

Eccentrically Braced Frame Construction—A Case History

JAMES R. LIBBY

The building described in this paper is the proposed new Bank of America Regional Office and Branch Bank Building located in San Diego, California. The building is currently under construction. Completion is scheduled for December 1981.

The building has a tower incorporating frames having joints with intentionally built-in eccentricities. The joint eccentricity is illustrated in Figs. 1 and 2. In Fig. 1, one sees an elevation of a frame having joints referred to in this presentation as "eccentric." In Fig. 2, a detail of an eccentric joint is shown. Providing this eccentricity is, of course, exactly what engineers were taught to avoid during their years of formal education. The motivation for providing the eccentricity is to achieve a framing configuration that will deform in a highly energy-absorbing manner when subjected to severe ground shaking of the type associated with great earthquakes. Concentrically braced frames do not have the ability to absorb large amounts of energy when subjected to seismically-induced forces. The means of achieving the large energy-absorbing capability consists of detailing the eccentric link in a manner that assures an inelastic moment capacity that is larger than the inelastic shear strength.

For interesting and comprehensive descriptions of the technical aspects of this type of framing, how it performs under very high loads and the principles used in their design, the works of Roeder, Popov and Teal are recommended references.¹⁻⁴

In aseismic structural design this is a relatively new and unique design concept. One might wonder why it would be used. The principal purpose of this paper is to describe why eccentrically braced frames were selected for the San Diego Bank of America building.

GENERAL REQUIREMENTS OF THE CLIENT

The architect on this project is the firm of Tucker, Sadler & Associates of San Diego. The firm is well known for their outstanding work in major buildings. Their work has included high, medium and low rise buildings of all uses. Their mark will be recognized and appreciated in San Diego for many years to come.

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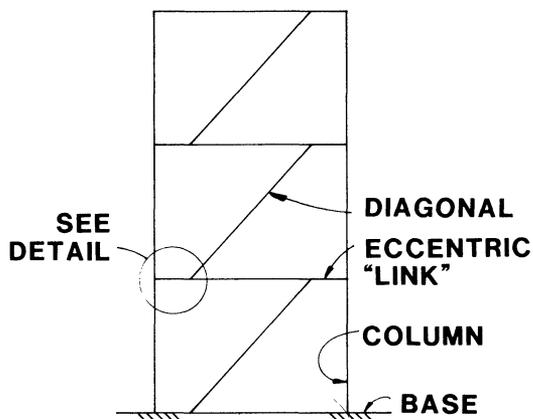


Figure 1

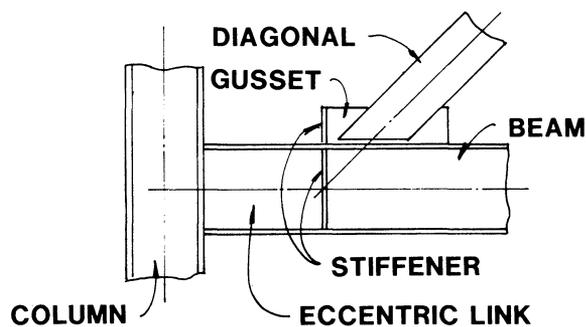


Figure 2

The configuration the architects selected for this structure provides the 192,000 sq ft of office space the Bank requires in a tower that extends out of a five-story concrete base. The base contains three levels of subterranean parking accommodating 377 automobiles, a semi-subterranean branch bank and a semi-above grade plaza-terrace level. The branch bank is approximately 200 by 300 ft in plan. It accommodates 24 tellers. The plaza-terrace level is a public area which will be park-like in a downtown setting, a feature that is certain to significantly enhance the beauty of the area. The building, as depicted by a model, is shown in Fig. 3.

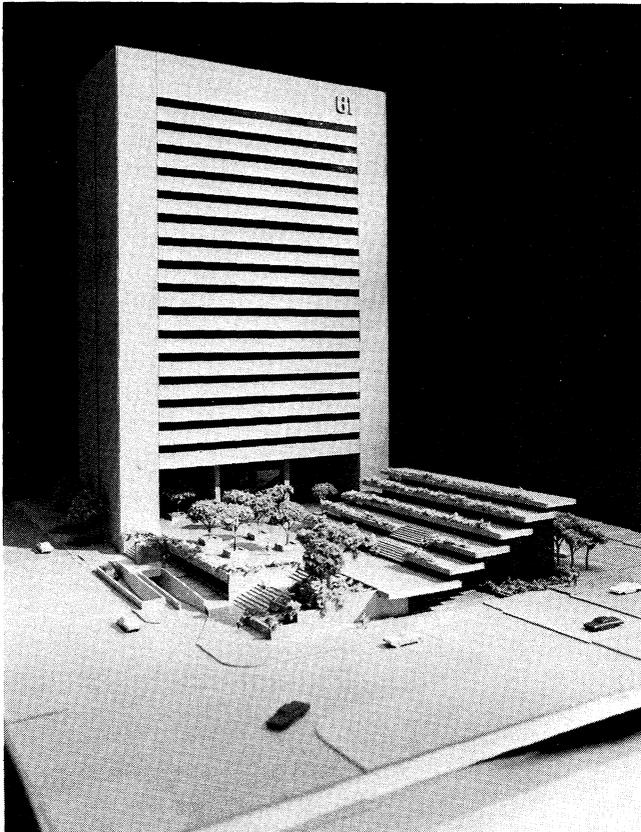


Figure 3

DETAIL OF THE TOWER STRUCTURE

The tower contains 16 floors of office space, with the first office level being 41 ft above the plaza level. Hence, the building has the height of a 19-story building. In plan, the tower is 86 ft wide in the north-south direction and 192 ft long in the east-west direction. It consists of two cores about 21 ft long at each end, separated by the office area as shown in Fig. 4. The office area contains only four columns, which are set back from the facade of the building to make them less visible at street level.

At the outset, the architectural designers did not want any columns at the plaza level in the open space between the cores. The structural problems and costs associated with a span of 148 ft between the cores convinced them the four interior columns should be accepted.

RATIONALE FOR USING ECCENTRICALLY-BRACED FRAMES

San Diego is located in seismic zone 3 as defined in the Uniform Building Code.⁵ This document contains aseismic design provisions stipulating that buildings over 160 ft high must incorporate ductile moment-resisting frames in their lateral-load-resisting systems. Complete ductile frame systems are permitted, as are dual systems. Dual systems

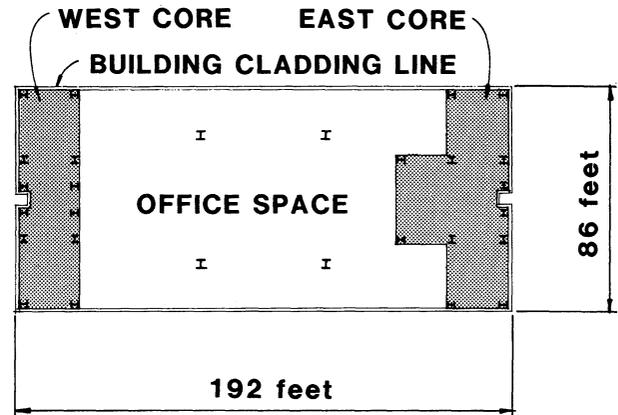


Figure 4

must be composed of shear walls or braced frames capable of resisting 100% of the code-mandated lateral forces combined with a system of completely independent ductile frames capable of resisting not less than 25% of the code-mandated lateral forces. In addition, dual systems must have a substantially complete frame for resisting vertical loads.

The configuration of the building as envisioned by the architect did not lend itself to an efficient complete vertical moment frame nor to a complete ductile moment frame solution. The close column spacings in the core areas could perform efficiently as components in moment frames, but the long beam spans in the large area between the cores would be relatively flexible and could not be efficiently used in moment-resisting frames. The temptation to use a ductile moment frame solution, with moderate beam spans above the first level with transfer girders at the first level and only four interior columns between the first level and the plaza level (Fig. 5), was easily resisted by simply recalling the results observed with the "soft-first-story" Olive View Hospital in the 1971 San Fernando Earthquake. The performance of the Imperial County Services Building during the October 15, 1979 earthquake confirmed the wisdom of this decision. A dual system was not feasible with this configuration because it simply was not possible to provide both a ductile moment frame and a braced frame with the building layout desired.

These facts led to the conclusion that the eccentrically braced frame would be the best system for this particular structure. The eccentrically braced frame is a "dual system" in a sense, but it is certainly not a "dual system" in the UBC sense. An appealing characteristic of the eccentrically braced frame is the fact that under low-to-moderate ground shaking it will perform as a braced frame rather than as a moment frame. Hence, a structure using this mode of construction should experience small drifts, little if any architectural damage, and no structural damage under such shaking. On the other hand, under the ground shaking associated with a great seismic event, the "links" in the

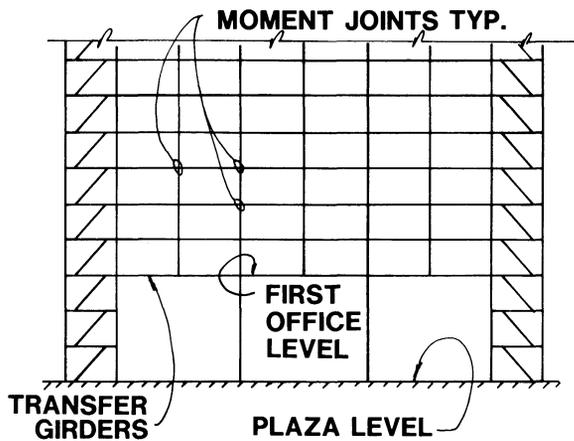


Figure 5

eccentrically braced frames are expected to deform inelastically and absorb great amounts of energy. Although the building may require extensive structural repairs, or even be abandoned after such an event, it would be expected to sustain such shaking without collapse.

At the time it was decided the eccentrically braced frame was the best configuration for this building, it was apparent special approval of the San Diego Building Inspection Department would be required for its use. This was necessary because the structure is over 160 ft high and the framing scheme proposed did not conform to the schemes described in the code for high buildings. To obtain permission to use the system, application was made to the Board of Appeals. Prior to allowing consideration of the request to use this principle, the Building Inspection Department requested the Structural Engineers Association of San Diego (SEAOSD) to render an opinion relative to the use of eccentrically braced frames in areas subject to earthquake-induced ground shaking. Subsequently, a conditional permit to use eccentrically braced frames using a *K*-coefficient of unity was granted. One condition was that Professor Popov be engaged to review the design. In addition, reference was made in the approval to the SEAOSD report and the thoughts contained therein.

STRUCTURAL DETAILS OF THE LATERAL-LOAD-RESISTING SYSTEM

A typical tower-floor plan is shown in Fig. 6. An examination of the plan will reveal there are eight frames provided in each direction. It will also be observed that they are arranged in such a way that there is no intentional eccentricity in the layout of the lateral-load-resisting elements in either of the two orthogonal directions. To provide additional redundancy, all frames in each direction are made identical in dimension and member size. Architectural details, such as the recessed windows at each end of the building, and functional details, such as the stair and elevator penetrations, resulted in diaphragms in the core areas

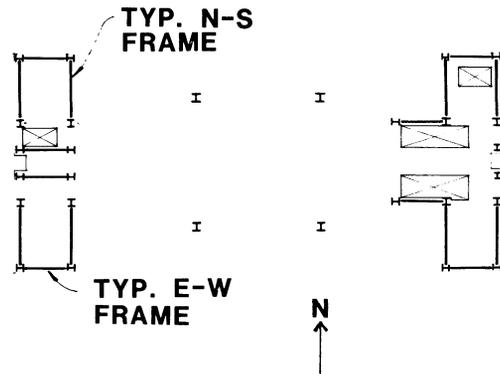


Figure 6

that are quite complex and not as straightforward as would have been preferred.

In elevation, as was explained previously, the east-west frames are all identical, as are the north-south frames. This is illustrated in Fig. 7. It was felt that making the frames in each direction identical would add redundancy to the structure and reduce cost through repetition of details. Note all of the frames are "single-diagonal-frames" with eccentric connections at both ends of the beams, as opposed to being "single-diagonal-frames" with an eccentric connection at only one end or a "two-diagonal-frame." These three configurations are shown in Figs. 8 through 10.

The details of the connection of the diagonal to the beam are shown in Fig. 11. Note that the diagonals are structural tubes and that the connections are all welded. Earlier designs of the connections were made with high strength A490 bolts. The bolted connections were abandoned as a result of industry feedback indicating the welded connection to be less costly.

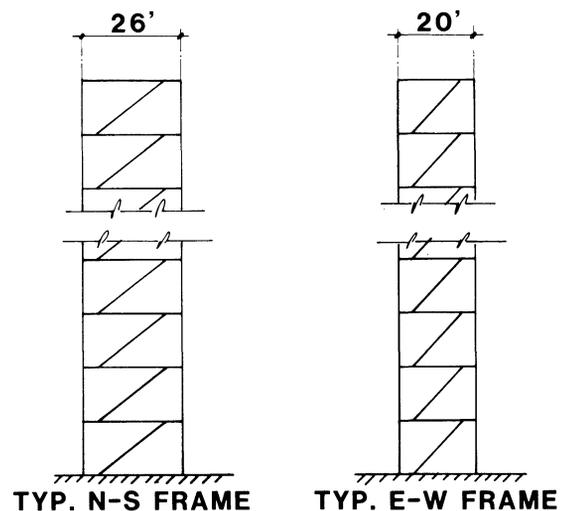


Figure 7

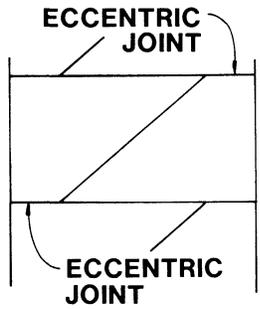


Figure 8

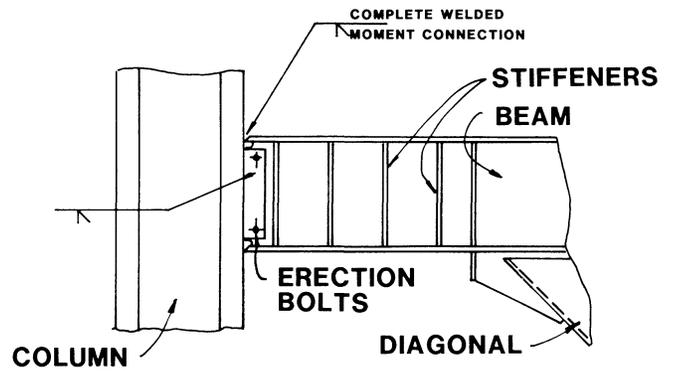


Figure 12

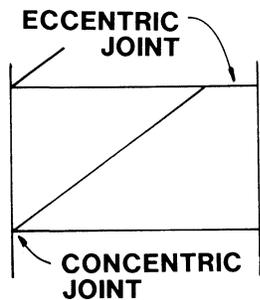


Figure 9

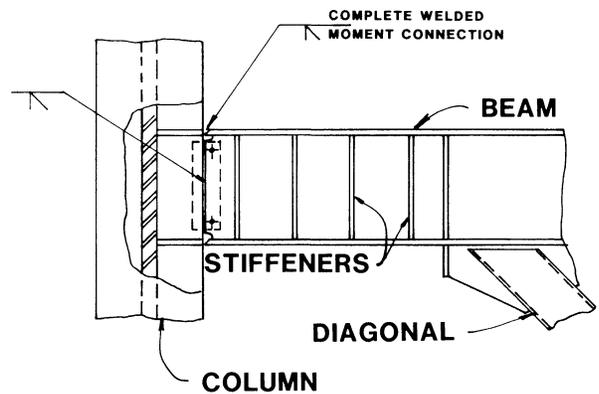


Figure 13

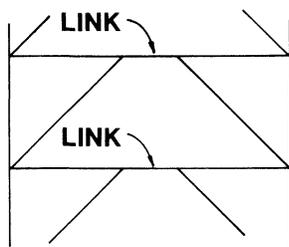


Figure 10

A beam-to-column-flange connection detail is shown in Fig. 12 and a beam-to-column-web connection is shown in Fig. 13. Note the provision of web stiffeners in the eccentric "links" in both figures. These were provided at the suggestion of Prof. Popov to guard against the possibility of web buckling. The tests which Prof. Popov has made on the eccentric connections had been made with specimens having beams of much shallower depth than the W18 beams specified for the Bank of America building. He was concerned that tests had not been made on specimens having an 18-in. depth and that the possibility of web buckling

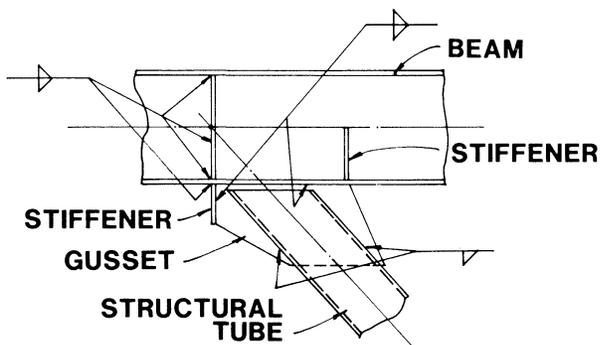


Figure 11

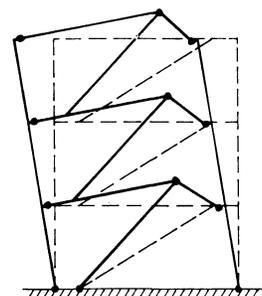


Figure 14

could exist when the links of the deeper beams become stressed above the yield strength. The web stiffeners should prevent this possibility.

Another modification made at Prof. Popov's suggestion was to place the beam-to-column-web connections at the low end of the diagonal, rather than the high end. The reason for this is twofold. First, tests have not been made on beam-to-column-web connections of the type used. In addition, the testing that has been done to date has shown the links at high end of the diagonal are always stressed into the elastic range; the links at the low end remain elastic. This is illustrated in Fig. 14 (from Ref. 1).

ANALYTICAL STUDIES

The computations for the Bank of America building followed the recommendations of Roeder and Popov,¹ i.e., the sections were first proportioned by plastic-analysis methods. Subsequently an elastic analysis was made. The minimum sizes were generally controlled by the elastic design. The effects of loads from floor beams framing into the sides of the beams of the braced frames were found to be significant. The provisions for lateral load, drift and stresses contained in the UBC⁴ were used as limiting values. Because some of the columns are common to braced frames in orthogonal directions, they were investigated under the conditions of having 100% of the lateral load forces in either direction acting simultaneous with 30% of the lateral load forces in the orthogonal direction. This combination was selected because it is the procedure suggested in *Tentative Provisions for the Development of Seismic Regulations for Buildings*,⁶ commonly referred to as ATC 3-06. The stresses under these combinations of loadings were found to be within allowable limits.

The secondary effect, commonly referred to as the *P*-delta effect, was also studied. This effect, when combined with the primary effect, was found to yield results within acceptable stress levels.

SUMMARY OF QUANTITIES AND COST

The following data may be of interest to the reader:

1. The total amount of structural steel is 3,750 tons.
2. The cost of the structural steel is \$4,725,575, which amounts to \$1,260 per ton.
3. The amount of the structural steel is 26.7 lbs/sq ft. The quantity of steel required was, of course, greatly influenced by the long spans in the office areas and the 41 ft between the plaza and the first office levels.

CONCLUDING REMARKS

The eccentrically braced frame is a workable structural configuration that has some significant advantages.

It is not overly complex.

The question of the large inelastic deformation that can occur in the links of the floor beams during a great earthquake was discussed in the report of SEAOSD mentioned above. The concern that such deformation or "kinks" in the floor system might result in the building exits being unusable was expressed. It is recognized that this is a possibility and it certainly should be considered. However, it should also be recognized that inelastic deformations occurring as the result of strong ground shaking can make the exits unusable in buildings constructed with other modes of structural framing as well. This problem is not unique to the eccentrically braced frame.

It is recognized that many other framing options are possible with eccentric joints. These can result from combining frames into multiple frames, rather than using single-bay frames as was done on the Bank of America. The use of frames having eccentric joints only at the upper end of the diagonal of single diagonal braces is intriguing and should result in some savings. The use of frames with two diagonal braces, rather than a single diagonal brace, should offer advantages in certain cases.

Based upon the experience to date, the opportunity of doing another building with this new and interesting method of construction would be welcomed.

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DISCUSSION

Eccentrically Braced Frame Construction—A Case History

Paper by JAMES R. LIBBY
(4th Quarter, 1981)

Discussion by Egor P. Popov

The author describes a challenging application of eccentrically braced frames for a spectacular building. As a consultant for this project on eccentric links, limited test data at the time of designing this structure required the advice for adopting closely spaced double stiffeners along the webs as shown in Fig. 13 of the paper. Subsequent research¹ has shown that the number of such stiffeners along the webs could have been reduced. Moreover, the use of web stiffeners on one side of the webs only appears to be sufficient to inhibit web buckling well into the plastic range.²

It is important to note that, according to the author, the sections were first proportioned by plastic analysis methods, and subsequently examined for compliance with elastic criteria. Precisely such a procedure was recommended for design for the assumed static loads by Roeder and Popov.^{3,4} Since eccentrically braced frames are currently being considered for a number of new buildings, it would be of great value to the profession if the author would offer the benefit of his experience on the following two items.

1. How effective is eccentric bracing in helping control elastic drift of slender frames such as used in this

building? A comment on the associated problem of column anchorage would also be of interest.

2. Was there any tendency in the frame columns for going into single curvature over several stories in the collapse mechanism (Ref. 3 or 4); and, if so, how was this problem resolved? If such conditions develop, columns may buckle in the global manner, but an appropriate choice of floor beams can prevent this. The design of eccentrically braced frames on the elastic basis only can be misleading, although it would satisfy current code provisions.⁵

REFERENCES

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