Provisions for Numerical Investigation of Utilizing Simultaneous Performance of Supplementary Concrete on Steel Structure Connections Retrofitting

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SUMMARY:
In this paper, the provisions for numerical investigation of utilizing supplementary concrete in retrofitting steel structure connections have been presented. Earthquake experiences such as Bam, Zarand and Silakhor Plain (Iran) reveal that the most important rupture section in steel structures occur due to the inefficiency of connections. Employing stiffness augmentation and applying re-welding are among the factors contributing to ineffective performance of connections subsequent to retrofitting. Using FRP composite plates or simultaneous performance method of connecting via supplementary concrete is among the efficient methods. Utilizing composite plates necessitates a special technique caused by their performance and sensitivity of their application. Yet utilizing simultaneous performance method of supplementary concrete requires applying appropriate analytical and numerical techniques in order to desirably estimate behavior of this collection. The utilized elements, appropriate non-linear analysis, proper consideration of supports and calibration of numerical and experimental data are among determining factors. Through carrying out a numerical analysis in a case study and estimating the aforementioned issues, an efficient numerical model has been proposed to analyze simultaneous performance of concrete and connection.

Keywords: Retrofitting, Supplementary concrete, Steel structures

1. INTRODUCTION

Steel-concrete composite systems have seen widespread use in recent decades because of the benefits of combining the two construction materials. Reinforced Concrete (RC) is inexpensive, massive, and stiff, while steel members are strong, lightweight, and easy to assemble. The structural performance of the complete frame is affected by the behavior of the joints, which should be considered in any global analysis of the structure. In many cases, the concrete slab rests on steel beams. When the presence of steel in the concrete slab is considered in the design of a connection, it is called a composite beam-to-column connection. It has been proven by experimental investigation that for service load, composite connections offer rigidities similar to those of rigid frames. Composite connections offer significant gains in stiffness and strength for all structural members. However, as in the non-composite connections case, they have had limited practical application due to the lack of analytical and design guidelines. In conventional analysis and design of steel and composite frames, beam-to-column connections are assumed to behave either as pinned or as fully rigid joints (Jones et al., 1981). Although the pinned or rigid assumption significantly simplifies analysis and design procedures, in practice, the actual joint behavior exhibits characteristics from a wide spectrum between these two extremes.

Stiffness relationships between cross-section stress resultants and generalized strains are distinct from the beam-column formulation and can be equally applied to displacement or force-based elements. Takeda et al. (1970) presented one such model, which employs a nonlinear cyclic moment-curvature equation and has been widely implemented for RC frames. Other techniques that consider the interaction of axial and bending effects employ adaptations of associated plasticity theory (Porter and Powell 1971; Hilmy and Abel 1985; Zhao 1993; Hajjar and Gourley 1997; Hajjar et al. 1997). Silva et al. (2001a) extended the component method that was widely accepted in a cold design to predict the response of steel joints under fire loading. Silva and Coelho (2001b) presented a model for evaluation of the ductility of steel connections loaded in bending. Also, Silva and Coelho (2001c) presented an equivalent elastic model to evaluate the response of steel joints under bending and axial forces. Further, Del Savio et al. (2009) presented a generalized component-based model for semi-rigid beam-to-column connections including axial force versus bending moment interaction, and described a detailed formulation of the proposed analytical model. Song et al. (2000) developed two component beam-column formulations of adaptive non-linear analysis, and described the details of the requirements for the automatic mesh refinement of elastic elements into an appropriate mesh of elastic and elasto-plastic elements, in the context of both fire and explosion analysis. Liew and Chen (2004) developed a numerical approach for the inelastic transient analysis of steel frame structures subjected to an explosion loading followed by fire.

For the modeling of the composite constructions, limited research has been done so far. Ayoub and Filippou (2000) presented an inelastic beam element for the analysis of steel-concrete composite girders with partial composite action under monotonic and cyclic loads. This element is derived from a two-field mixed formulation. Fiber discretization of section and hysteric material models for the constituent materials are used to achieve the nonlinear response. Sebastian and McConnel (2000) developed another beam element with layered steel beam and layered concrete slab for the analysis of steel-concrete composite girders. In this element, they also included the modeling of profiled steel sheeting. Baskar et al. (1996) built up a three-dimensional (3-D) FE model using ABAQUS to analyze the steel-concrete composite plate girders under negative bending and shear loading. Ahmed (1996) proposed a two-dimensional (2-D) model for the analysis of composite connections and composite frames using ABAQUS. In order to overcome the convergence problem of the concrete slab simulation, they ignored the concrete slab under negative bending and analyzed the steel beam with multipoint constraint to behave like a composite girder.

In order to investigate the utilized method for retrofitting the existing steel structure, a method for structure modeling and analysis is required. In this section the analysis method and the process employed for modeling will be introduced. Modeling connections is the best method for studying the effects of applying concrete in steel structures for two reasons: first, modeling the entire structure is cumbersome and of little futility and second, modeling other components of structure cannot adequately indicate the effects of using concrete in structure reinforcement. Yet, given that connection is the most sensitive section of a structure and given that the utilized method which includes applying concrete and stiffener inside the column as well as applying floor concrete and etc, will exert its influence on the connection, therefore, modeling the connection and investigating its behavior is indicative of structure’s behavior. Considering that connection is investigated as a component of the structure, it is obligatory to consider its interaction with other components; this is due to the fact that investigating connection without applying compatibility conditions with other structural components such as beam, column and etc, one cannot claim that the modeled connection exhibits the same behavior as the real connection. To this end, it is essential to consider loading conditions, supporting conditions and boundary conditions so that connection is seen in contrast with other components. These conditions will be discussed later. It is worth noting that the conducted studies were numerical while experimental studies would also be advantageous for a more detailed investigation of connections.

2. THE CONCEPT OF NON-LINEAR ANALYSIS

For nonlinear seismic response analysis of steel frames, proper consideration of the rigidity is essential, no matter how difficult the analysis procedure becomes. With modern computing technologies rapidly enabling the practical application of nonlinear analysis in design; there are increasing needs for accurate and efficient inelastic element formulations. Reliable nonlinear analysis tools are, for example, essential in performance-based earthquake engineering that involves accurate predictions of inelastic limit states up to structural collapse.

Inelastic frame analysis models can be broadly categorized by their resolution in modeling nonlinear behavior of beam-columns and their connections. Assuming one adopts the standard flexure-theory assumptions of an Euler-Bernoulli element (plane sections remain plane and normal to the centroid), element formulations can be distinguished by how they model spread of plasticity through the cross section and along the member length. 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)].

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 $0.5\acute{f_c}$, where $(\acute{f_c})$ 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 elasto-plastic 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 $P-\delta$ and $P-\Delta$ effects, may be incorporated in global models following basic procedures used in nonlinear frame analysis. 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 (Spacone and El-Tawil, 2004).

Another factor to be considered in non-linear solution is loading such that if the load is not entered correctly, the obtained solution will not be correct either. In ANSYS software, it is possible to define the load in different stages under the title of loading step. Namely, it is feasible to apply the load to the structure in several stages from zero to the ultimate load and to examine structural response in each of these stages. In each loading step, time is a symbolic parameter for indicating each loading stage. Each loading step can further be divided into several sub-steps; this is done in modeling with the intention of obtaining accurate answers. Still, it should be taken into account that the number of sub-steps is required to be increased in vicinity of the ultimate load. The number of steps and sub-steps is of significant influence in the ultimate answer such that altering their numbers could even lead to convergence or divergence of the solution.

3. APPLYING FORCES TO MODELS ACCORDING TO LOADING

In order to compare the obtained results, it is essential to consider their loadings equal. In these models the supposition is that the concerned connection is an intermediate one from a four-floor building with a span of four meters and each floor of three meters height. Fig. 1 shows the desired building and the discussed connection.

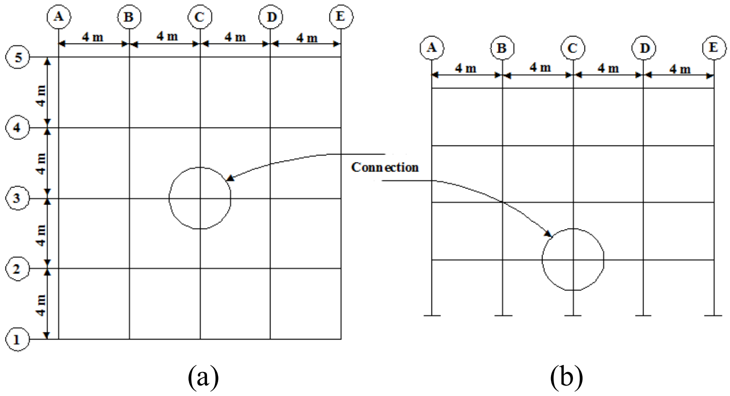

Figure 1. Describing the investigated building (a: Plan, b: Section)

Dead and live loads are founded on UBC-97 code. Moreover, in order to introduce the dead load, the ceiling is taken as a hollow-tile floor with wall paneling. Besides, live load for each floor is taken 200 kg/m^2. The earthquake load was also applied to the structure based on Iranian seismic code (code No. 2800) and through taking the parameters of static base shear force including A = 0.35 and I = 1. Subsequently, the forces applied to the connection were determined using equivalent static method and portal analysis, and then the connection and its components were designed at a basic level.

For the purpose of modeling, anchored in St. Venants principle, the sections continued up to three times the height of each section toward four sides of support, and then forces and boundary conditions were also applied in this distance to eliminate the effect of stress concentration and to depict stress distribution more realistically. Given that in ANSYS, it is essential to define the loads in nods, in order to apply shear force, this force was divided as the ratio of load share to the nods located in shear effective areas; that is shear force of beams were distributed along the beam web while shear force of columns was distributed along its effective section in shearing. Also, for applying moment in beams, the outcome of the momentum was applied on the wings above and below the beam as tensional and compressive forces; the same process was also repeated for the columns.

4. DEFINING OF SUPPORTS ACCORDING TO COLUMN'S STIFFNESS

Unless there is no need for utmost accuracy, it is possible to assume column base section consoled, then to carry out the analysis. Yet, in this project, to approach real supporting conditions of column, elastic supports containing a set of horizontal springs (along length of beam) and vertical springs (along the column) were employed at the column base.

Horizontal strings stiffness was considered with respect to the shear stiffness of column while vertical strings stiffness was considered with respect to the axial stiffness of column. Eqn. 4.1 and 4.2 state vertical and horizontal stiffness, respectively.

$$k_v = \frac{EA}{L} \tag{4.1}$$

$$k_h = \frac{12EI}{L^3} + \frac{GA}{L} \tag{4.2}$$

In the above relations, (A) and (I) represent the area and inertia moment of the concerned section, respectively, while (E) and (G) represent Shear Modulus and Modulus of Elasticity, respectively. For those models where concrete was not used inside the steel section and where the section was of a single type, materials properties and geometric properties of the utilized steel were applied, however, for concrete-using sections, separate characteristics for steel and concrete have been obtained and applied. Besides, in Eqn.4.2, shear effective area was applied as (A) parameter in order to calculate horizontal strings stiffness. For those nods having passive freedom in horizontal and vertical directions, spring support was not considered. Fig. 2 represents a view of utilizing springs in supports in the conducted models as four real connections.

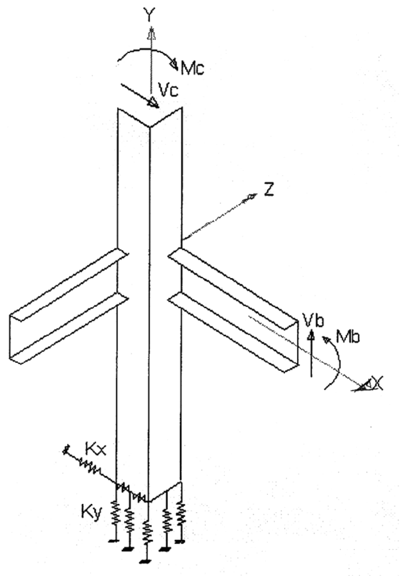

Figure 2. Using springs to model supports of the building

5. THE UTILIZED MATERIALS

5.1. Steel Modeling

The utilized properties for the consumed steel have been given in Table 1.

Table 1. The Properties of the Used Steel

Poisson's ratio	Shear modulus (N/m^2)	Elastic modulus (N/m^2)
0.3	78.65×10^9	205×10^9

5.2. Modeling of Reinforced Concrete

Concrete is a substance of a very sophisticated performance. This sophistication could be due to heterogeneous and non-isotropic nature of the concrete as well as cracking in the process of its analysis. For this aim, when modeling reinforced concrete, entering cracking of elements in the solution process is of significant importance. So far, various methods have been applied for crack modeling. One such a method is to enter cracking in calculations via pre-defined models; i.e., in each loading stage, the nods whose applied tension exceeds the tension concrete is capable of sustaining, are separated to two different groups at a distance equivalent to crack width. This method is extremely time- consuming (Malaska, 2000). Another more commonly applied method is to consider the cracked concrete as an orthotropic substance where elasticity module that is applied vertically on the created cracks in the element is taken zero (ANSYS User's Manual, 2010). The same method is utilized in ANSYS software.

5.2.1. Utilized coefficients and parameters for modeling concrete

The most important parameters utilized for concrete modeling which ANSYS also uses include:

1. Module of Elasticity (E)
2. Maximum uniaxial compressive stress of concrete $\left(f_c'\right)$

3. Maximum tension stress of concrete (f_r)
4. Poisson's ratio (υ)
5. Shear transfer coefficient in open cracks (βt)
6. Shear transfer coefficient in close cracks (βc)
7. Uniaxial compressive stress-strain curve of concrete

Regarding parameters βt and βc, it is worth noting that their value remarkably affects the solution, however, no clear relation is offered for determining these two parameters to date; even though an accumulation of studies have been conducted in order to determine them (Ashour and Rishi, 1997). Considering the carried out studies, values 0.05 to 0.25 are suitable values for βt, however, given that in small values for βt, the solution would not converge (concentric) (Ashour and Rishi, 1997); to this end, values 0.2 to 0.25 are appropriate values for βt. Also despite the fact that research suggests value 1 for βc, since ANSYS User's Guide proposes values less than 1, hence its value was taken 0.99.

6. ASSESSING THE ACURACY OF THE UTILIZED METHOD

In order to ensure the accuracy of the utilized method, it is essential to compare the results obtained from a sample tested and analyzed through the utilized method with experimental results. Regarding steel modeling, owing to the massive use of ANSYS software and ensuring the accuracy of the answers, there is no need to be anxious; yet, in the case of reinforced concrete, it is obligatory to control for it. To this aim, a reinforced concrete beam tested by Ashour and Rishi (1997) was modeled and analyzed using the proposed method. The obtained results should correspond to experimental results to a desirable extent.

6.1. Characteristics of Investigated Sample

The investigated sample is a reinforced concrete beam with a 3.0 m length, 12 cm width and 42.5 cm height. Longitudinal bars were set on beam section in three rows: above, middle and below the beam. The overall shape of the tested beam is indicated in Fig. 3. The steels employed in beams enjoyed high strength and were of 6, 10 and 12 mm diameters. Compressive and tension strengths of the utilized concrete was equal to 267 and 48.5 kg/cm^2, respectively.

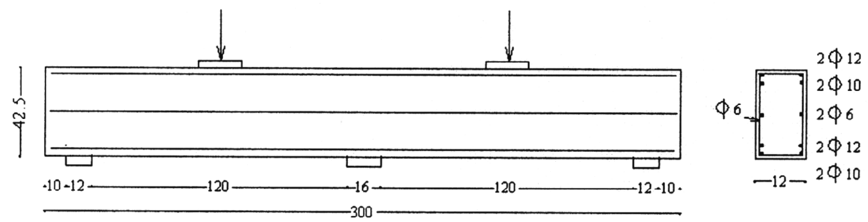

Figure 3. Dimensions and bars of the tested beam

7. BEAM MODELING AND ANALYSIS VIA THE PROPOSED METHOD

The reinforced concrete modeling was conducted employing Solid 65 element where the element was divided into several parts and the consumed bars were modeled as the mass of each part. Subsequently, the investigated beam was meshed with each element of approximately 5 cm size. In order to define the concrete properties, the criteria presented in section 5.2.1 were utilized. Modulus of Elasticity of concrete was taken 24900 MPa. βt and βc were taken 0.2 and 0.99, respectively. Additionally, according to Table 2, a three-line diagram was employed for stress-strain curve of steel.

Table 2. Stress-Strain Curve Properties of Used Steel

Point Number	Stress (*MPa*)	Strain
First Point	275	0.0013
Second Point	310	0.0018
Third Point	340	0.0047

The loading applied to the model was conducted in an area of 20×12 cm^2 dimensions based on the tested model. As a result, all the existing nods in this area were considered as the area load. Ultimately, the applied load was applied in the form of time-history analysis and the loading results were obtained.

8. COMPARING EXPERIMENTAL AND ANALYTICAL RESULTS

Ultimate load of discussed beam was obtained equal 445 *KN* experimentally. Whereas this ultimate load was obtained equal 398 *KN* analytically. It shows 90% of accuracy. Then load-displacement curve of investigated beam obtained experimentally and analytically were compared with each other. It has been shown in Fig. 4 which demonstrates that used method in this paper is efficient and precise. Furthermore, Mann-Whitney test performed for comparison between experimental and analytical data.

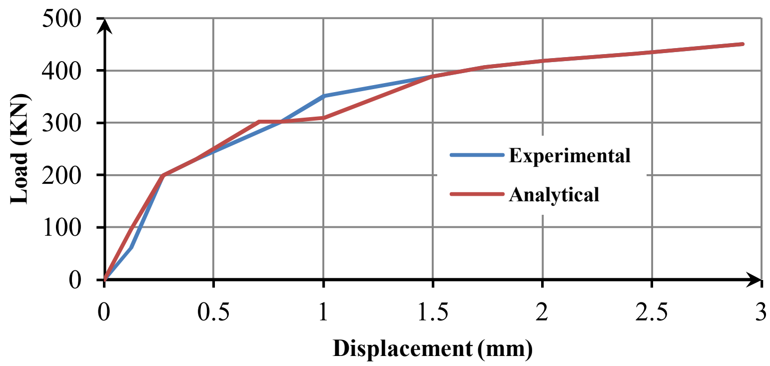

Figure 4. Load-displacement curve of the beam

9. INTRODUCING AND EVALUATION OF 2 NON-LINEAR MODELS

Models have initially been analyzed linearly and subsequently nonlinearly. Arriving at a solution in non-linear models is subject to numerous difficulties due to the nature of non-linear solution in ANSYS where convergent solution is obtained scarcely. The models considered for analysis are as follows:

9.1. A-1 Model

This model has generated for evaluation of connection performance in steel structures with no strengthening method. Based on the selected structure the connection includes an IPE270 beam and a box column of four welded plates (thickness: 1 cm, width: 25 cm). Stiffeners with dimension of $30\times20\times1$ cm^3 have used at base and top of the beam, and have connected to beam and column with corner and penetration welds, respectively. In addition, $60\times60\times6$ cm^3 angle in 15 cm has welded to both sides of the beam web to sustain the shear forces. This angle has welded to beam and column with corner weld. The detail of this connection has illustrated in Fig. 5. In finite element modeling, a quarter of connection has modeled by application of boundary conditions. One of the major steps in finite element modeling is the arrangement of elements that is a fundamental issue in achieving a proper solution.

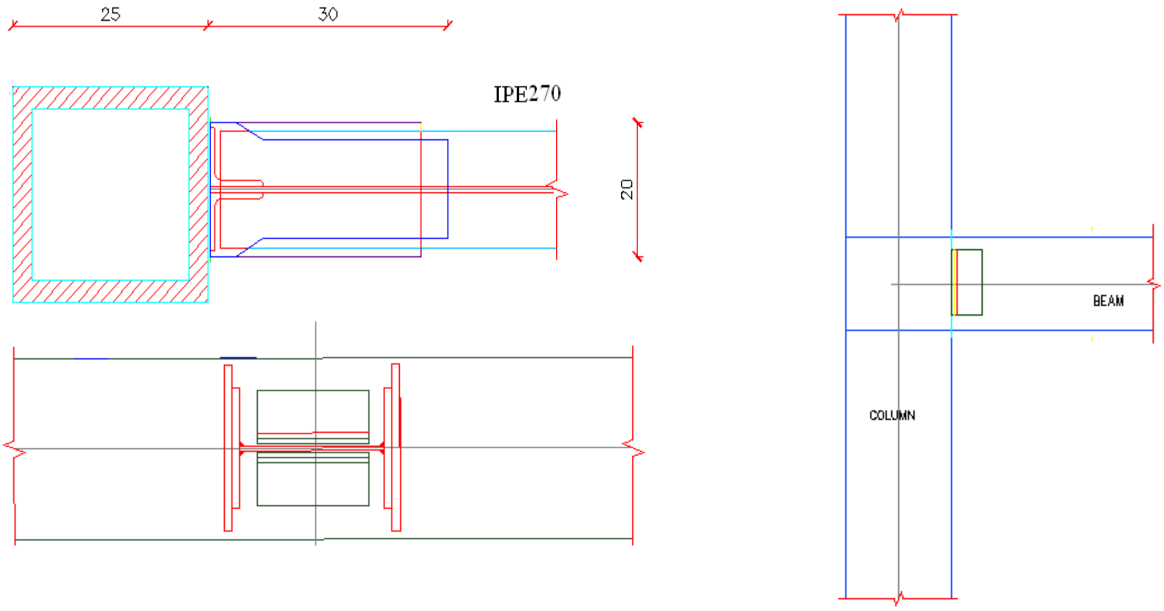

Figure 5. Connection details of A-1 model

9.2. Evaluation of A-1 Model

This is a rigid connection in a steel structure with box columns and no strengthening method. Von Mises stress in the ultimate stable loading step has illustrated in Fig. 6. As shown in the figure, the maximum stresses occur around the column's connection to base and top plates; however, the highest stress occurs in the weld joint of beam-column base plate, which reveals the critical condition of this joint. Stress evaluations led to the following results:

- For the stress distribution in the direction of the beam (S_{xx}): Stresses of the beam-to-column gusset plate's edge and column, in the place of connection have reached the critical level.
- For the stress distribution in the vertical axis (S_{yy}): critical condition only occurs in the beam-to-column connection.
- For the stress distribution in timber direction (S_{zz}): tensile and compressive forces are the causes of column distortion (corrugation) in the connection.
- For shear stress (S_{xy}): in the place of top and bottom weld joints, the maximum negative and positive stresses occur in the column.
- For plastic stresses, weld joints and especially the weld of base plate are the weakest points of the connection.

9.3. A-2 Model

This model has selected to evaluate the effects of the floor concreting with the usage of shear resisting elements on column's wing. In this model, a 120×95 cm^2 floor slab with minimum thermal reinforcement is included. In addition, two shear-resisting elements with U50 channel sections have welded to top wing of the beam with 35 and 60 cm distances from the column edge.

9.4. Evaluation of A-2 Model

In this model, floor concreting and concrete filled column methods have used for strengthening, and shear-resisting elements have used for improvement of shear transmission in floor slab and beam.

Evaluating the Von Mises stress in this model shows that, the stress amounts of the welds are high. The usage of concrete increased the rigidity of the upper part of the connection, which in turn provoke the stress concentration in the base plate. Therefore, in the location of beam-column base plate, the stress is substantially increased. Stress evaluations led to the following results:

- For stress distribution of (S_{xx}), (S_{yy}), (S_{zz}): a uniform distribution with no major stress concentration.
- For shear stress (S_{xy}): the stress distribution in the beam is uniform but in the case of column, similar to the other models, a considerable shear stress occurs due to tension and compression in the base (Fig. 7).
- For plastic stress: like other models, the amount of stress is maximal in the welds. Therefore, they are the weakest points of the connection.

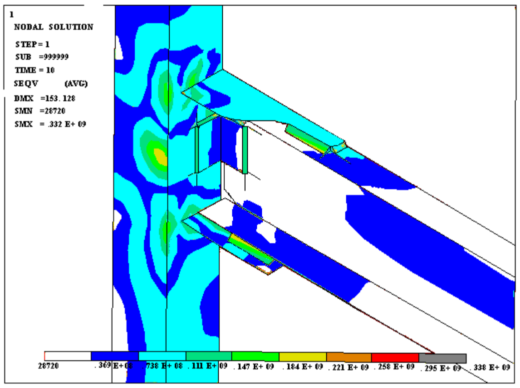

Figure 6. Von Mises stress of A-1 model

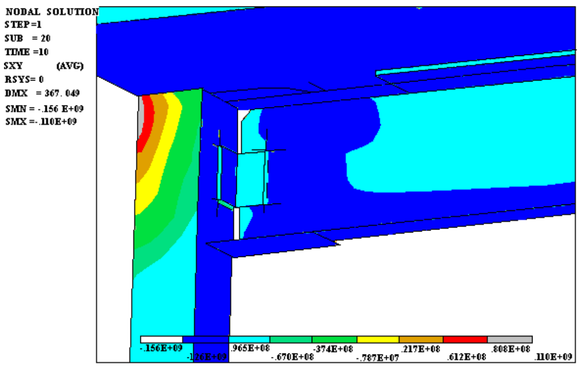

Figure 7. S_{xy} stress distribution of A-2 model

10. CONCLUSSION

Generally, in this paper it has been found that:

- Utilizing concrete and stiffener in column results uniform stress distribution as well as stress concentration prevention.
- The analyses emphasize the fact that welds are the most sensitive connection points for their design especial care is essential.
- Utilizing column and floor concrete considerably affects rotational stiffness and bending moment capacity.
- Utilizing simultaneous performance of concrete and steel is a suitable method for steel structure connections retrofitting.
- In order to analyze cases such as the utilized elements, it is vital to consider the appropriate non-linear analysis, to properly consider the supports and to calibrate numerical and experimental data.
- Maximum difference between numerical and experimental data is 10% which reveals that the conducted modeling enjoys adequate accuracy.
- In order to diminish destructive effects of earthquakes on steel structures, a suitable method is to employ the precise analysis mentioned in this paper for utilizing concrete and steel simultaneously.

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