

Sustainable composite beam behaviour with deconstructable bolted shear connectors

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ABSTRACT

Composite structures using steel joists mechanically connected to concrete slabs serving as the flooring and decking systems are commonly used in buildings and bridges respectively. The use of the state-of-art precast geopolymer concrete slabs coupled with innovative deconstructable pretensioned bolted shear connectors under service loads in a life-cycle composite beam design can reduce CO₂ emissions, thereby enhancing the sustainability in and recyclability of the construction industry. Most previous research has been conducted on composite beams with welded shear studs that cannot be deconstructed easily. A number of push tests are reported in the current study to establish the interface strength and stiffness characteristics of the bolted shear connectors. The application of the push test data in predicting complex full-scale composite beam behaviour with three distinctive regions that are delineated by the shear flow force at the interface, by the coefficient of friction at the interface and bolt pretension, and by the size of the clearance hole relative to the diameter of the bolt is explained. An initial theoretical investigation based on a simple mechanics-based “full-zero-full” shear interaction mechanism is presented and discussed.

1. INTRODUCTION

Composite structures using steel joists mechanically connected to concrete slabs are commonplace in buildings and bridges. Concrete is normally strong in compression but weak in tension, whilst steel in compression is prone to buckling effects, and so their symbiotic attributes in composite beams make the composite beam an efficient member. The mechanical shear connectors are designed to bring about composite action in these complementary materials to increase the strength, ductility and stiffness of composite beams above those of the individual materials themselves. Normally, the weakest component governs the strength of the composite structure. Ordinary Portland cement (OPC) has been traditionally used as a paste to produce concrete for many years. It is widely-

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known that OPC is a major contributor to anthropogenic greenhouse gas production, with large CO₂ emissions being associated with the manufacture of OPC, with one tonne of OPC producing approximately one tonne of CO₂ (Davidovits 1994).

The use of geopolymer concrete (GPC) as a replacement for OPC is seen as one means of mitigating the excessive CO₂ emissions associated with OPC, and is a topic of current research in cementitious materials science. Fly ash is a waste material from coal burning power stations around the world, and has been used and experimentally tested as a source of geopolymer for GPC that is reported to exhibit comparable if not better material properties to OPC-based concrete (Hardjito et al. 2004). The use of a waste material such as fly ash to replace OPC to produce high strength GPC is producing a new era of more sustainable concrete industry. However, GPC is not readily batched on-site by comparison to concrete based on OPC, and so its use in precast applications is an important research issue. The slab systems considered in this paper are precast GPC ones.

There are many types of mechanical shear connectors (Oehlers and Bradford 1995) as shown in Fig.1, but the most commonly used ones are the stud connectors normally welded to the top flanges of the steel joists before casting the concrete slabs (solid or profiled with steel sheeting) because they are economical and easy to install. Clearly the welding and casting of the concrete around headed stud connectors is highly problematic in formulating de-constructability and recycling technologies for composite beams. High-strength bolted steel shear connectors (Fig.1(b) or/and 1(h)) provide a viable alternative to headed connectors in terms of deconstructability. Despite this, and despite the potential for retrofitting composite bridge beams whose studs have failed under fatigue loading, surprisingly little has been reported in the open literature on high strength bolts as shear connectors.

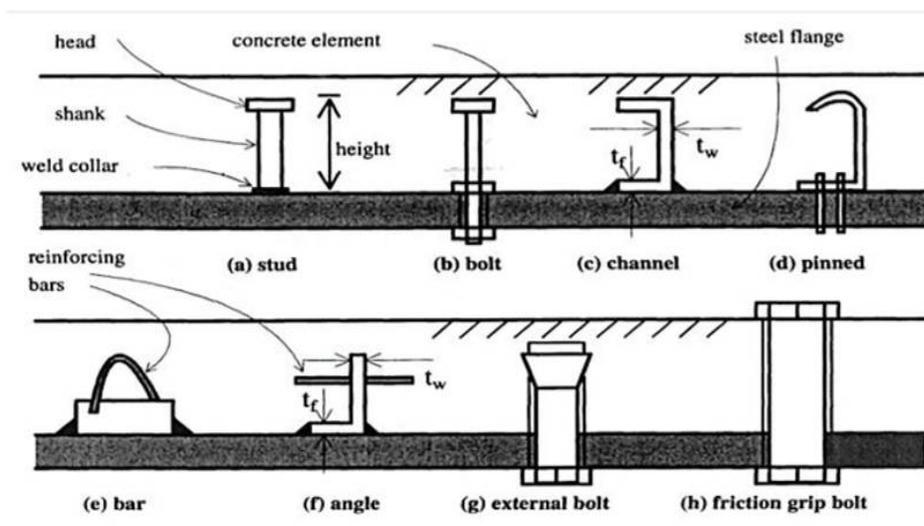
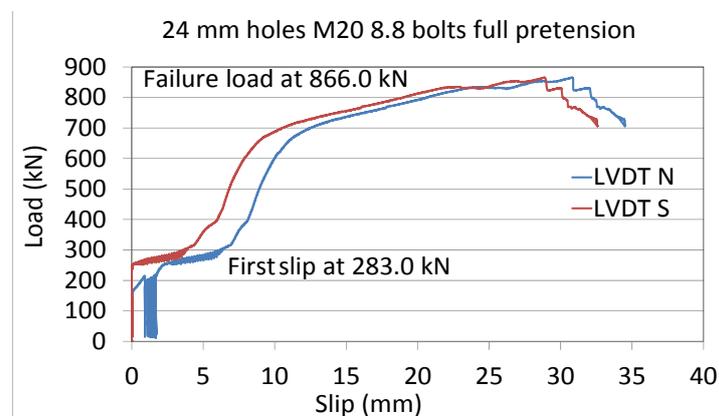


Fig.1 Mechanical shear connectors (Source: Oehlers and Bradford 1995)

Most of the previous research of composite beams has been based on stud shear connectors for normal steel joists and with OPC slabs. Of the limited studies on bolted shear connectors in composite steel-concrete beams, Dallam (1968) and Dallam and Harpster (1968) reported push tests and beam tests undertaken with bolts embedded in the concrete, Marshall et al. (1971) undertook tests on embedded and post-installed bolts and recently Kwon et al. (2010) investigated the use of post-installed shear connectors under static and fatigue loading. However, none of those studies offered a complete understanding of the structural behaviour of these types of structures nor considered the issues of deconstructability and sustainability. Therefore, more detailed investigations of the structural behaviour of bolted shear connectors, bolted steel joists and precast GPC slabs, all of which can be recycled at the life end of a structure, is much-needed. The results of five initial push tests have been reported in details elsewhere by the authors (Lee and Bradford 2013). The potential of applying the push test data to predict the structural response of full-scale sustainable composite beams with deconstructable bolted shear connectors using a mechanics-based model (Rowe and Bradford 2013) is discussed and the full scale experimental program has also been reported in detail (Lee and Bradford 2013). This paper reports the results of four additional push tests, and presents an initial theoretical investigation based on a simplified mechanics-based model. This understanding allows a codified design approach.

2. SUMMARY OF PUSH TEST RESULTS

The results of five initial push tests confirm one of the research hypotheses in which there are three distinctive stages of the load-slip curves of pretensioned bolted shear connectors: viz. a region of “full interaction”, a region of “zero interaction” and a region of “partial interaction”, that are delineated by the shear flow force at the interface, by the coefficient of friction at the interface and bolt pretension, and by the size of the clearance hole relative to the diameter of the bolt shown in Fig. 2.



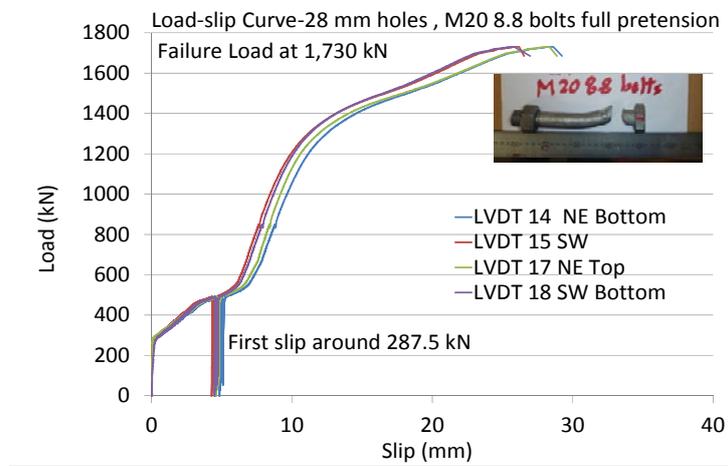


Fig 2 Load-slip evolution for typical single and double panel push tests.

The double panel push test confirmed that the shear force could be transferred successfully across the panels which replicate a similar situation in the full scale test beams. One of the push tests was also performed to confirm deconstructability of the shear connectors under service loads with a typical push test failure mode as shown in Fig. 3. The deconstructability of the bolts under service loads was confirmed. The bolts were then unloaded and then re-pretensioned and reloaded to failure. The test specimen failed at 890 kN with a maximum slip of 24.8 mm with a bolt fracture in addition to shear cracks in the concrete and three fully yielded bolts. One of the possible reasons that it failed at a higher load with a bolt fracture could be due to the increased concrete strength to 48 MPa at 90 days rather than 43 MPa at 21 days.

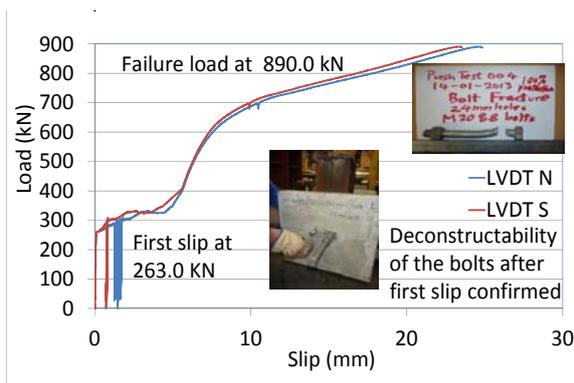


Fig. 3 Push test 004 conducted to confirm de-constructability of the shear connectors under service loads and a typical failure mode

Table 1 Key push test results.

Test	P_{sp} (kN)	P_u (kN)	μ	s_f (mm)	s_u (mm)
001	283	866	0.49	4	30.8
002	80	808	0.25	6	25.8
003	100	609	0.26	8	31.5
004	263	890	0.45	4	24.8
005	288	1730	0.50	8	28.6

P_{sp} = load for first slip; P_u = ultimate load; μ = coefficient of friction; s_f = slip at first bearing; s_u = ultimate slip.

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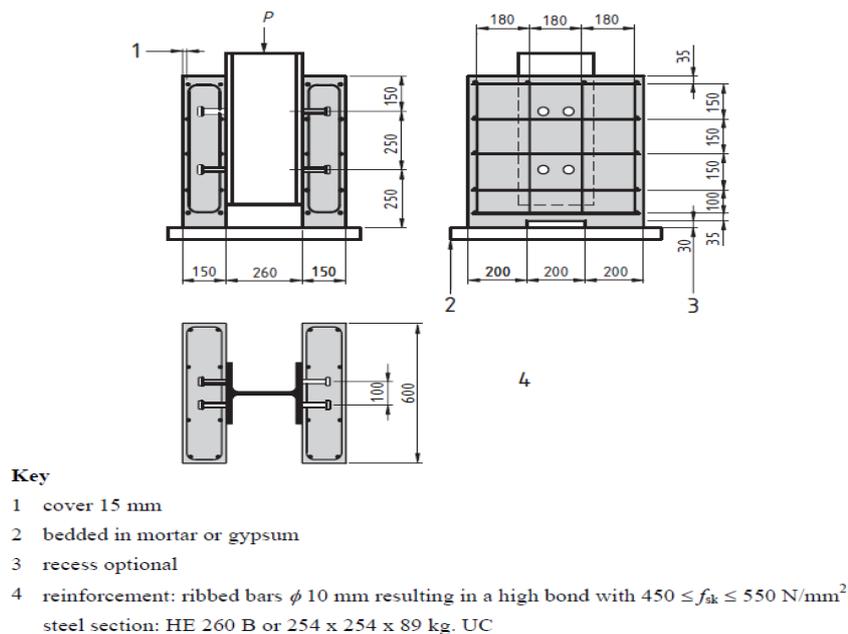


Fig. 4 Dimensions of standard push test specimens except the shear studs are replaced either with pretensioned single nut bolts or high-tension friction bolts (source: EC4)

The key results of five initial push tests are summarised in Table 1.

Four additional push tests (006-009) designed to EC4 standards (8 bolts and cycled 25 times between 5% and 40% of the estimated failure load) were carried out with concrete strength of 48 MPa at 90 days (a confirmatory compression test was carried out to confirm the strength which is comparable to that for the initial push tests at 90 days).

Push tests 006 and 007 were designed using eight M20 8.8 single nut bolts embedded in the two 600x600x150 mm thick GPC panels with 15 mm cover (see Fig. 1(b)) tightened manually. The pretension was calibrated as 130 kN. These two push tests are material tests for the full scale Control Beam.

Push tests 008 and 009 were designed using eight M20 8.8 bolts pretensioned to 145 kN using the calibrated electric wrench and squirter washers through eight 24 mm precast holes as per the five initial push tests except push test 002. The load-slip evolution of push tests 006 and 007 are presented in Fig 5.

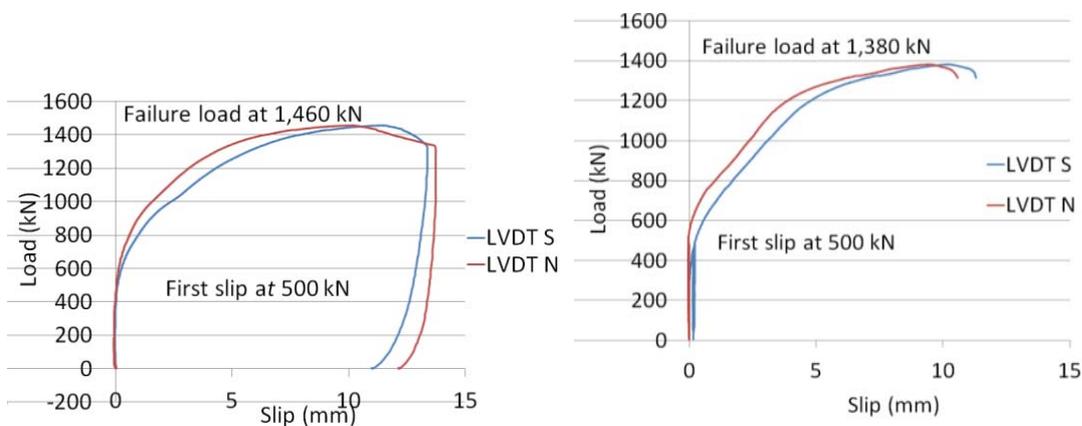


Fig. 5 Load-slip evolution of push tests 006 and 007

The load-slip curves of the single nut bolts embedded in the GPC panels in Figure 5 show two rather than three distinctive stages of composite action at the steel-concrete interface. The reason is that the single nut bolts were embedded in the concrete panels during the pre-casting process and therefore there is no hole clearances between the bolts and the surrounding GPC concrete. The first stage is of a frictional nature and once the friction is overcome, the bolts start to bear on the concrete. Push tests 006 and 007 recorded similar load of 500 kN at first slip and then a failure load of 1,460 kN and 1,380 kN respectively with a similar ultimate slip of around 10 mm (c.f. to 6 mm specified by EC4 to ensure ductile behaviour of composite beams). These results are comparable to those presented by Kwon et al (2010) in terms of the general shape of the load-slip curves and the loads at first slip and at failure. Push test sample 006 appears to be somewhat less stiff than sample 007 during the second stage of the steel-concrete interface behaviour i.e. the bolts started to bear onto the concrete.



Fig. 6 Test set up and failure mode for push tests 006 and 007

Both push tests failed by bolt fracture and concrete shear cracks as shown in Fig. 6. There was also evidence of friction being overcome in the failed samples coupled with loud banging noise during the first stage of loading before the first slip. The load-slip curves of push tests 008 and 009 are shown in Fig. 7. The general shape of the curves is similar to previous push test 005 i.e. three distinctive stages of steel-concrete interface behaviour i.e. fully composite (held by friction), near non-composite (with 4 mm total slip clearance between the bolts and the surrounding GPC concrete as demonstrated in Fig. 7 and then finally, partially composite as the bolts start to bear onto the GPC concrete. Push tests 008 and 009 recorded a load of 560 kN and 585 kN at first slip and a failure load of 1,715 kN and 1,770 kN respectively. The ultimate slip at failure is in the order of 20 mm showing again a very ductile behaviour of these types of shear connection. These test results are comparable to those of Kwon (2010). One interesting observation is that the load at first slip of push tests 008 and 009 (single panel with 8 bolts) appear to be twice that of previous push test 005 (double panels with 8 bolts). This will be a subject of further investigation and it is likely that the bolts in these two tests start to slip more or less the same time giving a much higher load at first slip.

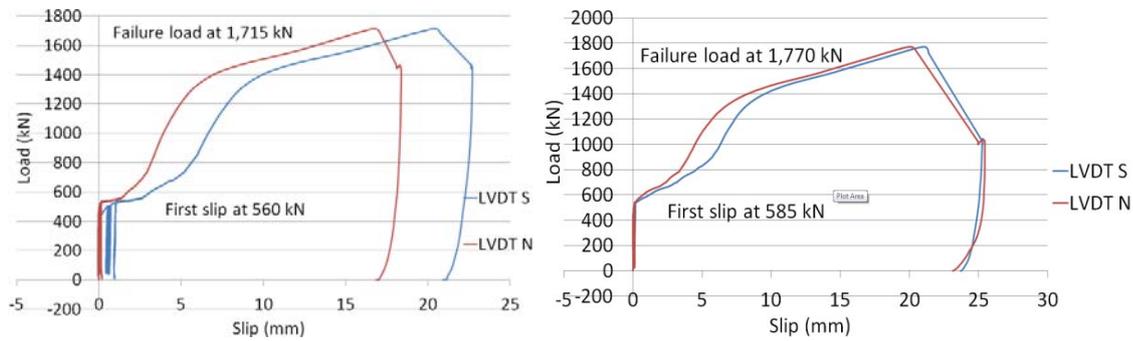


Fig. 7 Load-slip evolution of push tests 008 and 009



Fig. 8 Test set up and failure mode for push tests 008 and 009

The test set up and failure mode observed for push tests 008 and 009 are shown in Fig. 8. Both push test specimens failed by bolt fracture with no apparent concrete shear cracks on the non-contact side of the GPC concrete panels and this could be due to the increased concrete strength of 48 MPa at 90 days.

3. QUASI-ELASTIC MECHANICS BASED MODEL

3.1 Framework of Theoretical Model

A theoretical model of the response of a beam with pretensioned bolted shear connectors at service loading can be developed, which uses the empirical results from the push testing (Rowe and Bradford 2013). This can be illustrated with reference to a simply-supported geometrically symmetrical beam of length $2L$ with the x origin at mid span.

Initially, the beam has full interaction because of the prestress provided by the bolts, which has a capacity of $p = \mu nP/L$, where P is the prestress in each of the n bolts and μ is the coefficient of friction. When the shear flow force at the ends of the beam (qLB/I) reach p , the bolts in that location start to slip, i.e. when $q = q_1 = pI/(LB)$. As q increases from q_1 , the bolts start to slip with a limiting value of s_f , in this case, it refers to the size of the total clearance between the bolts and the holes. When q reaches a value of q_2 , the bolts then start to bear on the concrete. This phenomenon of three regions will start to propagate from the end supports where the shear forces are maximum towards the mid-span. These regions of different behaviour of shear connection are illustrated by the load-slip relationship shown in Fig. 9. A beam-type finite element model (Ranzi et al. 2004) can be cast in an adaptive mode using the engineering principle of geometric compatibility and equilibrium of forces. Typical values of k_1 , k_2 and k_3 of the pretensioned bolted shear connectors based on the initial push tests (4 bolts) are in the order of 0.0, 27,000 N/mm, 107,500 N/mm respectively. For the purpose of this paper, a simpler model of “Near to Full”-“Near to Zero”-“Near to Full” shear interaction (Rowe and Bradford 2013) was used to illustrate the concept and to predict the general load slip behaviour of the full scale composite beams with pretensioned bolted shear connectors having a span of $2L$ and simply supported under a uniformly distributed load q . For this model in Fig. 9 $k_1 = k_3 \rightarrow \infty$ and $k_2 = 0$

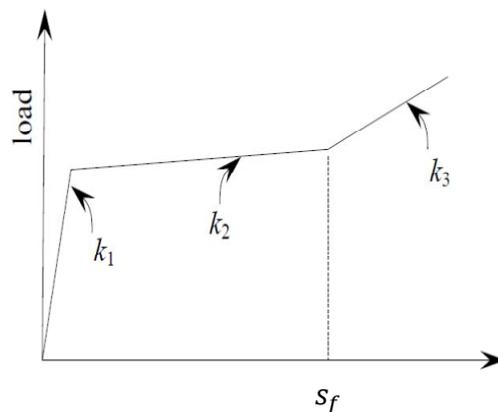


Fig. 9 Idealised response in elastic range (“Near to Full”-“Near to Zero”-Partial Shear Interaction).

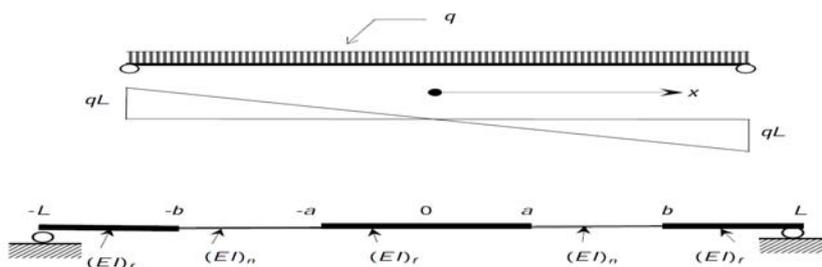


Fig.10 Simplified “Full-Zero-Full” shear interaction Model of a composite beam with span length of $2L$ under uniform load q

3.2 Refinement of Theoretical Model

The equation for bending in the composite beam in Fig. 10 under a uniformly distributed load q can be applied to different regions i.e. Full interaction $(-a, a)$, Zero interaction $(a, b) \cup (-a, -b)$ and Full interaction $(b, L) \cup (-b, -L)$ of the composite beam shown in Fig. 10. The equation for bending is given as

$$EIv'' = -M = q(x^2 - L^2)/2, \quad (1)$$

In which M is the bending moment, EI is the stiffness of the composite beam and the subscripts f and n indicate the region of the beam has either full or zero shear interaction respectively, v is the transverse deflection and $v''(x)$ is the curvature. Eq. (1) can be applied to three different regions of the composite beam in Figure 10. Applying the displacement and slope compatibility and static boundary conditions at $x = 0, a$ (where first interface slip occurs), b (where first bearing on the holes occurs) and L produces, after some algebra,

$$\frac{v(0)}{\bar{v}(0)} = \left(\frac{1-n}{5}\right) [(3\alpha^3 - 4\alpha^2 - 6\alpha + 12)\alpha - (3\beta^3 - 4\beta^2 - 6\beta + 12)\beta] + 1, \quad (2)$$

where

$$\bar{v}(0) = \frac{5q(2L)^4}{384(EI)_f}, \quad \alpha = \frac{a}{L}, \quad \beta = \frac{b}{L}, \quad n = \frac{(EI)_f}{(EI)_n}.$$

For the domain (a, b) and noting that $s(a) = 0$ by definition, the slip $s(x)$ is

$$s(x) = (v'(x) - v'(x)_a)h, \quad (3)$$

where s is the slip deformation in domain (a, b) and h is the distance between the centroids of the two elements (say in this case, the GPC concrete slab and the steel joist) of the composite beam. The location of the region of bearing can be determined using $x = b$, which produces the cubic equation

$$\beta^3 - 3\beta = \frac{6(EI)_n s_f}{qhL^3} + \alpha^3 - 3\alpha, \quad (4)$$

where the load at which bearing commences is defined in

$$q_2 = q_1 + \frac{6s_f(EI)_n}{hL^3(3\alpha - \alpha^3 - 2)}. \quad (5)$$

Fig. 11 below shows the potential effect of the limiting slip, s_f on the ratio of $\frac{q_2}{q_1}$ for one of the full scale composite beams as part of the ongoing current research.

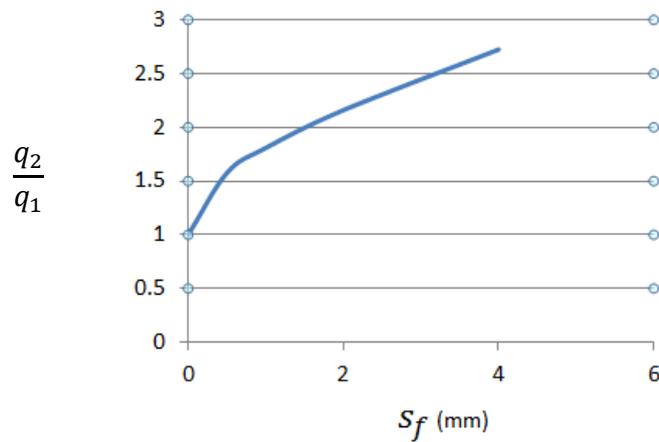


Fig. 11 Potential effect of the limiting slip on ratio of first load to bear to first load to slip

4. CONCLUSION

Additional push test data have been determined for composite beams with pre-tensioned high strength bolts as shear connectors with precast geopolymer concrete slabs, and added to the repository of five previous initial push test results. The potential of applying the push test data to predict the full scale beam behavior using a simplified quasi-elastic mechanic based composite beam model has been further refined to illustrate the concepts based on a “Full-Zero-Full” shear interaction model, proposed in earlier work.

ACKNOWLEDGEMENT

The work in this paper was supported by the Australian Research Council through an Australian Laureate Fellowship. Zeobond provided the GPC and Hobson Engineering Pty Ltd provided technical advice on the bolt installation. The support of the technical staff in the UNSW Heavy Structures Research Laboratory is acknowledged with thanks.

REFERENCES

- Dallam, L.N. (1968), "Pushout tests with high strength bolt shear connectors," Report for Missouri State Highway Department, Department of Civil Engineering, University of Missouri – Columbia.
- Dallam, L.N. and Harpster, J.L. (1968), "Composite beam tests with high-strength bolt shear connectors," Report for Missouri State Highway Department, Department of Civil Engineering, University of Missouri – Columbia.
- Davidovits, J. (1994), "High-alkali cements for 21st century concrete," In P. Kumar Metha (ed), ACI SP 144: 383-397. Detroit: American Concrete Institute.
- Eurocode 4 (2004), "Design of composite steel and concrete structures —Part 1-1: General rules and rules for buildings."
- Hardjito, D, Wallah, S.E., Sumajouw, D.M.J. and Rangan, B.V, (2004), "On the development of fly ash-based geopolymer concrete," *ACI Materials Journal*, 101(6), 467-472
- Kwon, G., Engelhardt, M.D. and Klinger, R.E. (2010), "Behavior of post-installed shear connectors under static and fatigue loading," *Journal of Constructional Steel Research*, 66, 532-541.
- Lee, S.S.M. and Bradford, M.A. (2013a), "Sustainable composite beams with deconstructable bolted shear connectors," SEMC 2013: The Fifth International Conference on Structural Engineering, Mechanics and Computation, Cape Town, South Africa.
- Lee, S.S.M. and Bradford, M.A. (2013b), "Sustainable Composite beam behaviour with deconstructable bolted shear connectors-Full scale experimental investigation," Concrete 2013, Gold Coast, Queensland, Australia.
- Marshall, W.T., Nelson, H.M. and Banerjee, H.K. (1971), "An experimental study of the use of high-strength friction-grip bolts as shear connectors in composite beams," *The Structural Engineer*, 49(4), 171-178.
- Oehlers, D.J. and Bradford, M.A. (1995), "Steel and Concrete Composite Structural Members: Fundamental Behaviour." Oxford: Pergamon.
- Ranzi, G., Bradford, M.A. and Uy, B. (2004). "A direct stiffness analysis of composite beams," *International Journal for Numerical Methods in Engineering*, 61(5), 657-672.
- Rowe, M. and Bradford, M.A. (2013), "Partial shear interaction in deconstructable composite steel-concrete composite beams with bolted shear connectors," International Conference on Design, Fabrication and Economy of Welded Structures, Miskolc, Hungary, 585-590.