

SEISMIC PERFORMANCE ASSESSMENT OF COMPOSITE FRAMES INCLUDING SHEAR PANEL EFFECTS

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SUMMARY

Shear panel deformations are known to have a significant influence on the response of steel structures under seismic loading conditions. The behaviour of the shear panel has a direct influence on both lateral stiffness and capacity of a moment-resisting steel frame. These effects are of even more significance in the case of composite steel-concrete moment-resisting frames. However, whereas well-established analytical models are available for representing the shear panel behaviour in steel frames, it can be shown that these models cannot be reliably used in the case of composite systems due to considerable differences in fundamental behavioural aspects.

This paper assesses the seismic response of composite steel-concrete moment-resisting frames with particular emphasis on the influence of shear panel effects. Firstly, analytical models available in the literature for representing shear panel behaviour in steel frames are reviewed, and their limitations in terms of application to composite frames are demonstrated. This is followed by the presentation of a new modelling approach specifically developed for panel zones in composite frames. The proposed model, which considers realistic boundary conditions in the panel zone, provides a more accurate and reliable representation of the response. The new approach is implemented within the advanced program ADAPTIC, which accounts for material and geometric nonlinearities. Details of the model are described and comparisons with available experimental data and alternative modelling techniques are presented for validation.

INTRODUCTION

Panel zone deformations play an important role in the seismic response of moment resisting frames. Under lateral loading conditions, the joints become subjected to unbalanced bending moments causing significant deformations in the panel zones. This effect can have significant influence on both the stiffness and strength of the frame. Additionally, panel zones usually contribute to energy dissipation under seismic loading, hence it is important to consider these components in analytical and design studies.

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Research in the past [1,2,3,4,5,6] has shown that the panel zone has a very ductile and stable behaviour. Also, the concentration of some inelasticity in this region can relieve the demand imposed in the beams [7]. However, some control must be applied during the design in order to limit the yielding strains in the panel zone. Excessive deformations in this element may also impair the global structural behaviour, and introduce additional second order effects. In order to address this issue, recent publications such as FEMA 350 [7] suggest that the initiation of yielding both in the beam and in the panel zone should occur nearly at the same lateral load level, in order to control the extent of inelastic distortion in the panel.

The inclusion of panel zones in numerical models of moment resisting frames is thus essential for accurate seismic response assessment. Different techniques have been suggested by several researchers [1,8,9]. All available models have been mainly derived for steel joints for which the unbalanced moment to be transmitted from the beam(s) to the column can be converted into a couple of forces subjecting the panel zone to idealised pure shear strain conditions. While this approach is valid for steel joints, it is not representative of a composite joint in which the panel zone is subjected to more irregular stress conditions which depend on the location of the neutral axes in the composite beam(s) connected to the joint. A more realistic approach is proposed in this paper to represent the panel zone response in both steel and composite joints where the key parameters associated with the joint type are incorporated.

PANEL ZONE BEHAVIOUR

In a joint under unbalanced bending moments (Figure 1), a complex stress state develops in the panel zone which is composed by normal stresses from the column and by shear stresses due to bending moment transmitted from the connected beam(s). At initial stages, the panel behaviour is governed by shear deformations until yielding in shear takes place. After that stage, extra resistance is provided by the "boundary frame" consisting of the column flanges and of the web stiffeners if these are present. When the column flanges yield, kinks form in the intersection with the beam and the strain-hardening stage begins. Due to its typical dimensions, these elements exhibit inelastic buckling for very large level of deformations which are usually not reached due to strict drift limitations imposed by design codes.

According to experimental observations, panel zones have very stable behaviour for cyclic loading conditions which makes them effective components for energy dissipation in a frame and also in reducing the ductility demand to the beams. Such stable behaviour must however be considered with care since large panel deformations may impair the overall structural response to lateral loading scenarios since it is coupled with significant second order effects.



Figure 1 – Panel zone region and boundary conditions

In general, the inelastic behaviour of the panel zone can contribute to relieving the demand on other structural elements. Nevertheless, the design should ensure that yielding in the panel occurs almost concurrently with flexural yielding in the connected beams [7].

AVAILABLE MODELLING TECHNIQUES

As mentioned before, various modelling techniques for representing panel zones have been suggested by several researchers. Two main approaches are usually adopted. The first one is termed the *scissors* model (Figure 2) and consists of introducing a rotational spring at the beam to column intersection in order to model the relative rotation of the two members that occurs due to the panel deformation. Additional rigid links may be used in order to represent the stiff region of the beam and the column within the joint region. The second approach, termed in this paper as the *frame* model, consists of a more realistic representation of the panel zone where the actual dimensions are considered as shown in Figure 3. An assemblage of links is used and a diagonal axial spring is adopted representing the panel region properties. This second modelling technique is more realistic as it can reproduce both the relative rotation between the beams and the column as well as the relative vertical translation between the beams connected to the joint.



For both the approaches, force-displacement relationships have to be assigned to the springs. Expressions were derived by a number of researchers [1,2,10,11,12] mainly differing from each other in addressing the post-elastic range. Most of these expressions were largely derived on the basis of a common assumption for the mechanism of moment transmission from the beam to the column. The panel is effectively assumed to have rigid boundaries and to behave under a pure shear stress state. This assumption allows the conversion of the unbalanced bending moment into a couple of horizontal forces which simplifies the problem and gives rise to sets of simple analytical expressions for the idealised springs. In subsequent sections, a brief review of existing representations for moment-distortion (M- γ) relationships available for both the elastic and post-yield range is presented. These relationships can be adopted for use in both the scissors and frame models.

Elastic Range

For the elastic behaviour of the panel zone, the following expression is commonly suggested [2]:

$$M = \frac{G * A_v * d_b}{1 - \rho} * \gamma \tag{1}$$

where G is the shear modulus of the material, A_v is the shear area, d_b is the steel beam height and ρ is a parameter that accounts for the beneficial effect of the shear force in the column which is defined by $V_{col}*d_b / M$. The main difference between the various proposals for the elastic stiffness is concerned with the shear area. For example, Krawinkler *et al* [2] used $A_v=(d_c-t_{cf})*t_{cw}$, while Fielding and Huang [1] proposed $A_v=d_c*t_{cw}$. It is worth noting that important differences may arise using the two expressions when deep columns are considered.

Based on Equation (1) and limiting the yielding shear stress to:

$$\overline{\tau_{y}} = \frac{f_{y}}{\sqrt{3}} * \sqrt{1 - \left(\frac{P}{P_{y}}\right)^{2}}$$
(2)

in which the presence of the axial load in the column (P) is accounted for by employing the von Mises yield criterion (P_y represents the axial capacity of the column), it is possible to define the unbalanced moment in the joint that causes yielding of the panel ($M_{y,pz}$):

$$M_{y,pz} = \frac{A_v * d_b}{1 - \rho} * \overline{\tau_y}$$
(3)

Regardless of the minor differences described, the available proposals for defining the elastic stiffness of the panel are generally accepted as a good representation of steel panel zones.

Post-Elastic Range

While for the elastic range the behaviour is expressed in largely similar terms by various researchers, based on elasticity principles, some differences are evident when describing the post-elastic stage. Different proposals, with increasing refinement, have been suggested in the literature.

Fielding and Huang [1] proposed a bi-linear relationship (Figure 4a) for the panel zone behaviour in which the post-elastic stiffness was, assuming v=0.3, defined by:

$$\Delta M = \frac{5.2 * G * b_c * t_{cf}^3}{d_b * (1 - \rho)} * \Delta \gamma$$
(4)

where b_c and t_{cf} are respectively the width and thickness of the column flange. No limits were considered for the post-elastic range which is unrealistic since at a certain stage yielding takes place in the column flanges. Based on experimental and analytical results, Krawinkler *et al* [2], later proposed a tri-linear representation (Figure 4b) in which the post yielding stiffness was defined as:

$$\Delta M = \frac{1.04 * G * b_c * t_{cf}^2}{(1-\rho)} * \Delta \gamma$$
(5)

An inelastic distortion equals to four times the yield distortion was assumed, after which a strainhardening stiffness based on material properties was suggested.



Figure 4 – Bi-linear and tri-linear relationships

More recently, Kim and Engelhardt [12] proposed a refinement to existing models which consisted of a quadri-linear model including both bending and shear deformation modes. Consideration for the contribution of the column flanges to the resistance of the panel zone at the onset of yielding in the panel was made, clearly taking into account the fact that full yield of the panel does not take place at the same load level. The model derived was also extended to account for cyclic conditions. Comparisons with experimental test results demonstrated that the proposed refinement gave better accuracy than previous models for joints with thick column flanges.

Composite joints

While the techniques described above give generally accurate results for steel joints, its application to composite joints have not been adequately assessed nor validated. Suggestions [4,5,12] have been made for a modification of the panel zone depth in order to account for the presence of the slab. This empirical approach has not been fully justified and its validity has not been undertaken due to limited availability of relevant experimental test data.

In the next section, a new approach is suggested for modelling the panel zone region in composite joints followed by validation with available experimental results as well as detailed numerical simulations.

PROPOSED APPROACH FOR STEEL AND COMPOSITE JOINTS

General

The assumptions made for deriving the models for steel joints are realistic as the moment transmitted to the joint is mainly carried by the beam flanges, hence its conversion into a couple of forces. For the case of a composite joint, the moment develops through bending in the steel beam but a significant component arises from the axial forces present in the slab and in the steel beam. This makes the establishment of a moment-distortion relationship a more difficult task. Recent suggestions for modifying the panel depth are attractive but experimental observation shows that the physical geometry of the panel zone is not different in a composite joint from its steel counterpart. The increase in strength observed in composite joints [5] can therefore only then be explained by the shear stress magnitude and distribution in the panel zone and not through a change of the panel dimensions.

In the proposed model presented in this paper, realistic stress distributions in the edge of the panel are considered so that a more accurate assessment can be obtained about the shear stress distribution and magnitude through the panel depth. The approach also incorporates both shear and bending deformations of the panel when deriving the spring stiffness. The contribution of the column flanges to the extra

resistance of the panel zone is also accounted for and is dependent on the column flange thickness as well as the column depth.

Main Assumptions and Considerations

In the proposed approach, the joint is represented by a modified version of the frame model, as shown in Figure 5. An additional assemblage of links is included on top of the panel zone in order to model the column region in contact with the slab. The panel zone is considered to have the physical dimensions (d_bxd_c) and the "top panel" is assumed to behave elastically. Small rotations are assumed to occur in the column, which allows the assumption that diagonally opposite nodes of the panel do not have relative vertical translation. Regarding the composite beam, preliminary analyses have shown that in most cases the beam remains elastic when the panel yields. Because of this, the composite beam is assumed to be elastic, hence enabling the location of the neutral axis to be defined considering a linear normal stress distribution. The bending moment developed in the slab is ignored in the current approach, since it is thought to have an insignificant effect on the behaviour. Concerning the effective width of slab in the vicinity of the joint, that is assumed to be equal to the column flange width (b_c) for the case where the beam is under sagging moment since that is the contact area of the slab with the column. For the hogging moment case, the slab is not considered as the reinforcement is not anchored to the column. However, different effective widths should be considered for scenarios when the slab extends further from the column. Nevertheless, the cases under study in this paper refer to arrangements where the slab ends at the contact with the column face.



Figure 5 – Modified version of the frame model

The basis of the proposed procedure is essentially to determine the spring properties for both the panel zone and the "top panel". As mentioned above, the latter will behave elastically, hence its stiffness can be found using expressions available in the literature. Regarding the panel spring, this is assigned a tri-linear curve describing the elastic, post-elastic and strain-hardening range. The properties are derived analytically, in such a way that it can be implemented in frame analysis programs. The procedure establishes a parallelism between the analytical model consisting of the real joint and the corresponding numerical model (i.e. the finite element representation of the joint). Figures 6a and 6b illustrate these models for the case of an external composite joint under the transmission of a positive bending moment.



Description

The approach is described for an external composite joint under positive moment but can be easily extended for negative moment and for the case of an internal joint. As mentioned above, the objective is to determine the properties of the diagonal spring representing the panel zone which will have a tri-linear behaviour.

Elastic Range

For a given bending moment to be transmitted to the column, knowing in advance the neutral axis location on the beam, a stress distribution in the edge of the panel can be defined. With that distribution, it is then possible to describe the bending moment and shear force distribution throughout the panel depth and, applying the virtual work method, to calculate the drift (Δ_{rel}^{AB}) between the top and bottom edge of the panel (points A and B in Figure 6a). In the calculation of the drift, both bending and shear deformations are included $(\Delta_{rel}^{AB} = \Delta_{rel,shear}^{AB} + \Delta_{rel,bend}^{AB})$.

On the other hand, the same moment in the numerical model, develops an equivalent shear ($V_{eq,num}$) which is then resisted by the diagonal spring. This equivalent shear is given by:

$$V_{eq,num} = \frac{M_s}{d_b} + \frac{N_s}{2} - V_{col} \tag{6}$$

where $V_{col} = M/h_s$ is the actual shear in the column and h_s is the storey height. The elastic stiffness of the panel zone can then be calculated:

$$K_{el,num} = \frac{V_{eq,num}}{\Delta_{rel}^{AB}}$$
(7)

It should be noted that the constant shear that develops in the numerical model is not necessarily the same as the peak shear force that occurs in the analytical model. A ratio (R_V) relating the equivalent and peak shear can be defined:

$$R_{V} = \frac{V_{analytical}^{\max}}{V_{eq,num}}$$
(8)

This ratio is important as it can be used to define the yield drift of the panel in the numerical model $(\Delta_{y,num})$. Knowing the shear capacity of the panel zone $(V_{y,panel})$:

$$V_{y,panel} = \overline{\tau_y} * A_v \tag{9}$$

it is possible to calculate the drift of the panel at onset of yielding in the numerical model:

$$\Delta_{y,num} = \frac{\frac{V_{y,panel}}{R_V}}{K_{el,num}}$$
(10)

The panel zone diagonal spring properties for the elastic range are then calculated as follows:

$$K_{el,spring} = \frac{K_{el,num}}{\cos^2 \alpha}$$
(11)

$$\Delta_{y,spring} = \frac{\Delta_{y,num}}{\cos \alpha} \tag{12}$$

where α is the angle of the spring in the numerical model. In the next section, the procedure for determining the spring properties for the post-elastic range is described.

Post-Elastic Range

After yielding of the panel zone in shear, extra resistance is provided by the boundary frame consisting of the column flanges and eventually, of the web stiffeners. The post-elastic stiffness ($K_{p-el,num}$) can be calculated based on the drift components obtained before:

$$K_{p-el,num} = \left(\mu * \frac{V_{eq,num}}{\Delta_{rel,shear}^{AB}}\right) + \left(\frac{V_{eq,num}}{\Delta_{rel,bend}^{AB}} * \frac{2 * I_{T-sec}}{I_{col}}\right)$$
(13)

where μ is the strain-hardening parameter, I_{col} is the second moment of area of the column and I_{T-sec} is the second moment of area of one T-section consisting of the column flange and a small part of the column web suggested to be $0.9 * t_{cf} + 0.05 * d_c$.

The relative drift of the panel at hinging formation in the flanges is assumed to be three and a half times the relative drift for yield of the panel ($\Delta_{y,num}$). This yielding point is very complex to assess and the value suggested here was found to give good results for a number of analyses performed within the validation stage. The panel zone spring properties are found in a similar way as that of the elastic range by using equations (11) and (12).

Strain-Hardening Range

The panel zone stiffness for the strain-hardening range (K_{sh,num}) is straightforward and is given by:

$$K_{sh,num} = \mu * \frac{V_{eq,num}}{\Delta_{rel,shear}^{AB}}$$
(14)

The corresponding spring stiffness is again found using Equation (11).

The procedure described above can easily be applied to internal joints. The main difference is related to the moment distribution at the joint. This can be easily found by considering that the ratio between positive and negative moment is proportional to the second moment of areas of the beams connected to the joint. After that, the procedure follows the same sequence of steps and the panel zone spring properties are obtained.

The approach is described in more detail elsewhere [13] where a discussion of the main parameters affecting the panel zone behaviour is made, particularly regarding the R_V factor that should be adopted in different joint types.

The whole procedure described in this chapter can be easily implemented in a spreadsheet or, preferably, in a mathematical programming package. In the next section, a number of comparisons are made with both numerical and experimental results in order to validate this proposed approach.

VALIDATION STUDIES

The proposed approach is validated against detailed numerical as well as available experimental test results of substructures as indicated in Figure 7. Firstly, to validate the applicability of the approach to steel joints, a comparison is made for a specimen tested by Krawinkler *et al* [2]. Then, focus is given to composite joints and comparisons are made both for external as well as internal joints. The new approach is applied to an external composite specimen tested by Lee and Lu [5] followed by two additional comparisons made against 3D detailed substructures prepared in ANSYS [14].



Figure 7 – Types of substructures used for validation

The validation is made for external and internal substructures as these systems represent typical arrangements within a moment-frame under lateral loading conditions where hinges are assumed to form at mid-span of the beams and of the columns. The analyses are carried out either controlling the vertical end displacement of the beams or by controlling the top horizontal displacement of the column. Table I

lists the substructures used in the validation, Table II gives the mechanical properties, and Table III presents the material properties adopted for the models.

Model	Joint type	Location	Node controlled	Model type	Beam(s) span (m)	Storey height (m)
INT_SA2	Steel	Internal	Column	Experim.	4.064	2.032
EXT_CFC	Composite	External	Beam	Experim.	2.3	3.4
EXT_CL45	Composite	External	Beam	Num.	4.5	3.0
INT_CL45	Composite	Internal	Column	Num.	4.5	3.0

Table I – Models adopted for validation

Table II – Mechanical properties of the models

Model	Boom	Column	Slab	
Model	Deam	Column	d _s (mm)	t _{slab} (mm)
INT_SA2	10 B 15	8 WF 24	-	-
EXT_CFC	W 18x35	W 10x60	76	89
EXT_CL45	UB 457x191x82	UC 356x368x177	0	120
INT_CL45	UB 457x191x82	UC 356x368x177	0	120

Table III – Material properties of the models

		Steel	Concrete		
Model	E (kN/mm²)	f _y (N/mm²)	μ (%)	E (kN/mm ²)	f _c (N/mm ²)
INT_SA2	208	282.7+	2.185	-	-
EXT_CFC	219 ⁺	244 ⁺	1.0 [*]	23	43.8
EXT_CL45	210	275	1.0	30	30.0
INT_CL45	210	275	1.0	30	30.0

* assumed value

⁺ value based on column properties

Numerical Models

The new approach introduced in this paper is implemented in the advanced structural analysis program ADAPTIC [15], which accounts for both material and geometrical nonlinearities. The substructures are modelled with 1D finite elements. Both the steel and the column members are modelled using Eulerian cubic plastic elements based on the fibre approach. For composite substructures, the composite beam is modelled with two lines of the same type of elements connected by rigid links representing both the steel beam and the concrete slab located at their centroidal axis. This technique has shown to represent accurately the behaviour of a composite beam [16]. Concerning the slab, simplifications are required both in terms of the effective width and with the interaction with the column. For the latter, rigid plastic

behaviour is considered and a confinement factor is applied for the case of internal joints. Regarding the effective width, this is taken as the column flange width on the vicinity of the joint and the full width elsewhere according to evidence from current research [13]. The models prepared in ANSYS consist of detailed meshes where solid elements (SOLID65) are adopted to represent the slab and shell elements (SHELL43) are used for modelling the steel beam and the column. Material nonlinearities are included through the adoption of a bi-linear model with strain-hardening for steel and a tri-axial model with smeared cracking for concrete. Full interaction between the steel beam and the slab is assumed for both ADAPTIC and ANSYS numerical models.

Steel Joints

For verifying the applicability of the proposed approach to steel joints, a comparison is made with the results obtained by Krawinkler *et at* [2] for specimen A2 consisting of an internal substructure of a steel frame. An axial load of the order of 30% of the axial capacity of the column is applied as in the test. The results obtained for both the global and local response are plotted in Figure 8. The plots clearly demonstrate the accurate predictions using the approach proposed herein.



Figure 8 – Global and local response of model INT_SA2 (Krawinkler A2 specimen)

Composite Joints

An application of the new approach to a specimen (EJ-FC) tested by Lee and Lu [5] is made. The model (EXT_CFC) is prepared in ADAPTIC and the spring properties are derived. As the model is to be loaded in a way that sagging and hogging moment occur in the beam, an asymmetric tri-linear curve is used for the spring representing the panel zone. For the positive moment scenario, the spring is assigned properties based on a composite joint while for the negative moment, the properties are based on those for a bare steel joint as no composite effect takes place due to the lack of anchorage for the reinforcement.

The comparisons between the numerical response and the experimental test data are illustrated in Figure 9. The results indicate good agreement with the test. The only differences occur at large drifts where the capacity of the specimen is higher than that obtained in the model. This is mainly attributed to the fact that the experimental response consists of an envelope of the peaks observed during the cyclic test. Hardening effects that occurred in the test are not included in the numerical analysis and hence the differences observed. The figure also shows that the results obtained by the application of the expressions available for steel joints described before do not give an accurate prediction of the response.



Figure 9 – Global and local response of model EXT_CFC (Lee and Lu EJ-FC specimen)

A comparison is also made for an external substructure (EXT_CL45) modelled in ANSYS. Figure 10 depicts both the global and local response. Figure 10a shows the response of the system for the case where the panel zone is assumed to be rigid and without limiting the concrete strength on the contact with the column. The results show that the inclusion of panel zone flexibility is of extreme importance in order to estimate the maximum moment transmitted to the column. Regarding the local response of the panel, the predictions from the new approach fit very well that obtained from ANSYS. For this particular case, it is interesting to note that the application of the approach for steel joints also provides good predictions.



Figure 10 – Global and local response of model EXT_CL45

An additional comparison is made for an internal substructure (INT_CL45) also modelled in ANSYS. The results are illustrated in Figure 11. As for the previous model, the global response shows the significant differences obtained when the flexibility of the joint is not considered. The local response of the panel is again well predicted by the proposed approach while the application of the approach for steel joints is clearly inadequate in this case.



Figure 11 – Global and local response of model INT_CL45

CONCLUSIONS

In this paper, a new approach based on previous analytical models for representing panel zone response, is introduced. Realistic stress boundary conditions applied to the panel are taken into account which allow the study of the response of this element in the context of composite joints. Both shear and bending deformation modes are considered for assessing the panel stiffness, and shear stress distributions within the panel zone are considered when assessing its yield capacity. A more realistic model is suggested for assessing the post-elastic response of the panel. The approach is then tested for both steel and composite joints and very good predictions are obtained.

The comparisons made have shown that the application of existing analytical models suggested for steel joints may be inadequate when applied to composite joints. The behaviour of panel zones in composite joints is relatively complex and depends on many factors such as stress boundary conditions and interaction between the slab and the column. These issues can be more readily dealt with by separate modelling idealisations of the steel beam and the slab. This facilitates the identification of the internal forces that are transmitted to the joint hence enabling appropriate assessment of the joint behaviour.

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