International Journal of
Advanced Structures and Geotechnical Engineering
ISSN 2319-5347, Vol. 03, No. 04, October 2014

Characterization of hybrid structural components

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Abstract: The steel-concrete hybrid structural elements rely on the transfer of force between the two materials in order to realize the benefits of hybrid action. And the problems in design are related to the problem of force transmission between concrete and embedded steel profiles. So it creates a situation in which it is not known how to combine the resistances provided by stud and plate connectors, and how to reinforce the concrete in the transition zone. This study covers the design and analysis of some hybrid specimen for push-off test to transfer forces from the steel profile to the concrete wall and gives an idea about behavior of various connectors. Rigid connectors like plate bearing shows relatively more resistance by creating struts and tie effects inside concrete where flexible connectors allows more slip. And it is not a good idea to use both type of connector together because of their unlike load-slip behavior.

Keywords: *Connection, hybrid structure, load transfer, load-slip behavior.*

1. Introduction:

Steel concrete hybrid structure is composed of the composite structure and the mixed structure. Hybrid steel and concrete elements can take many forms. Examples include the steel girder with concrete slab, concrete pier with steel girder, steel framing of a building with the concrete floor slabs, the encasement of a steel element with concrete, or the filling of a steel hollow section with concrete. These hybrid elements rely on the transfer of force between the two materials in order to realize the benefits of hybrid action. Benefits can include an increase in strength and stiffness as well as the restraint of buckling instabilities in the steel or confinement of the concrete. Hybrid action can be achieved through mechanical connection between the steel and concrete members or elements. The problem with those hybrid structures is that they are neither reinforced concrete structure in the sense of Eurocode 2 or ACI318, nor composite steel concrete structure in the sense of Eurocode 4 or ASCI2010. The problems with hybrid element design are mostly related to the problem of force transmission between concrete and embedded steel profiles, a situation in which it is not known how to combine the resistances provided by bond, by stud connectors and by plate bearings, and how to reinforced the concrete in the transition zone.

To investigate the complicated behavior of the hybrid structures or its components, the experimental investigation is the key resource. Besides the experimental investigation, numerical evaluation also plays a significant role to examine the structural behavior and mechanical properties of hybrid structures. To conduct experiment with varying geometric properties is a time consuming matter, whereas numerical analysis can easily check the effect of any variation. In this case my study involves numerical analysis of some hybrid structural component.

2. Definition of work:

The aim of this report is to design and numerical analysis of composite structural elements to study the transfer mechanism of compression or tension forces from the steel to the concrete. It includes theoretical design with existing codes and numerical modeling of some composite specimens. The study mainly involve with the investigation of the load transfer from the steel profile to the concrete wall without creating local disturbances, like transverse cracking or splitting of concrete around the steel

In this case we consider some reinforced concrete specimen with embedded steel profile. The failure load should be less than the concrete and steel profile capacity to confirm the failure occurs in the connection between steel and concrete. In this study two types of connection is used, one is stud connector and another is plate bearing. To achieve the goal of this thesis, this study has been done with some similar specimen with different configuration of shear connectors, plate bearings.

Five tests specimens are taken into consideration as described below: (Figure: 2.1)

A. steel profile strong axis perpendicular to wall face with flexible connectors on total length of the steel encased profile; polystyrene at the end of the steel profile (Configuration-A)
B. steel profile strong axis perpendicular to wall face with stiff connectors (Configuration-B)
C. steel profile weak axis perpendicular to wall face with flexible connectors (Configuration-C)
D. steel profile weak axis perpendicular to wall face with stiff connectors (Configuration-D)
E. steel profile weak axis perpendicular to wall face with flexible and stiff connectors (Configuration-E)

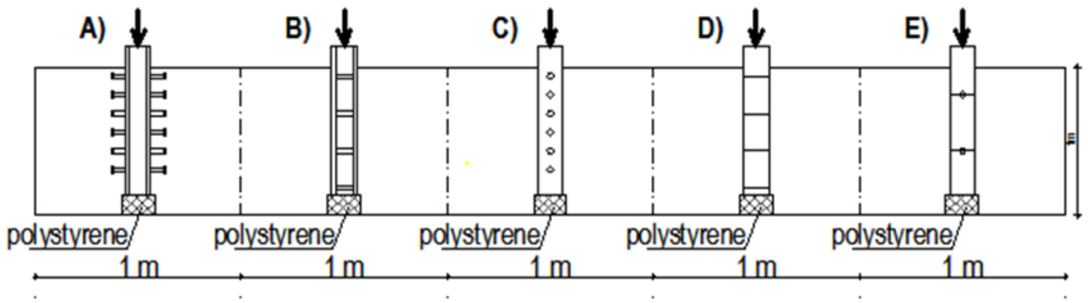


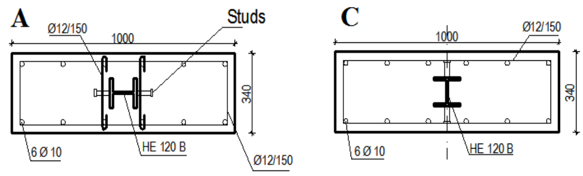
Figure 2.1 specimen configuration

3. Design of Specimens:

In the entire specimen, similar dimension of concrete section of C40/50 grade is used while designing those specimens. A steel profile HE120B of S355 steel grade is embedded inside every specimen with connector; the specimens are reinforced by 10mm and 12 mm diameter reinforcement of S500 grade.

The theoretical designs of all the specimens are done according to Eurocode. As in all the specimens the same concrete section and steel profile is used, so there capacity is remain same. The compression capacity of concrete is 13587kN and plastic resistance of the embedded steel profile is 1207kN. The design capacity for the connectors of each specimen is taken in design is $N_{max} = 1000kN$, in order to achieve the failure in the load transferring mechanism.

3.1 Specimen with stud connectors (A&C):



EC 4.1.- §6.6.3.1. (1) gives the individual shear connector characteristic strength:

$$P_{Rk} = \min\left(\frac{0.8 \cdot f_u \cdot \pi \cdot d^2/4}{\gamma_V}, \frac{0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} \right) = 80.43kN$$

Where,

d = 16 mm – diameter of the shear stud;

h_{sc} = 65 mm – stud height; 3d = 48 mm ≤ h_{sc} ;

s_c = 200 mm – longitudinal spacing; 5d = 80 mm ≤ s_c ≤ min(6h_{sc}; 800mm)=390mm;

f_u = 500 MPa – maximum stud tensile strength;

f_{ck} = 40Mpa – Concrete Characteristic strength

$\gamma_v = 1.00$;

$$\alpha = \begin{cases} 0.2\left(\frac{h_{sc}}{d}+1\right) & \text{for } 3 \le \frac{h_{sc}}{d} \le 4 \\ 1 \text{ for } \frac{h_{sc}}{d} > 4 \end{cases} = 1;$$

The necessary number of shear studs is:

$$\frac{N_{max}}{P_{Rk}} = \frac{1000kN}{80.43kN} = 12.43.$$

The strength capacity of n_{studs} = 12 shear studs is equal to: $N_{Rd} = n_{studs} \cdot P_{Rk}$= 965.1 kN

3.2 Specimen with plate connectors (B&D):

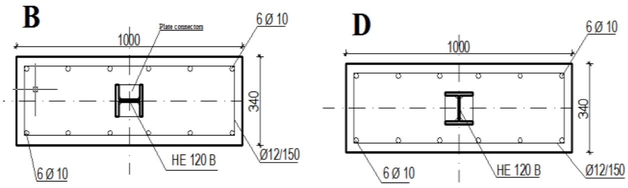


Plate connectors are welded to the web of the steel profile HE 120B shape. They have the following geometrical characteristics:

- Width of the plate: $a = \frac{b_f - t_w}{2} = 56.75mm$
- Length of the plate: $b^* = h - 2 \cdot t_f = 98mm$
- Width of the clipped corners: $c = 15mm$

Area: $A_{plate} = a \cdot b - c^2 = 53.36cm^2$

The plate connector strength is determined considering the following strut & tie model, as shown in Figure 3.2.1 It is considered that the struts are formed assuming an angle 45°

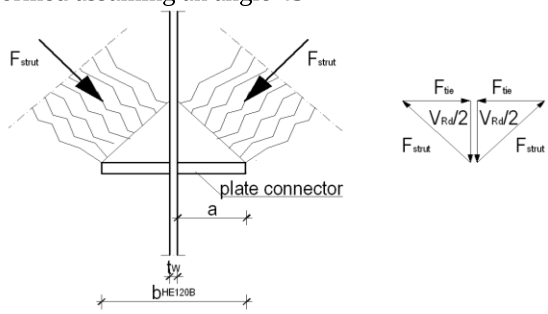


Figure 3.2.1: Strut & tie model to determine plate connector strength

Strut width is equal to:

$$\frac{a}{\cos\theta} = \frac{a}{\sqrt{2}/2} = 80.26mm$$

The strut resistance:

$$F_{Rd} = \frac{a}{\sqrt{2}} \cdot \sigma_{Rd,max} \cdot b^* = 158.56kN$$

where:

$$\upsilon' = 1 - \frac{f_{ck}}{250} = 0.84$$

$$\sigma_{Rd,max} = 0.6 \cdot \upsilon' \cdot f_{cd} = 20.16MPa$$

f_{cd} = 40MPa

For one plate:

$$V_{Rd,1plate} = F_{Rd} \cdot \cos\theta = 112.12kN$$

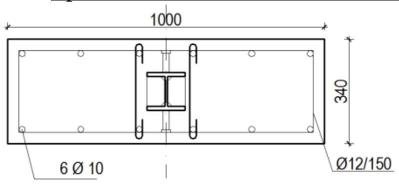
The necessary plate number is:

$$n_{plates} = \frac{N_{max}}{V_{Rd,1plate}} = 8{,}9$$

The strength capacity of n_{plates} = 8 plate connectors, 4 plates on each side, is equal to:

$N_{Rd} = n_{plates} \cdot V_{Rd,\ 1plate}$ = 896.96 kN.

3.3 Specimen with both connectors (E):



It is considered that the compressive axial force N_{max} = 1000kN is resisted by both shear studs and plate connectors. The total number of plate connectors is obtained by the formula:

$$\frac{N_{max}/2}{V_{Rd,1plate}} = 4.46$$

where:

$$V_{Rd,1plate} = 112.12kN$$

The number of shear studs needed is:

$$\frac{N_{max}/2}{P_{Rk}} = 6.217$$

where:

P_{Rk} = 80.45 kN;

For $n_{plates} = n_{studs}$ = 4 the resistance of connectors is:

$$N_{Rd} = n_{plates} \cdot V_{Rd,1plate} + n_{studs} \cdot P_{Rk} = 770.18kN$$

3.4 Summary:

Specimen	Nos. of connectors	N_{Rd} [kN]	Failure mode
Config. A & C	12 studs	965.10	Connection
Config. B & D	8 plate	896.96	Connection
Config.E	4 studs+ 4 plate	770.18	Connection

4. Numerical Investigation:

In numerical analysis a program for nonlinear analysis of two-dimensional reinforced concrete membrane structures VecTor2.0 is used. In this program the stress-strain relationship for concrete is define according to Popovics (1973).

4.1 Specimen with stud connectors (A&C):

To simulate the effects of stud a reinforcing bar is used in mdeling the Specimen A and C in VecTor2.0. And the properties of the steel bar are determined from the properties and behavior of the shear stud by bi-linearization. The load-slip relationship of shear studs is obtained from an article (ISSN 0974-5904, Volume 04, No-06 SPL) by Makki Abbass. The failure load is considered as the same as theoretical design.

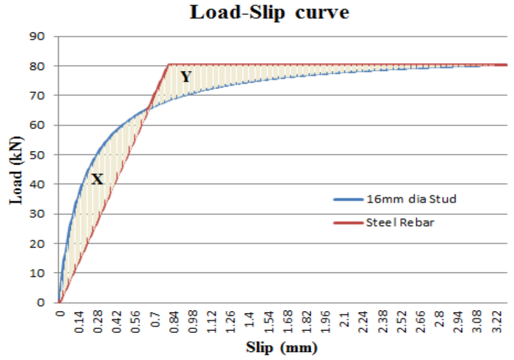


Fig 4.1.1 Bi-linearization of steel rebar

Now in the specimen a push-out test is performed by applying load in the steel profile vertically and that load is transmitted through the connector inside the concrete.

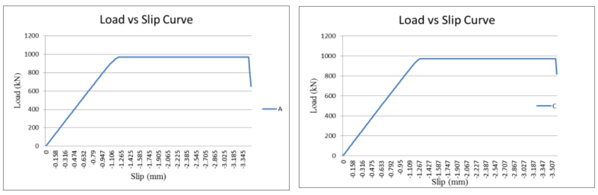


Fig 4.1.2: Load-slip curve for A and C

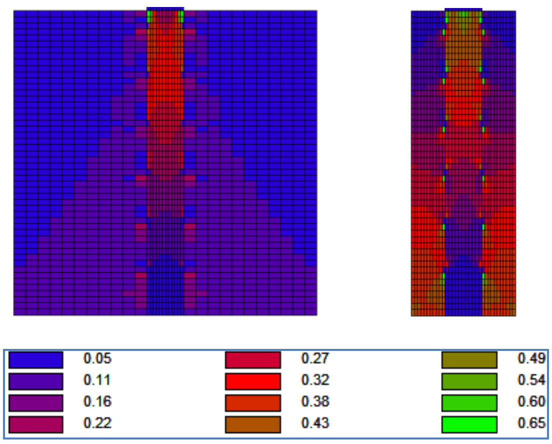


Fig 4.1.3: Force-Capacity ratio for A and C

From the result it can be justify if the concrete fails before the studs. As the stresses in concrete are less than its capacity so it is clear that the failure will be in the stud connection.

4.2 Specimen with plate connectors (B&D):

It is considered that concrete is perfectly bonded 60mm with flange at 190mm spacing, because the spacing between the plates is 250mm. There are 4 bonds of 60mm to represent 4 plates in each side. To define the concrete and steel profile here layer of element one top of another has been used.

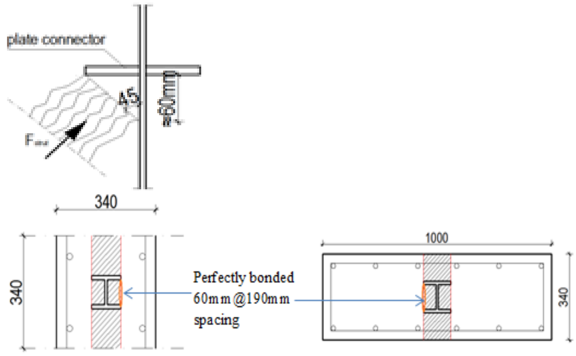


Fig 4.2.1: Modeling of plate connector

Now there is a problem with this model is, if I use one layer of material in top of another layer of material then both elements are perfectly bonded in those place. To solve this, the concrete parts where the load expected to transmit are separated from the profile and connected in the bonded part. Now it become like a strip of material which connected in the 60mm bonded zone. So only bonded part is transmitting the load to concrete.

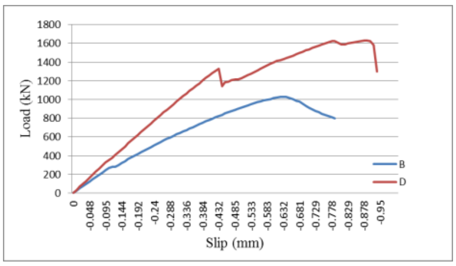


Fig 4.2.2: Load-slip curve for B and D

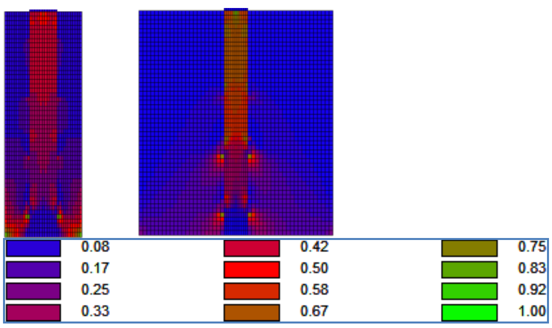


Fig 4.2.3: Force-Capacity ratio for B and D

From the result above its clearly seen that stresses in concrete is considerable higher in the cracked zone and it reached its capacity on the other hand the stresses reinforcement is much lesser than its capacity. So in both case the failure is occurring in concrete

4.3 Specimen with both connectors (E):

This specimen has both plate connector and shear stud. It has total 4 plate connector and 4 shear studs, where 2 in each side. To define different types of connector in different side in 2D in this proposed model, the specimen for the plate connector and shear studs are modeled separately. The analysis for each type of connector is done in separate model and later calculation is done together to get the combine effect. For the stud here the modeling of the specimen is done as it is described before in case of specimen A and B.

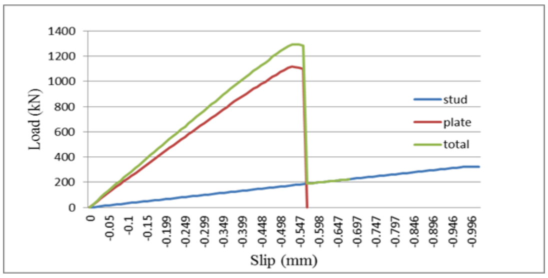


Fig 4.3.1: Load-slip curve for E

The plate connectors are stiffer then shear studs. So from the load-slip curve it is clear that specimen with plate connector fail at very high restraining force but in small slip value and specimen with stud fails at high slip value. Now for the combine effect, the failure of the specimen is occurred at the same slip value when the specimen with plate connector fails. So the total restraining force is the sum of the failure load of plate connectors and load corresponding to the same slip value for stud failure.

4.4 Summary of result:

Specimen	Theoretical design (kN)	FEM capacity (kN)
A	965	970
C	965	970
B	897	1027
D	897	1629
E	707	1295

5. Conclusion:

From the above study following statement can be conclude:

- Failure occurs in studs in both configuration of flexible connector before concrete and reinforcement. This type of connector dose not exhibits similar load-slip relationship like steel profile and concrete.
- Rigid connector shows more restrain capacity because of similar load-slip relation with concrete and configuration with higher concrete section has more capacity.
- In configuration with rigid connector probability of failure occurring in the connection by plate itself
- Flexible connectors cannot actually take part in load transfer mechanism as much as rigid connector where both type of connector is used.

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