Nonlinear Analysis of Steel-Concrete Composite Structures: State of the Art

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Abstract: This paper presents the current state of the art of nonlinear analysis of steel-concrete composite structures. The focus is on frame elements, which are computationally faster than continuum finite element models. First, section models are presented, with a review of resultant and fiber models and a discussion of possible practical applications. The presentation of frame elements follows. Models with lumped and distributed inelasticity, as well as models with perfect and partial connections are covered. Rigid and partially restrained joints are then reviewed and discussed at length. A discussion of the analysis of structural walls completes the presentation of the models. Modeling applications to the analysis of composite frames are also presented. This state-of-the-art review focuses on developments that have stemmed from the recently completed National Science Foundation sponsored U.S.-Japan program on composite and hybrid structures.

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Introduction

Steel-concrete composite systems (also called mixed or hybrid systems) have seen widespread use in recent decades because of the benefits of combining the two construction materials. Reinforced concrete is inexpensive, massive, and stiff, while steel members are strong, lightweight, and easy to assemble. In decks, composite systems eliminate the need for formwork. In columns, two systems are commonly used, steel reinforced concrete (SRC), where a steel section is encased in concrete, and concrete filled tubes (CFTs). One important advantage of composite systems is that construction is accelerated through separation of trades. Initially, a bare steel frame is erected to carry the gravity, construction, and lateral loads during construction. As erection of the building progresses, concrete is cast in lower-level columns to form the composite system that will resist the total gravity and lateral loads (Griffis 1992).

The inelastic behavior of composite members and systems, which is particularly important in limit state calculations for earthquake resistant design, is not yet thoroughly understood. As a result, design provisions for composite structures have generally been extrapolated from provisions for traditional reinforced concrete or steel structures [for example, ACI-318 (American Concrete Institute (2002) and AISC LRFD 2001 AISC (2001)]. Nevertheless, the growing body of experimental and analytical research, and the recent development of specifications addressing composite construction [Building Seismic Safety Council (BSSC) 1994; AISC (1997)], are increasingly providing engineers with guidance on the analysis and design of composite members and systems.

Nonlinearities in the response of steel-concrete structures stem from inelasticity of the materials or from changes in the geometry of the structure. The sources of material inelasticity are related to the components of a composite system, namely, concrete and steel. Concrete is a brittle material with distinctively different responses in tension and compression. Its tensile stiffness and strength are small, and design codes typically neglect them. Under compressive stresses, the concrete stiffness decreases significantly for stresses larger than about $0.5f'_c$, where f'_c is the concrete strength in uniaxial compression. After reaching its compression strength, concrete softens at a rate that depends on the amount of lateral confinement. Steel exhibits elastoplastic behavior in both tension and compression. Moreover, steel members contain residual stresses due to the fabrication or erection processes. Connections between steel and concrete components contribute to the nonlinearity of a composite system because the stress transfer mechanisms between the different components may exhibit complicated and highly nonlinear behaviors.

Geometric nonlinearities are generally classified into global and local nonlinearities. Global geometric nonlinearities, often referred to as P-δ and P-Δ effects, may be incorporated in global models following basic procedures used in nonlinear frame analysis (McGuire et al. 1999). Although usually neglected in frame analysis, local geometric nonlinearities, such as local buckling of steel components, are considered in more refined finite element (FE) analyses that warrant the inclusion of such behavior.

This paper discusses the state of the art of nonlinear analysis of steel-concrete composite structures. The focus is mainly on macromodels, for example, line (frame) elements and spring connection, rather than on micromodels (continuum FE models).

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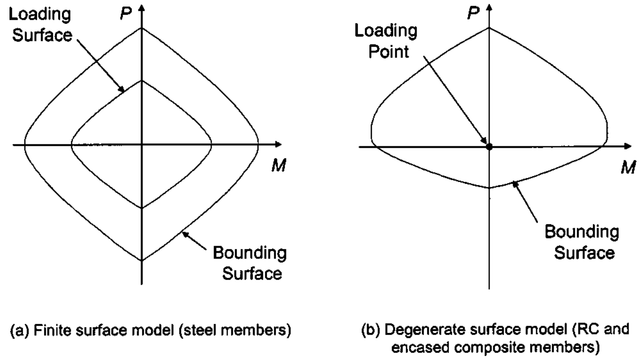

Fig. 1. Bounding surface model in force space (El-Tawil and Deierlein 2001a,b)

Since composite systems are comprised of steel as well as concrete components, it is natural that analytical methods for composite structures draw upon techniques used for reinforced concrete and/or steel systems. The objective of this paper is to review the most pertinent of these analysis techniques and to point out the special challenges posed by the presence of composite action. The paper is comprised of four main sections, namely, analysis of sections, members, joints, and structural systems including moment frames and wall systems. The material presented focuses on building applications and does not address the effects of fire and time (i.e., time dependent effects such as creep or shrinkage) on the behavior of composite structures.

Analysis of Composite Sections

Methods of analysis of composite sections have two main applications: (1) computing the response of a section to different load histories; (2) carrying out the state determination of a section (or integration point) in a frame element. In the first application, the section model typically returns the moment-curvature response of a given section under constant axial load. In the second application, the section model returns the section forces that correspond to given section deformations (in uniaxial bending the axial strain and the curvature). Two basic approaches are generally used to find the response of a composite section: resultant models and fiber section models.

Resultant Section Models

Resultant models explicitly define section responses in terms of moment-curvature response, axial load–axial strain relation, etc. The simplest resultant model decouples flexural and axial responses, with each following linear or nonlinear relationships such as the Takeda et al. (1970) model relating section moment and curvature. A more advanced resultant model, better suited for the analysis of beam columns, considers axial-bending interactions. Following the work by Hilmy and Abel (1985) and Hajjar and Gourley (1997), El-Tawil and Deierlein (2001a,b), developed a bounding surface plasticity model implemented in the stress-resultant space. The model was developed in a general manner so as to be applicable to steel, reinforced concrete, or composite members. As shown in Fig. 1, two variations of the plasticity model are considered: a finite-surface and a degenerate-surface version. The former explicitly considers a fully elastic response region to exist within the inner surface [Fig. 1(a)] and is thus

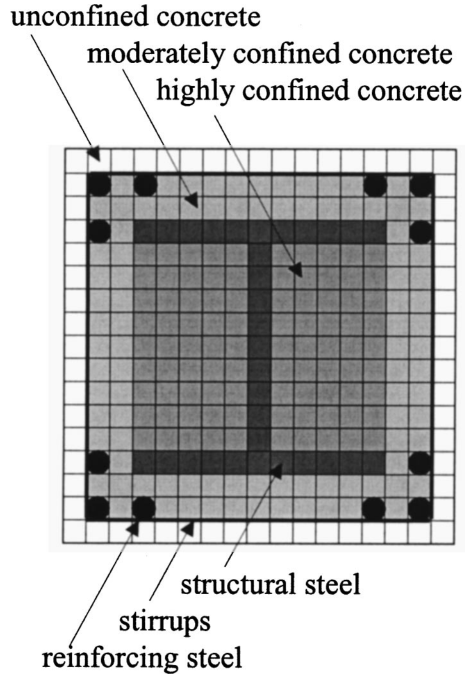

Fig. 2. Composite section fiber discretization

applicable to steel members, which typically have such behavior. The degenerate-surface model shrinks the elastic region to a point and thus the section behavior starts out as inelastic in any loading direction. This version is suitable for sections that have little or no elastic response region, such as reinforced concrete and composite sections. Stiffness degradation is accounted for as a function of the plastic strain energy absorbed by the composite member.

Fiber Section Model

The concept behind the fiber section model is rather simple. The section is subdivided into n fibers (not necessarily of equal area) and the stresses are integrated over the cross-sectional area to obtain stress resultants such as force or moment. The fiber section model generally makes use of a number of assumptions. (1) Plane sections remain plane after bending. It is generally accepted that this assumption is reasonably accurate even well into the inelastic range. (2) Shear and torsion stresses are neglected. For this reason the fiber method is generally used for the analysis of flexure dominated members, where the Euler-Bernoulli beam theory can be reasonably applied. (3) Although constitutive relations are typically defined as uniaxial, multiaxial stress states (such as those due to confinement effects) can be included by increasing the concrete strength and by modifying the concrete postpeak response. (4) Concrete cracking is generally accounted for. However, the cracking is considered to be smeared and normal to the member axis as a result of the plane section assumption. (5) Local buckling of the steel components and initial stresses resulting from either erection loads or thermal residual effects can be included. Local buckling is incorporated either by assuming a fixed effective width or by degrading the structural properties of the steel elements that reach a critical buckling stress (Liang and Uy 2000).

Each fiber in the section can be assigned concrete, structural steel, or reinforcing bar material properties (Fig. 2). Making use of the ‘‘plane sections remain plane’’ assumption and from relevant constitutive models, fiber stresses are calculated from the fiber strains. There are different approaches to finding the fiber strains as the load history on the section progresses. Spacone et al. (1996a,b) define the section axial strain and curvatures with respect to a fixed reference system and do not need to trace the evolution of the position of the neutral axis. El-Tawil and Deierlein (2001a,b), on the other hand, follow the migration of the section neutral axis during the load history.

Uniaxial constitutive models for concrete and steel are needed to compute the fiber stresses and moduli of elasticity. For the *concrete models*, the Kent and Park (1971) model, later enhanced by Scott et al. (1982) to include the confinement effects, has been extensively used for the analysis of reinforced concrete and composite sections. Mirza and Skrabek (1992) used a modified form of the Kent and Park (1971) model for the analysis of encased steel sections. The major drawback of the Kent and Park (1971) model is that the concrete initial stiffness is not one of the model parameters and cannot be explicitly controlled. Based on an equation suggested by Popovics (1973), Mander et al. (1988) proposed a unified stress-strain model applicable to confined concrete. The model is based on a single equation that describes both confined and unconfined concrete, and accounts for the increase in strength and ductility of concrete due to confinement. El-Tawil and Deierlein (1999) used a modified version of Mander's model to account for the behavior of concrete with unconfined strength of up to 110 MPa (16 ksi). Applications of the fiber section method to composite sections are found in Mirza et al. (1996), Hajjar and Gourley (1996), El-Tawil and Deierlein (1999), and Lee and Pan (2001).

The assumption that concrete has no strength or stiffness upon initiation of cracking has been frequently used in fiber analysis of composite sections. There have been attempts to account for the tension stiffening effect in composite sections, mostly based on models developed for reinforced concrete (El-Tawil et al. 1995; Hajjar and Gourley 1996; Mirza et al. 1996). Another approach is possible whereby the strength and stiffness characteristics of the reinforcing bars are modified instead of the concrete properties. Tension stiffening mostly affects the section response up to and immediately after cracking, and does not affect the section response at failure.

Most *steel models* used for fiber section analysis are uniaxial stress-strain relationships. Several studies have analyzed composite structures using a simple bilinear relationship with or without strain hardening after yielding and have obtained satisfactory correlation between experimental and analytical responses (among others, Bursi and Ballerini 1996; Salari et al. 1998). Alternatively, more accurate models such as the Ramberg-Osgood (1943) or Menegotto-Pinto (1973) model have also found wide application. Even though fiber analysis requires only uniaxial constitutive relationships, the response of steel in composite structures is the result of complex multiaxial effects that include local buckling and residual stresses due to cold-forming and welding. For their study on concrete-filled tubes, Hajjar et al. (1998a) derived the uniaxial steel constitutive model from a multiaxial constitutive law proposed by Shen et al. (1995).

Practical Application of Fiber Section Analysis

The fiber section method is a powerful tool that can be used to estimate the cross-sectional strength for design purposes. In this application, the maximum concrete compression stress in the constitutive model is usually set equal to a fraction of f_c'. The reasons for using a reduced compressive capacity instead of f_c' are well established in the literature. The reduced strength accounts for (1) the difference between concrete in a test cylinder and concrete in a structural member; (2) variation in concrete strength throughout the member due to variations in concrete compaction, water-cement ratio, and curing conditions; and (3) differences in rate of loading.

There is, however, disagreement in the literature over the precise value of maximum *unconfined* compressive stress, especially when higher-strength concretes are used. Specifications such as ACI-318 (ACI 2002) and AISC LRFD (AISC 2001) recommend $0.85f_c'$. Yong et al. (1988) observed a strength reduction factor of 0.92 for high-strength concrete in the 75–90 MPa (11–13 ksi) range. Work by Martinez et al. (1984) on 25–70 MPa (3.5–10 ksi) concrete has shown that the ratio of unconfined column strength to cylinder strength is about 0.85, irrespective of the concrete strength. An extensive test program by Cusson and Paultre (1994) on concrete with f_c' in the range of 59–117 MPa (8.5–17 ksi) resulted in an average reduction factor of 0.88. Collins et al. (1993), on the other hand, cite test evidence that supports maximum compression stress values ranging from $0.77f_c'$ to $1.0f_c'$.

There also seems to be little consensus about the behavior of the descending portion of the stress-strain curve. Ahmad and Shah (1982) observed that high-strength concrete [69 MPa (10 ksi) concrete] could be as ductile as low- to medium-strength concrete for the confinement levels they studied. The work by Yong et al. (1988) with concrete strengths in the range of 76–90 MPa (11–13 ksi), however, does not show such a trend. Research done by Martinez et al. (1984), with concrete in the 48–69 MPa (7–10 ksi) range, indicated that the stress-strain curves of high-strength concrete dipped sharply after the peak was reached and then flattened out again at a relatively high axial stress.

The confined compressive strength of concrete is generally determined through the confining pressure calculated at yield of the transverse hoop reinforcement, steel column flanges (Mirza and Skrabek 1992; El-Tawil and Deierlein 1999), or steel tubes (Hajjar and Gourley 1996). It is implicitly assumed that the confining pressures calculated in this manner are active pressures that exist without change throughout the loading history, which is clearly not the case. However, results obtained using this assumption have been reported in the literature as very good for concrete encased steel sections (Roik and Bergmann 1992; El-Tawil et al. 1995; El-Tawil and Deierlein 1999) and concrete filled steel tube sections (Hajjar et al. 1998a).

Analysis of Composite Members

Several frame elements, some applicable to any structural system, others developed specifically for steel-concrete composite structures, are available in the published literature. From the formulation standpoint, these elements can be classified into two general families, one based on the *displacement method* of structural analysis, the other on the *force method*. An alternative classification is possible whereby the elements are derived using *distributed* or *lumped* approaches. Another important classification applies mainly to composite elements and distinguishes between elements *with* and *without slip* between the steel and concrete components. The following discussion presents lumped versus concentrated models and elements with and without bond slip.

Lumped versus Distributed Models

Elements based on the *lumped* approach concentrate all inelasticity at the member ends [Fig. 3(a)], and thus deal with inelastic

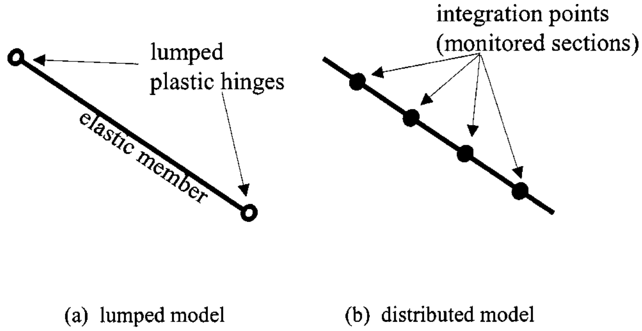

Fig. 3. Lumped versus distributed frame models

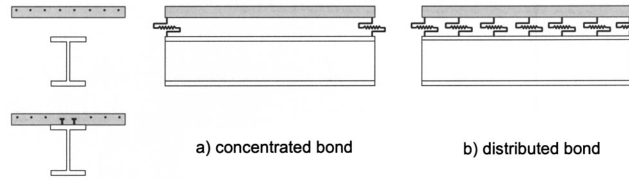

Fig. 4. Concentrated versus distributed bond models

material behavior in an approximate yet computationally efficient manner. Although lumped plasticity models imply behavior that is a physical impossibility, they have the advantage of being conceptually simple in addition to the computational convenience of having a stiffness matrix in a concise form. Hajjar and Gourley (1997) presented a lumped plasticity model for concrete filled tube members.

Distributed models, on the other hand, are more accurate and rational than concentrated plasticity models. As shown in Fig. 3(b), the behavior is monitored along the member length as opposed to only at the ends; thus distributed models are computationally more expensive. In the classical two-node, Euler-Bernoulli *displacement-based* frame element, the beam displacements are expressed as functions of the nodal displacements using shape functions (McGuire et al. 1999). Displacement-based frame elements are quite simple and easy to implement but they are not very accurate, because the assumption of cubic displacements (and thus linear curvatures) is exact for an Euler-Bernoulli beam only in the linear elastic range and for constant cross sections, while it is only an approximation if the cross section is not constant and, more importantly for the case of composite beams, if the material response is nonlinear. The issue is common in finite element analyses and is solved by using several elements in a single structural member, thus increasing the number of global degrees of freedom.

In the two-node, Euler-Bernoulli *force-based* frame element, the beam section forces are expressed as functions of the nodal forces through force shape functions (Spacone et al. 1996a,b; El-Tawil and Deierlein 2001a,b). The force-based element is rather attractive because it is exact within the small-deformation Euler-Bernoulli beam theory. In a frame member the bending moment diagram is linear and the axial load constant if no distributed loads are present, irrespective of the beam cross section or material response. This implies that one force-based element per structural member can be used. The complexity of force-based elements derives from their implementation in a finite element or frame analysis program, which requires the element to compute the stiffness matrix and the resisting forces corresponding to nodal displacements, while the force-based elements would naturally compute the flexibility matrix and nodal displacements corresponding to nodal forces. Spacone et al. (1996a,b) propose an iterative method for the force-based element state determination. While the element stiffness matrix is found by inverting the element flexibility matrix, the element forces are found by adjusting the section forces and deformations until the section deformations are compatible with the end displacements. The iterative procedure, although complex, is very robust for both strain-hardening and strain-softening section responses.

Fig. 3(b) schematically shows the integration points (or monitored sections) in a distributed model. The most widely used integration scheme for displacement-based elements is the Gauss scheme. Its precision is $2n-1$, implying that polynomials of degree up to $2n-1$ are integrated exactly. The Gauss-Lobatto integration scheme is preferred in the case of force-based elements (in spite of its lower precision, $2n-3$), because the end sections always correspond with the end nodes. The accuracy of a distributed model increases as the number of points at which nonlinear behavior is monitored increases. However, the demand for computer memory also increases since the additional points being monitored require additional storage space for the variables involved. Inelastic analyses by Sfakianakis and Fardis (1991) indicate that the use of five Gauss (sampling) points along the element length result in sufficient accuracy for most practical purposes while maintaining a reasonable demand on computer memory requirements. It should also be noted that the use of a large number of Gauss points along the member may cause lack of objectivity in the response of softening elements. As soon as a section starts softening (due to crushing of some concrete fibers) the inelastic response of the member tends to localize in this section, and different postpeak results are obtained if the number of Gauss points is changed. Regularization techniques are available from the finite element literature (among others, de Borst et al. 1994, Bazant and Planas 1998), while a specific study on localization issues in force-based beam elements is presented by Coleman and Spacone (2001).

Partial Bond Composite Members

While elements with perfect bond are general-purpose models that can be applied to steel, concrete and composite members, elements that consider bond slip between the member components are typically special-purpose models for the analysis of composite structures. Partial bond action between steel and concrete is an important issue in composite construction because of the implications it has on serviceability limit states, energy dissipation under cyclic loads, local stress distributions, and ultimate limit states. Connection enhancements in the form of embossments, ribs, and shear studs are typically used in composite slabs, while such devices are not always added in composite columns. The number of and distance between the shear studs depend on the desired degree of connection. Full connection is reached when the number of studs is sufficient to provide full shear transfer up to the beam ultimate state. In this case the cross section can be assumed as monolithic with perfect bond between steel and concrete components.

The simplest model for the description of partial bond uses different elements for the concrete and steel components and uses *concentrated springs* to model the connection. The springs can model either the action of the shear stud connectors (for example in a composite slab) or the friction effects in a concrete filled tube. This model is schematically shown in Fig. 4(a). The con-

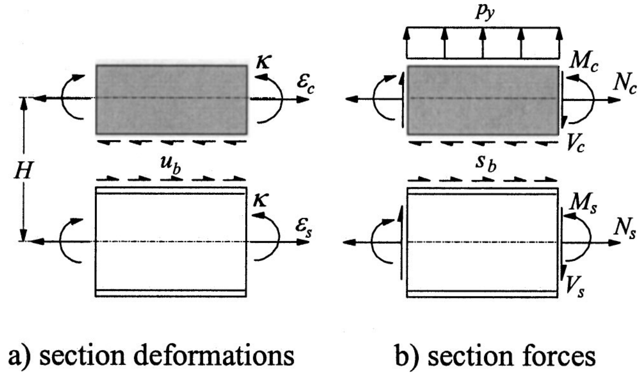

Fig. 5. Composite beam section forces and deformations

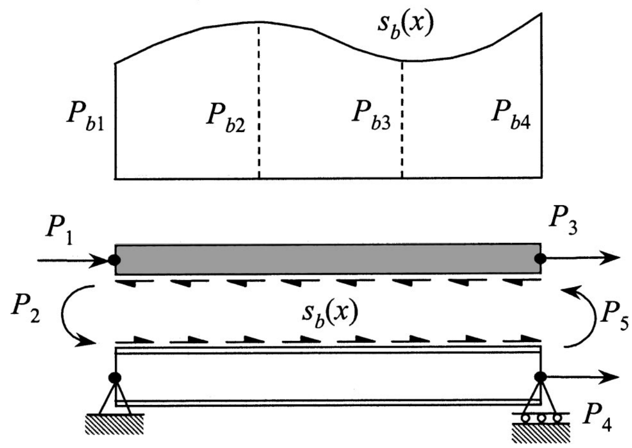

Fig. 7. Composite force-based element with distributed bond

centrated spring model is simple to use but presents a number of disadvantages. First of all, it requires a large number of elements and, therefore, of degrees of freedom. In the case of elements with shear connectors, the nonlinear springs are typically located at the location of the connectors, which implies very short elements for the steel and concrete components. In this case, if one uses fiber elements and accounts for the softening behavior of concrete under large compressive strains, the slab in compression may lead to strain localization problems.

More efficient models for members with partial connection are based on *distributed bond*. A prototype of this model is shown in Fig. 4(b). The model assumes that bond stress and bond slip are continuous along the contact surface. Uplift is typically neglected; thus the steel beam and the concrete slab have the same vertical displacement and curvature. Most of the steel-concrete elements with slip proposed to date use fiber section models for the beam components. The steel and concrete constitutive models previously discussed apply to these sections. As for the bond-slip model, the simplest model is a linear elastic model (if bond failure is not an issue) or an elastic–perfectly plastic model (if bond can fail). A more refined law is, however, needed if one needs to model cyclic bond degradation, energy dissipation, or shear stud failure. Several publications adapt the bond law developed by Eligehausen et al. (1983) for steel ribbed bars anchored in concrete. The main drawback of this law is numerical: after an ascending branch the law reaches a plateau of zero stiffness before entering a softening, descending curve. The zero stiffness plateau may lead to ill-conditioned stiffness matrices. To avoid the above problem, Salari and Spacone (2001b) propose a new law without a flat plateau. (See Fig. 5.)

Displacement-based, force-based, and mixed elements have been proposed to model distributed bond. *Displacement-based* elements typically assume separate displacement fields in the concrete and steel components, and bond slip is automatically derived from compatibility. The reference displacement-based element for composite members is shown in Fig. 6. Cubic polynomials are used for the vertical deflection and quadratic functions are used for the axial displacements in the concrete slab and steel beam. These assumptions lead to a quadratic bond-slip distribution. Because of their simplicity and ease of implementation, displacement-based composite elements have been successfully used in a number of analyses. Amadio and Fragiacomo (1993) used the element of Fig. 6 to study the creep and shrinkage effects in composite beams with deformable shear connections. Daniels and Crisinel (1993a,b) conducted an extensive study on the monotonic response of composite slabs to determine not only their strength but also their load-displacement response under monotonic loads. Hajjar et al. (1998b) use the same displacement-based model for the analysis of square and rectangular CFT columns with bond slip between steel tube and concrete. Both P-Δ and P-δ effects within the CFT columns are retained, thus making the element applicable to problems with small rigid body rotations and incremental strains. Based on the experimental results from Shakir-Khalil (1993a,b) on rectangular CFTs with and without mechanical connectors, Hajjar et al. (1998b) decided to use an elastic-perfectly plastic bond-slip relationship. Salari and Spacone (2001a) extended the original model of Amadio and Fragiacomo (1993) to the cyclic analysis of composite beams. Displacement-based elements are simple to formulate and implement, but they are not very accurate when the materials are nonlinear. It is a well-known fact that the assumed displacement fields are not accurate for a good description of the actual nonlinear structural response and therefore several elements need to be used for an accurate analysis.

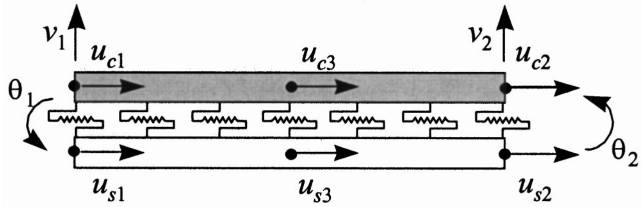

Fig. 6. Composite displacement-based element with distributed bond

An alternative to the displacement-based element is the *force-based* model. The motivation for such a model stems from the experience gained in the analysis of reinforced concrete and steel members with perfect bond, where the force-based formulation is exact within classical beam theories. The extension of force-based models to elements with partial bond is a natural step. The bond force is treated as a distributed force acting on the element components. If the bond force is known, its effect on the member components is derived from equilibrium. In the partial bond case, however, the element is not exact, because the bond force distribution along the beam is not known and cannot be derived exactly solely from equilibrium conditions. Fig. 7 shows the geometry, bond force distribution, and nodal forces of the two-node force-based element without rigid body modes proposed by Salari and Spacone (2001a). The bond force along the element is approximated by a cubic function. This distribution was selected because it closely follows the bond distribution in beams with double curvature normally encountered in frames under lateral loads. The cubic bond distribution results in fourth-order distributions for the bending moment and the axial force along the beam, which can be obtained through equilibrium. Explicit expressions of the above force interpolation functions for the beam of Fig. 7 are found in Salari et al. (1998). The force-based element state determination algorithm from the original scheme by Spacone et al. (1996a,b) which was developed for a beam with perfect bond, extends to the scheme presented by Salari and Spacone (2001a). The procedure adjusts the element forces until element compatibility is satisfied. The procedure maintains pointwise equilibrium between nodal and section forces.

Finally, Ayoub and Filippou (2000) propose a *mixed formulation* for the problem, where both the displacements and the forces are approximated along the element. Their element has the nodal displacement degrees of freedom of Fig. 6 and the nodal force degrees of freedom of Fig. 7. Ayoub and Filippou (2000) use the Hellinger-Reissner two-field mixed formulation to derive the element matrix equations. Similarly to force-based elements, the mixed element requires a special state determination procedure.

Further details of the displacement-based, force-based, and mixed formulations and the relevant implementation steps are discussed in detail by Ayoub and Filippou (2000), Salari and Spacone (2001a), and Limkatanyu and Spacone (2002a,b). The three methods are very robust and work well even when material softening is encountered in the concrete or in the bond-slip law. As for their accuracy, the force-based and the mixed elements are much more accurate than the displacement-based element; thus fewer elements are needed to study the response of a frame.

Composite Joints

When inelastic analysis is used, either for design practice or in research, it is important to accurately represent both joint deformations and finite size effects in composite structures. This is particularly critical for analyses involving lateral seismic loads where inelastic behavior often concentrates in or is adjacent to joints. Modeling the joint response is complicated by internal force-transfer mechanisms that involve composite action between steel and concrete and exhibit strength and stiffness degradation under cyclic loading.

Rigid Frame Joints

In this section, emphasis is placed on composite SRC and CFT rigid joints in which steel beams pass through reinforced concrete columns or concrete filled tube columns, respectively. These joints have been studied in some detail in both the United States and Japan. A description of common types of SRC rigid joints may be found in Sheikh et al. (1989) and Kanno and Deierlein (1996, 2002). Further details of the observed inelastic behavior of different types of CFT rigid joints may be found in Azizinamini et al. (1992), Ricles et al. (1997), and Schneider (1997).

Subassembly experiments of RCS joints by Sheikh et al. (1989) and Kanno and Deierlein (1996) show that, when carefully designed and detailed, the joints exhibit strength and deformation characteristics that make them well suited to seismic applications. Previous research has identified two basic failure modes in the joints: (1) panel shear, and (2) bearing of steel against concrete. Panel shear failure is similar in some respects to that observed in steel or reinforced concrete joints, except that in mixed steel-concrete joints both structural steel and reinforced concrete elements participate. Bearing failure occurs at locations of high compressive stress and permits rigid rotation of the steel beam within the concrete column. As discussed by Kanno (1993), the actual behavior usually involves deformations associated with both failure modes. However, separating the two components of deformation is helpful for understanding and quantifying the strength and deformation characteristics of the joint. National Earthquake Hazard Reduction Program (BSSC 1997) and AISC LRFD (AISC 1997) recommend that the total shear strength of fully encased steel connections may be calculated as the sum of contributions from the reinforced concrete and steel shear panels. Further details regarding the proportioning and detailing of SRC joints may be found in Deierlein et al. (1989) and ASCE (1994).

Sheikh et al. (1989) proposed a fairly simple multilinear model for the joint moment-rotation behavior of mixed steel concrete joints applicable to cases with monotonically increasing loads. The model considered only the overall joint distortion and did not differentiate between panel shear and bearing modes of deformation. Subsequently, Kanno (1993) improved upon this model with one that treated the two components of deformation separately. The panel shear distortion model was trilinear whereas the bearing deformation model was composed of two parts, an infinitely stiff region at the beginning followed by a smoothly degrading part. However, like the earlier model by Sheikh et al. (1989), Kanno's (1993) model considered only behavior for monotonically increasing loads.

Building on Kanno's idea of splitting the joint deformation into bearing and panel shear, El-Tawil et al. (1997, 2001a,b) developed a cyclic model for composite SRC joints based on two inelastic relationships corresponding to the two components of deformation. The two inelastic relationships are combined together to calculate the total response of the joint panel. Stiffness degradation in both panel shear and bearing modes is assumed to be a function of an evolving damage index. Finite joint size effects are included through use of the mechanical idealization, which involves rigid bars connected together by pins allowing panel distortions in each of the two vertical planes, but not in the horizontal plane.

Azizinamini et al. (1992) conducted detailed finite element analyses to investigate the performance of a through-type connection between steel beams and concrete filled tubes. The three-dimensional finite element model was analyzed using the program *ANSYS* (Swanson Analysis Systems, Houston, Pa., 1989). Concrete was modeled using brick elements which accounted for concrete cracking and crushing. The steel tube was modeled using quadrilateral shell elements for which the steel model was based on bilinear kinematic hardening. Contact elements were introduced to allow the steel and concrete elements to bear upon one another or to separate, but prevented the elements from piercing one another. Gap elements were provided at selected locations to allow slip between steel and concrete components. Chiew et al. (2001) conducted a similar study and used their model to study the effect of various connection details on strength. El-Tawil et al. (2002) also used a continuum finite element model to study the response of the connection between embedded steel beams and reinforced concrete walls.

Parra-Montesinos and Wight (2001) presented a model to predict the shear strength versus shear distortion response of hybrid connections between reinforced concrete columns and steel (RCS) beams. The model assumes a state of plane strain throughout the joint and is capable of predicting the shear force, and stirrup and concrete strains at any level of joint shear distortion for exterior joints.

Partially Restrained Frame Joints

Information pertaining to the behavior of partially restrained composite connections can be found in Leon and Ammerman (1990), Leon and Forcier (1992), and Leon and Shin (1995). It is important to account for the nonlinear behavior of semirigid composite connections in the design of composite frames (Liew et al. 2001). Serviceability limit states often govern design because of the lower flexibility associated with the semirigid connections. Further, low connection strength generally leads to weak connection–strong column mechanisms under lateral loading, which tend to increase second-order effects. Therefore geometric nonlinearities must also be included in the analysis of composite frames.

There have been few attempts at including the inelastic behavior of composite connections in frames subjected to cyclic loading. Leon and Shin (1995) developed a moment-rotation curve for semirigid composite connections that accounts for cyclic stiffness degradation. The moment rotation model was based on a trilinear backbone curve and a set of hysteresis rules governing cyclic behavior. The moment-rotation relationships were assigned to joint elements that exist at the end of beam elements, and the condensation technique was employed to remove the additional degrees of freedom belonging to the beam elements. The model was incorporated into a frame analysis program for materially and geometrically nonlinear analyses, which was used to better understand the inelastic response of composite frames subjected to seismic loading.

More recently, Alemdar et al. (1999) used a multispring model to represent the inelastic cyclic behavior of partially restrained composite connections. The model is comprised of many springs in series and parallel. Each spring represents one component of the connection including bolts, steel angles, steel reinforcement, concrete compression struts, etc. The model gives good results compared to test data and is implemented in a computer program for the analysis of composite frame systems.

Composite Systems

The models described in the previous sections for composite beams, columns, and joints can be combined and used to investigate the global behavior of composite systems. Of course, the whole system could be modeled using an assembly of continuum finite elements. However, this is rarely done because of the computational expense involved. In the following, emphasis is placed on two main types of systems; composite moment frames and composite shear wall systems.

Composite Frame Systems

One of the earliest inelastic analyses of composite systems is that presented by Hasegawa et al. (1988). They conducted a feasibility study of composite frames comprised of reinforced concrete columns and steel beams. The analyses were conducted using what was termed a shear type lumped mass model. In such a model, the structure is represented by a multiple-degree-of-freedom cantilever in which the structure mass is lumped at the story levels. The structural characteristics of the stories were calibrated to experimental results obtained from cruciform beam-to-column subassemblages.

More recently, Kim and Lu (1992) modified the computer program *DRAIN-2D* (Kanaan and Powell 1973) to analyze two-dimensional composite frames comprised of encased composite columns and steel beams with composite slabs. Column elements were formulated using a concentrated fiber element model in which only the ends were allowed to respond in an inelastic manner with the middle portion of the element remaining elastic. The column elements were capable of reproducing the descending portion of the moment-curvature response and had rigid ends to represent the panel zone region. The composite beam element made use of hysteretic relationships based on a set of predefined rules, and was capable of simulating the nonsymmetric, degrading behavior of a composite beam subjected to cyclic loading. Composite joints were included in the analysis as inelastic springs. Cyclic analyses of a series of experimentally tested one-story frames yielded reasonable results, although the degradation in advanced inelastic cycles was not accurately represented.

Hajjar et al. (1998b) performed the analysis of a four-story unbraced composite CFT frame subjected to gravity and wind loads. They used elements with no bond slip for the beams and elements with bond slip for the columns. A lumped plasticity model that accounts for bond slip was used. Twelve elements were used to discretize each beam and column. All elements used a fiber model for the section description. The main purpose of this study was to investigate the effect of slip between steel tube and concrete on the frame response. Gravity and lateral loads were increased monotonically up to failure. The results of this study indicated that even though bond slip played an important role in the load transfer mechanism at the beam-column connections, bond slip did not affect the global load-deformation response of the frame.

Mehanny and Deierlein (2001) used the models developed by El-Tawil and Deierlein (2001a,b) to evaluate the seismic performance of composite steel-concrete moment frames. They proposed a seismic damage index based on cumulative member ductility that employs the concept of primary and follower load cycles to distinguish loading history effects. The damage index is incorporated in a methodology that combines nonlinear time history and gravity load stability analyses to evaluate collapse prevention performance as a function of earthquake ground motion intensity.

Liew et al. (2001) investigated the behavior of two- and three-dimensional steel frames with composite floor beams subjected to the combined action of gravity and lateral loads. Composite beams were modeled using a distributed model based on a stress-resultant section model, while steel columns were modeled using a concentrated plastic hinge model. Slip in composite beams was indirectly accounted for by reducing the cross-section stiffness as a function of the degree of composite action. Salari and Spacone (2001a,b) analyzed a steel frame with composite floors previously studied without considering the effect of the concrete slab. Including the concrete slab greatly increases the stiffness and strength of the frame, while a study of the effect of different degrees of composite connection in the girders revealed that commonly used connection details basically lead to a full composite behavior. Additional information on the inelastic analysis of frames with semirigid composite connections between columns and steel beams with composite slabs may be found in Jarrett and Lennon (1992), Zandonini and Zanon (1992), and Leon and Shin (1995).

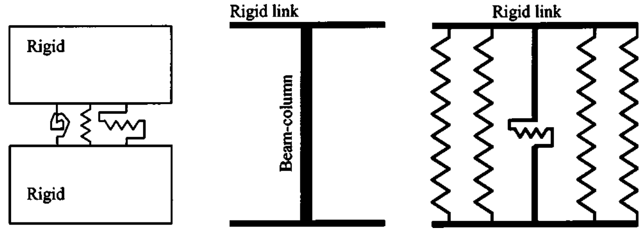

Fig. 8. Commonly used wall models

Composite Wall Systems

The term ''composite wall system'' refers to a number of possible configurations including (1) cantilever composite walls, where steel components are embedded in or attached to reinforced concrete walls, (2) hybrid coupled walls, where steel beams are used to couple two or more RC or composite walls in series, and (3) hybrid dual systems, where reinforced concrete walls are placed in parallel with steel moment frames.

Four kinds of analysis models are usually used to model composite wall systems: (1) equivalent frame models, (2) multi-spring models, (3) fiber section models, and (4) continuum finite element models. In the equivalent frame model, the finite width of the walls is generally represented using rigid elements, while wall behavior is modeled using an equivalent beam column placed at the wall centroid. In multispring models, the behavior of the wall is represented using a number of series/parallel springs to simulate the inelastic axial, shear, and bending behavior of the wall panels, while rigid elements are used to represent the physical size of the wall.

Many equivalent beam-column and multispring models (Otani 1980; Charney 1991; Kunnath et al. 1992; Cheng et al. 1993; Colotti 1993) have been developed to represent the behavior of reinforced concrete shear walls. Examples of these are shown in Fig. 8. These models suffer from the following drawbacks. They require extensive preanalysis to determine element structural properties and are generally inadequate when different interacting mechanisms take place simultaneously. Since the location of the wall neutral axis changes during an analysis, equivalent beam-column elements (which are generally placed at the centroid location) can be inaccurate unless they account for axial-flexural interactions. Although equivalent frame and multispring models yield useful information about system behavior, their results are often of a more qualitative rather than quantitative nature. However, in spite of their shortcomings, these models are computationally efficient and have been used in the inelastic dynamic analyses of composite wall systems (e.g., Shahrooz et al. 1993; Harries et al. 1998).

Fiber section analyses of walls are reported by Pilakoutas and Elnashai (1995). The advantage of the fiber section analysis is that it can easily account for the presence of structural steel, reinforcing bars, and concrete. As previously discussed, although constitutive relations are typically defined as uniaxial, multiaxial stress states (such as those due to confinement effects) can be indirectly considered. The major drawback of fiber analyses is that smeared cracking normal to the member axis is implied, i.e., diagonal tension effects cannot be directly considered. In addition, fiber models are computationally expensive.

Continuum analyses of reinforced concrete walls are reported by Bolander and Wight (1991), Chesi and Schnobirch (1991), and Sittipunt and Wood (1995), and analyses of hybrid coupled walls are reported by El-Tawil and Kuenzli 2002; El-Tawil et al. 2002. Compared to beam-column models, continuum elements offer several distinct advantages. While continuum element models require larger amounts of input data than equivalent models, the input parameters are easier to specify. They can be easily used to model three-dimensional situations. Continuum models provide a more physical description of the nonlinearities that occur in RC shear walls. The possibility of modeling the distribution of diagonal cracking and local crushing makes such models more realistic. Continuum models are able to describe local behavior at reentrant corners and at other discontinuities more accurately. For example, bearing between steel and concrete components can be simulated using contact elements. Continuum finite element models can account for local reinforcing details such as diagonal reinforcement, edge reinforcement, etc., and can model concrete crushing, cracking, and steel yielding. They also capture important behavioral responses such as axial-flexure interaction, inelastic shear deformation, steel confining effect on concrete behavior, concrete compression softening, and concrete tension stiffening.

Concluding Remark

Some of the system analyses surveyed in this paper utilized a large number of elements and degrees of freedom. These sophisticated analyses, unthinkable until a few years ago, are now performed on relatively inexpensive personal computers. Codes and design guidelines, such as FEMA-356 (Federal Emergency Management Agency 2000), are creating a demand for such analysis techniques by permitting and codifying nonlinear analysis as a design/evaluation option. As computer technology continues to advance and as more robust and efficient models become available, it is inevitable that nonlinear analysis tools will move from the realm of research into the hands of designers.

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