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# Numerical analysis and punching shear fracture based design of longitudinal plate to concrete-filled CHS connections

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## HIGHLIGHTS

- Mechanical behaviour of concrete-filled CHS plate connections was numerically investigated.
- A criterion for ductile fracture under low stress triaxiality conditions was employed into FEA.
- Theoretical and analytical models were established for connections under various loadings.
- Ductile fracture based design methods were proposed considering effect of inner concrete.

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## ABSTRACT

The mechanical behaviour of longitudinal plate-to-concrete-filled circular hollow section (CHS) connections under axial tension, eccentric tension and in-plane bending is extensively studied by the experimentally validated finite element analysis (FEA) in this paper. A total of 336 connections with a wide range of parameters on geometrical configurations, material properties and load positions was conducted to investigate (a) the general applicability of the experimental conclusion for the governing limit state, (b) the shear stress profile on the failure face and (c) the design equations based on fracture analytical models under various loading conditions. FEA extended the validity of experimental conclusion that the only governing limit state was punching shear failure instead of the deformation limit of 3% chord diameter (*D*). With an aim of proposing design equations based on ductile fracture mechanics, the stress distributions on the fracture failure face and the inner concrete were investigated by the parametric study, and then were adopted in the analytical models. Finally, design equations based on semi-theoretical models for the ultimate strength of longitudinal plate-to-concrete-filled CHS connections under three investigated loads were proposed. It is found the connection-capacity predictions agreed with both test and FEA results well.



## 1. Introduction

Concrete-filled steel circular hollow section (CHS) member is a popular alternative to hollow structural section (HSS) one especially when subjected to compression. It is due to the efficient utilization of the material strengths for both concrete and steel, leading to a better local buckling resistance and a higher ultimate strength, especially for those thin-walled members [1,2,3]. A simple and efficient way to connect those circular hollow section (CHS) members with branch members is to use a gusset plate, including longitudinal and transvers ones which are directly welded on the tube wall surface. Fig. 1 gives an example of a gusset plate-to-tube connection used in a large-span transmission tower in China. The longitudinal plate connection original from branch plate-to-I-beam connection is a traditional and common connection type especially for those with lightly loaded branch members. Because of the flexibility of the thin tube-wall face which often causes the excessive deformation combined with chord ovalization, the capacity of those branch plate-to-CHS connections are thus often governed by the deformation limit state. Therefore special precautions, such as connection stiffening, are always taken into consideration in practical design.

Previous studies have proposed relevant methods to stiffen those tubular connections with a flexible face, such as a through plate connection [4,5,6], an annular ring stiffened connection [7,8,9], a concrete-filled connection [6,10]. It is found that for the

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**Nomenclature**

$A$ parameters of material strain hardening;
$c_1$ friction coefficient in Mohr-Coulomb model;
$c_2$ shear stress resistance in Mohr-Coulomb model;
$D$ outer diameter of the chord member;
$e$ load eccentricity;
$e_{con}$ flow potential eccentricity in the concrete damaged plasticity model;
$E_s$ Young's modulus of steel;
$E_c$ Young's modulus of concrete;
$f_c$ concrete compressive design strength;
$f_{cu}$ concrete compressive cube strength;
$f_{u,v}$ ultimate shear strength of steel;
$f_{v,max}$, $f_{v,min}$ maximum and minimum shear stress on the punching shear face in the case of tension;
$f_{v,cmax}$, $f_{v,tmax}$ maximum shear stress on the punching shear face at the compressive and tensile side faces in the case of in-plane bending respectively;
$f_{\tau,AX}$, $f_{\tau,IB}$ shear stress distribution on the punching shear face;
$F_{EXP}$, $F_{FEA}$ ultimate strength obtained from tests and FEA;
$F_{u,AX}$, $F_{u,AX\_weld}$ ultimate strength obtained from design equations in case for axial tension with and without the weld effect respectively;
$F_{u,E}$ and $F_{u,E\_weld}$ ultimate strength obtained from design equations in case eccentric tension with and without the weld effect respectively;
$F_y$ yield strength of steel;
$F_u$ ultimate strength of steel;
$F_{V1,IB}$, $F_{V2,IB}$ sum of shear stress on the tensile side for side and end failure faces in case of in-plane bending, respectively;
$k_{AX}$ ratio of $f_{v,min}$ to $f_{v,max}$ in the case of axial tension;
$K_c$ the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian in the concrete damaged plasticity model;
$L$ length of the chord;
$\overline{L_b}$ position parameter defined in Fig. 10(a);
$l_b$ longitudinal length of plate;
$l_{bp}$ bolt-plate length;
$h_b$ plate height;
$M_{EXP}$, $M_{FEA}$, $M_{u,IB}$, $M_{u\_weld}$ ultimate moment strength obtained from tests, FEA and proposed design equations (with and without the weld effect);
$n$ parameters of material strain hardening;
$O_{c,IB}$, $O_{t,IB}$ the resultant point of tension and compressive side in the case of in-plane bending;
$\overline{T_b}$ a relative position parameter $(\Delta t_b)/t_b$ defined in the same way as $\overline{L_b}$;
$t$ chord-wall thickness;
$t_b$ longitudinal plate thickness;
$t_{bp}$ bolt-plate thickness;
$Z_{c,IB}$, $Z_{t,IB}$ distance from compressive and tensile resultant points to the neutral axis respectively;
$Z_{IB}$ internal lever arm of the moment section;
$\varepsilon_f$ elongation (tensile strain) after fracture based on a gauge length of 50 mm;
$\beta$ ratio of plate thickness to chord outer diameter;
$\eta$ ratio of plate longitudinal length to chord outer diameter;
$\gamma$ ratio of chord outer radius to chord thickness;
$\eta_b$ ratio of length of loading area to plate in the longitudinal direction;
$\theta$ angle shown in Fig. 10(a);
$\tau$ ratio of plate thickness to chord thickness;
$n_0$, $n_1$, $n_2$ the position of $f_{v,cmax}$, neutral axis and $f_{v,tmax}$ in the shear stress profile in case of in-plane bending;
$\mu$ viscosity parameter in the concrete damaged plasticity model;
$f_{b0}$, $f_{c0}$ initial equibiaxial compressive yield stress and initial uniaxial compressive yield stress in the concrete damaged plasticity model;
$\Phi_s$ dilation angle measured in the p-q plane in the concrete damaged plasticity model.

first two mentioned stiffened connections (for hollow tubes), though the tube wall deformation was decreased and the flexibility of connection face was mitigated, the capacity was still governed by the deformation limit failing at the state of chord-wall plastification; while for the concrete-filled connections, Voth [6] and Xu, et al. [10] showed that punching shear failure of chord-wall was the dominant limit state. They also indicated that the branch plate-to-CHS connections utilized the full strength of steel material. However the design recommendations for punching shear strength in CIDECT-1 [11] for non-concrete-filled plate connection, is generally conservative when applied to the inner concrete stiffened ones especially in the case of in-plane bending [10].

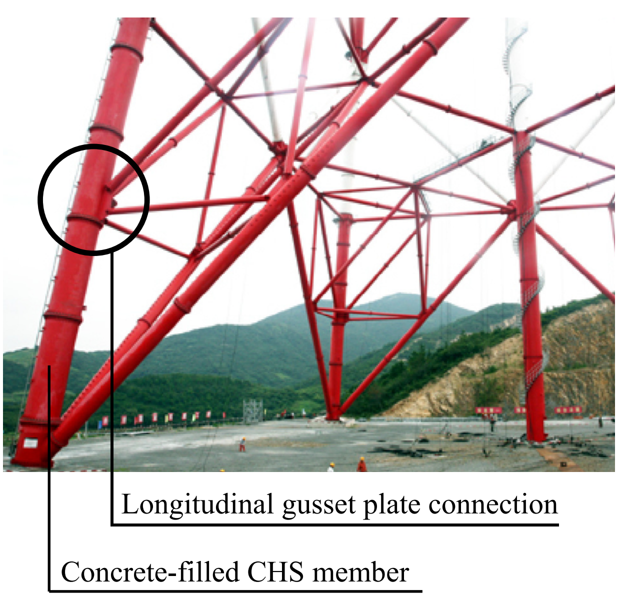


**Fig. 1.** Large-span transmission tower in China.

Significant research has been conducted on the behaviour and design methods of branch plate-to-HSS connection [5,12–18], some of which have been adopted in current design guidelines, such as CIDECT-1 [11] and CIDECT-3 [19] and codes of practice, API [20] and AISC 360–10 [21]. Unfortunately, limited experimental and numerical research [6,10] are available for the behaviour of plate-to-concrete-filled CHS connections, therefore there is a need to conduct complementary numerical simulation to propose design equations which are based on both experimental and numerical data. Thus, the method of finite element analysis (FEA) is employed to extend the scope of test specimens for both geometrical configurations and material properties. By a combined way of experimental and numerical investigations, the design recommendations for branch plate-to-concrete-filled CHS connections can be proposed with both a wider validity range and considerable reliability.

In this study, the mechanical behaviour of longitudinal plate-to-concrete-filled CHS connections under axial tension, eccentric ten-