upper	cover				
Fastener holes do not need to be therefore $A = 1950 \times 20 = 390$	20 mm^2	ed (unless oversize ho	oles are allo	wed),					
$N_{\rm pl,Rd} = \frac{Af_{\rm y}}{\gamma_{\rm M0}} = \frac{3900 \times 345}{1.0} \times 10^{-10}$	$0^{-3} = 13$	46 kN Satisfactory							
The maximum spacing of bolts Table 3.3 as the smaller of $14t$	is 110 m (= 280 i	m and the limiting sp nm) and 200 mm. Th	acing is giv ne spacing i	ven by s satisf	actory.	3-1-1 Table	l/ e 3.3		
Web Consider the stresses in the cover	er plate o	on a line through the	vertical rov	v of bo	lts.				
The moment on each cover plate The axial force = $609/2 = 305$ The shear force = $834/2 = 417$	e = 201 kN kN kN	/2 = 101 kN							
The stress at the bottom of the c	cover pla	te is thus:							
$(101 \times 10^6) / (10 \times 925^2/6) + (3)$	05×10^{3})	/(925 × 10)							
$= 71 + 33 = 104 \text{ N/mm}^2$									
The value of $p_1/t = 115/10 = 1$ be checked. Using a buckling le the slenderness is:	1, which of the second se	h is greater than 9ε ($0.6p_1 = 66 \text{ mm and } \delta$	3-1-8 Table 3-1-1	3/ e 3.3 l/6.3.	1.3				
$\lambda = \frac{L_{\rm cr}}{i\lambda_1} = \frac{66}{2.98 \times 76.5} = 0.30$									

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 12: Main gi	irder two-sj rder splices	pan brid	ge			
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009	
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From buckling curve a, $\chi = 0.98$ which is satisfactory. The spacing	B, so the g also cor	limiting stress $= 0.98$ nplies with the limit of	× 355/1.1 f 14 <i>t</i> (= 14	= 316 N 0 mm).	J/mm²,			
The shear stress is:								
$417 \times 10^3 / (10 \times 925) = 45 $ N/n	nm ²							
This is satisfactory and is low e. to be reduced.	nough th	at the resistance to di	rect stress (does not	need			

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 13: Transver	rder two-sp rse web stil	oan bric ffeners	lge			
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009	
	SCI		Checked by	JMS	Date	Sep 2	2009	
13 Transverse web stiffe	eners							
The intermediate stiffeners are r	eauired 1	to have adequate stiff	ness and st	ength.				
Choose flat stiffeners 200×20	nm for l	both the pier girder ar	nd span gir	der.				
The limiting outstand to prevent stiffeners this equates to a limit	torsional of $h_s/t_s \leq$	1 buckling is given by 13ε (see P356, Secti	y 9.2.1(8) a ion 8.3).	nd for	flat	3-1-: P356	5/9.2. 5 ^[4]	.1
For the yield strength of the stif	fener (f _y	$= 345$), $\varepsilon = 0.825$ a	and the limi	t is:				
$h_{\rm s}/t_{\rm s} \le 10.7$ - satisfactory								
Stiffness The effective section is 15 st $15 st200$								
For the web, $f_y = 355$ and, $\varepsilon =$	0.81							
pier girder $15 \varepsilon t$ 170 Area of Tee 9043 I_{st} 39.0 × 106	spar (30.	n girder 122 5630 9× 106						
Since, for the web panels, $a/h_{\rm w}$	= 1967	$/1000 = 1.967 > \sqrt{2}$	the stiffnes	s requi	rement	3-1-5	5/	
is $I_{st} \ge 0.75 h_w t^3$						9.3.3	3(5)	
Required $I_{\rm st} = 0.75 \times 1020 \times 14$	$4^3 = 2.0$	$6 \times 10^6 \mathrm{mm}^4$ (for the	pier girder)					
The stiffener is satisfactory for	ooth gird	ers.						
Strength The stiffener is required to sustain $V_{\rm Ed} - \frac{1}{\overline{\lambda}_{\rm w}^2} \frac{f_{\rm yw} h_{\rm w} t}{\sqrt{3} \gamma_{M1}}$	an axial	force, applied in the pl	ane of the v	veb, giv	en by:	3-1-5	5/9.3	.3(3)
Here, for the panel adjacent to t	he suppo	ort at the pier, $\overline{\lambda}_w = 0$.934			Shee	t 35	
Take max shear at support (the	value 0.5	$5h_{\rm w}$ from the support	may be use	d):				
$v_{\rm Ed} = 2511$ kN								
$\frac{1}{\bar{\lambda}_{w}^{2}} \frac{f_{yw} h_{w} t}{\sqrt{3} \gamma_{M1}} = \frac{1}{0.934^{2}} \times \frac{355 \times 10}{\sqrt{3}}$	$\frac{000 \times 14}{\times 1.1}$	$\times 10^{-3} = 2990 \mathrm{kN}$						

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 13: Transve	irder two-sj rse web sti	pan bri ffeners	dge			
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009	
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Therefore, the stiffener does no	t need to	be designed for an a	xial force.					
Since the web is Class 3, there the requirements of 9.2.1(5) do	is no des not need	stabilising effect of the local terms of ter	e web on th	e stiffe	ener, so			
The intermediate stiffeners are s	satisfacto	ry.						
13.2 Bearing stiffeners								
Consider the adequacy of double at the intermediate support.	e flat stif	ffeners, 250×25 mm	on both sid	des of t	the web			
The outstand/thickness ratio is 1 satisfactory.	0, as for	r intermediate stiffene	ers and is th	erefore	2			
$z \xrightarrow{275}$ y $z \xrightarrow{15ct} (=170)$ For the web, $t = 14 \text{ mm}, f_y =$ pier	25 15 28 355 and, girder 170	$\varepsilon t (=170)$ $\varepsilon t (=170)$ $\varepsilon = 0.81$						
Area of effective 3 stiffener I_y 907 I_z 566 i_z	34310 1 × 10 ⁶ 1 × 10 ⁶ 1 128	mm² mm⁴ mm⁴ mm						
For buckling out of the plane of (taken to mid-thickness of flanger	the web es)	o, the critical buckling	g length $L_{\rm cr}$	= 105	0 mm			
$\overline{\lambda} = rac{L_{cr}}{\lambda_1 i}$						3-1-1	l/(6.5	50)
$\lambda_1 = 93.9\varepsilon$ and for the stiffener	$r f_y = 34$	45 and $\varepsilon = 0.825$, so						
Thus $\overline{\lambda} = \frac{1050}{77.5 \times 128} = 0.11$						3-1-1 6.3.1	[/ [.2(4])
Since this value is < 0.2 , buck	ling can	be ignored						

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Mult Section 13: Tran	ti-gi sver	rder tv se wel	vo-sp b stif	oan bri ffeners	dge			
CALCULATION SHEET	Client			Made b	ру	DCI	Date	July	2009	
	SCI			Checke	ed by	JMS	Date	Sep 2	2009	
The reactions at the support due	to the c	onstruction stages	are	:						
ULS Stage 1 894 Stage 2 636 Stage 3 533										
The maximum reaction due to v and the following effects:	ariable lo	oad is 1976 kN an	nd oc	ccurs v	with	gr5 loa	ading			
<i>M</i> _Y <i>F</i> _× <i>F</i> _z Construction −8329 −295 102 Traffic gr5 −2476 269 874 −10805 −26 190	Bottom W (10 ⁶ mm ³ 9 4 45.67 3	flange Top flange W σ (10 ⁶ mm ³) σ -193 192 -54 66.63 37 -247 230	(10 3 7 4 0	Top reb <i>W</i> ⁹⁶ mm ³) 1.84	σ σ 138 59 197	Axial <i>A</i> (10 ³ mm ³ 94.3	(steel) ²⁾ σ 2 -3 -1			
Although there should be no thermal movement longitudinally (the bridge is a symmetric integral bridge and the bearing is at the mid-length of the bridge) and the transverse movement is very small (with one of the inner main girders restrained laterally), allow for an eccentricity of 10 mm in each direction.										
$N_{\rm Ed} = 894 + 636 + 533 + 19'$ $M_{\rm Ed} = 4039 \times 0.010 = 40.4 \text{ k}$ $W_z = 907 \times 10^6 / 257 = 3.53 \times$ $W_y = 566 \times 10^6 / 150 = 3.77 \times$ $W_y = 566 \times 10^6 / 331 = 1.70 \times$	76 = 402 Nm $< 10^{6}$ mm $< 10^{6}$ mm $< 10^{6}$ mm	 39 kN ³ (tip of stiffener) ³ (at stiffener) ³ web (at edge of 	sect	ion)						
Interaction criterion		-								
$\frac{N_{\rm Ed}}{N_{\rm Rd}} + \frac{M_{\rm y, Ed}}{M_{\rm y, Rd}} + \frac{M_{\rm z, Ed}}{M_{\rm z, Rd}} \le 1.0 $ (b)	iaxial in	teraction)								
Use the value of $M_{y,Rd}$ based on	the mod	ulus at the stiffene	er, n	ot on	the v	veb.				
$N_{\rm Rd} = \frac{Af_{\rm y}}{\gamma_{\rm M0}} = \frac{34310 \times 345}{1.0} \times 10^{-100}$	$0^{-3} = 118$	340 kN								
$M_{y,Rd} = \frac{W_y f_y}{\gamma_{M0}} = \frac{3.77 \times 345}{1.0} =$	1301 kNi	n								
$M_{\rm z,Rd} = \frac{W_{\rm z} f_{\rm y}}{\gamma_{\rm M0}} = \frac{3.53 \times 345}{1.0} =$	1218 kNr	n								
$\frac{4039}{11840} + \frac{40.4}{1301} + \frac{40.4}{1218} = 0.34 + $	0.03 + 0	.03 = 0.40 OK								
A separate verification should a subject to axial force and uniaxi	lso be ma al bendir	ade for the extrem ng. The interaction	ne fil n val	bre in lues fo	the vor that	web, w it case	hich is are			
0.34 + 0.07 = 0.41, which is a	also satis	factory.								
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SCI	Job Title	Composite highway	exampl	es						
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-gi Section 13: Transve	irder two-sj rse web sti	pan bric ffeners	lge					
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009			
	SCI		Checked by	JMS	Date	Sep 2	2009			
No verification of the interaction and shear stresses in the web is	n of thes called fo	e vertical stresses wit or in EN 1993.	nal stres	sses						
13.3 Bearing at loaded end o	ng at loaded end of the stiffener									
There is no explicit verification effective bearing stiffener but it										
Web/flange interface If the web is not fitted to the fla transferred through the weld. To strength of fillet welds loaded tr	inge (wh o transfe ansverse	ich is the usual case) r the full strength of t ly to their length.	the force m he web, co	nust be onsider t	the					
Using the simplified method of the weld, the resistance of a 6 n	3–1–8/4 nm throa	5.3.3 and neglecting t fillet weld is:	the longitu	dinal fo	rce on					
$F_{\rm w,Rd} = a \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} = 6 \times \frac{470/\sqrt{3}}{0.9 \times 1.3}$	$\frac{\overline{3}}{25} = 145$	0 N/mm								
As noted above, the maximum u vertical stress of 141 N/mm ² . T $141 \times 14 = 1974$ N/mm. The t	itilisation he design wo 6 mn	n in the web is 0.41, y n force in the web at n welds are adequate.	which is eq that positio	uivalen n is the	t to a refore					
Stiffener/flange interface The ends of the 25 mm flats sho impractical to provide a sufficie checked for fatigue, as follows.	ould be f ntly heav	itted and welded to th y fillet weld. The fill	le flange, b let weld mu	ecause ist then	it is be					
Range of reaction due to passag	e of FLN	M3 = 293 kN (lane 1)) and 251 k	kN (Lan	ne 2)					
The stress range at the tip of the	e flat due	e to this range is:								
293000/17610 + 2930/1100 =	20 N/mi	n^2								
The force per unit length = 25	× 20 =	500 N/mm								
The fatigue resistance should be the root of the weld.	checked	l at the toe of the wel	d (on the st	tiffener)) and a	t				
At the toe of the weld, the detai stress range is satisfactory by in	l categor spection	ry is 71 (Table 8.1, fo	or 60 mm f	lange) a	and the					
At the root of the weld, the stre the weld throat (see 3-1-9/Figu detail 3)	ss range re 5.1) a	is given by dividing and the detail category	the force/up is $36 (3-1)$	nit leng –9/Tab	th by le 8.5,					
Since there is no longitudinal or	transve	rse shear force, for a	6 mm throa	at fillet	weld,					
$\sigma_{\rm wf} = \sigma_{\perp f} = 500/(2 \times 6) = 42 \text{ N}$	/mm ²									

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Multi-g Section 13: Transve	irder two-sj erse web sti	oan bric ffeners	lge			
CALCULATION SHEET	Client		Made by	DCI	Date	July	2009	
	SCI		Checked by	JMS	Date	Sep 2	2009	
As for intermediate support regi $\lambda_1 = 1.73$	ons in S	ection 11.1:				Shee	et 44	
$\lambda_2 = 0.62$ $\lambda_3 = 1.037$								
$\lambda_4 = \left(1 + \left(\frac{251}{293}\right)^5\right) = 1.08$								
Design value is: $42 \times 1.73 \times 0.0$	52×1.02	$37 \times 1.08 = 50 \text{ N/m}$	m ²					
Fatigue strength = 36 N/mm^2 (Table 8.5	5, constructional deta	il 3)			3-1-	9/	
Design value of fatigue strength	$= 36/\gamma_{\rm N}$	$_{\rm Af} = 36/1.1 - 33 {\rm N/m}$	nm ²				le 8.3	
The weld must be increased by	about 50	% - to 9 mm throat (13 mm leg)					
(If the return weld around the enpossible to show that an 8 mm v	nd of the weld wou	stiffener were taken ald be sufficient but the	into accour hat is not ev	nt, it mi valuated	ight be l here.)			

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SCI	Job Title	Composite high	nway	bridges: W	orked	exampl	es		
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 1: Mu Section 14: Bra	ilti-gi icing	rder two-sj	pan bri	dge			
CALCULATION SHEET	Client			Made by	DCI	Date	July	2009	
	501			Checked by	JMS	Date	Sep	2009	
14 Bracing									
The configuration of the interme below.	ediate bra	acing systems bet	tweer	i girder pai	irs is a	s showi	1		
Assume the use of 120 × 120 ×	12 angle	sections.							
Consider requirements for stiffn	ess and s	strength							
To perform as a fully effective is the support, the stiffness needs	ntermed to be at 1	iate restraint to the east the value give	he bo ven b	ttom flange y:	e adjac	ent to			
$C_{\rm D} = \frac{4N_{\rm E}}{L} = \frac{4\pi^2 EI}{L^3}$									
where is the lateral second m	oment of	area of the effec	otivo	hottom fla	ngo (in	the			
simplified method cons	sidered in	a Section 9.1)	clive	Dottoini nai	ige (in	ule			
<i>L</i> is the length of flange	restraine	d by the bracing							
$C_{\rm D} = \frac{4\pi^2 \times 210000 \times 1.08}{5900^3}$	$\times 10^{9} =$	44 kN/mm							
The stiffness of the bracing syst model that reflects the actual ge effective section of the intermed simple triangulated system below	em can t ometry, iate stiff v:	be determined from including eccentr eners or a value	om a ric en can b	simple plan d connection e determin	ne fram ons, an led from	ne nd the m the			
Unit F		H Unit F	$B = H = D = \emptyset = \emptyset$	3700 mm 1100 mm 3860 mm 0.289 rad					
(The use of a diagonal system b greater flexibility than that with inclination of the angles.)	etween to the more	op and bottom fla e detailed plane f	anges frame	will gener model and	cally gi 1 the sh	ve a hallowe	r		

WORKED EXAMPLE 2:

Ladder deck three-span bridge

Index

Section

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4	Girder make-up and slab reinforcement	77
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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder d Section 2: Design bas	eck three-sp sis	oan bridg	ge			
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009	
	SCI		Checked by	RJ	Date	Sep 2	009	
2 Design basis The bridge is to be designed in a National Annexes.	accordan	ice with the Eurocode	s, as modif	ied by t	he UK	C		
The basis of design set out in El	N 1990 i	s verification by the p	partial facto	or metho	d.			
Design situations to be considered not repeated here; see Sheets 2 a	ed are as and 3 in	given in Example 1 Example 1 for details	and for bre	vity, the	ey are	Exa	ample	: 1
2.1 Partial factors and combination factors								
For a full summary of factors for all types of action, see Example 1.								
The values of the principal facto	rs used	in this Example are:						
At ULS: $\gamma_{\rm G} = 1.35$ for concrete self weig load (factors for adverse effects)	ght, 1.20 $\gamma_Q = 1$) for steel self weight 1.35 for traffic loads,	and superin 1.55 for th	mposed aermal lo	dead bads.			
At SLS, factors of unity apply. A combination factor $\psi_0 = 0.75$ accompanying actions and $\psi_0 =$ actions.	applies 0.60 to	to traffic actions whe thermal actions wher	ere they are they are a	accompa	nying			
The values tabulated in Section 7	7 are aft	er application of the 1	elevant fac	tors.				
2.2 Structural material prop	erties							
It is assumed that the same struc	tural ma	aterial grades as in Ex	ample 1 wi	ill be us	ed:			
Structural steel:S355 to ENConcrete:C40/50 to EReinforcement:B500 to EN	10025-2 N 206-1 10080 a	nd BS 4449						
For structural steel, the value of	$f_{\rm y}$ dependence	nds on the product sta	ndard.			3-1	-	
(Use 355 N/mm ² for $t \le 16$ mm 335 N/mm ² for 40 mm $< t \le 63$; 345 N/ 3 mm)	t^2 mm ² for 16 mm > t	≤ 40 mm;	and		1/N 2-1 Tab	IA.2. -1, ple 3	4
For concrete, $f_{ck} = 40$ MPa							, 10 5.	1
For reinforcement $f_{yk} = 500 \text{ N/m}$	mm ²							
The modulus of elasticity of both structural steel and reinforcing steel is taken as 210 GPa (as permitted by EN 1994-2, 3.2).								

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Silwood Park, Ascot, Berks S Telephone: (01344) 636525 Fax: (01344) 636570	L5 7QN	Subject	Example 2: L Section 2: De	adder d sign bas	eck three-sp sis	oan bridg	ge					
CALCULATION SHEET	Ī	Client			Made by	DCI	Date	July 2	.009			
		SCI			Checked by	RJ	Date	Sep 2	009			
For concrete, it is assum Example 1 and thus the long-term shrinkage strai	ed that t values of in are:	the avera	age age at firs odulus of elasti	t loadin city of	g is the san the concret	ne as in e and		Exa	ample	2 1		
r.	Short	term	Long term	Shrin	kage (long-te	erm)			· ·			
E _{cm} Modular ratio Drving shrinkage	$n_0 =$	зРа 6.0	$n_{\rm L} = 16.7$	Ecd	$n_{\rm L} = 15.4$	- 5						

	Job No.	BCR113		Sheet 4	· of	Jð Kev A				
SCI	Job Title	Composite highway b	ridges: Wo	rked exa	mples					
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder de Section 3: Actions on	eck three-sp the bridge	oan bridg	ge					
CALCULATION SHEET	Client SCI		Made by	DCI	Date	July 2	009			
			Checked by	RJ	Date	Sep 2	009			
 3 Actions on the bridge 3.1 Permanent actions Self weight of structural elem The 'density' of steel is taken as taken as 25 kN/m³. The self weight of structural elem 	enents 77 kN/i ights are	n ³ and the density of based on nominal dir	reinforced mensions.	concrete	e is	1-1- A.1	1/Tat	ole		
Self weight of surfacing The total nominal thickness of the surfacing, including waterproofing layer is 130 mm. Assume that the 'density' is 23 kN/m ³ for the whole thickness. The self weight generally produces adverse effects and for that case the self weight is										
The self weight generally produces adverse effects and for that case the self weight is based on nominal thickness $+55\%$. Thus:										
$g_k = 1.55 \times 0.13 \times 23 = 4.0$	63 kN/m	2								
Self weight of footway construction The nominal thickness of the footway (comprising concrete fill and a thin surfacing) is 200 mm and a uniform density of 24 kN/m ³ is assumed. The self weight is based on the nominal dimensions and thus:										
$g_k = 1.0 \times 0.2 \times 24 = 4.00$	KIN/111									
Self weight of parapets A nominal value of 2 kN/m is as	ssumed f	or each parapet.								
3.2 Construction loads										
Construction loads are classed as	s variabl	e loads.				1-1-	6/2.2			
For global analysis, a uniform c during casting. The use of perma extra load for formwork. Addition 1 kN/m^3 greater than that of har adds $Q_{cf} = 0.25 \text{ kN/m}^2$	onstructi anent pre onally, w dened co	on load of $Q_{ca} = 0.75$ exact planks is assume yet concrete is assume oncrete; for a slab thic	5 kN/m^2 is 2d and thus 2d to have a 2kness of 2	assumed there is a density 50 mm	d no y of this	1-1- Tab	6/ le 4.2	2		
The total construction load is the	us: $Q_c =$	0.75 + 0.25 = 1.01	kN/m ²							
3.3 Traffic loads Road traffic										
Normal traffic is represented by	Load M	odel 1 (LM1).				1-2/	2 16	1 0		
For the road carried by this brid traffic be represented by special	ge, the h vehicle	ighway authority spec SV100, as defined in	cifies that a the UK Na	abnorma tional A	l .nnex.		2.10.	.1.2		
Pedestrian traffic Pedestrian traffic is represented BS EN 1991-2, Table NA.3 and $(= 0.6 \times 5.0 = 3 \text{ kN/m}^2)$. The	by the re clause N reduction	educed value given by NA.2.36. Thus $0.6q_{\rm fk}$ n for longer loaded lend	the NA to is applied ngths is no	t made.		1-2/ NA.	Table 3	X		

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder de Section 3: Actions on	eck three-sp the bridge	oan bridg	ge				
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	009		
	SCI		Checked by	RJ	Date	Sep 2	009		
Fatigue loads For fatigue assessment, Fatigue as recommended by 3-2/9.2.2	Load M	odel 3 (FLM3), define	ed in 1-2/4	.6.4, is	used,				
3.4 Thermal actions									
Shade temperatures Maximum and minimum shade temperatures, based on a 50-year return period are defined in BS EN 1991-1-5 NA.2.20. For this bridge location, the values are: Maximum 33°C									
Minimum –17°C									
Thermal range (for determination of extreme value of thermal movement) For determination of the maximum movement at ULS, the values for a 120 year design life are relevant but according to EN 1990:A2, these are determined by applying $\gamma_0 = 1.55$ to characteristic values for a 50 year return period.									
The values of maximum/ minimum these are referred to as $T_{e,min}$ and	um unifo 1 <i>T</i> _{e,max}	orm bridge temperatur	es are give	en by El	N 1991	; 1-1-	1/6.1	.3.1	
For Type 2 deck						1-1-	5/		
$T_{e,max} = T_{max} + 4 $ (Figure $T_{e,min} = T_{min} + 5$	e 6.1)					Figu	1		
The characteristic value $\Delta T_{\rm K}$ is t	hus ½[((-17 + 5)	5)] = 49/2	= 24.5	°C		(.)		
The design value of temperature	differen	ce is given by							
$\Delta T_{\rm d}^* = \Delta T_{\rm K} + \Delta T_{\gamma} + \Delta T_0$						3-2/	(A.0)		
Assuming that bearing installation correction by resetting, $\Delta T_0 = 1$	n will b 5°C	e with estimated temp	erature and	1 withou	It	3-2/ Tab	le A.	4	
According to the UK NA, ΔT_{γ} =	= 5°C					NA.	.2.50		
Thus									
$\Delta T_{\rm d}^* = 24.5 + 5 + 15 = 44.5 ^{\circ}{\rm C}$									
For change of length in compositive 12×10^{-6} per °C.	te sectio	ns, the coefficient of l	inear therm	nal expa	nsion is	4-2/	5.4.2	.5	
Thus, if the fixed bearing is at one end of the bridge, the characteristic value of displacement at the second intermediate support is:									
$v_{\rm x} = (24500 + 42000) \times 44.5 \times 10^{-10}$	12×10^{-1}	$^{6} = 35.5 \text{ mm}$							
						1			

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Silwood Park, Ascot, Berk Telephone: (01344) 6365 Fax: (01344) 636570	s SL5 7QN 25	Subject	Example 2: Ladder Section 3: Actions of	deck three-spon the bridge	pan bridg	ge			
CALCULATION SHEE	T	Client		Made by	DCI	Date .	July 2	009	
		501		Checked by	RJ	Date S	Sep 2	009	
Vertical temperature The vertical temperature difference will be com- recommended in NA.2 100 mm, interpolate in Surfacing thickness (mm) 100 150 Interpolating for slab t $\Delta T = 12.7$ °C. (The 55% increase ov- ignored.) Only the heating differ difference situation. For temperature differe 10×10^{-6} per °C	e differen ire differen sidered to 2.12, if that in Table B. $\Delta T f$ 200 13 10. hickness 2 er nomina rence is co ence in con	ce nce giver act simu at is mor 2, as follor slab th 5 50 mm, s 1 thicknes onsidered mposite s	h in Table 6.2b will Itaneously with unif e onerous. For surfa- lows: ickness (mm) 300 16 12.5 surfacing thickness 1 ss, where surfacing here; it is more on ections, the coefficie	Checked by be used and orm tempera acing thickne .30 mm, give load is adve erous than th ent of therma	RJ temperature char ess other es rse, is ne coolin al expans	Date S ature ange, as than ng sion is	Sep 20	009 5/ le 6.2	2b

			Job No	BC	CR113				Sheet 7	7 of	58	Rev	A
Job Title Composite highway bri						ridges: Wo	rked exa	mples					
Silwood F Telephone Fax: (013	Park, Ascot, Berl e: (01344) 6365 344) 636570	(s SL5 7QN 525	Subject	Ex Se	ample ction	e 2: I 4: Gi	Ladder d rder mal	eck three-sp ke-up and sl	an bridg lab reinf	ge forcem	ent		
CALCU		ET	Client					Made by	DCI	Date	July 2	009	
			SCI					Checked by	RJ	Date	Sep 2	009	
4 G 4.1 M	irder make ain girders	-up and	slab r	einfo	orce	men	it	<u> </u>					
54500		16.5 m span girder	800 × 40 12	800 × 50	B20 @ 150 crs	B20 @ 150 crs	abutments.						
*		16 m pier girder	800 × 90 20	800 × 60	B20 @ 150 crs	B20 @ 150 crs	in midspan and at the						
42000		26 m span girder	a00 × 40 12	800×50	B20 @ 150 crs	B20 @ 150 crs	ediate supports to 1200 i e to XC3.	ler.					
*		16 m pier girder	800 × 90 20	800 × 60	B20 @ 150 crs	B20 @ 150 crs	200 mm at the interm is 35 mm, appropriat	ars will be slightly high					
54500 - 24500		16.5 m span girder	auu × 40 12	800 × 50	B20 @ 150 crs	B20 @ 150 crs	depth varies from 2 pp longitudinal bars	or upenies, ine cove ork is used, some ba					
		- - -	Web	Bottom flange	Top rebars	Bottom rebars	The overall girder The cover to the to For cross section i	permanent formwi					
The mak thickness Cross gi main gir	<i>xe-up shown a</i> ses. In praction rders are post rders by boltin	bove uses ce, more itioned at ng to flat	only tw variation 3500 m transver	no difj n mig m cen se we	<i>ferent</i> <i>ht be</i> ntres eb stif	<i>com</i> <i>used</i> in all fener	<i>bination</i> <i>for gre</i> three spread	es of flange eater econol pans, conne e main gird	<i>sizes an</i> my. ected to ers.	nd well	,		

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CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009	
	501		Checked by	RJ	Date	Sep 2	009	
4.2 Cross girders								
Intermediate cross girders								
c-L main				_{c-∟} main				
ginder	11 7	00		girder				
Overall depth 750 mm at the end	ds, 896 i	mm at the centre						
Flanges: 300×25								
Web: 15 mm			1					
girders need to be braced for the	e constru	at the cross girder million.	d-span if th	e cross				
Pier crosshead								
At the intermediate supports, a	2000 mn	n deep crosshead gird	er is provid	led, with	1			
crosshead is not covered in the o	ain girde example.	ers for bearing replace	ement. The	design	of the			



	Job No. B	CR113			Sheet]	LO of	58	Rev	Α		
SCI	Job Title C	ompos	amples								
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525	Subject E	xample	e 2: Ladder de	eck three-sp	oan bridg	ge					
Fax: (01344) 636570		ection :	5: Beam cross	s sections		1					
CALCULATION SHEET	Client SCI			Made by	DCI	Date	July 2	:009			
				Checked by	RJ	Date	Sep 2	009			
For the span girder, the values f for build up of stresses in the sp the total stresses will be at least	For the span girder, the values for the elastic moduli of the gross section will be used for build up of stresses in the span girder during construction, since classification for the total stresses will be at least Class 3.										
For the pier section, gross section properties will be used for the build up of bending stresses (since the section is Class 3 in bending) but an effective area will be used the effects of any axial compression at this stage, since the bare section is Class 4 in compression (for which $A_{\rm eff} = 105000 \text{ mm}^2$)											
Bare steel cross sections - ef	fective se	ection	properties i	n bending	l						
The values for the effective section girder at the bare steel stage (wh	on moduli ien the cros	are ne ss secti	eded for veri ion is Class 4	ification of in bendin	the spa g).	n					
The effective breadth of the Clas	ss 4 web is	given	by Table 4.1	, with:			3-1-	5/4.4			
$\psi = -499/611 = -0.817$	(and thus	$k_{\sigma} = 1$	9.5 and $\overline{\lambda}_{\rm p}$ =	=0.911), w	hich giv	ves:					
$\rho = \left[\overline{\lambda}_{\mu} - 0.055(3 + \psi)\right]/\overline{\lambda}$	$\bar{l}_{a}^{2} = [0.911]$	1 - 0.5	$5 \times 3.911]/0.9$	$911^2 = 0.93$	53						
$b_{\mu} = c \overline{b} / (1 - w) = 0.953 \times 10^{-10}$	1110/1 815	7 = 0.9	$53 \times 611 = 58$	82 mm							
$v_{\text{eff}} = \rho v / (1 - \phi) = 0.935 \times$	502	20	55 × 011 – 50	1.012	1						
I here is thus a noie in the web 61	1 - 582 =	29 mn	n long centrec	1 913 mm a	bove the	somt					
			Span girder								
Height of NA			547		(mn	ר)					
Second moment of area		lу	2.510E+10		(mn	1 ⁴)					
Elastic modulus, centroid top flang	e	Wbf,y	3.965E+07		(mn	1 ³)					
Elastic modulus, centrola bottom i	lange	VV tf,y	4.000E+07		(1111	1)					
Composite cross sections (sh	ort term)	$(n_0 =$	6.0)								
			Span girder	Pier girde	er						
Area		Α	305100		(mn	1 ²)					
Height of NA			1108	1735	(mn	ר)	Value	$of M_{\rm I}$	pl		
Second moment of area		lу	6.336E+10	2.332E+	11 (mm	1 ⁴)		lated i	ising		
Elastic modulus, top of slab		Wc	1.112E+09		(mn	1 ³)	Jy' /M) vanue 0 85f	s joi		
Elastic modulus, centroid top flang	е	₩bf,y	8.800E+08		(mn	1 ³)	for c	oncrete	ск ⁷ 7С ?		
Elastic modulus, centroid bottom f	lange	₩tf,y	5.850E+07		(mn	า ³)					
Plastic bending resistance		$M_{ m pl}$	22519		kNi	m					
The cross section of the span girder is Cl within the spacing limits in 4-2/6 6 5 5 (in	ass 1, provide	ed that th	e top flange is re g 730 mm max	estrained by sl	hear conn 299 mm)	ectors					
Uncracked pier girder section properties	are needed fo	r calcula	tion of shear flov	v.	200						
Composito cross soctions (lo	na torm)	6000	ina midena	n(n - 1)	6 7)						
		- sayy	Span girder		0.77						
Area		Δ	164300	(mm ²)							
Height of NA		~	922	(mm)							
Second moment of area		<i>L</i>	5.028F + 10	(mm ⁴)							
Elastic modulus, top of slab		Ŵc	1.590F + 09	(mm ³)							
Elastic modulus, centroid ton fland	e	W _{tf} ,	1.949E+08	(mm ³)							
Elastic modulus, centroid bottom f	lange	W _{bf,z}	5.605E+07	(mm ³)							
	÷			<u></u>							

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	Job No.	BCR113	5		Sheet]	l1 of	58	Rev	Α
SCI	Job Title	Compos	ite highway b	ridges: Wo	rked exa	amples	5	1	
Silwood Park, Ascot, Berks SL5 7QN	Subject]	Example	e 2: Ladder de	eck three-sp	an brid	ge			
Telephone: (01344) 636525 Fax: (01344) 636570		Section	5: Beam cross	s sections		-			
CALCULATION SHEET	Client SCI			Made by	DCI	Date	July	2009	
				Checked by	RJ	Date	Sep 2	2009	
Cracked composite sections	(hogging) - pier	girder						
			Gross	Effective	*				
Area		A	145200	120660) (mn	1 ²)			
Height of NA			1174	1160	(mn	n)			
Second moment of area		/y	1.390E+11	1.380E+	11 (mn	n ⁴)			
Elastic modulus, top rebars		W	1.129E+08	1.129E+	08 (mn	n ³)			
Elastic modulus, centroid top flang	ge	₩tf,y	1.389E+08	1.391E+	08 (mn	n ³)			
Elastic modulus, centroid bottom t	flange	₩bf,y	1.215E+08	1.197E+	08 (mn	n ³)			
Section class			4	4					
Cracked composite sections	(hogging) -at fi	rst cross gir	der in mai	in span	1			
			Gross	Effective	e				
Area		A	141300	122640) (mn	า ²)			
Height of NA			1032		(mn	n)			
Second moment of area		/y	1.033E+11		(mn	n ⁴)			
Fleatic medulus, ten rehere		W	9.861E + 08		(mn	n ³)			
Elastic modulus, top rebars					(• /			
Elastic modulus, top rebars	je	<i>W</i> tf,y	1.234E+08		(mn	n ³)			
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f	je flange	Wtf,y Wbf,y	1.234E+08 1.031E+08		(mn (mn	n ³) n ³)			
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b	ge flange oundary in be	Wtf,y Wbf,y	1.234E + 08 1.031E + 08 3* is class 4 under a	4 axial compres	(mn (mn sion.	n ³) n ³)			
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Temperature difference For calculation of primary effect $E_{cs} = 35$ GPa (For steel, E	ge flange oundary in be Derature o ts, use the = 210 GF	Wtf,y Wbf,y ending. It differen short-t Pa)	1.234E + 08 1.031E + 08 3* is class 4 under a nce & shrink erm modulus	4 axial compress age for concre	(mn (mn sion.	n ³)	She	et 2	
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Temperature difference For calculation of primary effect $E_{cs} = 35$ GPa (For steel, E Note: For each element of section, determine force and centre of force	ge flange oundary in be Derature o ts, use the = 210 GH <i>calculate su</i> <i>for that ar</i>	Wtf,y Wbf,y ending. It differen short-t Pa) tress as tea.	1.234E + 08 1.031E + 08 3* is class 4 under a nce & shrink erm modulus strain × modul	4 axial compress age for concre lus of elastic	(mn (mn sion.	n ³)	She	et 2	
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Temperature difference For calculation of primary effec $E_{cs} = 35$ GPa (For steel, E Note: For each element of section, determine force and centre of force For a fully restrained section comp and moment in the span girder due noted on Sheet 6 are:	ge flange oundary in be berature o ts, use the = 210 GH <i>calculate su</i> <i>for that ar</i> prising the e to the cha	Wtf,y Wbf,y ending. It differen e short-t Pa) tress as ea. full half	1.234E + 08 1.031E + 08 3* is class 4 under a nce & shrink erm modulus strain \times modul f-width of the tic values of ta	4 axial compress age for concre <i>lus of elastic</i> slab, the resemperature	(mn (mn sion. sion.	n^{1}	She	et 2	
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Temperature difference For calculation of primary effect $E_{cs} = 35$ GPa (For steel, <i>E</i> Note: For each element of section, determine force and centre of force For a fully restrained section comp and moment in the span girder due noted on Sheet 6 are:	ge flange oundary in be Derature o ts, use the = 210 GH <i>calculate si</i> <i>for that ar</i> prising the e to the cha	Wtf,y Wbf,y ending. It differen short-t Pa) tress as ea. full half aracteris	$1.234E + 08$ $1.031E + 08$ 3^* is class 4 under a nce & shrink erm modulus strain × modul f-width of the tic values of ta Centr	4 axial compress age for concre- lus of elastic slab, the res emperature e of force	(mn (mn sion.	n ³) n ³)	She	et 2	
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Temperature difference For calculation of primary effec $E_{cs} = 35$ GPa (For steel, E Note: For each element of section, determine force and centre of force For a fully restrained section comp and moment in the span girder due hoted on Sheet 6 are:	ge flange oundary in be perature o ts, use the = 210 GH <i>calculate su</i> <i>for that ar</i> prising the e to the cha	Wtf,y Wbf,y ending. It differen e short-t Pa) tress as ea. full half aracteris	1.234E + 08 1.031E + 08 3* is class 4 under a nce & shrink erm modulus strain \times modul S-width of the tic values of ta Centr Below top	4 axial compress age for concre <i>lus of elastic</i> slab, the res emperature e of force Above NA	(mn (mn sion. sion. ete: ete: straint fo differen (kN	n ³) n ³) orce ce	She	et 2	
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Temperature difference For calculation of primary effec Ecs = 35 GPa (For steel, <i>E</i> Note: For each element of section, determine force and centre of force For a fully restrained section comp and moment in the span girder due noted on Sheet 6 are: Top part of slab 0.	$\frac{ge}{flange}$ flange oundary in be oerature o ts, use the = 210 GF calculate su for that ar prising the to the cha w strain 000084	Wtf,y Wbf,y ending. It differen e short-t Pa) tress as ea. full half aracteris Ford (kN 315	1.234E + 081.031E + 083*is class 4 under ais class 4 under ance & shrinkerm modulusstrain × modulastrain × modulace k shrinkce k shrinkshrinkce k shrinkshrinkce k shrinkshrinkce k shrinkshrinkce k shrink	4 axial compress age for concre <i>lus of elastic</i> slab, the res emperature e of force Above NA 236	city, then sion. ete: city, then straint for differen Mon (kN	n ³) n ⁴	She	et 2	
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Fermperature difference For calculation of primary effect E_{cs} = 35 GPa (For steel, E Note: For each element of section, determine force and centre of force For a fully restrained section compand moment in the span girder due noted on Sheet 6 are: Top part of slab 0. Bottom part of slab 0.	$\frac{1}{1000084}$	Wtf,y Wbf,y ending. It differen short-t Pa) tress as ea. full half aracteris Ford (kN 315 90	1.234E + 081.031E + 083*is class 4 under ais class 4 under ance & shrinkerm modulusstrain × modulCentreCentreBelowtop36211	4 axial compress age for concre- lus of elastic slab, the res emperature e of force Above NA 236 100	tinn (mn (mn sion. ete: eity, then straint fo differen (kN 74 9	n ³) n ³) orce ce hent lm) 44 0	She	et 2	
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Femperature difference For calculation of primary effect Electron class For calculation of primary effect For calculation of primary effect Ecs = 35 GPa (For steel, E Note: For each element of section, Ideermine force and centre of force For a fully restrained section comp and moment in the span girder due noted on Sheet 6 are: Top part of slab 0. Bottom part of slab 0. Top flange 0.	$\frac{1}{1000084}$	Wtf,y Wbf,y ending. It differen short-t Pa) tress as ea. full half aracteris Ford (kN 315 90 20	1.234E + 081.031E + 083*is class 4 under ance & shrinkerm modulusstrain × modulstrain × modulc-width of thetic values of tac-width of tatic values of tac-width of tatic values of tac-width of tatac-width of tat	4 axial compress age for concre- lus of elastic slab, the res- emperature e of force Above NA 236 100 28	(mn (mn sion. ete: eity, then straint fo differen (kN 74 9 6	$\frac{1}{n^3}$ $\frac{1}$	She	et 2	
Elastic modulus, top rebars Elastic modulus, centroid top flang Elastic modulus, centroid bottom f Section class * The cross section is on the Class 3/4 b 5.2 Primary effects of temp Femperature difference For calculation of primary effect \mathcal{E}_{cs} = 35 GPa (For steel, E Note: For each element of section, Idtermine force and centre of force For a fully restrained section comp and moment in the span girder due noted on Sheet 6 are: Top part of slab 0. Bottom part of slab 0. Top flange 0. Web (to 400 below slab) 0.	$\frac{ge}{flange}$ flange for that ar for that ar for the cha for the cha flange	Wtf,y Wbf,y ending. It differen e short-t Pa) tress as ea. full half aracteris Ford (kN 31E 90 20 9	1.234E + 081.031E + 083*is class 4 under ais class 4 under amce & shrinkerm modulusstrain × modulStrain × modulStrain × modulCentrCentrBelowtop36211982270410	4 axial compress age for concre- lus of elastic slab, the res- emperature e of force Above NA 236 100 28 -112	(mn (mn sion. ete: etty, then straint fo differen (kN 74 9 6	$\frac{1}{n^3}$ $\frac{1}$	She	et 2	

	Job No.	BCR113		Sheet	12	of	58	Rev	А
SCI	Job Title Composite highway bridges: Worked examples								
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Example 2: Ladder deck three-span bridge Section 5: Beam cross sections								
CALCULATION SHEET	Client		Made by	DCI	Da	ite	July 2	2009	
	501		Checked by	RJ	Da	ite	Sep 2	009	
The strains and forces are illustrate	ed diagra	mmatically below. Strain 12.7 × 10 ⁻⁵ 4.0×10^{-5}							



- NA

For the above calculation, short-term section properties for the full width of slab are used, not those tabulated on Sheet 10. (Area = 383200 mm^2 , steel units)

Diagrammatically:



		Job No. B	CR1	13			Sheet	t 13 o	of	58	Rev	А
SCI		Job Title C	omp	osite highw	vay br	idges: Wo	orked of	example	es		-	
Silwood Park, Ascot Telephone: (01344) Fax: (01344) 63657	, Berks SL5 7QN 636525 0	Subject E	xamj ectio	ple 2: Lado n 5: Beam	ler de cross	ck three-s sections	pan br	ridge				
CALCULATION	SHEET	Client			I	Made by	DCI	Date)	July 2	2009	
		SCI			[Checked by	RJ	Date)	Sep 2	:009	
The release of the restraint moments is applied along the span, in the uncracked regions, as a separate loadcase. Since the girder depth varies along the span, the value of the restraint moment will vary along the span. For simplicity, a uniform 'average' value has been applied to the model in this example. Note that the omission of restraint moments in cracked regions is not mentioned in												
<i>EN 1994-2 but the view has been taken that the omission permitted for shrinkage (see EN 1994-2, 5.4.2.2(8)) may be used for the calculation of secondary effects of temperature difference.</i>												
Shrinkage For complete verification, shrinkage effects should be calculated at the time of opening to traffic and at the end of the service life and the more onerous values used. Here, primary and secondary effects are calculated only for the long-term situation (the values are greater than those at opening) and where the total effects of shrinkage are advantageous, they are neglected.												
The characteristic and the modular r effects generally a properties will be For a fully restration beam, due to the	value of shrin ratio is $\eta_L = 1$: and for determined for both. ined section, the characteristic value of the section of the	kage strain 5.4. (This i ning the se) e restraint ralues of sh	is g is ve cond force urink	iven on Shery close to dary effect e and mon age strain	the the volume of the volume o	as $\varepsilon_{cd} =$ value for one set c n the spar	33.1 long-t of long	× 10 ⁻⁵ erm g-term er, inne	er			
				Centre	of forc	e	mor	ment	7			
	Strain F	Force (kN)	В	elow top	Abc	ove NA	(kl	Nm)				
Slab	-0.000331	-8049		125	3	329	-2	647				
	_	-8049	=			=	-2	647	=			
The release of the separate loadcase. moment will vary to the model in this Hence the primar	restraint momer Since the girder along the span. s example. y effects are:	nts is applied depth varie For simplic	d alo es alc ity, a	ng the span ong the span a uniform 'a	n, in th n, the averag	ne uncrack value of ti ge' value h	ted reg he rest has bee	gions, as traint en appli	s a ed	a 4-2/.	5.4.2.	.2(8)
	W (steel unit	ts) (<i>E</i> cd <i>E</i> cd	int m)	Rele Bending	ase of (<i>M/W</i>)	restraint Axial (F/A)	Total				
Top of slab	1.22E+08	4.5		-1.4	4	-2.	6	0.5				
Bottom of slab	2.72E+08	4.5		-0.0	6	-2.	6	1.3				
Top of top flange	2.72E+08	0.0		-9.1	7	-40	.0	-49.7				
Bottom flange	-5.56E+0	7 0.0		47.0	6	-40	.0	7.6				
For the above cal	culation, sectio	n propertie	es fo	r the full v	vidth	of slab ar	e used	d.				

Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Job Title Comp Subject Exam	posite highway br	idges: Wor	ked exa	mples			
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Exam	nple 2. Ladder de						
	Ink, Ascot, Berks SL5 7QN (01344) 636525 (4) 636570SubjectExample 2: Ladder deck three-span bridge Section 5: Beam cross sections							
CALCULA FION SHEET	Client		Made by	DCI	Date	July 2009		
	SCI	-	Checked by	RJ	Date	Sep 20	09	
Diagrammatically:		'						
1600								
1400								
1200								
1000								
800								
600								
400								
400								
200 -								
-60.0 -40.0 -20.0	0.0 20.0							
	1750	 						
	z	896 at	midspan					
For the effective composite sec effectively simply supported an each side. Assume that there is The effective section properties	tion, taking acc d thus $L_e = 11^{\circ}$ only a single re of the cross give	ount of shear lag 700 mm and b_{ei} and b_{ei} ow of connectors rder at midspan	g, the cross = 11700/8 s on the bea are:	girder = 143 am cent	is 8 mm treline			
	Bare steel	Short-term comp	Long- te	erm		Value	s of M	I _{pl}
Area 🖌	27690	149600	7147	7 (r	nm²)	$\int \frac{calcul}{f_u/w_{ro}}$	ated i value	ising s for
Height of NA	448	915	799	(r	nm)	steel,	0.85f	$k/\gamma_{\rm C}$
Second moment of area /	v 3.603E+09	1.163E+10	9.404E+	-09 (r	nm⁴)	for co	ncrete	?.
Top of slab		3.021E+08	4.526E+	-08				
Elastic modulus, centroid top	of,y 8.273E+06	-3.692E+08	1.113E+	-08 (r	mm³)			
Elastic modulus, centroid	f,y 8.273E+06	1.289E+07	1.196E+	-07 (r	nm³)			
bottom nange	2 (bending)	1 (sagging)	1 (sanni	na)				
Section class		, ougging/	, (Juggi	······		1		
Section class Plastic bending resistance M	^{pl} 3207	6232 (sagging)	k	Nm			

	Job No.	BCR113		Sheet 1	5 of	58	Rev	Α
SCI	Job Title	Composite highway b	mples					
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder de Section 6: Global ana	eck three-sp lysis	oan bridg	ge			
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009	
	501		RJ	Date	Sep 2	009		
 6 Global analysis 6.1 3D FE model A 3D finite element model of th The steel girders were represent for the web. The deck slab was the deck slab was divided longit midway between; transversely th main girder and one for each ca was intended to give an accurate for the application of empirical a Cracked section properties were intermediate support (approxima elements were given anisotropic transversely). 	e structu ed by be represen udinally ne mesh ntilever e represe allowanc used for tely 15% properti	are was created, as she cam elements for the f ted by a mesh of shel by lines of nodes alor was divided into six e portion of slab. This of ntation of shear lag ef es in the model. The slab elements eit of the span either sid es (cracked longitudir	Checked by own below. langes and l elements. ng each cro elements be division for ffects, with her side of de of the su hally, uncra	RJ shell eld The me ss girde tween ea the dec out the upport). cked Span 3	Date ements esh for r and ach k slab need The	Sep 2	009	
CG9 CG8 Span 1 Span 1 Analysis model, showing main of The global analysis effectively g in the model. The software then forces on notional composite bear rules. Each notional composite bear rules. Each notional composite bear plus a width of deck slab. For the cross beams, the notional the forces and moments are base For the main beams, the notional slab to the bridge centreline.	girders, of ives a co converts ams, for beam cor al beam id beam id beams	Key: CG8 Cross girder and cross girders and slab comprehensive pattern is the stresses into equiverification in accord nprises the steel element includes half the width e stresses in one deck include the cantilever	reference us o mesh on s of stresses ivalent mor ance with t ents of flam h of slab ei slab element element an	ed in tex span 3 c in all el- nents an he Euro ges and ther side nt either nd the w	et ement ad web e, i.e. side. vidth o	s		

	Job No. BCR113		Sheet]	6 of	58	Rev	Α			
SCI	Job Title Composite highway									
Silwood Park, Ascot, Berks SL5 7ΩN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Example 2: Ladder Section 6: Global an	Subject Example 2: Ladder deck three-span bridge Section 6: Global analysis								
CALCULATION SHEET	Client	Made by	DCI	Date	July 2	2009				
	501	Checked by	RJ	Date	Sep 2	009				
Because shear lag effects are aut the moments and forces per bear any notional pattern of shear lag	tomatically accounted for in t m thus depend on the 'actual s, such as that given by the ru	the global and shear lag r ales in EN 1	nalysis n ather th 1994-2.	nodel, an on						
The extracted effects on each ma of moment and axial force. The being modelled separately but al girders (there is a small longitud loading is not symmetrical).	ain girder section (as defined axial forces arise mainly as so as a result of unequal load linal shear across the bridge	above) wer a result of the ling on the centreline w	te combine edge two mai then the	nation beam n	s					
The application of the extracted (allowing for shear lag) is a real variation of axial force in the ed longitudinal shear which can be (see Section 10.1)	moment plus axial force on t istic combination of effects of ge beam gives rise to (relativ taken into account in verifyin	the effective on those sector (ely small) and the shear	cross setions. The additionation	ections ne al tion	5					
6.2 Construction stages										
For simplicity, it is presumed th whole of span 1, followed by the The edge beams will be concrete therefore provided for:	at the deck will be concreted e whole of span 3, followed ed after span 2. Separate ana	in three sta by the whol lytical mode	iges - th e of spa els are	e n 2.						
Stage 1 All steelwork, wet con Stage 2 Composite structure in Stage 3 Composite structure in Stage 4 Composite structure in Stage 5 Composite structure (s (For simplicity, the weight of the includes the long-term properties, model. The difference between the design of the main beams.)	acrete in span 1 a span 1 (long-term properties a spans 1 & 3, wet concrete i a both spans (long-term properties) both term properties) e edge beams is applied to the s of the edge beams, rather the the two approaches is negligible	s), wet conc n span 2 erties) e stage 4 m han introduc ble, in relati	odel, wh ce anoth con to th	span 3 nich er e						
A further model, a modification determine the rotational stiffness	of Stage 3, without the wet a soft the beams at that stage.	slab, was ar	alysed t	0						
6.3 Analysis results										
The following results are for des appropriate partial factors on cha- load cases for shrinkage and tem given).	sign values of actions, i.e. af aracteristic values of actions, perature difference (for which	ter applicati except for ch character	on of the indivisitic val	vidual ues ar	e					
For construction loading, results construction stages. For traffic 1 traffic and pedestrian loading for intermediate support and at the r resistance adjacent to the interm coexistent effects at the position	are given for the total effect oading the results are given for r worst bending effects at two middle of the central span. For ediate support, additional ress of the first cross girder in the	ts at each of for the combound o locations or verification ults were ex- be central sp	the bination at an on of bu tracted an.	of Ickling for	;					

		Job N	o. BCR1	13			Sheet	L7 of	58	Rev	Α
		Job Ti	itle Comr	osite highy	vav bri	dges: Woi	rked exa	amples	5	1	
Silwood Bark App	ot Borko SIE -		nt Exam	nlo 2. Lod	lor doo	lt throa an	on hrid	- I	·		
Telephone: (01344) Fax: (01344) 6365	636525 570		Sectio	on 6: Globa	l analy	sis	an orid	ge			
CALCULATION	SHEET	Client SCI			Ν	lade by	DCI	Date	July 2	2009	
		ber			c	hecked by	RJ	Date	Sep 2	009	
Stage 1 Self weight of a Self weight of a Construction loa	steelwork concrete on ads on span	span 1 1									
Position		ULS				SLS					
FUSICION	<i>M</i> _y (kNm)	$F_{\rm X}$ (KN)	Fz (KN	I) /// _y (ki	vm)	$F_{\rm X}$ (KN)	Fz (K	N)			
At pier	-4432	4	311	-34	76	3	25	5			
At CG8	-3291		309	-254	18	0	250	5			
At CG13	121		82	16		0	61				
Note: F_x is axial force CG8 is the first inter-	ce, <i>F</i> _z is vertical rmediate cross	shear girder on the	main span s	side of the pie	r. CG13	is at midspa	n.				
Stage 2											
Self weight of a	concrete on	span 3									
Construction lo	ads on span	3									
Removal of con	struction loa	ads on spar	n 1								
		111.5				515					
Position	$M_{\rm y}$ (kNm)	F_{x} (kN)	<i>F</i> z (kN	l) <i>M</i> v (kl	Nm)	$F_{\rm x}$ (kN)	<i>F</i> z (k	N)			
At pier	1307	-1	-77	96	3	-1	-5	7			
At CG8	1004		-95	74	1	0	-70	C			
At CG13	-584		-91	-43	3	0	-6	7			
Stage 3 Self weight of a Construction lo Removal of con	concrete on ads on span struction loa	span 2 2 ads on spar	n 3								
		ULS				SLS					
Position	<i>M</i> y (kNm)	<i>F</i> _x (kN)	<i>F</i> z (kN	l) <i>M</i> y (kl	lm)	<i>F</i> _x (kN)	<i>F</i> _z (k	N)			
At pier	-9374	48	1209	-694	13	35	89	5			
At CG8	-5267	38	1083	-390	2	28	802	2			
At CG13	4371	40	2	323	/	30	2				
Stage 4											
Self weight of a	concrete edg	ge beams									
Self weight of p	parapets	. .									
Self weight of a	carriageway	surfacing									
Self weight of a	otruction 1	struction	. 1								
Kenioval of con	su uction 102	us on spar	1 4								
		ULS			SL	S					
Position	<i>M</i> y (kNm)	F _x (kN)	Fz (kN)	<i>M</i> у (kNm)	<i>F</i> _x (k	(N) Fz	(kN)				
At pier	-6944	303	936	-5693	25	7 7	69				
At CG8	-3825	227	828	-3130	19	5 6	81				
At CG13	2904	-457		2388	-37	0	0				
	_										

			J	Job No. BCR113					Sheet]	8 of	58	Rev	Α
	SCI		J	Job Title Composite highway bridges: W					rked exa	mples			
Silwood	SCI Park, Aso	ot. Berks SL	5 70N S	ubject	Exam	nle 2	Ladder de	eck three-sn	an bride	ve.			
Telephor Fax: (01	ne: (0134 344) 636	4) 636525 570			Sectio	on 6:	Global ana	lysis	un orrag	50			
CALCU	ILATIO	N SHEET	C	lient				Made by	DCI	Date	July 2	2009	
				CI				Checked by	RJ	Date	Sep 2	009	
.							•			1			
Long te	erm shr	inkage (re	estraint i	nome	nts app	plied	in uncrac	ked region	s)		She	et 2	
The Ion	lowing (characteris	tic value	s appi	y at boi	un U.	LS and SL	S, since $\gamma_{\rm Sh}$	= 1.0		and		
Position		(haracteri	stic	(LNI)						Exa	mple	1
		<i>W</i> _γ (KNM)	<i>F</i> _× (KN)	F	2 (KIN) 15								
		-3414	235		-15								
At CG1	3	-3737	-35		0								
	-				-								
Stage	5 - vari	able actio	ons										
Traffic	loads f	or worst l	nogging	at inte	ermedia	ate s	upport						
Load			<u> </u>	LS				SLS					
Group	Position	ח <i>M</i> y (kN	m) <i>F</i> _x	(kN)	<i>F</i> _z (k	N)	<i>M</i> y (kNm)	<i>F</i> _x (kN)	<i>F</i> z (k	N)			
Gr1,	At pier	-1111	3 5	98	146	3	-8231	444	108	3			
Gr1,	At CG8	-603	4 5	32	140	0	-4469	395	103	7			
Gr5,	At pier	-1082	28 7	19	136	8	-8020	532	101	3			
Gr5	At CG8	-596	96	61	137	9	-4421	490	102	1			
Traffic	loads f	or worst s	hear at	interr	nediate	e sup	port						
Load		ULS					SLS						
Group	Positio	ר <i>M</i> y (kNr	n) <i>F</i> _x (k	N)	Fz (kN)		<i>М</i> у (kNm)	<i>F</i> _× (kN)	<i>F</i> _z (kN)				
Gr1	At pier	-8190	492		2029		-6021	371	1509				
G. 1	At CG8	8 –2405	341		1206		-1753	259	885				
Gr5	At pier	-7719	563		2169		-5675	422	1612				
	At CG8	-1493	351		1419		-1084	265	1046				
Traffic	loads f	or worst s	agging a	ıt mid	ldle of	cent	re span						
Load		ULS	00 0				SLS						
Group	Positio	ר <i>M</i> y (kNr	n) <i>F</i> _x (k	N)	Fz (kN)		<i>М</i> у (kNm)	<i>F</i> _× (kN)	<i>F</i> z (kN)				
Gr1	At CG1	3 8120	-172	28	-113		6015	-1278	-83				
	At CG1	2 7052	-155	53	488		5224	-1149	361				
Gr5	At CG1	3 8585	-188	30 1 2	-69		6359	-1391	-51				
CG13 is	at midspa	2 7434 n CG12 is at	the adiace		airder		5507	-1132	490				
221010		,			9								
Effects	of ther	mal action	IS										
Effects	due to t	he charact	eristic va	lues (ions)	of vertic	cal te	emperature	difference	(restrain	nt			
	- appin		har '										
Position		(M. (kNm)	naracteri	stic	- (kNI)								
At nier		1233			[∠] (KIN) 5								
At CG8		1224	-89		-1								
At CG1	3	1352	37										
						-							

	Job No.	BCR113		Sheet	19	of	58	Rev	Α		
SCI	Job Title	bb Title Composite highway bridges: Worked examples									
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	^{oject} Example 2: Ladder deck three-span bridge Section 6: Global analysis									
CALCULATION SHEET	Client		Made by	DCI	Dat	е	July 2	009			
501			Checked by	RJ	Dat	е	Sep 20)09			

Worst shear at midspan

	U	LS	SI	S
Position	<i>F</i> z (kN)	⊿F _x over 3.5m	Fz (kN)	⊿F _x over 3.5m
Gr 1 traffic	672	427	498	316
Gr5 traffic	755	530	559	393

Range of effects due to passage of fatigue vehicle

Worst bending effects

	Pier		Mid-span	
	$M_{\rm y}$ (kNm)	<i>F</i> _× (kN)	My (kNm)	F _× (kN)
Lane 1 pos	-1560	146	1278	-289
Lane 1 neg	243	-14	-179	56
Range	-1803	161	1457	-344
Lane 2 pos	-1109	50	891	-227
Lane 2 neg	170	-7	-126	33
	-1279	57	1017	-260

Worst shear effects

	Pier	Mid-span	Associate	d change of F_x
	Fz (kN)	Fz (kN)	Pier	Mid-span
Lane 1 pos	403	105	41	-99
Lane 1 neg	-21	-114	0	45
Range	423	219	41	-144
Lane 2 pos	227	68	20	-57
Lane 2 neg	-14	-76	1	23
	241	143	19	-80

The change of axial force F_x is the value over a panel of 3.5 m

Effects on intermediate cross girder

With the traffic load positioned for worst effects on the cross girders, the worst sagging occurs in the middle cross girder in the central span. The values are:

	At mid	-span of	CG13
	Mу	Fx	Fz
Dead Loads at Stage 1 (ULS)	48	0	0
Dead Loads at Stage 2 (ULS)	0	0	0
Dead Loads at Stage 3 (ULS)	523	2	
Dead Loads at Stage 4 (ULS)	272	104	
	843	106	0
Gr1 worst sagging (ULS)	1960	-93	43
Gr5 worst sagging (ULS)	2192	-78	184

		Job N	o. BCI	R113					Sheet	20	of	58	Rev	Α
SCI		Job T	itle Con	nposite	highw	ay b	ridį	ges: Wo	orked e	xamp	ples			
Silwood Park, Ascot, Berks SL Telephone: (01344) 636525 Fax: (01344) 636570	5 7QN	Subje	ct Exa Sect	mple 2 tion 6:	: Ladd Global	ler de l ana	eck lysi	three-s is	pan bri	dge				
CALCULATION SHEET		Client					Ma	ide by	DCI	Da	Date July 2009			
			Checked by					RJ	Da	ate	Sep 2	009		
For consideration of the effects on the bolted end connection, the values at the first and second cross girders adjacent to the pier and the middle cross girder are:														
		CG8			CG9				CG13			1		
	Му	Fx	Fz	Mу	Fx	F	z	Mу	Fx	F	z			
Stage 1	-19	10	16	-10	7	15	5	0	0	16	6			
Stage 2	-1	-4	0	0	1	C)	0	0	(C			
Stage 3	-86	40	180	-27	11	180)	-17	1	180	C			
Stage 4	-106	29	99	-91	29	113	3	-68	-26	108	3			
Construction	-212	75	295	-128	48	308	3	-85	-25	304	1			
Gr1 traffic (for max shear)	-436	49	901	-281	-24	843	3	-201	-187	806	3			
Gr5 traffic(for max shear)	-447	94	933	-276	-22	879)	-184	-239	859	Э			
Total (gr5)	-659	169	1228	-404	26	308	3	-269	-264	1163	3			

				Job No.	BCR	.113				Sheet 2	21 of	58	Rev	Α
S	ci			Job Title	Com	posite hig	ghway	bridges:	Wo	rked exa	mples			
Silwood Pa Telephone: Fax: (0134	rk, Ascot, E (01344) 63 4) 636570	Berks S 36525	L5 7QN	Subject	Exar Secti	nple 2: L on 7: De	adder o sign va	leck thr lues of	ee-sp the e	an bridg ffects of	ge f comb	ined a	ctions	
	ATION SI	HEET		Client				Made b	у	DCI	Date	July 2	2009	
				SCI				Checke	ed by	RJ	Date	Sep 2	009	
	_					_								
7 De Design va central sp need to be	sign va llues of ei an. In pra e consider	ffects actice, red.	of the are giv , furthe	e effects ven below er situatio	for c ns for	combine ertain de other pa	ed ac esign si arts of	tions tuations the stru	s for cture	parts of would	f the also			
7.1 Eff	ects of c	const	ructio	n loads (ULS)									
Generally as constru on the bar cracked co stages 1, 2 term com	, the effe action pro- re steel se omposite 2 and 3 a posite sec	cts of gresse ection sectio re on etion.	constr es. For for sta on. For the ba	the pier ges 1, 2 the section the section the steel g	nds ap girden and 3 on at ross s	ply to di , on the , long ter the midd ection an	fferent main s rm effe le of th d stage	cross s pan sid cts are ne centr e 4 effe	ectio e, th on th al sp cts o	n prope e effects ne effect an, effe n the lo	rties, s are tive cts at ng-			
Stresses	at pier (main	span	side)				_						
				Bottom fla	inge	Top fla	nge	Top rel	bar	Axia A	al	Using	g steel	and
	М _у	Fx	Fz	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ (10 ⁶ mm ³)	σ	(10 ³ mm ²)	σ	crack	xed sec	tion
Stage 1	-4432	4	311	115.2	-38	102.0	43			105	0		incs.	
Stage 2	1307	-1	-77	115.2	11	102.0	-13			105	0			
Stage 3	-9374	48	1209	115.2	-81	102.0	92		~~	105	0			
Stage 4	-6944	303	936	119.7	-58	139.1	50	112.9	62	121	-3			
Shrinkage $(\gamma_{sh} = 1)$	-3414	176	-15	119.7	-29	139.1	25	112.9	30	121	-1			
	-22857	530	2364	-	-195		197	-	92	-	-4			
Stross at	CG8 (nic	r air	dor cr	·ackad sa	ction)									
Stress at		I gir	uci, ci	Bottom fla	unde	Ton fla	nae	Top rel	har	Δχίε	al	Using	g steel	and
				W	ingo	W	ngo	W	bui	A		crack	ced sec	tion
	Mу	Fx	Fz	(10 ⁶ mm ³)	σ	(10 ⁶ mm ³)	σ (10 ⁶ mm ³)	σ	(10 ³ mm ²)	σ	prope	ernes	
Stage 1	-3291	0	309	96.9	-34	85.3	39			105	0			
Stage 2	1004	0	-95	96.9	10	85.3	-12			105	0			
Stage 3	-5267	38	1083	96.9	-54	85.3	62	00.0	40	105				
Stage 4	-3825	227	828	103.1	-37	123.4	31	96.8	40	123	5 -2			
$(v_{sh} = 1)$	-3397	235	0	103.1	-33	123.4	28	96.8	35	123	-2			
(1	-14776	500	2125		-148		148		75		-4			
Stress at	mid-snar	ı (sna	n oird	er)										
	Spul	- (5 Pu	- 81 4	Bottom fla	inge	Top fla	nge	Top re	bar	Axia	al	Using	g steel	and
	M.	F	- - -	W	σ	Ŵ	σ	W	σ	A	σ	long sectio	ierm on	
Ct. 1	1019	,		$(10^6 \mathrm{mm^3})$	·	(10 ⁶ mm ³)		(10 ⁶ mm ³)	U	(10 ³ mm ²)	~	prop	erties	
Stage 1	121	(0 82	48.U	3	39.9 20.0	-3			85.3 05.3	0			
Stage 2	-584 1271	(//	ບ – ອ ເ ດ	40.U 10 0	-12	39.9 30 0	110 110			00.3 85.2	0			
Stage 3	4371 2904	40 _45	σ 2 7 0	-+0.0 56 1	52	194 9	-110	1590	_1	164	บ ว			
Shrinkage	(not	advers	. U	50.1	52	107.0	15		1.0	104	0			
$(\gamma_{sh} = 1)$	6812	_/117	_7	-	12/		_110	-	_1 0		3			
	0012	-41/	-1	-	134	-	-113	-	-1.0	<u>.</u>	3			
												1		

	Job No. BCR113	Sheet 22 of 58 Rev A
SCI	Job Title Composite highway bridges: Wo	orked examples
Silwood Park, Ascot, Berks SL5 7QN	Subject Example 2: Ladder deck three-s	pan bridge
Telephone: (01344) 636525 Fax: (01344) 636570	Section 7: Design values of the	effects of combined actions
CALCULATION SHEET	Client Made by	DCI Date July 2009
	Checked by	RJ Date Sep 2009
7.2 Effects of traffic loads	plus construction loads (ULS)	
Effects due to traffic actions an	d temperature difference are determined	from the
cracked cross section at the pie	r and the short term composite section in	midspan.
Loading for maximum hoggi	ng at pier	
The worst effects are due to gr not adverse.	l traffic loads. Effects due to temperature	e difference are
Effects at pier position		
	Bottom flange Top flange Top rebai	Axial (steel) Using cracked properties for
My Fx Fz	W W W (10 ⁶ mm³) σ (10 ⁶ mm³) σ (10 ⁶ mm³) σ	A the effects of variable actions
Construction -22857 530 236	4 – 195 197 9 2 110 7 02 120 1 80 112 0 0	2 -4 8 121 5
-33970 1128 382	7 -288 277 19	$\frac{8}{90}$ $\frac{121}{-9}$
Coexistent effects at CG8	Bottom flange Top flange Top rebai	Axial (steel)
M E E	W W W W	A (103) (ctool) properties for the effects of
Construction –14776 500 212	5 -148 148 (10°mm°) 6 (10°mm°) 7	75 -4 <i>variable actions</i>
Gr 1 traffic6034 532 140	0 103.1 -59 123.4 49 96.8	<u>62</u> 123 <u>-4</u>
-20810 1032 352	<u> </u>	37
Loading for maximum saggi	ng bending	
The maximum sagging moment	s on the composite beam occur at mid-sp	an.
	Bottom flange Top flange Top rebar	Axial (steel) Using short term composite
$M_{\rm y}$ $F_{\rm x}$ $F_{\rm z}$	(10^6 mm^3) σ (10^6 mm^3) σ (10^6 mm^3) σ	(10^3 mm^2) σ properties for the effects of
Construction 6812 –417 – Traffic gr5 8585 –1880 –6	7 134 -113 -1 9 58.5 147 880.0 -10 1112 -7	7 305 6
Temp 1257 34	0 58.5 21 880.0 -1 1112 -1	.1 305 0
difference*	6 302 -124 -10	.6 9
* $\gamma_{\rm Q}$ = 1.55 and ψ_0 = 0.6 applied to char	acteristic values	
Loading for maximum shear		
Shear at pier position		Using cracked
The value of the maximum she	ar is needed to verify the shear resistance	of the web the effects of
and to determine the longitudin	al shear on the stud connectors.	variable actions
	Bottom flange Top flange Top rebar	Axial (steel)
My Fx F	σ (10 ⁶ mm ³) σ (10 ⁶ mm ³) σ (10 ⁶ mm ³) σ	σ (10 ³ mm ²) σ
Construction –22857 529 23 Gr 5 traffic –7719 564 21	54 -192 189 8 59 123 -63 155 50 125 6	$\begin{vmatrix} 3 & -3 \\ 2 & 127 & -4 \end{vmatrix}$
-30576 1093 45	-255 239 14	

	Job No.	Job No. BCR113					23 of	58	Rev	Α
SCI	Job Title	Comp	osite hig	hway t	oridges: Wo	rked exa	mples			
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject]	Examp Section	ple 2: La n 7: Des	adder d sign val	leck three-sp lues of the e	oan bridg effects of	an bridge ffects of combined actio			
CALCULATION SHEET	Client				Made by	DCI	Date	July 2	.009	
	SCI				Checked by	RJ	Date	Sep 2	009	
7.3 Effects of traffic loads The values of effects at SLS are	plus cons	struc	tion loa fy crack	ds (S contro	LS) ol in the sla	b at the	pier.			
Effects at pier position										
	Bottom fla	ange	Top flar	nge	Top rebar	Axial (s	teel)	Using	e steel	and
M, F, A	$W_{10^6 \text{ mm}^3}$	σ	$W^{(10^6 \text{mm}^3)}$	σ (1	W 0 ⁶ mm ³) (7	A	σ	crack	ed sec	tion
Stage 1 -3476 3 25	5 115.2	-30	102.0	34	0 11111 / 0	105	0	prope	erties	
Stage 2 968 -1 -5	7 115.2	8	102.0	-9		127	0			
Stage 3 -6943 35 89	5 115.2	-60	102.0	68		127	0			
Stage 4 -5693 257 76	9 119.7	-48	139.1	41	112.9 50	121	-2			
Shrinkage -3414 175 -1	5 119.7	-29	139.1	25	112.9 30	121	-1			
Gr 1 traffic -8231 444 108	3 119.7	-69	139.1	59	112.9 73	121	-4			
-26789 913 293) -	-228		218	153		-7			
Effects at midspan The values at midspan would b resistance but, by inspection, th values. 7.4 Effects due to fatigue The range of bending effects du determined at the two locations At pier $M_y F_x F_z$ Range, lane 1 - 1803 161 22 Range, lane 2 - 1279 -7 Ratio lane 2/lane 1 moments = 0.709 At mid-span $M_y F_x F_z$ Range, lane 1 1457 - 344 -9 Range, lane 2 1017 - 260 - 13 Ratio lane 2/lane 1 moments = 0.698	e required e stresses a vehicle te to the parallel already co Bottom fla W (10 ⁶ mm ³) 5 121.5 – 121.5 – Bottom fla W (10 ⁶ mm ³) 58.5 58.5	if the at UL assage onside: ange σ 14.8 10.5 ange σ 24.9 17.4	ULS ef S are le of the f red for s Top fla W (10 ⁶ mm ³) 138.9 Top fla W (10 ⁶ mm ³) 880.0 880.0	fects e ss thar fatigue static 1 nge σ (13.0 9.2 nge σ (-1.7 -1.2	exceeded the h the elastic e vehicle in oading. Top rebar W 10 ⁶ mm ³ σ 112.9 16.0 112.9 11.3 Top of slab W 10 ⁶ mm ³) σ 1112.0 – 0.9	e elastic design each lar Axial(s A (10^3 mm ² 145.2 145.2 Axial(s A (10^3 mm ²) 305.1	te is σ σ σ σ σ σ 1.1 0.9			

				Job No. BCR113					Sheet 2	24 of	58	Rev	A
SCI				Job Title	Comp	osite hig	hway t	oridges: Wo	rked exa	mples		I	
Silwood Park, As	cot, Berks	s SL5 7	<u>а</u> м [Subject	Exam	ple 2: La	adder d	leck three-sp	oan bridg	ge			
Telephone: (0134 Fax: (01344) 636	4) 63652 570	25			Sectio	n 7: Des	sign va	lues of the e	effects of	f comb	ined ac	tions	
		т		Client				Made by	DCI	Date	July 2	2009	
OALOOLATIO		•	;	SCI				Checked by	RI	Date	Sen 2	009	
								Checked by	KJ			009	
Maximum and	minimu	ım effe	ects (for max	/min s	stresses i	in rein	forcement a	t pier)				
				Bottom f	lange	Top fla	nge	Top rebar	Axial(s	teel)			
	1.1	r	г	W 3	_	<i>W</i>	- 14	<i>W</i>	A	-			
Max offect	1VIy 1560	Г× 1/6	72 (205	(10° mm°) 121 F	0 12 Q	(10° mm°)	0 (1 11 2	1120128	(10° mm²)	<i>o</i> 1 0			
min effect	243	-14	205 -21	121.5	2.0	138.9	-17	112.9 - 2.2	145.2	0.1			
initiation of the second	210	• •	- ·	12110	2.0	100.0		112.0 2.2	110.2	0.1			
7.5 Effects	in inter	rmedia	ate c	ross gi	rders								
The worst sagg	ing und	ler trat	ffic a	ctions o	ccurs	in the ce	entral c	cross girder	•				
Worst saggin	g on ci	ross g	irder	r (ULS)									
The following	stresses	are el	astic	stresses	on the	e effecti	ve cro	ss section (a	allowing	g for			
shear lag)													
				Bottom	flange	Top fla	nge	Top of slab	Axial (steel)			
	11	r	r	<i>W</i>	_	<i>W</i>	_ //	<i>W</i>	A	_			
Stage 1	1VIy 18			(10° mm°)	σ	(10° mm°)	σ (* 6	$10^{\circ} \text{mm}^{\circ}) \sigma$	(10° mm²)				
Stage 7	40	0	0	8.27	0	8.27	0_		27.7	, 0			
Stage 2	523	2	0	8 27	63	8.27	_63		27.7	, 0			
Stage 3	272	104	0	11 96	23	111	-03	453 -0 6	27.7 5 71 5	1			
	8/3	104	0	11.00	92	••••	71	-0.6	<u>,</u> , , , , , , , , , , , , , , , , , ,	/			
Gr1 traffic	1960	_93	43	12 89	152	-369	5	302 -6 5	<u> </u>	<u>'</u>			
Gr5 traffic	2192	-78	184	12.89	170	-369	6	302 -7.3	150) 1			
Total (gr5)	3035	28	184		262		-65	-7.9	<u>-</u>	0			
_						-			-				
Values for SLS	would	be ne	eded	if the to	otal str	esses ex	ceeded	l the design	elastic	values			
but, by hispect	ion, me	y ale i		xceeueu	•								

	Job No.		Sheet 2	25 of	58	Rev	Α			
SCI	Job Title	Composite highway b	oridges: Wo	rked exa	mples					
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder d Section 8: Verificatio	eck three-sp on of bare st	oan bridg eel girde	ge er duri	ng con	struct	ion		
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	009			
	SCI		Checked by	RJ	Date	Sep 2	009			
8 Verification of bare s The two main girders are suscep	teel gi	rder during const lateral torsional buckl	ruction	he weig	ht of					
The beams are partially restrained girders. The cross girders provide	P35	6 ^[4] -1/6.3	3.2.2							
Use the expressions in Appendix C of P356 to determine the non-dimensional slenderness and thus the buckling resistance. Use gross section properties for determination of LTB slenderness, even where the section is Class 4.										
In this example, only the central of the three spans would need to deemed satisfactory by inspectio	l span is be cons on by cons	considered. In genera sidered, though the sh mparing with the large	al, the adeq orter spans er span.	uacy of would	each be					
8.1 Torsional flexibility of p In the global model, moments or end of the main span cross girde	baired n f 10 kNi ers in co	nain girders m about a longitudinal nstruction stage 3.	axis were	applied	at eac	h				
The total torque applied to each	beam is	thus:								
$10 \times 11 = 110 \text{ kNm}$										
The deflected shape determined	by the a	nalysis is shown below	w.							
The rotational displacements at t	the centre	ral gross girder, given	by the ana	lysis, w	ere	HAVERA				
1.600×10^{-4} rad		Annondin C in								
1 nus, the torsional flexibility, for $A_{\rm e} = 1.600 \times 10^{-4} / (110 \times 10^{6})$	or use in $(-1)^{44}$	Appendix C, 1S: $55 \times 10^{-12} \text{ rad/Nmm}$								
$O_{\rm R} = 1.000 \times 10^{-7} (110 \times 10^{-7})$	<i>,</i> — 1.4.	10 1au/1111111								

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SCI	Job Title	Composite highway b	oridges: Wo	rked exa	ample	es			
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder de Section 8: Verificatio	eck three-sp n of bare st	oan brid eel gird	ge er du	ırin	g cons	struct	ion
CALCULATION SHEET	Client		Made by	DCI	Date	9	July 2	009	
	SCI		Checked by	RJ	Date)	Sep 2	009	
CALCULATION SHEET 8.2 Evaluation of non-dimension of the formula	Client SCI nsional cross set $\frac{5}{200} = 7.4$ $\frac{7}{2.133} =$ $\times 0.44$ are need 0.44×10^7 $0.25 = \left[L_w^3 / [EI_z] \right]$ $1 + \frac{\pi^2}{\pi^2}$	slenderness - main ction at mid-span, the 0.44 - 1) = -0.12 ded: (1-0.44) +0.12 ² = 1 $\frac{84 \times 10^{9} \times 2.6}{\times 42000^{2}} = 1.46$ $\frac{2 \times 0.44 \times 1.46}{\times 42000^{2}} = 1.46$ $\frac{1.46}{\times 4200^{2}} = 1.46$ $\frac{1.46}{\times 4200^{2}} = 1.46$ 1	Made by Checked by girders section pro- 0 50 (using E = 0.719 .25)	DCI RJ operties	Date Date	2	July 2 Sep 20 P35 and P35	009 009 6/C.4 C.3.	4.3 2
The limiting (minimum) value of $k = 0.209$	f <i>k</i> is (1.	.7 - $0.7 V_{ m eq})L_{ m r}/L_{ m w}$							
The limiting (minimum) value of	f k is (1.	$1.7 - 0.7 V_{\rm eq})L_{\rm r}/L_{\rm w}$							
Taking $L_{\rm r} = 3500$, the limit is:		100 1 0.000	`						
$(1.7 - 0.7 \times 0.719) \times 3500/4200$	00 = 0.1	100, so use $k = 0.209$	Ĵ						

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SCI	Job Title	Composite highway b	ridges: Wo	rked exa	mples						
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder de Section 8: Verification	eck three-sp n of bare st	an bridg eel girde	ge er durii	ng cons	structi	ion			
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	009				
	501		Checked by	RJ	Date	Sep 2)09				
Assume $1/\sqrt{C_1} = 1.0$ (uniform $U = 1.0$ (welded section)	moment	- conservative assum	ption)			P356/C.4.2 and C.4.3					
$V = \left\{ \left[4a(1-a) + 0.05\lambda_{\rm F}^2 + \psi_{\rm a}^2 \right]^{0.5} + \psi_{\rm a} \right\}^{-0.5},$											
$= \left\{ \left[4 \times 0.4 (1 - 0.44) + 0 \right] \right\} \right\}$	0.05×7.4	$43^2 + 0.12^2 \Big]^{0,5} - 0.$	$12 \Big\}^{-0,5} =$	0.741							
Take D 1.2 (destabilising loads)											
$\lambda_{\rm z} = \frac{kL_{\rm w}}{i_{\rm z}} = \frac{0.209 \times 42000}{212.1} = 41.4$											
$\lambda_1 \qquad = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{345}}$	W _y from	and <i>M</i> n She	$I_{\rm pl}$ eets								
$\beta_{\rm w} = \frac{W_{\rm y}}{W_{\rm pl,y}} = \frac{3.970 \times 10^7}{15110 \times 10^6/345} = 0.906$ (effective section modulus $W_{\rm y}$)											
Thus: $\overline{\lambda}_{\rm LT} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w} =$	1×1×0.7	$41 \times 1.2 \times \frac{41.4}{77.5} \sqrt{0.906}$	$\bar{6} = 0.45$								
Slenderness derived from but Alternatively and less conservation buckling analysis of the structure $\overline{\lambda}_{LT}$ would be given by $\overline{\lambda}_{LT} = \sqrt{2}$	ckling an vely, slen e at the base $\frac{W_y f_y}{M_{cr}}$	nalysis nderness could be der are steel girder stage	rived from and then th	an elast he value	ic e of						
where $M_{\rm cr}$ is given by the analys	sis.										
8.3 Reduction factor											
Since $h/b < 2$, use buckling cur	eve c, α_{LT}	= 0.49				3-2/	6.3.2 1/ 6 3	2.2			
$\phi_{\rm LT} = 0.5 \left[1 + \alpha_{\rm LT} \left(\overline{\lambda}_{\rm LT} - 0.2 \right) + \right]$	$\overline{\lambda}_{LT}^2 =$	0.5[1 + 0.49(0.45 - 0	$(.2) + 0.45^2$] = 0.	66	3-1- NA	1/ .2.16				
Hence $\chi_{\rm LT} = 1 / \left(\phi_{\rm LT} + \sqrt{\phi_{\rm LT}^2 - \overline{\lambda}_{\rm LT}^2} \right) =$	1/(0.66+	$+\sqrt{0.66^2 - 0.45^2} =$	0.875								
8.4 Verification											
$M_{\rm b,Rd} = \frac{\chi W_{\rm el} f_{\rm y}}{\gamma_{\rm M1}} = \frac{0.875 \times 3.97}{1}$	$70 \times 10^7 \times 3$	$\frac{345}{2} \times 10^{-6} = 10900$	kNm			3-1-	1/6.3	3.2.1			
$M_{\rm Ed}$ = 121 - 584 + 4371 =	3908 kN1	m (Sheet 17) $< M_{\rm b,l}$	$_{\rm Rd} = 10900$) kNm -	OK						

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SCI	Job Title	Composite highway	bridges: Wo	rked exa	amples			
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder of Section 8: Verification	leck three-spon of bare st	oan bridg eel girde	ge er duri	ng con	structi	ion
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	.009	
	301		Checked by	RJ	Date	Sep 2	009	
8.5 LTB of cross girders The adequacy of the LTB buckli the main beams, also needs to b cross girders, unless they need t Geometrical parameters: $L_{\rm w} = 11700$ mm and for the $I_{\rm z,c} = 5.625 \times 10^7$ $I_{\rm z,t} = 5.625 \times 10^7$	ing resis e verifie o be pai central o	tance of the cross gir d. There are no inter red together at their 1 cross section, the sect	ders, spann mediate res nid-span. ion propert	ing betw traints to ies are:	veen o the			
$I_{\rm T} = 4.077 \times 10^{6}$ $i_{z} = 63.7 \text{ mm}$ h = 896 mm (at middle of 0) $t_{\rm f} = 25 \text{ mm}$ $d_{\rm f} = 871 \text{ mm}$ $\lambda_{\rm F} = \frac{L_{\rm w}}{i_{z}} \cdot \frac{t_{\rm f}}{h} = \frac{11700}{63.7} \cdot \frac{2}{89}$ $a = \frac{I_{z,c}}{I_{z,c} + I_{z,t}} = 0.5 \text{ (equ}$ $\psi_{a} = 0.8(2a - 1) = 0.8 \times (2)$	CG) $\frac{5}{96} = 5.12$ al flange $2 \times 0.5 \rightarrow 0.5$	2 es) $-1) = 0$				P35 and	6/C.4 C.3.2	4.3 2
For an unrestrained beam, $k =$	1.0							
Assume $1/\sqrt{C_1} = 1.0$ (uniform	momen	t - conservative assum	nption)					
U = 1.0 (welded section)								
$V = \begin{cases} [4a(1-a) + 0.05\lambda_{\rm F}^2] \\ = \begin{cases} [4 \times 0.5(1-0.5) + 0.05\lambda_{\rm F}^2] \end{cases} \end{cases}$	$+\psi_a^2 \Big]^{0,2}$ 05×5.2	$\left\{ 5 + \psi_{a} \right\}^{-0.5}$ $\left\{ 12^{2} + 0 \right\}^{0.5} - 0 = 0$	= 0.811					
Take D 1.2 (destabilising lo	ads)					P35	6/C.1	1
$\lambda_{z} = \frac{kL_{w}}{i_{z}} = \frac{1.0 \times 11700}{63.7}$ $\lambda_{1} = \pi \left[\frac{E}{E} = \pi_{1} \sqrt{\frac{210000}{210000}} \right]$	= 183	.7						
$\beta_{\rm w} = \frac{\sqrt{f_{\rm y}}}{W_{\rm pl,y}} = 8.273 \times 10^6$	9 / (3207	$7 \times 10^{6}/345) = 0.890$				W _y fror She	and <i>M</i> n et 14	$I_{ m pl}$
Thus: $\overline{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_w} =$	1×1×0.	$811 \times 1.2 \times \frac{183.7}{77.5} \sqrt{0.89}$	90 = 2.18					

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SCI	Job Title	Composite highwa	y bridges: Wo	orked exa	amples			
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder Section 8: Verifica	deck three-s tion of bare s	pan bridg teel gird	ge er durir	ig con	structi	ion
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	009	
	SCI		Checked by	RJ	Date	Sep 2	009	
8.6 Reduction factor Since $h/b > 2$, use buckling cur $\phi_{LT} = 0.5 \Big[1 + \alpha_{LT} (\overline{\lambda}_{LT} - 0.2) + $	rve d, α $\overline{\lambda}_{\rm LT}^2$ =	$f_{\rm LT} = 0.76$ = $0.5 [1 + 0.76 (2.18 - 10.10)]$	- 0.2) + 2.18	$\left[\frac{2}{2}\right] = 3.$	63	3-2/ 3-1- 3-1- NA	6.3.2 1/ 6.3 ·1/ .2.16	2.2 3.2.2
Hence $\chi_{\rm LT} = 1 / \left(\phi_{\rm LT} + \sqrt{\phi_{\rm LT}^2 - \overline{\lambda}_{\rm LT}^2} \right) =$	= 1/(3.0	$63 + \sqrt{3.63^2 - 2.18^2}$	$(\overline{2}) = 0.153$					
8.7 Verification								
$M_{\rm b,Rd} = \frac{\chi W_{\rm el} f_{\rm y}}{\gamma_{\rm M1}} = \frac{0.153 \times 8.27}{1}$	73×10 ⁶ ×	$\times 345 \times 10^{-6} = 397$	kNm			3-1-	1/6.3	3.2.1
$M_{\rm Ed} = 48 + 523 = 571 \rm kNm$	(Sheet	19) > $M_{\rm b,Rd}$ = 397	kNm - Not	satisfact	ory			
Consider pairing the cross girde is then classed as a beam with a	rs togetl central	ner with channel bra torsional restraint.	ncing at their	mid-spa	n. This			
8.8 Verification for paired of	ross gi	rders						
From global analysis, it is determined restraint is $\theta_{\rm R} = 4.73 \times 10^{-11}$ ratio	mined th ad/Nmm	hat the torsional flex	ibility of the	central				
Using the same values of $\lambda_{\rm F}$ and	l <i>a</i> as be	fore, the value of V	_{eq} is needed	to calcul	late k.			
For a bi-symmetric section, the	value of	f $V_{\rm eq}$ may be taken a	is equal to V_{i}			P35	6/C.4	4.6
Thus:								
$V_{\rm eq} = 0.811$								
Then the stiffness parameter V_{eq}	${}^{4}L_{\rm w}{}^{3}/[EI_{\rm c}]$	$d_{z,c}\theta_{\rm R} d_{\rm f}^{2}(1-a) = 3270$) and thus k	= 0.494				
Consider whether the cross girde indicated by the value of the 'ar: Figure C.1 in P356)	er would row' on	d then buckle into to the appropriate cen	vo half wave tral restraint	s, which curve (s	i is see			
The value of k at the position of	the arro	ow is given by:				P35	6/C.4	4.5
$k = \left[\frac{1+\pi^2\alpha}{4(1+4\pi^2\alpha)}\right]^{0.25}$								
Where $\alpha = \frac{V_{eq}^4}{\pi^2 \left(1 - V_{eq}^4\right)}$ for V_{eq}	≤0.999							
$\alpha = \frac{0.811^4}{\pi^2 (1 - 0.811^4)} = 0.07725 \text{ an}$	d thus <i>i</i>	$k = \left[\frac{1 + \pi^2 \times 0.077}{4(1 + 4\pi^2 \times 0.077)}\right]$	$\left[\frac{25}{725}\right]^{0.25} = 0$).574				

							_		
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SCI	Job Title	Composite highway bridges: Worked examples							
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Example 2: Ladder deck three-span bridge Section 8: Verification of bare steel girder during construction							ion	
CALCULATION SHEET	Client		Made by	DCI	July 2	July 2009			
	501		Checked by	RJ	Date	Sep 2	009		
Thus, since the value of k given by the stiffness parameter and the central restraint curve is less than the value of k at the position of the arrow on that curve, it is to the right of the arrow and the cross girder will buckle in two half waves. (<i>Note: Detailed evaluation of the expressions in P356 for this case show that the arrow is at a stiffness parameter of about 570.</i>)									
Curve for central t 1.00 0.90 0.80 0.70 0.60 0.50 0.40 0 500 1000	orsional	2000 2500							
For two half waves (node at $\lambda_{\rm F} = 2.56$ (half the previous $V = \left\{ \left[4 \times 0.5(1 - 0.5) + 0. \right] \\ \lambda_z = \frac{L_{\rm w}}{i_z} = \frac{5850}{63.7} = 91.8 \right] \\ \overline{\lambda}_{\rm LT} = \frac{1}{\sqrt{C_1}} UVD \frac{\lambda_z}{\lambda_1} \sqrt{\beta_{\rm w}} = 0.5 \left[1 + \alpha_{\rm LT} \left(\overline{\lambda}_{\rm LT} - 0.2 \right) \right] \\ Hence$ $\chi_{\rm LT} = \frac{1}{\sqrt{\left(\phi_{\rm LT} + \sqrt{\phi_{\rm LT}^2 - \overline{\lambda}_{\rm LT}^2} \right)}}{M_{\rm b,Rd}} = \frac{\chi W_{\rm el} f_{\rm y}}{\gamma_{\rm M1}} = \frac{0.36 \times 8.273 \times 3}{1.1}$	central value) a 05×2.5 $1 \times 1 \times 0.9$ $1 + \overline{\lambda}_{LT}^2$ = 1/(1) $10^6 \times 345$	restraint): and thus $56^{2} + 0 \Big]^{0,5} - 0 \Big\}^{-0,5} =$ $932 \times 1.2 \times \frac{91.8}{77.5} \sqrt{0.890}$ $= 0.5 \Big[1 + 0.76 (1.25 - 1.25^{2}) \Big]$ $1.68 + \sqrt{1.68^{2} - 1.25^{2}} \Big]$ $5 \times 10^{-6} = 934$ kNm >	$= 0.932$ $= 1.25$ $0.2) + 1.25^{2}$ $= 0.36$ $M_{Ed} = 5^{2}$] = 1. 71 kNm	68 OK	P35	6/C.:	3.2	
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SCI	Job Title	Composite hig	ghway b	oridges: Wo	rked exa	mples			
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: La Section 9: Ver	adder d rificatio	eck three-sp n of compo	oan bridg site gird	ge er			
CALCULATION SHEET	Client			Made by	DCI	Date	July 2	2009	
	SCI			Checked by	RJ	Date	Sep 2	009	
9 Verification of compo	osite a	irder							
9.1 In hogging bending with									
The elastic design bending resist design effects at the stages.									
The bare steel section is Class 3									
The effects (stresses) in the cross section properties for effects on composite section. (<i>This avoids a</i> <i>design situation, based on the zet</i>									
From Sections 7.1 and 7.2, the the total moment is 33970 kNm, (cracked) section is 21471 kNm.	design n which The str	noment on the means that the resses are as sh	steel se momer own be	ction is 124 at on the co clow.	499 kNr mposite	n and			
190	N/mm ²	-9 N/mm _							
122 N/mm ² 155 108 N/mm ² 180 N/m steel compo (bendi	$\begin{array}{c c} & & & & & & \\ 122 \text{ N/mm}^2 & 155 \text{ N/mm}^2 & - & \\ & & & & & \\ & & & & & \\ & & & &$								
The primary effects of shrinkage	e do not	need to be incl	luded.				4-2/	6.2.1	.5(5)
Resistance of cross section For verification of cross section stresses f_{yd} and f_{sd} . For this verification: $f_{yd} = f_{y}/\gamma_{M0} = 335/1.0 =$ $f_{sd} = f_{yk}/\gamma_{s} = 500/1.15 =$ By inspection, the stresses in bo The reinforcement should also b The design of the slab is not cov available. There is a margin between sufficient for inclusion of local e	resistan = 335 N = 435 N th are O e checke vered in ween glo effects.	ce, the stresses /mm ² for the 6 /mm ² for the re K ed for the comb this example a obal stresses an	s should 0 mm l einforce bined g nd loca ad stress	l not exceed bottom flan ement lobal and lo l design str s limits that	d the lin ge ocal effe esses ar t should	ects. e not be	4-2/	6.2.1	.5

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SCI	Job Title	Composite hig	ghway b	ridges:	Worke	d exa	mple	S				
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: L Section 9: Ve	adder der der der de rification	eck thre	e-span nposite	bridg gird	ge er					
CALCULATION SHEET	Client			Made by	, D	CI	Date	July	2009			
	SCI			Checked	dby R	J	Date	Sep	2009			
Buckling resistance For verification of buckling resists section (on which $M_{b,Rd}$ is based	stance in) has to 1	bending, the be determined	design i using:	resistan	ce of t	he cr	OSS					
$\mathcal{M}_{el,Rd} = \mathcal{M}_{a,Ed} + \mathcal{M}_{c,Ed}$ Where k is a factor such that a stress limit is reached												
In this case the bottom flange w	the bottom flange will reach its limit first and the limit is											
$f_{1} = f/\alpha_{1} = 335/1.1 = 305$ N	$1/mm^2$ (w	reach its limit first and the limit is: pm^2 (y_{re} is used since buckling is being considered)										
$J_{yd} = J_{y}/\gamma_{M1} = 333/1.1 = 303$ N	ses only:	M1 IS USED SILL		ing is t	eing c	UIISIC	lereu	,				
$M_{\rm el,Rd} = 12499 + \frac{(305 - 108)}{180} \times$	21471 =	only: 471 = 35990 kNm										
To evaluate $M_{b,Rd}$, determine the	slender	ness										
The slenderness of the length of beam in the hogging region could be evaluated considering the LTB of a composite section comprising the effective width of slab and the steel girder but it is much simpler and a little less conservative to use the simplified method of EN 1993-2, as recommended by EN 1994-2.												
Consider the lateral buckling of and one third of the depth of the compression as that under total of	an effect part of effects, i	tive Tee sectio the web in con ncluding axial	n comp npressi force.	rising t on. Tal	he both the the o	om f lepth	lange in	e 3-2	2/6.3.4	4.2		
Flange area is $800 \times 60 \text{ mm}$												
Height to zero stress:					Stress	Mid	-heigł	nt				
$(297/(297 + 268) \times (2175 - 3))$	30) + 30	= 1158 mm	Top fla	inge	268	2	175					
Height of web in compression	= 1098	mm	Bottom	n flange	297		30					
Area of Tee = $800 \times 60 + (109)$	$98 \times 20)$	/3 = 55320 m	m^2									
Lateral 2^{nd} moment of area = 80	$00^3 \times 60$	$/12 = 2560 \times$	$10^{6}{\rm mr}$	n ⁴								
Radius of gyration = $\sqrt{2560 \times 10^{-5}}$	0 ⁶ /55320	$D = 215 \mathrm{mm}$										
Initially, assume that the first cr flange, through U-frame action.	oss girde	er provides eff	ective la	ateral r	estrain	t to tl	he					
For a buckling length of 3500 m	ım (supp	ort to first cro	ss girde	er, CG8	3):					•		
$N_{\rm E} = \pi^2 \frac{EI}{L^2} = \pi^2 \frac{210000 \times 2560 \times 10^6}{3500^2} \times 10^{-3} = 433100 \text{ kN}$									2/(6.1	2)		
The lateral restraint is sufficient	ly stiff if	its stiffness C	C _d satisf	ies:								
$C_{\rm d} > \frac{4N_{\rm E}}{L}$								3-2	2/6.3.4 13)	4.2		

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SCI	Job Title Composite highwa	ay bridges: Wo	orked exa	amples								
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Example 2: Ladde Section 9: Verific	er deck three-spation of compo	pan bridg osite gird	ge er								
CALCULATION SHEET	Client	Made by	DCI	Date .	July 2	009						
		Checked by	RJ	Date	Sep 2009							
The stiffness of the U-frame is g applied at the two bottom flange expressed as:	given by consideration of e es. Following the guidance	qual and oppo in Table D.3,	osite unit this ma	forces y be	3-2/	D.2.4	4					
$\frac{1}{C} = \frac{h_v^3}{3EI_v} + \frac{h^2 b_q}{2EI_q}$		b_{q}	-									
At the first cross girder:												
$b_{\rm q} = 11700, h_{\rm v} = 1114 { m mm}$ $I_{\rm q} = 1.02 \times 10^{10} { m (short-term}$ $I_{\rm v} = 1.89 \times 10^8 { m (web plus flux)}$	h = 1979 mm, (to mean 1) a section, average value ov lat stiffener)	evel of short- er tapered cro	term NA ss girdei	x) :)								
Hence $C_{\rm d} = 210000 / \left(\frac{1114^3}{3 \times 1.89 \times 10^8} + \frac{1979^2 \times 11700}{2 \times 1.02 \times 10^{10}} \right) = 44800 \text{ N/mm}$												
The required $C_{\rm d}$ is:												
$\frac{4 \times 433100}{3500} \times 10^3 = 495000 \mathrm{N/mn}$	n											
Therefore the frame is not stiff	enough to be considered as	rigid										
Now consider the stiffness of the Since the half wavelength has do	e second frame, for buckli oubled,	ng over a leng	gth of 2	panels.								
$N_{\rm E} = 433100/4 = 108300$ kN a	and required $C_{\rm d}$ is then 4 ×	108300/7.0 =	= 61900	kN/m								
For this frame, $h = 1729$ mm, h	$h_{\rm v} = 864 \text{ mm}$											
$C_d = 210000 / \left(\frac{864^3}{3 \times 1.89 \times 10^8} + \right)$	$\frac{1729^2 \times 11700}{2 \times 1.02 \times 10^{10}} = 73600 \text{ N}$	l/mm (≡ kN/n	n)									
Which is sufficient to be conside	ered as rigid											
The verification may be carried	out using											
$\overline{\lambda}_{ m LT} = \sqrt{rac{A_{ m eff}f_{ m y}}{N_{ m crit}}}$					3-2/	(6.10))					
Where $N_{\rm crit} = m N_{\rm E}$												
$N_{\rm E}$ is the elastic critical buckling force and m is a parameter that non-uniform axial force. Expres M_2/M_1 and V_2/V_1 , as well as on	g load for the equivalent co allows for intermediate lat ssions are given in 6.3.4.2(the intermediate spring stil	lumn under u eral spring res 7) depending fness.	niform a straints a on the ra	xial nd for atio								
In this case, the girder tapers over $determine the value of m but the not overly conservative for this$	ver the buckling length and e limiting (minimum) value situation.	it would be c of 1.0 may b	omplex e used a	to ind is								

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CALCULATION SHEET	Client SCI	Made by	DCI	Date	July 2	2009					
		Checked by	RJ	Date	Sep 2	009					
Thus											
$\overline{\lambda}_{\rm LT} = \sqrt{\frac{A_{\rm eff}f_{\rm y}}{N_{\rm crit}}} = \sqrt{\frac{55320 \times 335}{108300 \times 10^{5}}}$	$\frac{5}{3} = 0.414$										
Since $h/b < 2$, use buckling curves	3-2/6.3.2.2										
$\phi_{\rm LT} = 0.5 \left[1 + \alpha_{\rm LT} \left(\overline{\lambda}_{\rm LT} - 0.2 \right) + \overline{\lambda}_{\rm LT}^2 \right] = 0.5 \left[1 + 0.49 (0.414 - 0.2) + 0.414^2 \right] = 0.638$											
Hence											
$\chi_{\rm LT} = 1 / \left(\phi_{\rm LT} + \sqrt{\phi_{\rm LT}^2 - \overline{\lambda}_{\rm LT}^2} \right) =$											
$M_{\rm b,Rd} = \chi M_{\rm el,Rd} = 0.890 \times 35990 = 32030 \rm kNm$											
For verifying the contribution of the same Tee section (and thus t	f axial resistance in the ir he same slenderness and	teraction criter reduction facto	ion, con r).	sider							
No effective section for axial for the effective Tee that would buck the slenderness with this amount Tee for bending. The same area	rce is given in EN 1993-2 kle laterally should comp t of web is very little diffe is used here for both cas	but it could be rise half the are rent from that o es.	argued a of the of the efj	that web: fective							
$N_{\rm b,Rd} = \chi A_{\rm Tee} f_{\rm yd} = 0.890$	$0 \times 55440 \times 305 = 150501$	xNm									
$N_{\rm Ed} = A_{\rm Tee} \times stress = 55320 >$	$49 = 498 \mathrm{kN}$										
This verification of resistance to largest moment given by $0.25L_k$	buckling may be carried (where $L_{\rm K} = L/\sqrt{m}$)	out at a distant	ce from	the	3-2 6.3	/ .4.2(7	7)				
Here, consider moment at 0.25 interpreted linearly between the linearly between values at the su	× 7000 from the support. values at the two ends (7 upport and first cross gire	Conservatively 000 mm apart) er.	this can or in th	n be is case							
At the support, $M_{\rm Ed} =$ At the first cross girder M Hence $M_{\rm Ed} =$	33970 kNm $M_{\rm Ed} = 20810$ kNm 27390 kNm at $0.25L_{\rm k}$										
The section is subject to combin be assumed since the buckling n	ed bending and axial for node is the same for both	e and a linear i	interaction	on wil							
In the M/N interaction verification reduction over the buckling lenge	In the M/N interaction verification, use $N_{\rm Ed} = 498$ kN (on the effective column) without reduction over the buckling length.										
The interaction relationship is th	ius:										
$\frac{M_{\rm Ed}}{M_{\rm b,Rd}} + \frac{N_{\rm Ed}}{N_{\rm b,Rd}} = \frac{27390}{32030} + \frac{498}{1505}$	$\frac{1}{0} = 0.855 + 0.033 = 0.88$	8 OK									
The buckling resistance is satisfa	actory.										
Interaction with shear must also	be considered (using cro	ss section resist	ances).								

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9.2 Maximum shear at sup	port	intermediate support	= 4533 kN	r		Sheet 22						
However, the bottom flange is in		01 22										
The total stress in the bottom fla												
Hence vertical component = (26)												
$V_{\rm Ed} = 4533 - 1197 = 3336$												
Assume that no transverse web s girders (3500 mm spacing).												
The web panel adjacent to the su end of the panel.	apport is	s tapered; base the slee	nderness or	the dee	eper							
$a_{w} = 3500 \text{ mm}$ $h_{w} = 2090 \text{ mm}$ t = 20 mm $f_{y} = 345 \text{ N/mm}^{2}$												
The factor $\eta = 1.0$ according to	the NA	L .				3-1-	3-1-5/NA.2.					
From equation (5.6):						3-1-	-5/5.3	3				
$\overline{\lambda}_{\rm w} = \frac{h_{\rm w}}{37.4t\varepsilon\sqrt{k_{\rm t}}}$ where $\varepsilon = \sqrt{23}$	$\overline{35/f_y} =$	$\sqrt{235/345} = 0.83$										
Since $a_{\rm w} > h_{\rm w}$ and there are no	longitud	linal stiffeners:				3-1-	-5/A.	3				
$k_{\rm t} = 5.34 + 4.0(h_{\rm w}/a)^2 = 5.34 +$	4.0(209	$(90/3500)^2 = 6.77$										
$\overline{\lambda}_{w} = \frac{2090}{37.4 \times 20 \times 0.83 \sqrt{6.77}} = 1$.294											
Since the girder is continuous, c	onsider	as a rigid endpost cas	e. Thus, fr	om Tabl	le 5.1:							
$\chi_w = 1.37/(0.7 + \overline{\lambda}_w) = 1.37/1.994$	= 0.687											
$V_{\rm bw,Rd} = \frac{\chi_{\rm w} f_{\rm yw} h_{\rm w} t}{\sqrt{3}\gamma_{\rm M1}} = \frac{0.687 \times 34}{\sqrt{3}}$	$\frac{5 \times 2090}{8 \times 1.1}$	$\times 20 \times 10^{-3} = 5200 \text{ kN}$				3-1-	-5/5.2	2				
This resistance is adequate, ever	n withou	t a contribution from	the flanges.									
Using the same web slenderness 4813 kN. The force in the comp flange force is also less, $V_{\text{Ed,net}}$ =	, the she pression = 2865 l	ear resistance at the sh flange is less at that p kN (calculations not sh	allow end osition and hown here)	of the p althoug . This is	anel is the the SOK.							
9.3 Combined bending and	shear											
The shear coexisting with maximinclined flange of 1325 kN, givi	num mo ng a net	ment is 3827 kN less value of $V_{\rm Ed} = 2502$	a contribut kN	ion fron	n the							

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$\overline{\eta}_3 = \frac{V_{\rm Ed}}{V_{\rm bw,Rd}} = \frac{2502}{5200} = 0.481$						3-1-	-5/7.1	l					
So interaction does not need to b	be consid	dered for this design	situation.	$\overline{\eta}_3 < 0.5$)								
For the maximum shear design s $N_{\rm Ed} = 1092$ kN) the net shear is	situation $V_{\rm Ed,net}$ =	$(M_{\rm Ed} = 30576 \text{ kNm})$ = 3336 kN and thus	$V_{\rm Ed} = 453$	33 kN,		She	et 22						
$\overline{\eta}_3 = \frac{V_{\rm Ed}}{V_{\rm bw, Rd}} = \frac{3336}{5200} = 0.642$													
So shear-moment interaction doe	es need t	to be considered. ($\bar{\eta}$	$\frac{1}{3} > 0.5$)										
For interaction, consider the val value must take account of axial	eraction does need to be considered. $(\eta_3 > 0.5)$ ider the value of $M_{\rm f,Rd}$. For this parameter, $\gamma_{\rm M0}$ applies and the unt of axial force.												
value must take account of axial force. The area of the bottom flange is smaller than that of the top plus the rebars and its compressive resistance is 16080 kN. Deduct half the compressive force (conservative, since that flange is smaller) and multiply by the lever arm between the two flanges (take $d = 2220$ mm). Thus $M_{f,Rd} = (16080 - 1092/2) \times 2.22 = 34500$ kNm													
Since $M_{\rm Ed} < M_{\rm f,Rd}$ the interaction combined effects are satisfactory	n criteri ′.	on of 3-1-5/7.1 does	s not apply a	ind the									
As noted in Example 1, it is sug from accumulated stress, rather shown here but the value would	gested in than as be less i	the sum of the momentation $M_{ m f,Rd}$, the sum of the momentation $M_{ m f,Rd}$,	_{Ed} should be ents. That co	determi alculatio	ned n is no	t							
9.4 In sagging bending													
The composite cross section is C can be utilised.	Class 1 (pna in the top flange	e) so the plas	stic resis	stance		. 10						
The plastic bending resistance of the total design value of bending 2263 kN. The presence of tensil covered by EN 1994-2; in this c mid thickness of the top flange, flange and there is negligible eff is satisfactory by inspection.	f the sho g effects e axial f ase, who the axia fect on the	ort term composite s is 16654 kNm, with force on the plastic b ere the pna of the co l force only moves the plastic bending re	ection is 225 an axial ter pending resis pomposite sector he pna withit esistance. Th	19 kNm asile ford tance is tion is a in the to e cross	a and ce of not t the p section	She	et 10 et 22						
It can also be seen that the stress in stages, are satisfactory, as fol	ses calcu lows:	es calculated elastically, taking account of construction ows:											



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Area provided = $2 \times 314 \times 367$	8/150 =	15400 mm ² - Satisf	actory					
Crack control Requirements relate only to the longitudinal stresses in the reinfor Global stresses due to permanen rebars. The tensile stress includi	She	et 23						
are: $\sigma_{\rm s} = \sigma_{\rm s,0} + \frac{0.4 f_{\rm ctm}}{\alpha_{\rm st} \rho_{\rm s}}$								
For this calculation, the followin $A_s = 129800, I_s = 1.160 \times 1000$ $A = 145200, I = 1.390 \times 1000$	She and	ets 9 11						
$A_{\rm ct} = 3678 \times 250 = 919500$								
$\alpha_{\rm st} = \frac{AI}{A_{\rm a}I_{\rm a}} = \frac{145200 \times 1390}{129800 \times 1160}$	= 1.340							
$\rho_{\rm s} = A_{\rm s}/A_{\rm ct} = 15400/91950$	00 = 0.016	67 (i.e. 1.67%)						
$\sigma_{\rm s} = 77 + \frac{0.4 \times 3.5}{1.340 \times 0.0167} =$	140 N/mr	m ²					/Tabl	. 7 2
From Table 7.2, maximum bar	spacing =	= 300 mm > 150 m	m provided	- OK		4-2)	1 auto	- 1.2
Limiting stresses at SLS Reinforcement stresses at SLS, i sections should also be considered than the design values at the pier satisfactory at SLS, by inspectio	ncluding t ed. The el r and at m n.	the effects of tension astic summations of hidspan and here it i	n stiffening stresses at s judged th	in crack ULS ar at stress	ked e less es are			
9.6 Verification of cross gir	ders							
As noted in Section 7.5 the stress loads considered in the FE analy prevent buckling of the slab whe Section 6.3.2, the cross girders of the main girders exceeds 30 t example.	esses in the resis. However ere it is in are requir imes the s	cross girders are we ever, these transvers compression. As no ed to perform this f slab thickness, which	within elastic se girders a boted in P35 function whe h is the case	e limits, re requir 6, en the sp e in this	for th red to pacing	e		
The cross girders need to be bot function in addition to the resista	h stiff end ance to the	ough and strong eno e effects already cal	ugh to perf	orm this	•			
There are no explicitly worded Eurocode clauses for such design of composite girders but the following evaluation is based on guidance in Hendy & Murphy. The cross girder is considered as a transverse stiffener on the deck slab plate.								

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The stiffness requirement may b	e expres	sed as:				D6.6-22					
$\delta + \Delta \le \frac{b}{300}$											
Where \triangle is the first order deflect deflection due to the longitudina	tion due l compre	to transverse loads a ession in the slab	nd δ is the	second	order						
From the analysis $w_{Ed} = 13.5$ n traffic loads. Strictly, Δ should to those either side of it, but ver normally have similar loading of deflection of the most heavily lo	nm at the be the ob- ry consen n adjacen baded cro	e centre of the middle oserved relative deflect rvatively (since the de nt cross girders) it cou oss girder. i.e. $\Delta = w$	cross girde tion of one sign situati 1ld be taken Ed	er under cross g on woul as the	gr5 jirder d						
For the case where there is no a reduces to:	xial forc	e in the cross girder,	the express	sion for	δ	Bas D6.	ed on 6-21	l			
$\delta = w_0' \left(\frac{\sigma_{\rm m} b^4}{\pi^4 E I_{\rm st} - \sigma_{\rm m} b^4} \right)$											
In which:											
$w_0' = w_0 + w_{\rm Ed}$											
w_0 is the initial imperfection, we (Essential tolerance D.1.6(5), we	th $L =$	be taken as $L/400$ ac 2×3500 and thus w_0	cording to $= 17.5 \text{ m}$	EN 109 m)	0-2,						
Hence $w'_0 = 17.5 + 13.5 = 31 \text{ mm}$	n										
The value of σ_m depends on the Section 7.2 the maximum stress effects) and the level of zero str The effective width of slab actin in the slab is:	longitud at the to ess is ap g with e	inal force in the slab. op of the slab is 9.1 N proximately at the sla each main girder is 39	From the 1/mm ² (bend b top flang 75 mm and	results i ding plu e interfa thus th	n s axia ice. e forc	1 e					
$N_{\rm Ed} = (9.1/2) \times 7950 \times 250 \times 10^{-10}$	$e^{-3} = 904$	40 kN									
Note: if plastic bending resistan been much greater.	ce had b	een utilised, the force	in the slat	would	have						
Assuming that the ratio of columnity (conservative) and with eq	nn-like b ual spac	uckling stress to plate	e-like buckl	ing stres	ss is	Bas D6.	ed on 6-5	l			
$\sigma_{\rm m} = \frac{2N_{\rm Ed}}{ab} = \frac{2 \times 9040 \times 10^3}{3500 \times 11700} =$	= 0.44 N/	'mm ²									
Thus $\delta = 31 \times \left(\frac{0}{\pi^4 \times 210000 \times 10^{-5}}\right)$	$.44 \times 117$ $.16 \times 10$	$\left(\frac{700^4}{10} - 0.44 \times 11700^4\right) =$	= 1.08 mm								
$\delta + \Delta = 1.08 + 13.5 = 14.58$	mm. wh	ich is less than $b/300$	(39 mm) -	OK							
$w_0' = w_0 + w_{\rm Ed}$											

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The strength of the cross girders destabilising effect of the slab ap	also ne oplies an	eds to be checked for additional moment of	this effect. f:	The		Base	ed on						
$(w'_0 + \delta) \frac{\sigma_{\rm m} b^2}{\pi^2} = (31 + 1.08) \times \frac{0}{2}$	D6.	6-20	L										
This results in an additional tens													
196×10^6 / $12.89 \times 10^6 = 15$ N	She	et 24											
This gives a total stress of 262 -	This gives a total stress of $262 + 15 = 277 \text{ N/mm}^2 - \text{OK}$.												
Note that, although the cross gir against buckling, verification of since the slenderness a/t (=3500 effects can be ignored (see 2-1-1 in this example.	ders are the slab 250 = 250	stiff enough to act as still needs to consider 14) may exceed the 1 Detailed design of th	r restraints r second or imit below e deck slab	to the sl der effe which s is not o	lab ects, such covered								

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 10 Longitudinal shear The resistance to longitudinal sh connectors and the transverse re <i>intermediate values would also b connectors</i>.) Since the composite beam mode from the 3D FE model does not depend on both the cross section the section) and on the variation transferred from the portions of 	lear is ver inforcemo be verified l for which include to properti of axial slab not	rified for the the the the the the the full with the full with the force along the	the web/fla pier and at nise the pro- tots and force idth of slab ling (applie ng the beam n the beam	t mid- ovisio ces ha o, the ed to n, due	veld, t span. <i>n of s</i> ve bee shear the she e to th on.	the sheat (<i>In prachear</i>) en extrac flow wi ear force e shear	r ctice ill e or	e, 1			
10.1 Effects for maximum sl	hear										
ULS values at pier											
			Shear		Axial	force					
			force Pier		CG8						
Shear on steel section (stages 1-3)	hear on steel section (stages 1-3)					38					
Shear on long-term composite sect	tion (crack	(ed)	936	30)3	227					
Shear on short-term composite sec	ction (crac	ked)	2169	56	63	351					
ULS values at midspan (CG1	3)										
			Shear		Axial	force					
			force	CG	13	CG12			Valu	ies fr	om
Shear on steel section (stages 1-3))		-7	4	0	40		Sect	$rac{10}{7}$	7.1	
Shear on long-term composite sect	tion		0	57	-407			anu	1.2		
Shear on short-term composite sec	ction		755		$\Delta F_{x} =$	530					
SLS values (Only values for composite section Shear on long-term composite section	on noted)			er 39	Spa	n (CG13 0)				
Shear on short-term composite sec	ction (wor	st effects)	16	12		559					
10.2 Section properties To determine shear flows elastic and stage.	cally, the	parameter	A_z/I_y is	need	ed for	each se	ectio	on			
For composite sections, uncrack used to determine shear flow.	ed unrein	torced co	mposite se	ction	prope	rties can	ı be	•	4-2/0	5.6.2	.1(2)

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Telephone: (01344) 636 Fax: (01344) 636570	525		Section 10:	: Longitudi	inal sh	ear						
CALCULATION SHE	ET	Clier	nt		Made	by	DCI	Date	July 2	2009		
					Check	ed by	RJ	Date	Sep 2	2009		
	50	otion	at plar	Saar	tion at	midar						
	Web/flang	ge	Flange/slab	Web/flai	nge	Flan	ge/slab					
	Az/I_y (m ⁻	⁻¹)	$A\bar{z}/I_{y}$ (m ⁻¹)	A_{z}^{-}/I_{y} (n	n ⁻¹)	Az/I	y (m ⁻¹)					
Steel section	0.392		0.803									
Long term section	0.443		0.276	0.797	7	0	.633					
Short term section	0.464		0.388	0.790)	0	.753					
Note that the use of t	he use of the $A\overline{z}/I_y$ parameter applied to the shear on the cross section is											
conservative at the pier because the inclined bottom flange also provides a component of the shear resistance (but, since the flange force reduces away from the support, it would not be appropriate to apply the parameter to the net shear carried by the web).												
would not be appropriate to apply the parameter to the net shear carried by the web). If the plastic resistance had been utilized, the shear flow would need to have been be determined by consideration of the forces in the slab, taking account of the non-linear resistance in accordance with 4-2/6.2.1.4.												
Additionally, the axial force in the section varies along the girder and this must be accompanied by a shear flow.												
10.3 Shear flow at	t ULS											
Force at web/flange	junction											
At pier 14	443 × 0392	+ 9	$936 \times 0.443 + 2$	2169×0.4	464 =	1867	kN/m					
Near mid-span	7×0	0.803	$3 + 0 \times 0.797 +$	-755×0.7	94 =	591 k	N/m					
Force at flange/slab j	junction											
At pier			$936 \times 0.276 + 2$	2169×0.3	888 =	= 1100	kN/m					
Near mid-span			0×0.633 +	755×0.7	753 =	569	kN/m					
At the pier, the diffe	erence in ax	tial f	orce between pi	ier and cro	oss gir	der is	:					
866 - 578 = 288 kl	N											
This is equivalent to	a shear flow	w of	288/3.5 = 82 1	kN/m								
The shear due to this ratio of steel area / c	variation c racked area	of ax (cra	ial force at the acked area used	slab/girder to be cons	r inter sistent	face d t with	epends FE moo	on th del):	e			
Shear flow $= A_s/A$	× 82 = 129	800/	/145200 × 82 =	= 73 kN/m	l							
The shear at the web	/flange jund	ction	depends on the	area of w	veb plu	us bott	tom flar	nge:				
Shear flow = $A_{w+f}/A \times 82 = 51 \text{ kN/m}$												
At midspan, the diff	erence in a	xial	force between C	CG12 and	CG13	is:						
50 kN long term and 530 kN short term	1											
This is equivalent to	a shear flow	WS O	f 50/3.5 = 14 k	xN/m and	530/3	.5 =	151 kN	/m				

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Section 10:	Ladder d Longitudi	eck three-sp inal shear	oan bridg	ge						
CALCULATION SHEET	Client			Made by	DCI	Date	July 2	:009				
	501			Checked by	RJ	Date	Sep 2	009				
The shear at the slab/girder interareas:	rface de	pends on the	ratio of s	steel area to	o compo	osite						
Shear flow = $85320/164300 \times 14$ = 7 kN/m (long) = $85320/305100 \times 151$ = 42 kN/m (short)												
The shear at the web/flange junction depends on the area of web plus bottom flange:												
Shear flow = $53320/164300 \times 14$ = 5 kN/m (long) = $53320/305100 \times 151$ = 26 kN/m (short)												
The total shear flows are thus:												
At the pier: slab/flange shear = At midspan: slab/flange shear =	= 1173] = 618 k	kN/m, web N/m, web	o/flange s o/flange s	hear = 191 $hear = 622$	l8 kN/m 2 kN/m	1						
10.4 Shear flow at SLS												
Force at flange/slab junction												
	1 (10	=	1									
At pier 769×0.276 At mid-span 0×0.633	+ 1612 × + 559 ×	$0.385 \ 837$ 0.753 = 42	kN/m 1 kN/m									
The shear flow at SLS is require	ed for ve	erification of	the shear	· connectors	5							
These values are 76% and 74% the different partial factors at UI apply to the shear flow due to variable.	respecti LS and S ariation	vely of the U SLS. It can b of axial force	ULS value be assume e.	s and the r d that simil	atio refl lar ratio	ects s						
10.5 Web/flange welds												
Design weld resistance given by	the sim	plified metho	od of EN	1993-1-8,	4.5.3.3	is:						
$F_{\rm w,Rd} = f_{\rm vw,d}a$ where $f_{\rm vw,d} = \frac{f_{l}}{f_{\rm vw,d}}$	$\frac{\sqrt{3}}{8\gamma_{M2}}$											
For 6 mm throat fillet weld (8.4	mm leg	(a = a)	6 mm									
For web and flange grade 355 in	n thickne	ess range 3 -	100 mm,	$f_{\rm u} = 470$) N/mm	2	EN	1002.	5-2			
From Table 3-1-8/4.1 $\beta = 0.9$												
Thus $F_{w,Rd} = \frac{6 \times 470/\sqrt{3}}{0.9 \times 1.25} = 144$	47 N/mn	n (kN/m)										
Resistance of 2 welds = 2890 k	N/m >	1918 kN/m	shear flow	w in pier gi	rder - C	ЭK						
10.6 Shear connectors												
Stud shear connectors 19 mm di assumed, with $f_u = 450 \text{ N/mm}^2$	ameter 1	50 mm long	(type SE	01 to EN IS	SO 1391	8) are						

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject Example 2: Ladder Section 10: Longitu	deck three-spudinal shear	pan brid	ge						
CALCULATION SHEET	Client	Made by	DCI	Date	July 2	2009				
	SCI	Checked by	RJ	Date	Sep 2	Sep 2009				
The resistance of a single stud is	s given by $4-2/6.6.3.1$ as the	e lesser of:		•						
$P_{\rm Rd} = \frac{0.8 \times f_{\rm u} \times \pi \times d^2 / 4}{\gamma_{\rm V}}$					6.6. Eq (3.1(1) (6.18))			
$P_{\rm Rd} = \frac{0.29 \times \alpha \times d^2 \sqrt{f_{\rm ck} \times E_{\rm c}}}{\gamma_{\rm v}}$					Eq ((6.19))			
$\alpha = 1.0 \text{ as } \frac{h_{\text{sc}}}{d} = \frac{150}{19} > 4$					Eq ((6.21))			
$P_{\rm Rd} = \frac{0.8 \times 450 \times \pi \times (19^2 / 4)}{1.25}$	$\frac{0}{2} \times 10^{-3} = 81.7 \text{ kN}$				Eq ((6.18))			
$P_{\rm Rd} = \frac{0.29 \times 1.0 \times 19^2 \times \sqrt{40}}{1.25}$	$\times 35 \times 10^3 \times 10^{-3} = 99.1 \text{ kN}$	ſ			Eq ((6.19))			
Therefore the design resistance of	of a single headed shear con	nector is								
$P_{\rm Rd}$ = 81.7 kN										
If studs are grouped and spaced reinforcement), then a row of 3	at 150 mm spacing along th studs has a design resistance	e beam (to s e of:	uit trans	sverse						
$F_{\rm Rd} = 81.7 \times 3 / 0.150 = 163$	34 kN/m									
This is adequate at the pier (F_{RG})	$_{\rm i} = 1634 > F_{\rm Ed} = 1173 \text{ kN/s}$	m)								
Alternatively, rows of 5 studs at	300 mm spacing would pro	wide 1362 k	N/m.							
The requirement at mid-span is a spacing or 2 studs at 150 mm sp	about 53% that at the suppopacing would be adequate	rt and thus 3	studs a	t 300						
Consideration of fatigue resistan	ce of the shear connection is	s covered in	Section	11.3.						
Resistance at SLS At SLS the shear connector resistence of the shear con	stance is limited to $k_{\rm s} P_{\rm Rd}$ wit	$h k_{s} = 0.75$			4-2/	NA.2	.11			
The SLS shear flows are 76% or requirement is thus only slightly inspection.	f the ULS value at the pier a more onerous and the prov	and 75% at 1 ision is satist	nid-spar factory l	n. The by						
10.7 Transverse reinforceme	nt									
Consider the transverse reinforce 5 studs at 300 mm spacing, i.e.	ement required to transfer th 1362 kN/m.	ne full shear	resistan	ce of						
For a critical shear plane around dotted above) the shear resistance	the studs (type b-b in 4-2/Fi e is provided by twice the are	gure 6.15 an ea of the both	d shown tom bars	n chain S.	4-2/	6.6.6	.1			

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder Section 10: Longit	r deck three-sj udinal shear	oan brid	ge								
CALCULATION SHEET	Client		Made by	DCI	Date	July 2009							
	SCI		Checked by	RJ	Date	Sep 2	.009						
The shear force to be resisted is	s given h	$x 4_2/(6.21) \approx 136$	$\frac{1}{2}/\cot \theta$										
Take $\cot \theta = 1$ hence required	resistanc	$y = -\frac{1362 \text{ kN/m}}{1362 \text{ kN/m}}$	12/0010			2-1-	1/6.2	.4					
Assume B20 bars at 150 mm sr	acing.	c = 1302 km/m											
$\begin{array}{c} \text{Assume B20 bars at 150 mm sp} \\ \text{Bosistence} = 4 \text{ fr} / \text{s} = (2 \times 2) \end{array}$	$14) \times (5)$	$00/1$ 15) /150 $\times 10^{-10}$	⁻³ - 1921 I-N	[/m		2-1-	1/6.2	.4					
The transverse bars are adequat	(30)	0/1.13) /130 × 10	-1021 Kiv	/ 111									
The underside of the boods of the	C.	mood to be at least	10 mm chave	the tree			665	4					
bars. In this case an overall stud	d height	of 150 mm should	be sufficient.	the trai	isverse	4-2/	0.0.3	.4					
	U												

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder d Section 11: Fatigue a	eck three-sp ssessment	oan bridg	ge						
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	009				
	SCI		Checked by	RJ	Date	Sep 2	009				
11 Fatigue assessment	al staal	details									
The design value of the stress ra	nge in s	tructural steel is:				3-2/	9.4.1	and			
$\gamma_{\text{Ff}} \Delta \sigma_{\text{E2}} = \gamma_{\text{Ff}} \lambda \phi_2 \Delta \sigma_{\text{p}}$ where $\phi_2 = 1.0$ and γ_{Ff} is given by	by the N	A as 1.0				9.5. 3-2/	1 NA.2	.35			
The value of $\lambda = \lambda_1 \ \lambda_2 \ \lambda_3 \ \lambda_4$ (b)	out not n	nore than λ_{\max})									
For intermediate supports where more than 30 m $\lambda_1 = (1.70 + 0.5)$	L, the $(L-30)$	ength of the critical in $)/50)$	nfluence lin	e (in m) is						
Here, $L = (24.5 + 42)/2 = 33.25$ m and thus $\lambda_1 = 1.73$											
For span regions, $\lambda_1 = (2.55 - 0.7 \times (L - 10)/70)$ and here $L = 42$ m and thus $\lambda_1 = 2.23$											
The value of λ_2 is given by $\lambda_2 = \left(\frac{Q_{m1}}{Q_0}\right) \times \left(\frac{N_{Obs}}{N_0}\right)^{1/5}$											
Where $Q_0 = 480$ kN and $N_0 = 0$	0.5×10^{-10}) ⁶									
From 3-2/NA.2.39, $Q_{m1} = 260$	kN										
From 1-2/Table NA.4, $N_{\text{Obs}} = 1$	10^{6}										
Hence $\lambda_2 = \left(\frac{260}{480}\right) \times \left(\frac{1.0}{0.5}\right)^{0.2} = 0.0$.62										
For a 120 year design life the va	alue of λ	l ₃ given by 3-2/Table	9.2 is 1.03	7:							
The value of λ_4 depends on the passage of FLM3 in the second	relative and	magnitude of the stres	ss range du	e to the							
$\lambda_4 = \left(1 + \frac{\text{effect in lane 2}}{\text{effect in lane 1}}\right)^{0.2}$											
Design stress ranges at pier											
At the pier, the stress range $\Delta \sigma_{\rm p}$	in top a	and bottom flanges (at	their mid	thicknes	s) is:	Shee	rt 23				
Bottom flange:15.9 N/mm²Top flange:11.9 N/mm²											
The ratio of lane 2/lane 1 effects	s = 0.70	09 and thus $\lambda_4 = 1.13$	3								
$\lambda = 1.73 \times 0.62 \times 1.037 \times 1.13 = 1$	1.242										
For support regions where $L >$	$30 \text{ m} \lambda$	$\max = 1.80 + 0.90 \times (L - 1.80)$	- 30)/ 50								
Thus, for $L = 33.25$, $\lambda_{max} = 1.8$	6										
Hence $\lambda = 1.242$											

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525	Subject	Example 2: Ladder d Section 11: Fatigue a	eck three-sp ssessment	oan bridg	ge				
Fax: (01344) 636570	Client		Made by	DCI	Date	July 2	2000		
CALCULATION SHEET	SCI		Checked by		Date	Son 2	009		
			oncekeu by	10	Dute		007		
The design stress ranges are thu	s:								
Bottom flange: $1.0 \times 1.242 \times$ Top flange: $1.0 \times 1.242 \times$	(15.9 = 11.9 =	20 N/mm ² 15 N/mm ²							
The worst detail category that might apply is for a bearing plate welded to the underside of the bottom flange, which, for a flange plate over 50 mm thick, is category 36 (3-1-9/Table 8.5, detail 6).									
Design value of fatigue strength									
Design stress ranges in mid-span At mid-span, there is negligible stress range in the top flange. The range in the bottom flange is 26.0 N/mm ² . The ratio of lane 2/lane 1 effects = 0.698 and thus $\lambda_4 = 1.112$									
$\lambda = 2.23 \times 0.62 \times 1.03 / \times 1.112 = 1$.599								
The design stress range is thus:	a ()	10 NI (2							
Bottom flange: $1.0 \times 1.599 \times$	26.0 =	42 N/mm ²				3-1-9/			
The most onerous detail would be 80 and the fatigue strength = 80 welded to the bottom flange.	be a trans $\frac{1}{1.1} = 7$	Sverse web stiffener, 73 N/mm ² . This is O	which is de K even for	stiffene	egory rs	Tab	e 8.4	ł	
11.2 Assessment of reinforc	ing stee	I							
The design value of the stress ratio $\gamma_{F,fat}$ is given by Table NA.1	nge in re as $\gamma_{F,fat} =$	einforcing steel is $\gamma_{\rm F,fa}$ = 1.0	_{at} ⊿σ _{S,equ} wh	ere the	value	2-1/	6.8.5		
$\Delta \sigma_{S,equ}$ is referred to in EN 1994	l-2 as ⊿σ	$r_{\rm E}$, given by:				4-2/	6.8.6	.1	
$\Delta \sigma_{\rm E} = \lambda \phi \left \sigma_{\rm max,f} - \sigma_{\rm min,f} \right $									
The value of $\lambda = \lambda_s$									
and $\lambda_{s} = \phi_{fat} \lambda_{s,1} \lambda_{s,2} \lambda_{s,3} \lambda_{s,4}$						2-2/	NN.2	.1	
Where ϕ_{fat} is a damage equivalent	nt impact	factor							
The value ϕ effectively duplicates	s ϕ_{fat} but	since $\phi = 1.0$, this is	not signifi	cant		4-2/	6.8.6	.1	
The value of the stress range du (in regions of intermediate suppo need to be increased for the effe	e to FLM orts) in a oct of tens	13 needs to be increat coordance with NN.2 sion stiffening in accordance.	sed by a fa 2.1(101). St ordance wit	ctor of f resses a h 4-2/7.	1.75 Iso .4.3	2-2/	NN.2	2.1	
Based on cracked section proper actions is 77 N/mm ² (see SLS va	ties, the alues in S	stress in the top reba Section 7.3).	rs due to p	ermanen	it	EN Ann	1991- ex B	-2	
The maximum tensile stress due (see Section 7.4) and this is incr 22.4 N/mm ² . Thus, ignoring ten	to the Fl reased by sion stiff	LM3 fatigue vehicle the 1.75 factor, givi ening, $\sigma_{max,f} = 99$ N	in lane 1 is ng a value /mm ² .	11.5 N. of	/mm ²				
The minimum stress (compressive) due to the FLM3 fatigue vehicle in lane 1 is 2.1 N/mm ² (see Section 7.4) and this is increased by the 1.75 factor, giving a value of 3.6 N/mm ² . Thus, ignoring tension stiffening, $\sigma_{\min,f} = 73$ N/mm ² .									

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder of Section 11: Fatigue a	leck three-sp assessment	oan bridg	ge					
CALCULATION SHEET	Client		Made by	DCI	Dat	е	July 2	009		
	SCI		Checked by	RJ	Dat	e	Sep 20)09		
To determine the effect of tension determined on Sheet 38:	on stiffer	ning, the following pa	arameters w	ere			4-2/7.4.3			
$\rho_{\rm s} = 0.0167$ $\alpha_{\rm st} = 1.34$ $f_{\rm ctm} = 3.5 \text{ MPa (for C40/50 c}$ $\Delta \sigma_{\rm s} = \frac{0.2 f_{\rm ctm}}{\alpha_{\rm st} \rho_{\rm s}} = \frac{0.2 \times 3.5}{1.34 \times 0.0167}$	= 31 N/s	mm ²					2-1-1 See 4 for 0.	/Table -2/6.8 2 fact	e 3.1 3.5.4 or	
Thus, the maximum and minimu	im stress	ses including tension	stiffening a	e:						
$\sigma_{s,\max,f} = \sigma_{\max,f} + \Delta \sigma_s = 99 + 31 = 130 \text{ N/mm}^2$ And $\sigma_{s,\min,f} = \sigma_{s,\max,f} \frac{M_{E,d,\min,f}}{M_{E,d,\max,f}}$.4	
Using the ratio of stresses, rathe	er than d	irectly using moment	s:							
$\sigma_{\rm s,min,f} = 130 \times \frac{73}{99} = 96 \mathrm{N/mm^2}$										
For intermediate support region	and span	n of 33.25 m, $\lambda_{s,1}$ =	0.988				2-2/1	Figur	e	
(The curve can be expressed app	proximat	ely, for $L < 50 m$ by	the express	ion			NN.	1		
$0.3(L/100)^2 + 0.25(L/100) + 0.8$	872.)									
For $N_{\rm Obs} = 1 \times 10^6$, medium dis	stance tra	affic and straight bars	$s(k_2 = 9):$	$\bar{Q} = 0.9$	94 ar	nd	2-2/7	Fable		
$\lambda_{s,2} = \overline{Q} \sqrt[k_2]{\frac{N_{\text{Obs}}}{2.0}} = 0.94 \sqrt[9]{\frac{1.0}{2.0}} =$	0.870							1		
For 120 year design life:										
$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{\text{Years}}}{100}} = \sqrt[9]{\frac{120}{100}} = 1.020$)									
For 2 slow lanes:										
$\lambda_{s,4} = k_2 \sqrt{\frac{\sum N_{\text{Obs},i}}{N_{\text{Obs},1}}} = \sqrt[9]{\frac{2.0}{1.0}} = 1.08$	80									
For road surface of good rought	ness $\phi_{\rm fat}$	= 1.2					1-2/2	Anne	хB	
Thus $\lambda = 1.2 \times 0.988 \times 0.87 \times 1.02$	×1.08 =	1.136								
$ \Delta \sigma_{\rm E} = 1.136 \times 1.0 \times 130 - 96 =$	1.136×	$34 = 39 \text{ N/mm}^2$								
$\gamma_{\rm F,fat} \Delta \sigma_{\rm S,equ} = 1.0 \times 39 = 39 \text{ N/}$	mm ²									
$\frac{\Delta \sigma_{\rm Rsk}}{\gamma_{\rm s,fat}} = \frac{162.5}{1.15} = 141 \text{ N/mm}^2 >$	> 39 mm	² OK					2-1-1	1/6.8	.5	

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CALCULATION SHEET	Client		Made by	DCI	Date	July 2	:009		
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Note: The above verification con is appropriate for regions close also be considered and then con a ladder deck the worst local efj effects are less. Design of the de	nsiders o to the m ubined g fects are eck slab	nly global stresses in pain girder. Fatigue un lobal and local effects midway between main is outside the scope of	the reinford nder local l s must be ev n girders, v f this exam	cement, oading s valuated vhere gl ole.	which should ; with obal	1			
11.3 Assessment of shear c	onnecti	on							
The design value of the stress ra	inge in s	shear studs is given as	$\gamma_{\rm Ff} \Delta \tau_{\rm ,E2}$ wh	nere		4-2/	6.8.7	.2	
$\Delta \tau_{\mathrm{E},2} = \lambda_{\mathrm{v}} \Delta \tau$									
In which $\Delta \tau$ is the range of shea	3-2/	NA.2	.35						
EN 1994-2 refers to EN 1993-2 for the value of $\gamma_{\rm Ff}$, which is given as 1.0									
The value of $\lambda_{v} = \lambda_{v,1} \lambda_{v,2} \lambda_{v,3}$	$\lambda_{\mathrm{v},4}$								
Since the span is less than 100 r	n, $\lambda_{\mathrm{v},1}$ =	= 1.55							
The values of λ_2 , λ_3 and λ_4 are but with an exponent of 1/8 rath	calculate er than	ed in the same manner 1/5	r as for stru	ictural s	teel				
Hence $\lambda_2 = \left(\frac{260}{480}\right) \times \left(\frac{1.0}{0.5}\right)^{0.125} =$	0.591								
$\lambda_3 = \left(\frac{120}{100}\right)^{0.125} = 1.023$									
The value of λ_4 depends on the passage of FLM3 in the second	relative lane and	magnitude of the stres l is given by:	ss range du	e to the					
$\lambda_4 = \left(1 + \frac{\text{effect in lane 2}}{\text{effect in lane 1}}\right)^{0.125}$									
Shear at pier									
The range of vertical shear force effects is $241/423 = 0.570$. The	e at the generation	pier is 423 kN and the on of axial force over	e ratio of la 3.5 m is 4	ine 2/lar l kN.	ne 1	Shee	rt 19		
As noted earlier, the axial force in the cross section is consequent upon both the separate modelling of the edge beam and of the unequal loading on the two main girders (the FLM3 is in lane 1). These cause longitudinal shears along both edges of the portion of slab acting with the steel girder; some of that shear has to be transferred to the steel beam (pro rata to area) and thus needs to be included in the design shear flow on the studs.									
At the pier, the studs are 19 mm 300 mm spacing)	n diamet	er, in rows of 3 at 15	0 mm spac	ing (or 6	5 at				

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Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder Section 11: Fatigue	deck three-sj assessment	pan bridg	ge						
CALCULATION SHEET	Client		Made by	DCI	Date	July 2	2009				
	501		Checked by	RJ	Date	Sep 2	009				
$\bar{Az/l_y} = 0.388 \text{ m}^{-1}$ hence shear	flow du	e to bending/shear =	= 423 × 0.38	38 = 16	4 kN						
Shear flow due to axial force =	41/3.5	$\times A_{\rm s}/A = 10 \ \rm kN/m$									
(cracked properties used, to be c	consister	nt with analysis mode	el)								
Total shear flow = $164 + 10 =$	= 174 kN	N/m									
The stress range per stud = 174	× 0.15	$0/(3 \times \pi d^2/4) = 30.$	6 N/mm ²								
$\lambda_4 = (1 + 0.570)^{0.125} = 1.058$											
$\lambda_{\rm v} = 1.55 \times 0.591 \times 1.023 \times 1.058 =$	0.991										
$\Delta \tau_{\rm E,2} = 0.991 \times 30.6 = 30 \mathrm{N/mm}$	\mathbf{n}^2										
The reference value of fatigue st	rength f	For a shear stud is Δa	$t_{\rm c} = 90$			4-2/6.8.3					
The partial factor on fatigue stre	angth γ_{Mi}	f = 1.1.				3-1- NA					
The design strength is thus $90/1.1 = 81 \text{ N/mm}^2 > 30.6 \text{ N/mm}^2 \text{ OK}$											
Additionally, since the flange is steel flange must be verified, us	in tensio ing:	on, the interaction w	ith normal s	tress in	the	4-2/	6.8.7	.2			
$\frac{\gamma_{\rm Ff} \Delta \sigma_{\rm E,2}}{\Delta \sigma_{\rm c} / \gamma_{\rm Mf}} + \frac{\gamma_{\rm Ff} \Delta \tau_{\rm E,2}}{\Delta \tau_{\rm c} / \gamma_{\rm Mf,s}} \le 1.3$											
With $\Delta \sigma_{\rm c} = 80$.											
Coexistent stresses should be use onerous values for each of $\Delta\sigma_c$ a	ed but cound $\Delta \tau_{\rm c}$	onservatively one car	n consider th	ne most							
$\frac{1.0 \times 15}{80/1.1} + \frac{1.0 \times 30}{90/1.1} = 0.57 \text{OK}$											
Shear at mid-span The range of vertical shear force effects is $143/219 = 0.653$. The	e at mid variatio	-span is 219 kN and on of axial force is 1	the ratio of 44 kN over	lane 2/la 3.5 m	ane 1	Shee	et 19				
$\bar{Az}/l_y = 0.753 \text{ m}^{-1}$ hence shear	flow due	e to bending/shear =	$= 219 \times 0.75$	3 = 16	5 kN						
Shear flow due to axial force =	144/3.5	$5 \times A_{\rm s}/A = 12 \ \rm kN$									
Total shear flow = $165 + 12 =$	= 177 kN	J/m									
At mid-span, if the 19 mm stud	s are in	rows of 3 at 300 mi	n spacing								
Stress range = $177 \times 0.300 / (3)$	× 284)	$= 62 \text{ N/mm}^2$									
$\lambda_4 = (1 + 0.653)^{0.125} = 1.065$											
$\lambda_{\rm v} = 1.55 \times 0.591 \times 1.023 \times 1.065$	5 = 0.99	8									
$\Delta \tau_{\rm E,2} = 0.998 \times 62 = 62 \mathrm{N/mm^2}$	<81 N/i	mm ² OK									

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SCI	Job Title	Composite highway b	oridges: Wo	rked exa	mples			
Silwood Park, Ascot, Berks SL5 7ΩN Telephone: (01344) 636525 Fax: (01344) 636570	Subject	Example 2: Ladder de Section 12: Cross gir	eck three-sp der to main	oan bridg girder c	ge connec	tion		
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12 Cross girder to main	girder	connection	t to the inte	ermedia	te			
support.	indens a	initiaspun und udjucon		ermoura				
12.1 Structural arrangement	of con	nection						
The structural arrangement of th a main girder is shown below. O stiffener on the main girder; the	e conneo Only the flanges	ction between an inter web of the cross gird are not connected.	mediate cro er is conne	oss girde cted to t	er and the			
		Depth of main gin 1200 mm at r 1894 mm at 0 1644 mm at 0 Depth of cross gi Web stiffener: M24 grade 8.8 b	rder: midspan CG8 CG9 irder: 750 300 × 30 olts in 26 m	mm mm m holes				
12.2 Design basis The cross girders behave essenti the main girders but there are sr Until the concrete slab has harde corresponding to the self weight approximately equal for the cross CG9, because of the lateral restric centroid of the bolt group this sr due to the end shear times the di- bolt group alone has to resist the should be designed not to slip at magnitudes of the effects are rel	ally as s nall end of the c s girders caint pro nall hog stance f e combin this stag atively r	imply supported beam moments due to perm ere is very little end m antilever. The magnit s at midspan and a litt vided by the deep pies ging moment is partly rom the main girder w ation of shear and mo ge but this is easily ac nodest.	ns, spanning nanent and noment, apa ude of this the greater a r crosshead y offset by the web. At this poment. The chieved bec	g betwee variable art from momen at CG8 a l. At the the sagg s stage, connect ause the	en loads that t is and ing the tion			
Once the slab has been cast, the A small hogging moment will an cantilever. Under traffic loading small and either hogging or sagg adjacent cross girders and warpi CG8 adjacent to the intermediate to the proximity of the deep cross	connect ise due , end mo ging (the ng stiffn e suppor sshead g	ion will act composite to the self weight of s oments at the midspan moment arises due to ess of the main girden t are larger and predo irder at the support. T	ely with the surfacing or a cross gird o unequal lo rs). End mo minantly he The values a	e deck sl n the er will b oading o oments o ogging, at CG9	ab. De on on due are			

between those for CG8 and for the midspan cross girders.

For the actions on the composite structure, it is assumed that vertical shear at the connection is resisted by steel web of the cross girder and moments are resisted by a composite section comprising a width of slab and the web of the cross girder.

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It is assumed that the vertical shear is shared equally by all the bolts and that the moment (at line of the centroid of the bolt group) is determined from an elastic stress distribution on an effective Tee section comprising slab and beam web. The horizontal force in the lowest part of the web due to the moment at the middle of the connection is calculated and the force is shared between the bolts in the bottom row.									
The bolted connections provide adjacent to the intermediate supp the effects on the connection mu the level of the main girder bott but it is better to use the same c intermediate cross girders.	restraint port and st incluc om flang onnectio	to the main gird are therefore de le an allowance ge. In midspan ro n detail and desi	ler ag signe for a egion ign ba	ainst LTB d for no sli lateral restr s, slip could asis for all t	in region p at Ul raint fo d be to the	ons LS and orce at lerated			
Strictly, the forces in the bolts due to bending moments should be derived by considering the three stages of bare steel, long-term composite Tee and short-term composite Tee, and adding the results from all three stages. However, since the connection is designed to be slip resistant at ULS and the effects at the bare steel stage are modest it is considered adequate to design the connection as a slip resistant connection for the total effects, applied to a short-term composite effective section.									
12.3 Design situations									
From a review of a range of dif most onerous design situation for vehicle) arranged such that the e are greatest were found to have design of the connection.	ferent lo r the con and shear a lesser	ading arrangeme mection is with is greatest. Situ end shear and th	ents, i gr5 lo lation lus to	t was concluding (the back where the be less one	luded f SV100 e end n erous f	hat the) noment or the	S		
The total ULS effects due to dea	id loads	are.					Effe	cts	
At the middle cross girder (CG At the end cross girder (CG8): At the second cross girder (CG9) (Moments are on the line of the	13): $M = 21$ M = 21 9) $M = 1$ main gin	= 85 kNm (hogg 2 kNm (hogging 128 kNm (hoggi rder web)	ging) g) and ing) a	and $V = 30$ V = 295 1 and $V = 30$)4 kN KN 8 kN		Shee	et 20	on
For CG13, the effects due to gr M = 184 kNm (hogging) and	5 traffic d $V = 8$	load at ULS are 59 kN	e:						
For CG8, the effects due to gr5 M = 447 kNm (hogging) and	traffic 1 d $V = 9$	oad at ULS are: 33 kN							
For CG9, the effects due to gr5 M = 276 kNm (hogging) and	traffic 1 d $V = 8$	oad at ULS are: 79 kN							

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12.4 Design resistance of in	dividual	bolt connections						
The connection is made with M	24 grade	8.8 preloaded bolts i	n 26 mm h	oles.				
Cross girder web thickness = Main girder stiffener thickness =	= 15 mm = 30 mm	n $(\sigma_y = 355 \text{ N/mm}^2)$ n $(\sigma_y = 345 \text{ N/mm}^2)$						
Slip resistance						3-1-	8/3.9	.1
$F_{\rm s,Rd} = \frac{k_{\rm s} n \mu}{\gamma_{\rm M2}} F_{\rm p,C}$						(3.6))	
$F_{p,C} = 0.7f_{ub}A_s = 0.7 \times 800 \times 353 = 198 \text{ kN}$ $\mu = 0.5 \text{ (class A friction surface)}$ $k_s = 1.0 \text{ (normal clearance holes)}$ $F_{s,Rd} = \frac{1.0 \times 198 \times 1 \times 0.5}{1.25} = 79.2 \text{ kN}$								
As the connection will be design determined by an 'elastic' distrib	ned again bution of	nst slip at ULS, the fo f stresses in the web o	orces in the of the cross	bolts m girder	ust be			
For information, the following c included. It shows that the resist This higher value could be used needed but, as explained earlier on all cross girders.	alculatio tance in where r , the sar	on of bearing/shear re bearing/shear is grea estraint of the main g ne connection detail w	rsistance at ter than ag irders agai vould norm	ULS is ainst slij nst LTB ally be i	p. is not used			
ULS shear resistance of a single	e bolt							
Resistance = $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$						3-1- Tabl	8/ le 3.4	Ļ
$A = A_s = 353 \text{ mm}^2$ $f_{ub} = 800 \text{ N/mm}^2$ $\alpha_v = 0.6 \text{ for grade 8.8 bolts}$						bolt 3-1-	stren 8/3.1	gth: .1
Resistance = $\frac{0.6 \times 353 \times 800}{1.25} \times 10^{-10}$	$0^{-3} = 136$	5 kN						
ULS bearing resistance of a sing	gle bolt o	on the cross girder we	eb					
Resistance = $F_{b,Rd} = \frac{k_1 \alpha_b f_u dt}{\gamma_{M2}}$	kN					3-1- Tabl	8/ .e 3.4	Ļ
$k_1 = \min\left(2.8\frac{e_2}{d_0} - 1.7; 2.5)\right)$	for edge	e bolts						
Here, the force under combined use the lesser 'edge' distance	shear a	nd bending is not perp	bendicular t	to an edg	ge, so			
$k_1 = \min\left(2.8\frac{45}{26} - 1.7; 2.5\right)$	= 2.5							

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$k_1 = \min\left(1.4\frac{p_2}{d_0} - 1.7; 2.5)\right)$	for inner	r bolts	mondioulou	to the 1		c			
bolts, so use the lesser spacing	i snear a	and bending is not per	pendicular	to the h	ines of				
$k_1 = \min\left(1.4\frac{70}{26} - 1.7; 2.5)\right)$	= 2.07								
$\alpha_{\rm b}$ is the smallest of $\alpha_{\rm d}$, $f_{\rm ub}/f_{\rm u}$ a	nd 1.0	1							
$\alpha_{\rm d} = \frac{e_1}{3d_0}$ for end bolts and $\alpha_{\rm d}$	$=\frac{p_1}{3d_0}-$	$-\frac{1}{4}$ for inner bolts							
$\alpha_{\rm d} = \frac{45}{3 \times 26} = 0.58$ for end bolt	s and $\alpha_{\rm d}$	$=\frac{70}{3\times 26}-\frac{1}{4}=0.65$	for inner bo	olts					
The bearing resistances in different locations in the splice will be different. Consider the bottom corner bolt as a end-edge bolt) although the component due to shear is away from the end, not toward it)									
$F_{\rm b, Rd} = \frac{2.5 \times 0.58 \times 355 \times 24 \times 11}{1.25}$	$\frac{5}{-} = 148$	kN							
So the ULS bearing/shear resista and this value must be used for	ance is d all the b	etermined by the sheat olts in the group.	ar capacity	of the b	olt	3-1-	8/3.7		
12.5 Design forces on bolt g	roup								
Midspan cross girder (CG13) Moment at centroid of bolt grou	р								
M = -(85 + 184) + (304 + 160) Shear = (304 + 859) = 1163 k	+ 859) > N	$\times (100 + 70) \times 10^{-3} =$	= — 71 kN	m (hogg	ging)				
Cross girder adjacent to pier	(CG8)								
Moment at centroid of bolt grou	р								
M = -(212 + 447) + (295) Shear = (295 + 933) = 1228 k	+ 933) N	$\times (100 + 70) \times 10^{-3}$	= -4501	xNm (ho	ogging				
Second cross girder from pie This cross girder provides the ef- assumed in determining the buck addition to moments from analys in the bottom flange over the bra Flange force = $297 \times 48000 \times$ Lateral force = 143 kN	r (CG9) ffective l cling res sis, inclu aced leng $10^{-3} =$	ateral restraint to the istance in Section 9.1 ide an allowance of 1 gth (here $L_k = \ell$). 14300 kN	bottom flar . For this c % of the m	nge that ross gir aximum	is der, in force	3-2/0	6.3.4	.2(5)	
Moment depends on height from	flange	to CG of the effective	Tee sectio	on (see b	elow)				

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Moment at centroid o M = -(128 + 27) Shear = (308 + 879) 12.6 Effective sect Number of vertical Number of horizontal	f bolt group 76) + (308 + a) = 1187 kN ion at conne icolumns' of the rows of bolts	879) × (100 +70) > ection bolts: 3 3: 8	$x 10^{-3} = -202 $ k	Nm (ho	ogging)			
Total number of bolts	$= 3 \times$	8 = 24						
Vertical spacing: Horizontal spacing:	80 mm 70 mm							
To determine forces i slab and the web of th web and the presence connected).	n the bolts, co ne cross girde of the flanges	onsider the stresses i r. Ignore the effect o s of the main girder	n a Tee section c of the notch at the and cross girder	omprisin top of (they ar	ng the the e not			
		(notch ignored)						
There rules in EN 199 cover this situation, w girder, rather than co- width of slab equal to	94-2 for deter where there is ntinuity. For to the width of	mining the effective an end moment from the design of the con the main girder top	width of the top n the warping res nection it was ju flange should be	flange d traint of dged tha used.	lo not f the at a			
Short term section Slab area = 800 × 22 Steel 'web' area = 7 Neutral axis of compo	properties for 50 / 6.0 = 33 $25 \times 15 = 61$ posite section =	br composite Tee s 3,300 mm ² (steel unit 13 mm ² = 245 mm below top	section: (s) of slab					
For composite section	n, $I = 2.600$	$\times 10^9 \mathrm{mm^4}$						
Offsets and section m	oduli (steel ur	nits) for key position	s in the section:					
	H below NA (mm)	Modulus (steel units (mm ³)) Modulus (conc. (mm ³)	units)				
Top of slab	-245		-63.6 × 10) ⁶				
Mid way rows 7 & 8	645	4.03 × 10 ⁶						
Bottom of web	730	3.56 × 10 ⁶						

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While this simple model is reasonable for determining the forces in the bolts (if the effective width of slab width were doubled, to 1600 mm, this would reduce bolt forces by only a few percent), it is not reliable for verification of stresses in the slab. Slab stresses are influenced by interaction with the surrounding regions, including the presence of the cantilever, and any verification of local transverse stresses in the slab is better served by examination of the FE model. The local slab stresses were considered acceptable in this example.								
Central cross girder (CG13) Share the vertical shear equally between all the bolts. Force/bolt = $1163/24 = 48.5$ kN								
Determine force (in web portion) at the level of the bottom row of bolts. Stress midway between lowest two rows, for $M = 71$ kNm (Sheet 54):								
$\frac{71 \times 10^6}{4.03 \times 10^6} = 18 \mathrm{N/mm^2}$								
Stress at bottom of web: $\frac{71 \times 10^{6}}{3.56 \times 10^{6}} = 20 \text{ N/mm}^{2}$								
Force = average stress × depth = $(18+20)/2 \times 15 \times (808-723) \times 10^{-3} = 24.0 \text{ kN}$ Force/bolt = 24.0/3 = 8.0 kN								
The total resultant force is thus:								
$F = \sqrt{48.5^2 + 8.0^2} = 49.2 \text{ kN} < 79.2 \text{ kN OK}$								
Note that the above simple meth for evenly spaced bolts and whe symmetrically disposed about the should be resolved into pure ber then applied separately to the bo	od for d re the be e bolt ro uding an olt group	eriving the force on ottom portion of web w. In other cases th d axial components o to derive the great	the lowest b is approxim e stresses in and the mon est bolt force	olt is ad ately the wel aent and 2.	lequate 5 ! force	2		
The stress in the concrete due to	the mo	ment is:						
Top of slab: $\frac{71 \times 10^6}{63.6 \times 10^6} = 1.2$ N/mm ² (tension)								
As noted above, this value is not a reliable value but is small and well within the tensile strength of the concrete f_{ctm} .								
Cross girder adjacent to pier Share the vertical shear equally	(CG8) between	all the bolts						
Force/bolt = $1228/24 = 51.2$	kN							

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CALCULATION SHEET	Client SCI		Made by	DCI	Date	July 2	009	
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Determine force (in web portion) at the level of the bottom row of bolts Stress midway between lowest two rows for $M = 450$ kNm (Sheet 54):								
$\frac{450 \times 10^6}{4.03 \times 10^6} = 112 \mathrm{N/mm^2}$								
Stress at bottom of web: $\frac{450 \times 10^{6}}{3.56 \times 10^{6}} = 126 \text{ N/mm}^{2}$								
Force = average stress × depth = $(112 + 126)/2 \times 15 \times (808 - 723) \times 10^{-3} = 151.8 \text{ kN}$ Force/bolt = $151.8/3 = 50.6 \text{ kN}$								
The total resultant force is thus:								
$F = \sqrt{51.2^2 + 50.6^2} = 72.4$	0 kN <	79.2 kN OK						
The stress in the concrete due to the moment is: Top of slab: $\frac{450 \times 10^6}{63.6 \times 10^6} = 7.08$ N/mm ² (tension)								
This stress is about double the tensile strength (which is the limit for using uncracked properties for determining flexural effects, according to 4-2/5.4.2.3). Although the surface stress given by the connection model is not considered reliable for verification of the slab, it is indicative that the local stress is not excessive. The tensile force for the stress distribution given by the model could be resisted by the transverse reinforcement (20 mm bars at 150 mm centres).								
The force in the slab (in this model) is introduced through the shear stud connectors on the flange of the main girder. Over a length of 800 mm the force transferred to the slab will probably be shared between at least 15 studs (3 rows of 5) and the resultant (from combined longitudinal and transverse shears) will be within the design capacity (81.7 kN, see Section 10.6). The verification of the connectors for the combined effects is not shown here.								
Separate checks will be needed for fatigue assessment of the stud connectors. The determination of appropriate loading and load effects is not covered here.								
Second cross girder from pier (CG9) Share the vertical shear equally between all the bolts Force/bolt = $1187/24 = 49.5$ kN								
Extra moment due to lateral restraint provided to bottom flange								
(take lever arm as mid-slab to mid bottom flange)								
$M = 143 \times (1894 - 60/2 - 125) /1000 = 214$ kNm (take as hogging)								
Total moment = $202 + 214 = 416$ kNm (hogging)								

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CALCULATION SHEET	Client SCI		Made by	DCI	Date	July 2	009	
	ber		Checked by	RJ	Date	Sep 2	009	
CALCULATION SHEET Determine force (in web portion Stress midway between lowest of $\frac{416 \times 10^6}{4.03 \times 10^6} = 103 \text{ N/mm}^2$ Stress at bottom of web: $\frac{416 \times 10^6}{3.56 \times 10^6} = 117 \text{ N/mm}^2$ Force = average stress × Force/bolt = 140/3 = 46.7 kM The total resultant force is thus: $F = \sqrt{49.5^2 + 46.7^2} = 68$. The stress in the concrete due to Top of slab: $\frac{416 \times 10^6}{63.6 \times 10^6} = 6.54 \text{ M}$ As noted for CG8, this is not a for As for CG8, the shear connector effects but that verification is not $416 \times 10^6 = 1.54 \text{ M}$	Client SCI) at the 1 wo rows depth = 1 kN < 1 kN < 1 the mon 1/mm ² (t <i>reliable</i> to t given h	level of the bottom ro $(103+117)/2 \times 15 \times (8)$ 79.2 kN OK ment is: ension) value for verification of to be verified for the of here.	Made by Checked by w of bolts $308 - 723) \times$ of the slab. combined lo	DCI RJ	Date Date	July 2 Sep 2	009	

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