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Response of steel-concrete composite panels to in-plane loading

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ABSTRACT

Steel-Concrete Composite (SCC) panels consist of steel faceplates with welded shear studs and concrete infill. The shear studs, which perform a similar function to bond of rebars in reinforced concrete, act as springs resisting the shear slip between the faceplates and concrete. In spite of extensive research in the 1980s and 1990s due to the interest in using SCC in offshore construction and in nuclear power plants, and recently related Codes, there is no analytical method to-date to predict the shear studs response to in-plane loading. Shear connectors are sized according to various criteria unrelated to their actual forces under in-plane loads, such as prevention of faceplate buckling or out-of-plane shear resistance. This paper presents a closed analytical solution of the equilibrium and compatibility differential equations for steel and concrete displacements of SCC panels, based on distributed shear springs idealization. Analytical results presented in this paper are validated by test results of SCC panels loaded by pure shear forces and can be used as practical design formulas for the in-plane portion of the design loads.

1. Introduction

Steel-Concrete Composite (SCC) panels are built as steel boxes with concrete infill. To act together as a composite structure, as bond between steel plates and concrete is neglected, the steel plates have an array of welded shear studs on the inside faces. The shear studs, which perform a similar function to bond of rebars in reinforced concrete, act as springs resisting the shear slip between the faceplates and concrete. Due to the interest in offshore construction during the 1980s, a number of extensive research programs into the behavior and failure mechanisms of composite structures have been carried out in Canada, Japan, Great Britain and United States (Fukumoto et al., 1987; Matsuishi and Iwata, 1987; Ohno et al., 1987; Akiyama et al., 1989, 1991). Renewed interest in SCC use in nuclear power plants resulted in extensive research in the 1990s, the bulk of it performed in Japan (Usami et al., 1995; Suzuki et al., 1995; Ozaki et al., 2004), and in the publication of an SCC Code of Practice (JEAG 4608, 2005), issued by the Nuclear Standards Committee of Japan Electrical Association. A similar Code is currently under preparation in US (Appendix N9 of AISC, 2010). Significant experiment research has been recently performed at Purdue University as part of Westinghouse AP1000 test program. In spite of this, there is no analytical method to-date to predict the shear studs response to in-plane loading. Shear connectors are sized according to various criteria unrelated to their actual forces under in-plane loads, such as prevention of faceplate buckling or out-of-plane shear resistance. An alternative method is to use Par. Q1.11.4 of AISC N690-94 (AISC, 1994), which applies to composite action of steel beams with concrete slabs under out-of-plane loading, requiring the shear studs to develop one half of the yield stress in the faceplate between the point of maximum positive moment and point of zero moment. Several studies, using "push-out" tests of shear studs, were performed to assess their shear-slip characteristics in composite structures (Viest, 1956; Ollgaard et al., 1971). They determined that the load slip curves are non-linear due to a combination of crushing of the concrete in the concrete wedge in front of the stud and flexural/shear yield of the stud. Nonetheless, for practical applications, the curves may be reasonably idealized by a linear spring coefficient K, the latter proportional to the concrete strength and the shear stud area. This paper presents a closed analytical solution of the equilibrium and compatibility differential equations for steel and concrete displacements of SCC panels, based on the shear studs being modeled as uniformly distributed shear springs over the steel plate area.

2. Analytical methodology

2.1. Smeared connectivity of the welded shear studs

Shear studs spacing is relatively small in comparison to the SCC panels plan dimensions, typically a few percent of the latter. As a result of this it is reasonable to treat the steel to concrete connectivity as evenly distributed (or "smeared"), in a similar manner to reinforced concrete connectivity to rebars. The elastic shear load-slip function of the distributed interface forces f may then be

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expressed by the relationship $f = ku$, where u is the slip and $k = K$ (linear spring coefficient)/(horizontal spacing × vertical spacing).

2.2. General bi-axial loading

SCC panels are subjected to a set of member forces TX^0, TY^0 and TXY^0 which are computed in the orthogonal X Y Z system of the structural FE model. As a result of this, the panels response should be assessed by first calculating the orientation of the principal axes, denoted X and Y in Fig. 1, and the corresponding principal forces TX and TY. The forces are shown in absolute values, their sign depending upon orientation, being either tension or compression. Depending upon the actual sign of the forces, three different configurations are shown in Fig. 2, namely bi-axial compression, bi-axial tension, and tension–compression. As the bi-axial compression case is usually treated as monolithic, only the second and the third mechanism are of interest to composite action. Pure shear is a special case of the latter, in which the compression and tension have equal absolute values.

2.3. Critical orientation and post-cracking composite mechanism

From a design viewpoint, the most severe principal axes orientation is the one that results in the highest tensile loading in the surface plates, as a result of the concrete cracking and shear studs deformations associated with tensile stresses in excess of the concrete tensile capacity. This critical orientation, that can be either X or Y, depending on the principal axes calculation, is referred to as "r" in the following (see Fig. 1). The analytical development is formulated by considering equilibrium and compatibility conditions in this direction, and the Poisson's ratio relationships to the stresses and strains in the principal axis normal to it, denoted by "n".

Monolithic elastic behavior, in which steel and concrete have compatible strains, is assumed up to the initiation of cracking. The crack spacing (see Fig. 3), which can decrease throughout the loading, is denoted by "a". Beyond this point, we will look at the deformation mechanism involving the composite post-cracking action between the steel plates, smeared shear studs and in-fill concrete. Once the cracks are initiated, the tensile force on concrete is transferred to the steel plate by the shear studs. As a result of this, the steel stress varies between the concrete crack planes from a minimum value at the mid-point between cracks to a maximum at cracks location. The applied axial tensile force is resisted by a combination of two mechanisms, namely the axial stiffness of the steel plates and the composite action of the in-fill concrete with the shear studs and the steel plate. For the n-direction, forces, stresses and strains are uniform, in case of monolithic response to compression, or varying between the concrete crack planes in case of tension, similarly to the r-direction.

The following general notation is used in the analytical development of the post-cracking mechanism:

T_r = r-direction of total tensile force per unit length, applied on steel plates at crack locations

T_n = n-direction of total force per unit length. If tensile, it is applied as tension on steel plates at crack locations. If compression, it is resisted monolithically by the SCC section.

Ψ = the ratio of T_n to T_r, constant throughout the loading

r = distance along the critical principal axis, measured from mid-point between cracks to point of stress or strain calculation.

υ_s = Steel Poisson's ratio.

The following notation is specific to the critical tensile r-direction

$T_{sr}(r)$ = steel plate tensile force, varying between the concrete crack planes from a minimum value at the mid-point between cracks to a maximum of $T_r/2$ at cracks location

$\sigma_{sr}(r)$ = steel plate stress = $0.5\,T_{sr}(r)/t_s$ where t_s = steel plate thickness.

$u_{sr}(r)$ = steel plate displacement

$\varepsilon_{sr}(r)$ = steel plate strain = $du_{sr}(r)/dr$

σ_{spr} = steel plate stress at crack location = $\sigma_{sr}(a/2) = 0.5\,T_r/t_s$

ε_{spr} = average steel plate strain

$\sigma_{cr}(r)$ = concrete stress

$u_{cr}(r)$ = concrete displacement

$\varepsilon_{cr}(r)$ = concrete strain = $du_{cr}(r)/dr$.

The following notation is specific to the n-direction

Steel plate force = variable value $T_{sn}(n)$ for tension case, varying between the concrete crack planes from a minimum value at the mid-point between cracks to a maximum of T_{spn} at $n = a/2$, or uniform value T_{spn} for compression

$\beta = T_{spn}/T_n$, is equal to less than 1.0 in case of monolithic response to compression, or 1.0 for tension

Steel plate stress = variable value $\sigma_{sn}(n) = 0.5\,T_{sn}(n)/t_s$, for tension case, or uniform value $\sigma_{spn} = 0.5\,T_{spn}/t_s$, for compression

Also, σ_{spn} =steel plate stress at crack location (for tensile case) = $\sigma_{sn}(a/2) = 0.5\,T_{spn}/t_s$

$u_{sn}(n)$ = steel plate displacement

$\varepsilon_{sn}(n)$ = steel plate strain = $du_s(n)/dn$

ε_{spn} = average steel plate strain

2.4. Poisson' s ratio effect

The r-direction stresses are related to the strains in both directions by:

$$\sigma_{sr}(r) = \frac{E_s\left[\varepsilon_{sr}(r) - \nu_s \varepsilon_{sn}(n)\right]}{\left(1 - {\nu_s}^2\right) t_s} \tag{1}$$

As a result of the Poisson's ratio effect being a secondary order of magnitude, the differential equations can be reduced to one variable by replacing $\varepsilon_{sn}(n)$ with a uniform value ε_{spn}, calculated from:

$$\varepsilon_{spn} = \frac{\sigma_{spn} - \nu_s \sigma_{spr}}{E_s} \tag{2}$$

where

$$\sigma_{spr} = \frac{0.5T_r}{t_s} \tag{3}$$

$$\sigma_{spn} = \frac{0.5T_{spn}}{t_s} \tag{4}$$

Substitution of Eqs. (3) and (4) into Eq. (2) results in:

$$\varepsilon_{spn} = \frac{T_r\left(\psi\beta - \nu_s\right)}{2t_s E_s} \tag{5}$$

where

$$\Psi = \frac{T_n}{T_r} \tag{6}$$

$$\beta = \frac{T_{spn}}{T_n} \tag{7}$$

By definition, in case of tension, we have $\beta = 1.0$. For compression (see Fig. 4), steel and concrete act monolithically, which results in:

$$\beta = \frac{1}{1 + \phi} \tag{8}$$

where

$$\phi = \frac{t_C}{2nt_s} \tag{9}$$

and $n = E_s/E_c$ where E_s and E_c are steel and concrete Young's Moduli.

The r-direction displacements of the steel plates and of the concrete in-fill are compatible at the mid-point $r = 0$ between cracks.