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Deconstructable flush end plate beam-to-column composite joints: component-based modelling

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Abstract

Within a paradigm of designing building structures for their end-of-life deconstruction, this paper addresses flush end plate beam-to-column composite joints that may be dis-assembled and reused elsewhere. The joints consist of steel beams bolted to steel columns, and these are made composite over the joint with precast concrete slabs attached to the top flange of the steel beams with post-tensioned high strength bolted shear connectors installed in clearance holes. Joints of this type experience partial shear connection, and accordingly their design needs to incorporate this effect. Experimental work reported elsewhere by the authors shows that a structural system of this type may indeed be deconstructed, even when loaded beyond the serviceability limit state, and that the moment-rotation response is both robust and ductile. A numerical modelling procedure using ABAQUS software is introduced in the paper, and the results of this are used identify the parameters most influential in the structural response, and to propose equations for the initial stiffness, moment capacity and rotation capacity of a joint. These equations are consistent with the component-based representation of the Eurocode 4 and Australian AS2327 composite structures standard.

Keywords: *Component method; composite joint; deconstructable; friction grip; FEM.*

1. Introduction

The favourable attributes of constructing steel-concrete composite framed buildings for deconstruction and material reuse are manyfold. It has been shown that by using precast concrete slabs with post-installed tensioned friction-grip bolted shear connectors (PFGBSCs) to connect them to steel beams allows for robust composite action, as well as for deconstruction by unbolting the shear connectors at the end of the structural life of the building [1-5]. In regions of hogging bending at a column (Fig. 1), the use of flush end plate bolted connections also allows for unbolting to expedite deconstruction [6-8].

While experimental tests have provided both proof of concept and data bases for calibrating associated numerical models of such framed structures [8], structural design guidance is needed in codified formats for engineering practitioners. A popular technique used in the Eurocodes EC3 [9] and EC4 [10] is the component-based method [11], which provides a good balance of accuracy and simplicity. The aim of the current paper is to develop such a model for deconstructable flush end plate beam-to-column composite joints, based on a numerical model validated by experiments. The equations developed fit well within the design procedures of EC4 [10] and the Australian AS2327 [12].

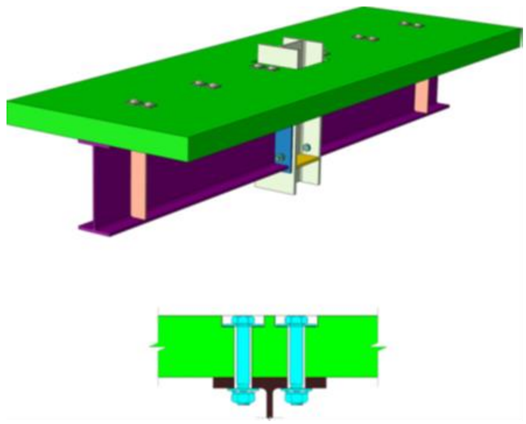

Fig. 1. Deconstructable flush end plate composite joint.

2. Computational modelling

The structural behaviour of full-scale beam-to-column joints with deconstructable PFGBSCs and precast concrete slabs subjected to static loading has been investigated with a three-dimensional FE model [8] using ABAQUS/CAE software [13], and the various components of the joint are shown in Fig. 2. Material and geometrical non-linearities as well as the non-linearity associated with contacts/interfaces were incorporated in the model. Because of the symmetry of the specimens and loading, only half of each joint was considered as illustrated in Fig. 3. The procedures for the modelling, material constitutive relationships, constraints, boundary conditions, meshing, contacts, and load application are presented in detail in [8], as well as validation of the model. Accordingly, the current paper focuses only on using the model in [8] to provide the data for validating the component-based approach.

Based on the parametric study used for the component model, the most influential parameters that influence the behaviour of the deconstructable composite joints were identified. A 67% shear connection ratio, 0.70% reinforcing ratio, 460UB82.1 steel beam, 250 UC89.5 steel column, 120 mm precast concrete slab, 10 mm flush end plate and Grade 10.9 M24 bolt were adopted for the standard joint considered in the parametric study, but each respective property was varied.

Shear connection ratios (SCRs) of 34%, 54%, 67%, 101%, 108%, 162%, 170% and 162% were considered to investigate their effect at the interface between the precast concrete slab and the steel beam. The ratio of the shear connection between the steel beam and precast concrete slab was obtained by changing the number and size of the bolt shear connectors and their spacing along the composite beam length. To gain insight in to the effect of the spacing of the bolted shear connectors on the behaviour of composite joints with PFGBSCs and a precast concrete slab, three different spacings of the shear connectors (275, 550 and 1100 mm) were considered. Different reinforcement ratios of 0.36%, 0.43%, 0.51%, 0.60%, 0.70%, 0.80%, 0.91% and 1.03% were used to investigate their effect on the moment-rotation responses. The effect of the bolt size was determined using nine diameters: 12, 14, 16, 18, 20, 22, 24, 26, 28 and 30 mm, which corresponds to respective end plate thickness to bolt diameter ratios t_{ep}/d_b of 0.83, 0.71, 0.63, 0.56, 0.50, 0.45, 0.42, 0.38, 0.36 and 0.33. Eleven plate thicknesses: 6, 8, 10, 12, 14, 16, 18, 20, 22, 26 and 30 mm were used to study the effect of the flush end plate thickness on the behaviour of composite joints with PFGBSCs and a precast concrete slab. Seven different steel grades, ranging from S235 to S960, were chosen to investigate the effect of this property on the behaviour of the composite joints, while seven different precast concrete slab thicknesses: 80, 120, 160, 200, 240, 280, 320 mm were employed to investigate the effect of this parameter on the behaviour of the composite joints. Six different steel column flange thicknesses: 10, 14, 18, 22, 26, 280, 30 mm were considered in this study in order to investigate their effect on the behaviour of the composite joints.

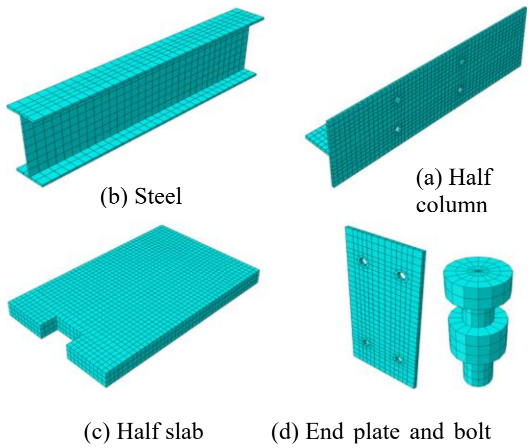

Fig. 2. FE mesh of joint components.

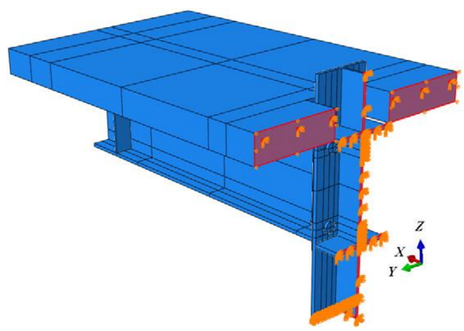

Fig. 3. Half joint considered in FE modelling.

3. Component-based modelling

The component-based method provided in EC4 [10] is extended herein to predict the initial stiffness of composite joints having PFGBSCs and a precast concrete slab. In this method, each component of the composite connection is represented by an elastic spring (Fig. 4) and the stiffness of the connection is assessed through the assemblage of the components. The rebars, flush end plate, bolts in the tension zone, column flange and column web are the main components contributing to the initial rotational stiffness. An effective stiffness k_{eff} for a bolt row can be obtained by combining the stiffness of the end plate in bending k_3, bolts in tension k_4, column flange in bending k_5 and column web in tension k_6, in series. The stiffness coefficients of the column web in compression k_1 and column web in shear k_2 are assumed to be infinite due to the stiffened steel column and by the symmetric loading on the connection. The stiffness of the rebars k_7 is combined with k_{eff} and a single equivalent stiffness coefficient k_{eq} and its equivalent lever arm z_{eq} is obtained. The initial stiffness of the composite joint can then be written as $S_j = Ek_{eq}z_{eq}^2$ (in kNmm/mrad).

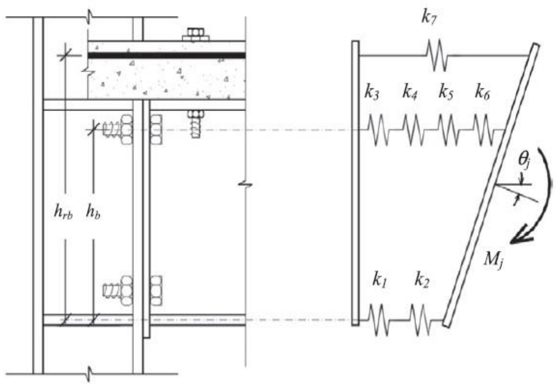

Fig. 3. Initial rotational stiffness model.

The initial stiffness derived from the component-based model developed is compared in Table 1 with the FE results. It can be seen that there is good agreement between these two approaches for most of the models, but a large discrepancy can be seen for a few cases such as those having a high reinforcement ratio. Liew et al. [14] reached similar conclusions after comparing their composite joint test results with those predicted by the component method, as the component-based method provided 26 to 83% higher initial stiffnesses compared to their test results [14]. Comparisons between the results predicted by Ahmad and Nethercot's model [15] and the component-based method shows that except for some cases, both techniques appear to be appropriate to use for estimating the initial stiffness of a deconstructable composite joint.

Table 1. Initial rotational stiffness predictions.

Variable	Model	FEM	Component method		Ahmed & Nethercot	
			kNm/rad	ratio	kNm/rad	ratio
Reinfor-cement ratio	0.36	62	54	0.87	51	0.82
	0.43	69	57	0.83	56	0.81
	0.51	70	60	0.86	62	0.89
	0.60	72	62	0.86	65	0.90
	0.70	73	67	0.92	66	0.90
	0.80	73	70	0.96	83	1.14
	0.91	74	73	0.99	91	1.23
	1.03	75	77	1.03	99	1.32
	1.16	75	80	1.07	107	1.43
	1.29	75	84	1.12	115	1.53
	1.43	76	88	1.16	124	1.63
Shear connector	2M16	46	63	1.37	53	1.15
	2M20	52	65	1.25	55	1.06
	4M16	70	66	0.94	66	0.94
	6M16	85	80	0.94	77	0.91
	4M20	72	67	0.93	56	0.78
	6M20	88	81	0.92	78	0.89
	10M16	111	97	0.87	107	0.96
	10M20	114	99	0.87	108	0.95
Slab thickness (mm)	80	53	61	1.15	52	0.98
	120	73	66	0.90	66	0.90
	160	85	70	0.82	67	0.79
	200	97	75	0.77	68	0.70
	240	121	80	0.66	68	0.56
	280	142	86	0.61	68	0.48
	320	167	91	0.54	68	0.41
Bolt size (mm)	12	58	58	1.00	66	1.14
	14	65	61	0.92	66	1.02
	16	65	63	0.97	66	1.02
	18	66	64	0.97	66	1.00
	20	68	64	0.94	66	0.97
	22	73	65	0.89	66	0.90
	24	73	66	0.90	66	0.90
	26	74	67	0.91	66	0.89
	28	82	67	0.82	66	0.80
	30	97	68	0.70	66	0.68
End plate thickness (mm)	6	27	41	1.52	66	2.44
	8	56	53	0.95	66	1.18
	10	73	67	0.92	66	0.90
	12	93	75	0.81	66	0.71
	14	112	83	0.74	66	0.59
	16	128	89	0.70	66	0.52
	18	146	94	0.64	66	0.45
	20	158	96	0.61	66	0.42
	22	170	97	0.57	66	0.39
	26	185	98	0.53	66	0.36
	30	196	98	0.50	66	0.34
Column flange thickness (mm)	10	36	55	1.53	66	1.93
	14	53	63	1.19	66	1.25
	18	73	66	0.90	66	0.90
	22	77	67	0.87	66	0.86
	26	83	68	0.82	66	0.80
	30	86	68	0.79	66	0.77
Average				0.90		0.93
Standard deviation				0.22		0.38

The moment capacities of composite joints with PFGBSCs and precast concrete slabs can be calculated using the concept of rigid plastic analysis. Anderson and Najafi [15] proposed an approach (Fig. 4) using this technique for the calculation of the moment capacity of traditional composite connections, and their model is extended herein for composite joints with PFGBSCs and precast concrete slabs. The bending strength of a composite joint can be determined from

$$M_j = F_{rb}\left(h_{rb} - 0.5t_{fb}\right) + F_b\left(h_b - 0.5t_{fb}\right) \quad (1)$$

if $F_{rb} + F_b \leq F_{fb}$, and from

$$M_j = F_{rb}\left(h_{rb} - 0.5t_{fb}\right) + F_b\left(h_b - 0.5t_{fb}\right) - \left(F_{rb} + F_b - F_{fb}\right)\cdot\left(0.5y_c + 0.5t_{fb}\right) \tag{2}$$

if $F_{rb} + F_b > F_{fb}$, in which F_{rb} is the tensile strength of the longitudinal rebars, F_{bl} the tensile force in the bolts at the top row, F_{fbc} the resistance force in the bottom flange of the steel beam, h_{rb} the distance between the centroid of the reinforcing bars and the centroid of the steel beam bottom flange, h_b the distance between the centroid of the top row of bolts and the centroid of the steel beam bottom flange and t_{fb} the thickness of the steel beam bottom flange (Fig. 4). In addition, y_c is the depth of web in compression, being obtained from

$$y_c = \frac{F_{rb} + F_b - F_{fb}}{t_{fb} f_{yb}}. \tag{3}$$

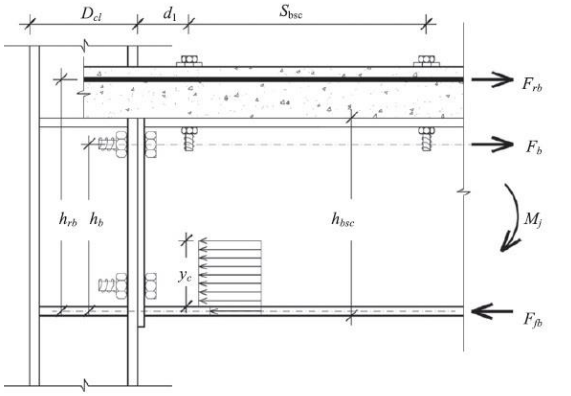

Fig. 4. Moment capacity model.

The moment capacity determined from this method is compared with the FE results in Table 2. It can be seen that good agreement between these two approaches is demonstrated for most of the models. However, there is a discrepancy for a few cases, such as those with very high reinforcement ratios or with very low degrees of shear connection. This is because the design method is not applicable to composite joints with an overly-low degree of shear connection, as its failure governed by fracture of shear connectors.

In order to incorporate the effects of partial shear connection in this method, the strength of the longitudinal reinforcing bars located in the precast concrete slab was limited by the shear strength of the bolted shear connectors. Because of this, based on the experimental results achieved by Ataei et al. [4-7], when the yield strength of the reinforcement (F_{rb}) is 1.5 times that of the shear strength of the bolted shear connectors (F_{bsc}), F_{rb} can be obtained from

$$F_{rb} = 1.5F_{bsc}. \tag{4}$$

When the effect of partial shear connection is taken into calculation of the moment capacity of a composite joint with PFGBSCs and a precast concrete slab, the analytical model provides much closer results (the values in brackets) to the FE predictions.

Table 2. Moment capacity predictions.

Variable	Model	FEM	Anderson & Najafi	
		kNm/rad		ratio
Reinforcement ratio	0.36	340	383	1.13
	0.43	377	412	1.09
	0.51	416	444	1.07
	0.60	456	479	1.05
	0.70	540	517	0.96
	0.80	589	558	0.95
	0.91	602	601	1.00
	1.03	573	647	1.13
	1.16	548	696 (654)	1.27 (1.19)
	1.29	548	747 (654)	1.36 (1.19)
	1.43	548	802 (654)	1.46 (1.19)
Shear connector	2M16	422	517 (449)	1.23 (1.06)
	2M20	530	517	0.98
	4M16	529	517	0.98
	6M16	506	517	1.02
	4M20	521	517	0.99
	6M20	486	517	1.06
	10M16	479	517	1.08
	10M20	479	517	1.08
Slab thickness (mm)	80	526	497	0.94
	120	540	517	0.96
	160	559	537	0.96
	200	577	558	0.97
	240	598	578	0.97
	280	616	598	0.97
	320	635	619	0.97
Bolt size (mm)	12	345	339	0.98
	14	404	365	0.90
	16	436	395	0.91
	18	465	412	0.89
	20	492	442	0.90
	22	520	483	0.93
	24	540	517	0.96
	26	548	559	1.02
	28	556	611	1.10
	30	562	620	1.10
End plate thickness (mm)	6	480	517	1.08
	8	507	517	1.02
	10	532	517	0.97
	12	544	517	0.95
	14	568	517	0.91
	16	596	517	0.87
	18	620	517	0.83
	20	642	517	0.81
	22	658	517	0.79
	26	665	517	0.78
	30	665	517	0.78
Column flange thickness (mm)	10	500	517	1.03
	14	527	517	0.98
	18	544	517	0.95
	22	546	517	0.95
	26	546	517	0.95
	30	546	517	0.95
Average				1.00 (0.95)
Standard deviation				0.13 (0.10)

The assessment of the ductility of a composite connection is conducted by determining its rotation capacity. In addition, according to guidance given in EC4, if the minimum rotation capacity of a composite joint is larger than 30 mrad, the joint is deemed to be ductile and plastic analysis and design are permitted. Therefore, calculation of the rotation capacity with sufficient accuracy is needed.

Ahmad and Nethercot [16] proposed a model for the rotation capacity of a connection based on the deformation of the reinforcement and the top row bolts and the slip at the interface between the concrete slab and steel beam, being written as

$$\theta_j = \frac{\delta_{rb}}{h_{rb} - y_c} + \frac{\delta_b}{h_b - y_c} + \frac{s}{D_b - y_c} \,, \tag{5}$$

where δ_{rb} is the elongation of the longitudinal reinforcing bars, δ_b the extension of the top row of bolts and s the final slip at the interface between the steel beam and concrete slab. These can be calculated from

$$\delta_{rb} = 0.01\left(\frac{D_{cl}}{2} + d_1 + s_{bsc}\right), \delta_b = \frac{F_b}{K_b}, s = \frac{F_r}{K_{bsc}} \tag{6}$$

where D_{cl} is the depth of the steel column, D_b the depth of the steel beam, d_1 the distance between the column face and the first row of the shear bolts and s_{bsc} the distance between the first row and the second rows of the shear bolts. The stiffness of the top row of bolts (K_b) can be taken as K_b = 155 kN/mm. The stiffness of the bolt shear connectors (K_{bsc}) can be taken as K_{bsc}=10n kN/mm according to the results observed from the tests conducted by Ataei *et al.* [4,5], where n is the number of bolt shear connectors present in the shear span.

The rotation capacities determined from this model are compared with the FE results in Table 3. Good agreement between these two approaches is observable for most of the models, but there is a large discrepancy for a few cases such as those with very high reinforcement ratios or with high degrees of shear connection. This may be because fracture of the bolt in the connection zone, fracture of the flush end plate and the effect of the degree of shear connection are not considered in Ahmad and Nethercot's model.

So as to incorporate the effect of partial shear connection in this method, the strength of the longitudinal reinforcing bars located in the precast concrete slab was limited by the shear strength of the bolted shear connectors. Accordingly, based on the experimental results obtained by Ataei et al. [4], δ_{rb} = 0 if $F_{bsc} \leq 0.67F_{rb}$. Table 3 shows that when the effect of partial shear connection is taken into account, the analytical model can provide closer results (values in brackets) compared to the FE predictions.

Table 3. Moment capacity predictions.

Variable	Model	FEM	Ahmed & Nethercot	
		mrad		ratio
	0.36	31	39	1.26
	0.43	35	42	1.20
Reinforcement ratio	0.51	39	45	1.15
	0.60	42	48	1.14
	0.70	57	53	0.93
	0.80	58	56 (63)	0.97 (0.91)
	0.91	60	61 (53)	1.02 (0.91)
	1.03	38	66 (39)	1.74 (0.91)
	1.16	32	71 (39)	2.22 (0.91)
	1.29	32	76 (39)	2.38 (0.91)
	1.43	32	82 (39)	2.56 (0.91)
	2M16	31	53 (40)	1.71 (1.29)
Shear connector	2M20	62	53	0.85
	4M16	52	53	1.02
	6M16	45	44	0.98
	4M20	50	44	0.88
	6M20	31	38	1.23
	10M16	28	36 (32)	1.29 (1.14)
	10M20	28	33 (28)	1.18 (1.00)
	80	63	55	0.87
Slab thickness (mm)	120	57	53	0.93
	160	55	52	0.95
	200	52	52	1.00
	240	49	51	1.04
	280	46	50	1.09
	320	44	50	1.14
	12	22	46	2.09
Bolt size (mm)	14	27	47	1.74
	16	38	48	1.26
	18	43	49	1.14
	20	44	50	1.14
	22	51	52	1.02
	24	57	53	0.93
	26	56	55	0.98
	28	58	58	1.00
	30	56	60	1.07
	6	49	53	1.08
End plate thickness (mm)	8	49	53	1.08
	10	58	53	0.91
	12	55	53	0.96
	14	53	53	1.00
	16	55	53	0.96
	18	54	53	0.98
	20	51	53	1.04
	22	48	53	1.10
	26	38	53	1.39
	30	36	53	1.47
	10	62	53	0.85
Column flange thickness (mm)	14	60	53	0.88
	18	59	53	0.90
	22	58	53	0.91
	26	57	53	0.93
	30	57	53	0.93
Average				1.18 (1.09)
Standard deviation				0.39 (0.21)

4. Conclusions

This paper has used the methodology of a FE modelling reported elsewhere by the authors to assess a proposed component-based technique for designing deconstructable flush end plate beam-to-column composite joints. The numerical procedure is underpinned by ABAQUS software and its validity was also confirmed elsewhere by comparisons with test results. The FE procedure allows for a substantial combination of parameters over a wide range of those met in practice.

It was shown that prescriptive models available in the literature and augmented appropriately provide a good balance of simplicity and accuracy. These models and their mathematical representation are consistent with techniques already used in codes of practice.

Acknowledgement

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