

SEISMIC JOINTS IN STEEL FRAME BUILDING CONSTRUCTION

C. MARK SAUNDERS



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BIOGRAPHY

Mark Saunders is President of Rutherford & Chekene, one of the major consulting engineering firms in Northern California. He is a graduate of Santa Clara University and has a Masters Degree in Structural Engineering from Stanford University. He has specialized in the design of major buildings in seismic regions for over 30 years. He is currently Vice Chair of AISC TC-9 (Seismic) and has been a member of that committee for over 10 years. He has served as president and board member of several major technical organizations. He is a licensed Structural Engineer in California, Illinois, and Hawaii.

ABSTRACT

Seismic joints are frequently required between adjacent buildings and are often introduced to separate two or more parts of the same building. This paper discusses the use and design issues of seismic joints in buildings, including answering the following questions:

- What is a seismic joint?
- How have seismic joints traditionally been used?
- How do seismic joints differ from expansion joints?
- When should seismic joints be used and when not?
- Where should seismic joints be located?
- How is the appropriate width of the joint determined?
- What are the major structural issues with seismic joints?
- What are the major architectural and fire issues with seismic joints.

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INTRODUCTION

Seismic joints occur naturally when one building is built adjacent to another, whether or not the buildings are linked functionally. Seismic joints are also frequently introduced to separate wings, or other parts of a single building. A seismic joint typically creates a separation between the adjacent buildings or parts of buildings that includes separation of walls, floors, roof and, in the case of joints within the same building, may also include separation, or accommodation for movement of piping, HVAC ducts, and other elements that have a functional need to cross the joint. The design of seismic joints is complex and includes efforts by all members of the design team to assure that the joint is properly sized, adequately sealed from weather and safe to walk on, as well as providing for adequate movement of other systems crossing the joint and means to maintain the fire ratings of the floor, roof and wall systems. Joints are costly and architecturally undesirable, so they should be incorporated with discretion.

TRADITIONAL USE OF SEISMIC JOINTS

The earliest use of seismic joints did not recognize them as joints at all. They were merely the space between adjacent buildings. As structural engineers in seismically active areas thought more about the lateral movements of buildings in seismic events, they began to develop rules of thumb (such as 2 inches per floor) for deciding how wide seismic joints should be. They studied earthquake damage and began to see evidence that buildings had collided and that sometimes very serious damage had occurred. Particularly serious damage sometimes resulted when floors of adjacent buildings did not align, or when one building was much taller than the other.

As seismic analysis evolved to the level of the static analysis methods of the 1950's and 60's, structural engineers began to recognize that certain building shapes resulted in potentially undesirable effects, such as torsion or high collector forces at reentrant corners, that their analysis methods were not yet adequate to deal with. It became a common practice to introduce seismic joints to divide a complexly shaped building into a group of smaller buildings with simple shapes that were easy to analyze and had predictable seismic performance. For example, an L-shaped building was often divided into two rectangles.

Another place where seismic joints have often been introduced is at locations where diaphragms are recognized to be weak, and it is felt to be better to introduce a joint than to suffer the damage that might occur during a seismic event. This is somewhat similar to the practice of introducing joints in sidewalks where they get narrower or change direction. A typical example where this might occur would be in the somewhat common H-shaped building, when the elevators and stairs are located in the narrow crossbar of the H. A diagram of such an example is shown in Figure 1.

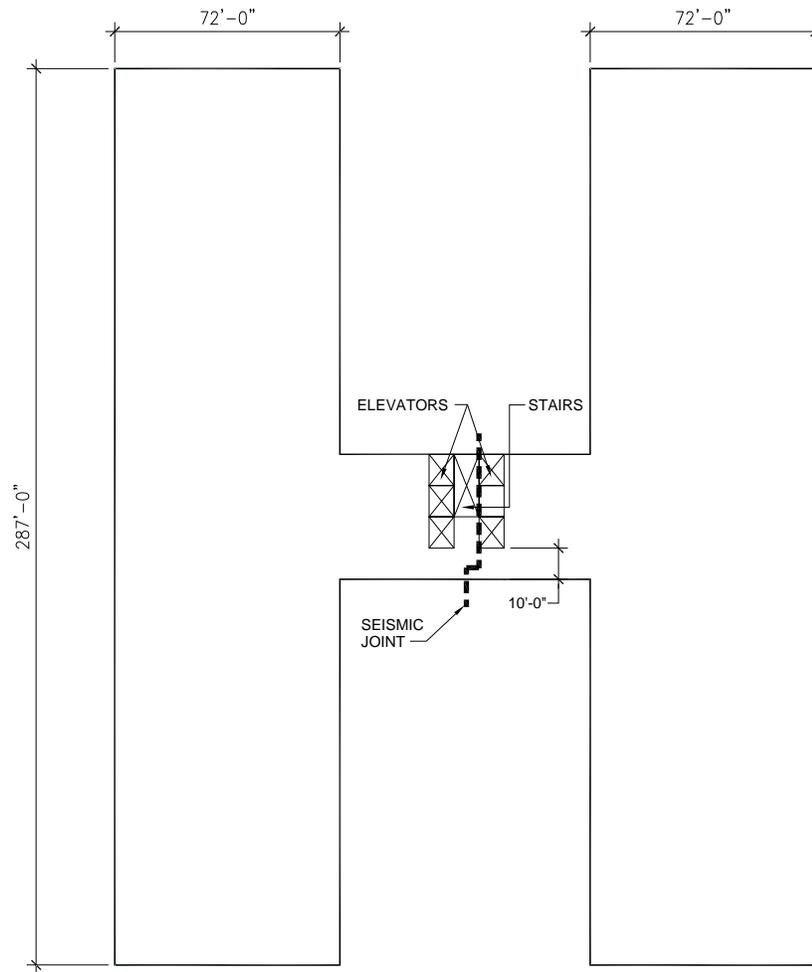


Figure 1. Example of Seismic Joint Location in H-Shaped Building.

Starting with the 1988 UBC, definition and design criteria applicable to various types of irregularities were introduced. The design criteria included force penalties for design of certain elements (e.g. one-third increase not permitted for collector design) when plan irregularities occurred. Once these provisions were adopted, there was additional incentive for engineers to utilize three-dimensional dynamic analyses to better define forces due to irregularities, and the use of seismic joints strictly for analysis ease was less frequent.

SEISMIC JOINT VS. EXPANSION JOINT

Seismic joints are similar to expansion joints, but at the same time very different. Expansion joints are introduced to accommodate building movements caused by shrinkage, creep, or temperature changes. They are often one-way joints, that is, they are primarily intended to accommodate movements in the direction perpendicular to the joint. Expansion joints are also commonly placed at some regular interval of length based on the expected rate of shrinkage or temperature movement expected to occur over the building length. Seismic joints, on the other hand, must accommodate movement in both orthogonal directions simultaneously and their spacing is not typically affected by building length or size.

COST ISSUES

In major urban areas, the largest cost of seismic joints may well be the cost of the land (or lost building area) needed to accommodate the joint. For example, in downtown San Francisco, even in the current poor office economy, office buildings sell for more than \$200 per square foot. If an owner wants to construct a 40 story building on a site with a 100 ft. long side adjacent to another building, and a 4 ft. wide building separation (seismic joint) is needed, the lost square footage would be worth $\$200 \times 100 \times 40 \times 4 = \$3,200,000$.



Seismic Joint Between Adjacent Buildings

For joints within a building, the cost is in the additional structure, the added cost of joint wall and floor covers, additional flashing and sealing, fire blankets, and piping and ductwork expansion devices. Cost estimators tell me that they estimate the cost of wider (2 ft. range) seismic joints at \$150-\$200/lineal foot. For an internal seismic joint of the same size as described above, this would result in a cost of around \$1M.



Seismic Joint Within a Building



WHEN TO USE SEISMIC JOINTS / WHEN NOT

Seismic joints or building separations between adjacent buildings that are constructed at different times, have different ownership, or are otherwise not compatible with each other are clearly necessary and generally unavoidable. Within a single building, however, it is desirable to avoid seismic joints whenever possible, for all of the reasons discussed above. Where seismic joints may have been routinely inserted in the past, modern analysis methods can often provide the engineer with sufficient information and confidence to eliminate the joints. Where a sufficiently detailed and sophisticated analysis is performed, the required forces at reentrant corners, narrow and weak diaphragm areas, etc. can be determined with a fairly high degree of confidence, and appropriate resistance can be provided.

Sometimes it is even possible and desirable to avoid joints between wings of a building that are to be constructed in two or more phases. To accomplish this generally requires that the owner and architect be very sure of what the future wing or wings will be like, and provide sufficient fee to the structural engineer to analyze the building for seismic force distributions that could be expected in the configuration occurring in each phase of its construction. When this is done, the first phase can be designed for the worst case forces of any of the levels of buildout, and accommodation can be made for the connection of the future wings. Obviously, caution is required in using this approach, since future designs could be affected by code changes that would render the assumptions used in the first phase invalid. Generally the agency having jurisdiction would require that the complete building, at each level of completion, meet the code applicable to the most recent stage of construction. In the case of significant changes in seismic codes, this could lead to a requirement to seismically retrofit the previously constructed phases of the building, or to introduce the previously avoided joint.

LOCATION OF SEISMIC JOINTS

When seismic joints are determined to be necessary or desirable for a particular building, the locations of the joints are often obvious and inherent. Often these inherent locations are also the most desirable from the standpoint of mitigating the appearance and cost of the joints. For example, if a joint is introduced because elevators, stairs, mechanical shafts and other floor openings conspire to so weaken the diaphragm that a joint is deemed necessary (see Fig. 1), the beneficial result is that there is very little floor area through which the joint needs to traverse. The various shafts and floor openings can be incorporated as part of the joint and only the narrow area of floor remaining (usually a corridor) requires an expensive joint cover. Similarly, joints that may be employed at reentrant corners and comparable locations have the advantage of being relatively easy to conceal in the exterior wall. Where there are several possible joint locations that would satisfy the functional purpose of the joint, similar considerations to the above should govern the choice of location and path of the joint.

WIDTH OF SEISMIC JOINTS

The width of seismic joints in modern buildings can vary from just a few inches to several feet, depending on building height and stiffness. Joints in more modern buildings (post-1988) tend to be much wider than their similar predecessors. This is due to several major factors, the most important of which is changes in the codes. Other contributing factors are the lower lateral stiffness of many modern buildings, and the greater recognition by engineers of the magnitude of real lateral deformations. This latter factor is, of course, behind the changes in the codes.

It is instructive to review the history of drift calculations and seismic joint width from the perspective of code development. The following table is based on assumptions for a 10-story Special Steel Moment Frame building in a basic Seismic Zone 4 (not near-fault) area, on good soil. Separations are calculated for two identical 10-story wings.

Code	V basis	C _d	Drift Limit	Relative Stiffness (Note2)	Bldg. Separation Criterion	Seismic Separation/ Joint Width (Note 4)
1982 UBC	V=ZIKCSW K=0.67	1.0 (Note1)	0.005	3.6 (Note 3)	2312(h) “..separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.” (Note 5)	16.2 inches (@0.005) 8.1 inches (@.0025) (Note 3)
1888/94 UBC	V=ZICW/R _w R _w =12	1.0*	0.03/R _w = 0.0025	13.2	2312 K 3R _w /8=4.5 times sum of drifts calculated by code.	36.5 inches
1997 UBC	V=C _v IW/RT R=8.5	.7R=5.95	0.020	10.1	1633.2.11 $\Delta_{MT}=\sqrt{(\Delta_{M1})^2+(\Delta_{M2})^2}$ The Δ_M values are calculated using C _d .	45.8 inches
2003 NEHRP 2002 ASCE-7	V=C _s W C _s =S _{DS} /R/I R=8	5.5	0.015 for SUG II	16.1	4.5.1 “..avoid damaging contact under total deflection, $\delta_{x...}$ ” δ_x is calculated using C _d .	48.6 inches (Note 6)

Notes:

1. C_d was not used, drift values were set lower and calculation was based on non-amplified elastic drift.
2. Relative stiffness of frames was based on the assumption that the frame design was governed by drift. The relative stiffness was calculated as C_dV/Drift Limit.
3. This value is lower than typical designs because most offices used their own more conservative rules of thumb and other criteria for wind and strength would likely have governed these buildings.
4. Assumes two identical 10-story wings, both designed at the drift limit. The joint width is calculated for the root of the buildings.
5. “Deflection from seismic action” was typically interpreted to mean deflection calculated at code forces without amplification.
6. This value is based on the sum of the absolute values of drift. Most firms would calculate an SRSS value, as explicitly prescribed in the 1997 UBC, or calculate a value from some type of dynamic analysis. See later discussion of research by Kasai et al.

Based on the table above, it is clear that, for the example shown, the required stiffness of drift-controlled moment frames has not changed dramatically over the years. However, for near-fault zones and other areas where seismicity has been increased, the design forces have been increased significantly, resulting in stiffer buildings as compared to earlier buildings in those areas.

As seen, prior to 1988, building separations or seismic joints were usually based on calculated code drift values, which were as much as 4 to 8 times less than those indicated by later codes.

The above discussion is based on the use of code approaches to building design, which are common and practical for low-rise buildings. Firms designing taller buildings commonly utilize more sophisticated dynamic analysis procedures that can often be used to justify smaller separations between buildings and smaller joints, if joints are used. In addition, researchers Kasai and Jagiasi (1993) have proposed a special method, referred to as the Inelastic Spectral Difference Method, that gives results indicating that the SRSS method of combining the two building drifts

may be too conservative for adjacent buildings or wings with similar natural periods. Use of the absolute sum of the drifts of the two buildings, or wings, is obviously the conservative upper limit for joint width.

STRUCTURAL DESIGN ISSUES

Seismic joints often result in somewhat complicated structural framing conditions. In the simplest of joints, separate columns are placed at either side of the joint to provide the necessary structural support. When double columns are not acceptable, the structure must either be cantilevered from more widely spaced columns, or seated connections must be used. In the case of seated connections, there is the temptation to limit the travel of the sliding element, as longer sliding surfaces using teflon sliders or similar devices are costly and the seat element may interfere with other elements of the building. It is strongly recommended that seated connections be designed to allow for movements that exceed those calculated for the code earthquake (475 year return period) to allow for the effects of greater earthquakes and because the consequences of the structure falling off of the seat may be disastrous. Where this is not possible, restraint cables such as are often used on bridges should be considered. Where restraints are used, impact forces must be considered in their design.

ARCHITECTURAL AND FIRE ISSUES

Architects have a strong dislike for seismic joints, with good reason. The joints require very significant design effort by the architect and, ultimately, no matter how well they are done, they are expensive and unattractive features of the building. In low to mid-rise buildings, architects will frequently ask that the joints be made the same width over the full height of the building, in order to simplify the design. This is, of course, acceptable from a structural engineering perspective, but can add to the expense of the joint because of the larger-than-needed joint covers that will be employed at the lower floors of the building.

Joint covers are costly specialty items that increase in cost and decrease in availability exponentially as they increase in width. The joints typically require very complicated detailing to achieve weather-tightness at the building exterior and to maintain through-floor fire ratings. Fire rating is typically accomplished through use of fire blankets that are hung slack across the joint. Reportedly, inadequate fire protection for seismic joints was a major contributor to the 85 deaths in the 1980 MGM Grand fire.

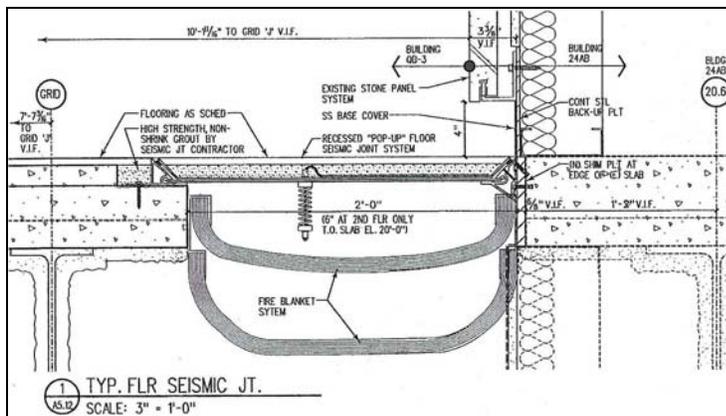


Figure 2. Example 2' 0" wide seismic joint



Photo of joint in Fig. 2

CONCLUSIONS

Seismic joints or separations between buildings have been used for many years. However, over the years the required width, and subsequently the cost, of the separations has grown. Seismic joints, within a single building, have been introduced by engineers, in the past, either to simplify analysis or to reduce the seismic effects of building irregularities. Because of the cost and architectural undesirability of seismic joints, modern computer analysis methods have been utilized to justify the elimination of joints in many buildings where they might previously have been used. Seismic joints, where used, require complex and careful detailing by all members of the design team to assure that the requirements for structural function, weather tightness, fire separation, appearance, and services distribution performance are all met.