Nonlinear Deformation of Satchel Connections

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ABSTRACT: *A type of connection, called Khorjeeni, has been widely used in Iran for low-rise steel frame buildings. The term describes the configuration with two beam profiles placed on the sides of the columns. Here we use the English equivalent and refer to this connection as satchel connection. The beam profiles are fastened to the columns by top and seat angles placed parallel to the beam. Depending on the relative stiffness of the angles, profiles, and the strength of the welds, the connection stiffness may range from very stiff to quite flexible. A large number of steel frames with satchel connections have been used in the earthquake zones, replacing masonry buildings destroyed by previous earthquakes. However, their effectiveness remains unclear under seismic forces. The stress in the connection is due to bending, shear, and torsion. Restraining in-plane forces are developed in the angles as they are typically welded to the flange of the beam profiles. These restrictions increase the bending capacity of the angle substantially, making the connection much stiffer. Excessive drift is often a major problem in flexible connections. Until recently building codes did not allow flexible connections in earthquake or high wind regions. The economic and practical advantages of flexible connections, however, have provided impetus for much research in recent years. Based on this research the Eurocode has already provided specifications for the use of flexible connections and the AISC Code is developing similar specifications. This paper discusses the nonlinear deformations of satchel connections. The purpose of the study is to develop a better understanding of the behavior of this type of connection under monotonic and cyclic loads, and to identify the areas requiring special attention during the design.*

Keywords: Satchel (Khorjeeni) Connection; Nonlinear analysis; Steel structure

1. INTRODUCTION

A steel frame connection that has been widely used for low-rise buildings in Iran is called Khorjeeni. The term is descriptive of the connection configuration with beam profiles placed on both sides of the columns. Here we use the equivalent English term satchel connection. The beam profiles are attached to the columns by top and seat angles placed on the sides of the columns, parallel to the beam profiles. The development and widespread use of this connection in Iran seems to have been dictated by a combination of economic and practical requirements. Since large sections are not readily available, the use of two smaller profiles, although not as efficient as a single profile, provides sufficient moment of inertia for design. The profiles are used as continuous members over several spans. This in addition to reducing the peak moments, would eliminate the need for much precision cutting and welding. The connection is made by welding the seat angles to the columns before their erection, providing a seat for the beam profiles during the construction. Top angles are then placed and welded to both the column and the beam profile. In the common configuration the seat angle has a larger width than that of the beam flange, while the top angle has a smaller width. This would eliminate the need for overhead field welding. Thus, erection of the frame can be done by average welders, without a need for highly skilled workers and precision work. Although satchel connections are commonly used in permanent structures, they can be used in temporary structures as well. In this case dismantling can be done by simply grinding the weld of the angles. Because, unlike standard connections, the beam profiles are not cut to fit between the columns, the profiles can be used again in other frame configurations.

It is well known that steel frames, because of their ductility reserve, can perform better than other structures in absorbing the energy of earthquakes. This is apparently the reason for the use of steel frames in the reconstruction of damaged buildings in Iran, such as those in Manjil. However, for a steel frame to be effective during earthquake, the members must reach their plastic capacity. In other words, failure of the members must not be due to instability and other premature causes. The concept of steel frames with strong-column-weak-beam recommended for earthquake regions is based on this idea. While extensive laboratory studies and post-seismic investigations have proved that adequately designed steel frames with certain standard connections do in fact perform well during earthquake, such results are not as yet available for frames with satchel connections. In spite of its widespread use in Iran, so far only a few investigations have been devoted to this connection. These include studies by Tahooni, S., 1991, Agha Koochak, A.A., 1991, Moghaddam, H., and Karami R., 1991, Simonian, W., and Kaffee, M.A., 1990, and Moghaddam, H., and Koohian, R., 1994. Much needs to be done in terms of understanding the behavior of this connection and the development of adequate design procedures.

In spite of its simple form the stress picture in satchel connections can be complex. One interesting aspect of this connection is that its strength and stiffness can vary drastically with the weld configuration. For example, if the seat angle is welded to the column only, then this angle provides little bending resistance, as the angle leg acts like a cantilever plate. In this case the connection provides little lateral resistance to drift of the structure under lateral loads. On the other hand, if sufficient weld is used to connect the seat angle to the beam profile, axial deformation of the seat angle leg is restrained, leading to a substantial increase in the bending strength of the angle. Although such welding would reduce the twisting of the connection, in general the connection does not seem to be very strong against drift. This strength may be enhanced by bracing or other shear resisting elements.

Still another aspect of the problem, that must be understood, is the inelastic behavior of the connection, an important effect for withstanding earthquake forces. Designers know well that it is not easy to justify the cost if the frame is going to remain elastic during earthquakes with rare frequencies of occurrence. On the other hand should such an earthquake happen during the life of the structure, it is not possible to justify its collapse and potential loss of life. Development of inelastic deformations, so long as they do not make the structure unstable, is a logical compromise solution to this dilemma. That is why the two track earthquake design procedure is finding general acceptance. In such a design, under small and common earthquakes, the structure remains elastic, while under rare and extreme earthquakes permanent deformation and damage can result. This damage may or may not be reparable after the quake. However, any loss of life will be avoided.

This paper presents some results of an investigation into the nonlinear behavior of satchel connections and the effect of the major parameters on the behavior of the connection. A good understanding of the post elastic behavior of satchel connections is the first step toward the development of a rational earthquake design procedure for this type of connection.

2. MODELING

In order to determine the effect of nonlinear deformations on the behavior of satchel connections, both linear and nonlinear analyses were performed on three-dimensional models of a typical connection. The nonlinear analysis was performed by the computer program Abacus, 1994, which provides much capability for both geometric and material nonlinearities. The complex geometry of the three-dimensional model would make the finite element modeling with three-dimensional elements extremely time consuming. To alleviate this problem another program, I-DEAS, Lawry, M. H., 1976, was used to define the geometry of the model. The model included the beam, the top and seat angles, as well as the welds. A three-dimensional mesh was generated by I-DEAS. Since we were interested in the relative rotation of the beam and the column, the column was taken as a rigid body, and thus did not need to be modeled. The analytical model replicated a half-scale physical model that was going to be tested for comparison, Figure 1. The prototype frame under lateral load develops zero moment at the mid-spans. Thus, this point acts as a hinge but with support. In the physical model the beam spans are extended to the mid-spans, where they would be supported, thus providing the same condition as a hinge. The beam profile for the model was an $S4\times 7.7$, the top angle $L\,2\frac{1}{2}\times 2\times \frac{1}{4}$ and the seat angle $L\,3\frac{1}{2}\times 2\frac{1}{2}\times \frac{1}{4}$.

The element used to model the geometry was a twenty-node quadratic brick element. Although a similar linear element was available, the quadratic element was chosen in order to provide a better picture of the stress, without requiring an excessive number of elements. This was especially important because we wanted to obtain the stresses in the 1/4 inch fillet welds around the angles. The force produced by the lateral loads on the connection is because of the shear force induced at the mid-span of

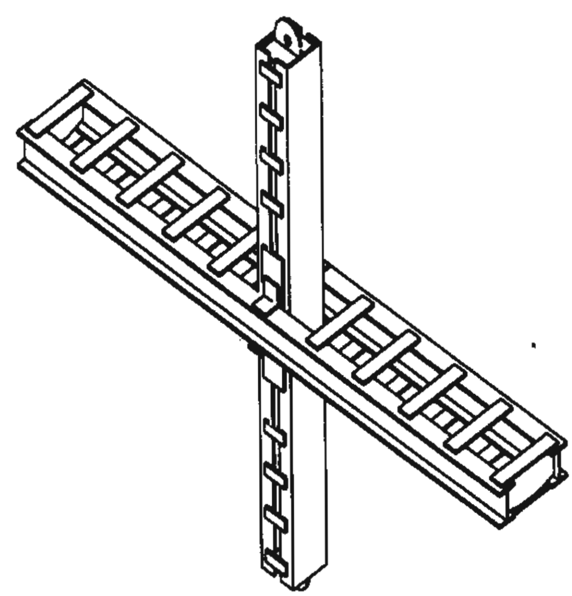

Figure 1. Satchel connection configuration.

the beam. This shear force was taken at 50 kips in the analytical model.The plastic limit for the angles and the beam was taken at 36 ksi, and that of the welds, 60 ksi. Both materials were assumed to be elastic-rigid plastic. Although in some situations welds can be brittle materials, it is not difficult to produce welds with significant inelastic deformations.

The static analysis was conducted for monotonically increasing loads. The nonlinear analysis was carried out by Abacus using Newton's method. Thus, the solution was obtained by a series of increments with iterations at each increment until equilibrium was reached. Automatic incrementation was used since the efficiency of the computation was dependent on the size of the increments.

In frames with satchel connections usually double channels are used for columns, Figure 1. The angles are connected to the column through a plate which is welded to the side of the column. As the plate is welded around its periphery only, under a tension force its interior points can move away from the column. On the other hand under a compression force the plate presses against the column. That is, the flexibility of the connection is much higher under tension than compression. To allow for this effect, gap elements were used between the interior nodes of the plate and the angle legs, and the adjacent nodes on the beam or column flanges. These gap elements allow the two nodes to move away from each other but do not let them pass each other.

An area of concern was characterization of the actual behavior of the welds. It has been shown, Heuser, A., 1987, that the different zones developed in the base material differ in stiffness. This is true in both elastic and plastic regions. Because of the complexity of the situation no attempt was made to include these different zones, nor was the effect of residual stresses included. When the weld is loaded parallel to its axis, it is capable of large ductility, albeit at the price of strength reduction. On the other hand when the weld is loaded perpendicular to its axis, its strength is increased, while its ductility is reduced. The AISC specifications require that the upper limit of shear strength be taken at 0.6 of the weld's specified yield stress. This is based on the assumption that the weld will fail in shear. Therefore, the AISC Code does not take into account the orientation of the weld. Regardless of the weld's orientation, fillet welds do display some amount of ductility, Salmon, C.G., 1990. In general, it is difficult to consider the effect of the weld orientation and this effect was not considered here. Care must also be exercised in selecting the strength of the weld material. Under tension, if the weld metal strength is larger than that of the base metal, plastic strain occurs in the base material resulting in necking and failure outside the weld area. On the other hand, if the weld metal strength is lower than that of the base metal, plastic strain occurs in the weld with a low tensile elongation, Phillips, A. L., 1993. Weld impurity is another parameter that was not considered in this study.

Because of symmetry, only half of the configuration needed to be modeled, Figure 2. This represents the case where two beam profiles are used on the sides of the column. However, the displacement of the beam profile out of the plane was not restricted. In this way the behavior of spandrel beams, with only one single beam profile, could also be examined. For the case of double profiles, out of plane displacements can take place unless large diaphragms are placed between the two beam profiles to restrict any motion due to imbalance in the load on the two profiles.

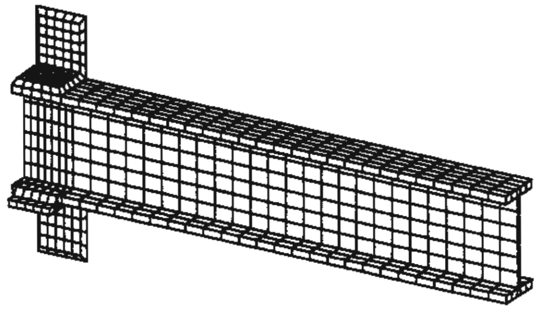

Figure 2. Finite element model of the connection.

3. RESULTS OF THE ANALYSIS

The model described above was first subjected to a monotonically increasing load. The analysis was terminated after 14 load increments, at a total load of 4175 pounds. The calculations could have continued for higher loads if the deflection had not become excessive.

Figure 3 shows the deformed configuration. It is clear from this figure that out of plane and rotational deformations of the profile take place. This should not come as a surprise, as the support provided by the angles with one leg attached to the column and the other to the beam is not symmetrical. Because of this lack of symmetry we expect unsymmetrical deformations. Figure 4 shows this unsymmetrical behavior. However, the stronger the angles, the less the unsymmetrical deformations. Thus, strong angles and welds can reduce the unsymmetrical

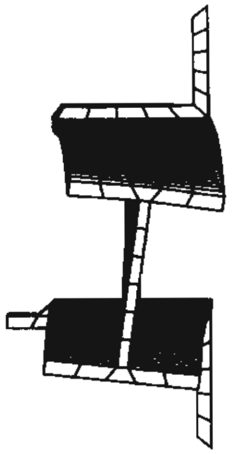

Figure 3. Deformed configuration of satchel connection.

deformations of the beam profiles. With beams of double profiles another important parameter is the lacing of the profiles. A substantially laced cross-section would force the two profiles to act together, reducing unsymmetrical deformations and the possibility of lateral-torsional buckling. Filling the area between the two beam profiles also increases the resistance against lateral-torsional buckling.

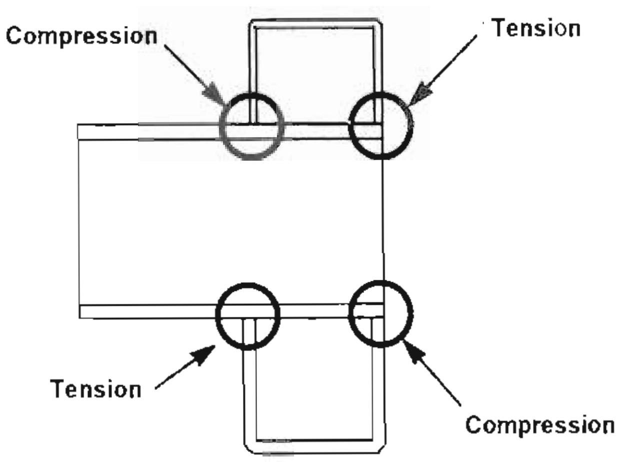

Figure 4. Back view of connection.

The stress distribution in the top angle is shown in Figure 5, and that in the seat angle is shown in Figure 6. Figure 7 shows the view of the stress distribution in the connection from the side of the beam. These stresses were obtained at the fourth load increment with an equivalent load of 2875 lbs.

Figure 8 shows the von Mises stresses at two points, one located in the weld and the other just below it in the flange of the beam at different load levels. The graph in this figure shows that the point in the flange and the adjacent point in the weld follow approximately the same loading pattern until the flange element reaches its yield stress. Beyond this point the load is carried by the weld until it reaches its yield stress. From the stress distributions at different load increments, Figure 8, it is clear that the areas of high stress originate in the weld near the junction of the angle and the beam. On the side of the connection, where the load is applied, the edge of the top angle is primarily in tension, while that of the bottom angle is in compression. The opposite condition is true for the side of the connection away from the load. This indicates that the angles, along with the beam, experience torsion. Figure 8 also shows that at about the sixth load increment the point on the flange becomes plastic, while the adjacent point in the weld, which has a higher capacity, remains elastic for another two load increments. This indicates that the connection will perform well if the weld is not brittle. We can see from these results that the areas of high stress are along the knees of the angles and the adjacent welds. It is good practice therefore to have continuous welds at the corners of the angles. If this is not possible, then end returns should be used for the welds in order to decrease the concentration of stresses near the knees of the angles. This is especially true if the welding is not inspected or if

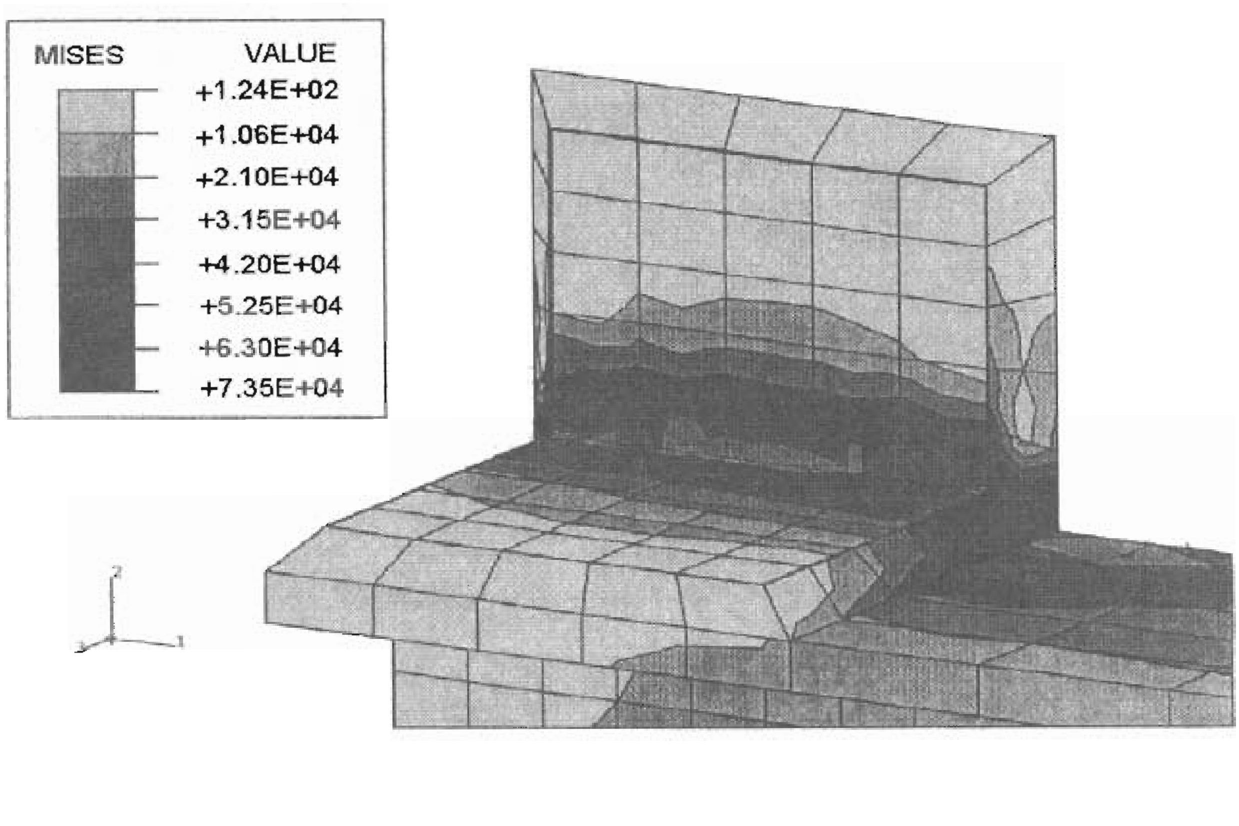

Figure 5. Von mises stress distribution in top angle.

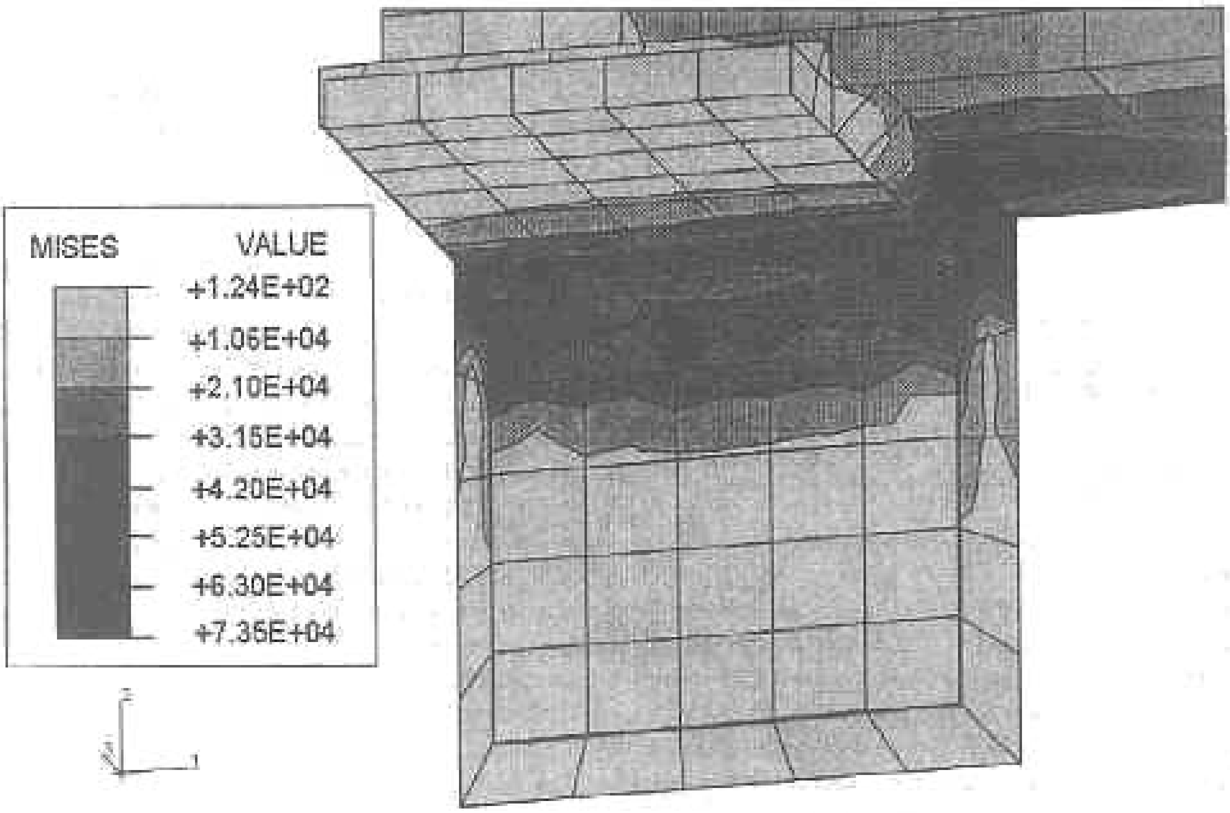

Figure.6. Von mises stress distributon in bottom angle.

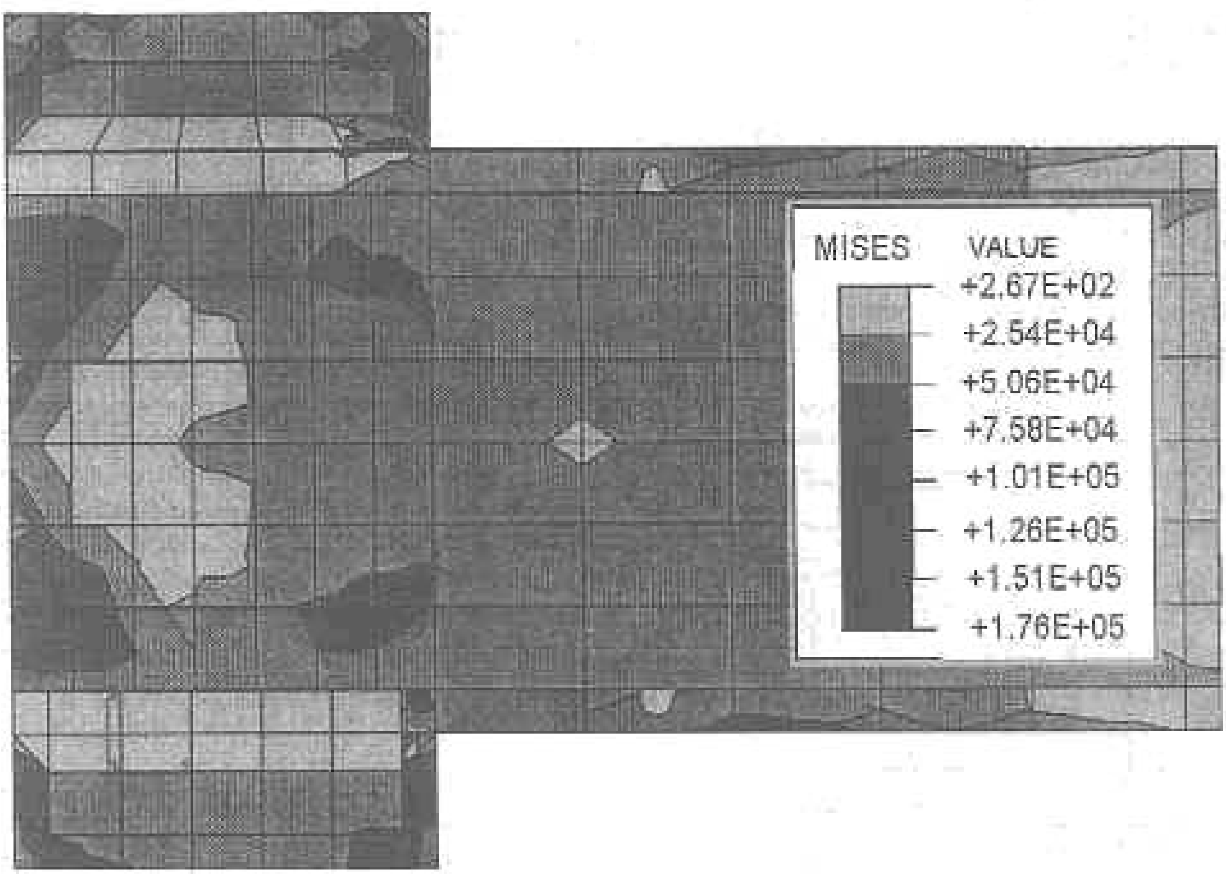

Figure 7. Von mises stress distribution in beam.

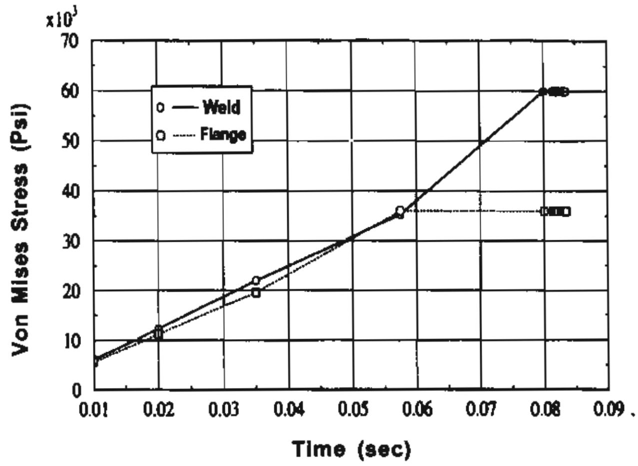

Figure 8. Von mises stress in weld and flanges.

not of the highest quality. The areas of concern for the beam are in the web near the edges of the connection angles. The size of the weld also has an influence on the behavior of the connection. The results of analyses with different weld sizes indicate that the material becomes plastic on the bottom of the connection. This may be attributed to the fact that the weld connecting the top angle was 53% smaller than the weld of the bottom angle.

The moment-rotation curve for monotonic loading is shown in Figure 9. We can see from this figure that the connection undergoes significant plastic deformations at the fifth load increment, corresponding with a 4000 pound load. The initial stiffness of this connection is found to be 74,000 kip-in/rad. Aziziamini, A., 1987, reports test results for standard semi-rigid connections with bolted top and seat as well as web angles. The initial stiffness measured for such connections ranged from 15300 to 57900 kip-in/rad. Therefore, satchel connections can provide a stiffness higher than that of some standard connection. It can be seen that for modeling the behavior of satchel connections in frames a bilinear moment-rotation curve would be appropriate. In fact good results may be obtained by using an elastic-rigid plastic spring to model the connection.

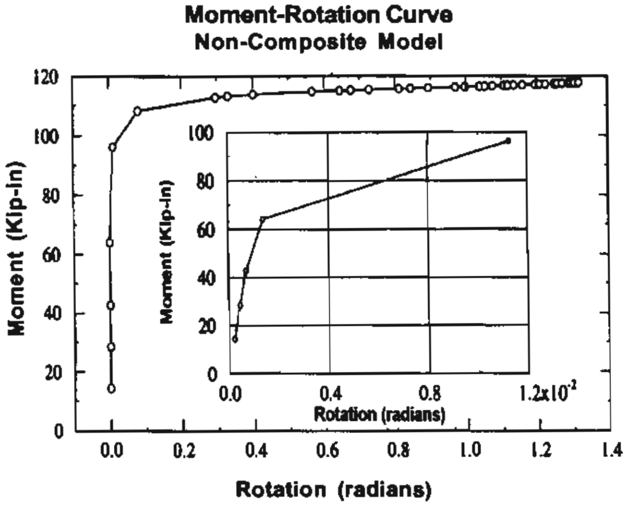

Figure 9. Moment-rotation curve for monotonic loading.

4. CYCLIC LOADING

The connection was also subjected to the cyclic loading of Figure 10, in order to examine its behavior under earthquake forces. The maximum equivalent load was set at 5000 pounds. This was done to insure that under cyclic loading the model would go through at least one elastic cycle.

The resulting hysteresis loop, at the end of the analysis, is shown in Figure 11. This figure shows that some plastic deformation occurs during the third and fourth cycles of loading, as indicated by the increased rotations. Large plastic deformations were occurring during the final cycle before the analysis was terminated. During the early part of the analysis the areas of high stress were concentrated at the base of the connection angles, where they are connected to the beam. As the magnitude of the load increased, the plastic zone spread through the web and to the leg of the angle connected to the column, just above its vertex. As with monotonic loading these are critical points. A large number of load reversals may lead to fracture and fatigue at these points.

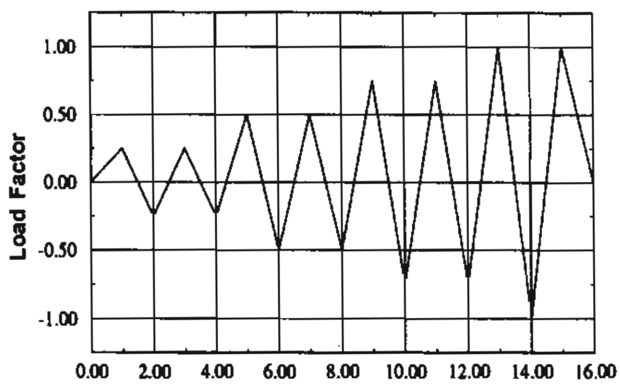

Figure 10. Load cycles.

5. MOMENT-ROTATION CURVE

From the results discussed above, the moment-rotation curves were established for both monotonic and cyclic loading, Figures 9 and 11. This was done by finding the rotation of the line along the middle of the beam web. The x-and y-displacements of the points along this line were recorded at each load increment during the analysis. The change in the slope was calculated for each increment, the moment arm was taken from the point of application of the load to the center line of the angles. These moment-rotation curves can be used in modeling satchel connections in frames. As mentioned earlier a bilinear curve or elastic-rigid plastic spring may be used for monotonic loading. For cyclic loading, such as produced by earthquakes, a hysteresis loop, Figure 11, must be used if the time history of the deformations is to be obtained from the analysis. In this figure the change from one loop to the next shows the amount of inelastic deformations that occurred during that cycle. Comparing the hysteresis loop obtained for satchel connections, Figure 11, with standard steel connections used for frames in earthquake regions, for example, Figure C-9.9 in the AISC-LRFD Code of 1994, we can conclude that satchel connections can absorb less energy, and therefore are not as efficient in dissipating the energy of earthquakes. The reason for this can be the smaller area of the angle, of which only a portion becomes plastic compared to much larger sections of I shaped profiles that become plastic.

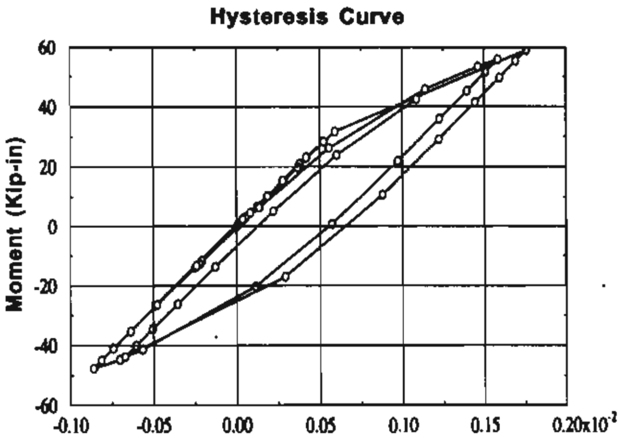

Figure 11. Hysteresis diagram.

6. CONCLUSION

The purpose of this study was to examine the nonlinear behavior of satchel connections. A nonlinear finite element analysis was conducted to determine the state of the stress within a typical connection. This analysis indicated that, if designed properly, satchel connections can exhibit good mechanical behavior as part of steel frames, with a high stiffness in the elastic range. However, their energy absorption capacity does not seem to be as high as standard steel connections used in earthquake regions. Further parametric studies may be needed to examine the effect of the weld orientation, angle and beam sizes on the behavior of the connection.

REFERENCES

1. Tahooni, S. (1991). "Stiffness of Khorjeeni Connections", Report Submitted to the Housing Foundation of Iran.
2. Agha Koochak, A. A., and Hossein Khani, A. (1991). "Investigation of the Behavior of Common Connections under Earthquake", Report Submitted to Islamic Azad University, Southern Branch, Iran.
3. Moghaddam, H., and Karami, R. (1991). "Investigation of the Mechanical-Structural Behavior of Khorjeeni Connections and their Role in Earthquake Design", Report Submitted to Sharif University of Technology, Iran.
4. Simonian, W., and Kaffee, M.A. (1990). "Inclusion of Scissors Connections in Matrix Analysis of Structures", Abdi Publications.
5. Moghaddam, H., and Koohian, R. (1994). "Strength of Steel Buildings with Semi-Rigid Connections (Khorjeeni Connections) Against Earthquake Loads", IIEES Report No. 10-96-74.
6. Abacus (1994). Standard User's Manual. Version 5.4, Hibbitt, Karlson & Sorensen, Inc., Rhode Island.
7. Aziziamini, A., Bradburn, J.H., and Radziminski J.B. (1987). "Initial Stiffness of Semi-Rigid Beam-to-Column Connections", Joint Flexibility in Steel Frames. W. F. Chen, ed., Elsevier Applied Science Publishers, Ltd., New York.
8. Heuser, A., Twickler, R., and Dahl, W. (1987). "Experimental Investigations of the Failure Behavior of Welded Joints and its Numerical Simulation by Using Elastic-plastic Finite Element Calculations", The Fracture Mechanics of Welds, EGF Pub. 2 (Edited by J.G. Blavel and K.-H. Schwalbe), 1987, Mechanical Engineering Publications, London. 97-124.
9. Hilleborg, A. M., Modeer, A., and Petersson P.E. (1976). "Analysis of Crack Formation and Crack Growth in Concrete by Means of Fracture Mechanics and Finite Elements", Cement and Concrete Research, **6**, 773-782.
10. Lawry, M. H. (1976). I-DEAS Master Series Student Guide. Structural Dynamics Research Corporation, Ohio, 1993.
11. Phillips, A. L. (1993). Editor. Welding Handbook: Fundamentals of Welding, 5th edition, Section One, American Welding Society, New York, 1963.
12. Salmon, C.G., and Johnson, J.E. (1990). "Steel Structures: Design and Behavior, Emphasizing Load and Resistance Factor Design", 3rd edition, Harper & Row Publishers Inc., New York.