The Traditional Type 1 Joint in Structural Steel - A Clear and Still Present Danger

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Abstract :

The design of girder to column joints have evolved over the years but essentially done by following standard prequalified details such as the ones published in the ASEP Guide 1991 [3] for standard Butt-welded Beam/Girder to Column Joints. This joint is now known universally as the "Type 1 Joint" in contrast to other joint types, which have been proposed to replace it.

But why was there a need to replace this Joint detail as proposed by respected US Technological Associations involved in Earthquake Engineering and Research even as early as 1994? Also, why does this detail keep on recurring in Building designs here in our country?

These questions and the reasons why this Type 1 joint should be replaced is the topic of this Paper. It is hoped that the Local Engineering profession or some fellow practitioners who are not aware of the problem associated with this detail would be able to understand the compelling need to change their details. In addition, it is hoped that ASEP would take the lead and issue a circular "outlawing" this joint for use in seismic detailing and withdraw this detail in the outdated 1991 Guide.

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3 ASEP Guide "*Earthquake Resistant Design of Structures*"

1.0 Introduction

As soon as the findings of the Northridge Earthquake of 1994 became common knowledge in the Engineering community both locally and abroad, serious questions have arose regarding the highly critical vulnerability of the Traditional Type 1 or the *Butt-welded Beam/Girder to Column Joint* in structural Steel construction. It was found out that significant failures occurred in localized regions of the joint and column flanges, which required very costly repairs. As a result, some buildings in California that otherwise appeared safe (At least in external appearance), had to be razed because of the uneconomic cost of repairs entailed by damage sustained by this Type 1 joint.

This Joint Type is illustrated below in 3D rendering taken from Ref [6] and shows the connection detail where the Beam or Girder Flanges and the web are butt-welded to the column Flange. The Column may or may not be reinforced with web stiffeners.

(a) Type 1 'traditional' moment connection.

Consultants who kept abreast of the State-of- Practice quickly abandoned this detail and adopted the Official recommendations published by various research organizations such as the SAC Committee [4]

However, and surprisingly as we have observed in the local Engineering community, some design houses were still very slow to adopt or have continued the use of the highly vulnerable Traditional *Type 1* joint detail despite almost 10 years since this joint was removed from the recommended details.

Why this poor state of affairs? Partially this is to be blamed on the lack of knowledge and information on developments in the structural engineering field due to lack of funds for keeping abreast of the *state-of practice* and/or lack of interest. Also, this could be partly because the ASEP [5] has not superseded the 1991 Guide which contained this originally "*recommended*" but otherwise banned Traditional *Type 1* joint detail.

However, both these two situations are not acceptable excuses particularly nowadays because of the easy access to FREE technical information from the *Internet* where most of the materials in this Paper have been obtained.

This *problem*, which prompted the writing of this paper, became glaringly evident when our office was asked recently to do a value engineering study for a 5-Storey structural steel commercial building, which was already in the bidding pipeline.

Aside from our findings that the building was over designed by as much as 30 to 40%

[4] Task Committee composed of the Structural *Engineers Association of California* (**SEAOC**), the ***A**pplied Technology Council* (**ATC**) and the ***C**alifornia Universities for Research in Earthquake Engineering* (***CUREE***) *).* Collectively known as the SAC Joint venture.

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for the primary structural framing system, ironically, failure could still ensue despite the *over design* due to the highly vulnerable Traditional *Type 1* joint detail provided in the plans. To compound the problem, the building was long but narrow in plan and required a column free interior. This resulted in dependence on only two column rows leaving no alternative stress paths. Thus a domino type collapse is possible with a failure in one of the joints as the girder spans are relatively large at 15 meters.

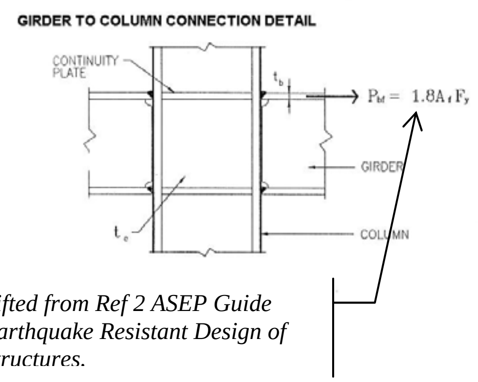
1.1 The ASEP Guide " *Earthquake Resistant Design of Structures*" 1991 Edition [6]

The ASEP Guide " *Earthquake Resistant Design of Structures*" 1991 Edition contained in Chapter 4 "*Recommended Structural Detailing Practices*" the Traditional *Type 1* joint detail as connection Detail 4.29 on page *182*.

In addition, page *178* of the same Guide required a Column to Girder Strength Ratio of 1.25. While this requirement could promote a "*Weak Beam Strong Column" (WBSC)* approach espoused in later studies, this pre-*Northridge Earthquake*" provision was not enough to prevent damage to this Type of Joint.

It should be noted that the ASEP Guide was published as the 1991 Edition. The *Northridge Earthquake* occurred in 1994 or 3 years after this publication. Our recent telephone inquiry with the ASEP Secretariat [7] indicated that this Edition has not been superseded by later publications.

The following detail was lifted from the ASEP Guide [4]



Lifted from Ref 2 ASEP Guide Earthquake Resistant Design of Structures.

Type 1 Joint in ASEP Guide

2.0 Historical Background

The Northridge earthquake resulted in 57 deaths, more than 5,000 injured and $20 billion in property damages, making it the costliest seismic disaster in U.S. history. Severe structural damage was seen in a wide variety of buildings. The engineering community was specially surprised by the poor performance of the highly regarded and widely used beam-to-column welded connections of Steel Moment Resisting Frames (SMRF).

After the *Earthquake* of January 17, 1994, a task committee was formed in the USA consisting of the ***S**tructural Engineers Association of California* (SEAOC), the ***A**pplied Technology Council* and the ***C**alifornia Universities for Research in Earthquake Engineering (CUREE)).* Collectively known as the **SAC** Joint venture Ref [7]. The **SAC** studied Post earthquake damage effects. A very disturbing or even alarming consequence is the discovery of numerous damages in beam to column Joint connections, which were based on what is now known as the "*Traditional Type 1 Connection*" for Moment framed joints.

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6 *ASEP Guide "Earthquake Resistant Design of Structures." 1991 Edition*

7 Telephone Inquiry June 10, 2005

Excerpts from the report are collected herein to shed more light on the criticality of this type of joint.

"Prevailing construction design codes take into account a strong inelastic behavior by the steel structure when exposed to earthquake ground motion. This is why ductile elements and connections are used in the SMRF. Based on research dating back to the 1960's and previous earthquake experiences the steel frame with moment resisting connections has been considered the most reliable seismic resistance design for low and high-rise buildings. The common usage of welded steel moment resisting frame is also a consequence of its versatility, economy, and its supposed high plastic deformation capacity.

In just 15 seconds, the Northridge Earthquake invalidated historic design approaches and proved wrong the theory of integral ductile response of the welded SMRF. In more than 250 buildings, brittle fractures were discovered in the welded beam-to-column joints. Fortunately not a single building collapsed and no death or injury occurred due to the unexpected mode of failure. The cracks were observed through the beam-to-column welds and/or through the base metal of the beam or column flanges. These cracks resulted in a loss of seismic moment resistance in the damaged connections; however, the connections still transferred gravity loads which may explain why there were no total collapses triggered by the brittle failure of welded joints."

The Northridge earthquake caused an unexpected brittle failure on welded SMRF constructed conforming to modern building codes and standards of practice. It was proved that those welded SMRF connections did not fulfill the design intent of providing reliability and safety. Research was initiated to improve these connections.

("The Northridge Earthquake and Welded SMRF ") Anon [3].

3.0 Girder to Column Moment Joint Details

3.1 Traditional Girder Column Joint Detail

The Figures below, taken from Ref [6] show the various components of the typical "*Type 1*" Pre Northridge Earthquake Traditional Type 1 Joint connection detail.

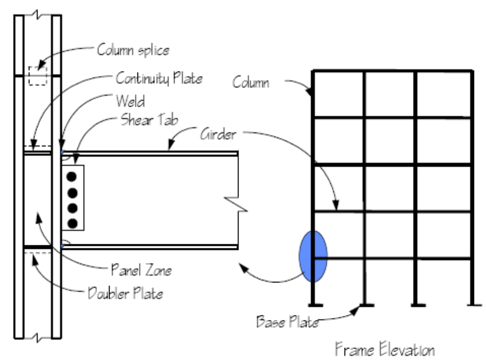


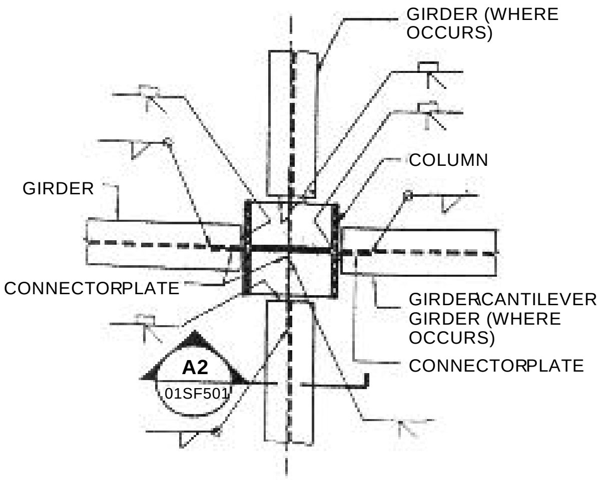
Figure 2-1 Elements of Welded Steel Moment Frame

The foregoing is the same detail incorporated in the ASEP Guide "*Earthquake Resistant Design of Structures 1991 Ed*" unfortunately; this guide has not been replaced nor superseded to reflect the current State of knowledge regarding the problems associated with this connection detail in the light of the Northridge Earthquake experience.

The failures are primarily attributed to a fundamental flaw in the standard code-prescribed welded-flange bolted-web connection and the extreme ground motion at the site. [8]

<u>As would be evident in this report, this type of detail would no longer be acceptable based on current state of practice due to the inherent lack of ductility and propensity for localized failure in the joint panel based on numerous recorded failures of this Type of Joint.</u>

Below is a Detail from a drawing for the 5 Storey Commercial building, which was the subject of the Value Engineering we conducted:



Based on the details as shown above, taken from a scan of the drawings, the following are main features of the Joint detail:

1. A plug weld is used to weld the Girder flanges supported by a backing bar or spacer directly to the column flanges.
2. Connection plates ("*Stiffener*") at the level of the top and bottom flanges were incorporated within the joint panel as stiffener plates.
3. The girder web is connected to the column by means of connector plates butt welded to the column using a *Vee* weld.

The foregoing figure shows that the joint connection details are similar to or identical to joint connection details in use prior to the *Northridge Earthquake*, which consists essentially of Girders being framed into columns by full welding of the Girder Flanges to the corresponding column Flanges or webs by butt or groove welds. These welds were very much in use pre 1994 until detailed post Northridge Earthquake damage evaluation indicated that something was terribly wrong with these joints.

3.2 Type 1 Joint Failure Mechanism

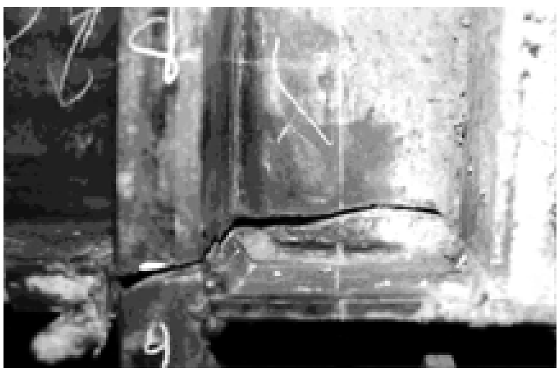
Simply stated, the problem with the traditional Type 1 Connection is the lack of Ductility in the Panel Joint connection details leading to brittle fractures. However, the crack initiation and propagation mechanism is not as simple. In all cases where the Type 1 joint was examined, failure was at the region of the connection between the top and/or bottom Girder flange/s and the column.

Failure was initiated in all instances by the incomplete fusion flaw as provided by the backing bar and its gap to the column flange. This constitutes a pseudo crack, which becomes a stress raiser during cyclic loading leading to crack initiation.

The open notch tip of the weldment where the backing bar is placed simulates a crack in itself. During cyclic dynamic loading, the crack propagates into the weld metal into the Heat affected zones (HAZ) and unaffected zones.

[8] David P. O'Sullivan et al "Repairs to Mid-Rise Steel Frame Damaged in Northridge Earthquake" ASCE *Journal of Performance of Constructed Facilities*, Vol. 12, No. 4, November 1998, pp. 213-220

Figure 4. Brittle failure in 'traditional' steel frame connection observed after Northridge Earthquake.



Damage to Column very Severe

Researchers have found that the stresses induced in the process although highly localized, are at least one order of magnitude higher than the stresses predicted by elastic analyses. These highly localized overstresses are concentrated at the Girder bottom flange connection within the critical joint panel connection.

This is compounded by the problem that this portion of the joint is the least accessible under field welding conditions thus; the quality of workmanship becomes an issue. This is highly undesirable, as we would not want the failure to initiate at the column or at the critical joint Panel connection as both would exhibit brittle failure modes.

It would be necessary to shift any failure to the connecting Girder (or Beam) away from the Joint. This would ensure that plastic hinging would occur at the Girder to allow flexural yielding rather than a brittle type of Failure. This is the basis for the recommendation Ref [9] to have a "*weak beam strong column concept*" (WBSC) in order to assure that the failure is not brittle. Providing a weaker beam (relative to column strength) assures that the failure would be that of plastic hinging of the beam, which ensures the extended ductility of the system. Formation of plastic hinges in the beam promotes a "*beam sway mode*" Failure mechanism, which is preferred over "*column hinging*", which could result in more catastrophic collapse modes.

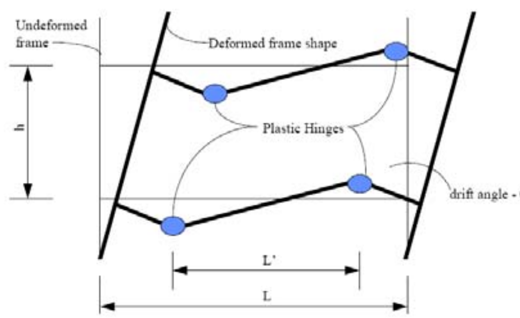


Figure 7-1 - Desired Plastic Frame Behavior

The **AISC** "*Seismic Provisions for Structural Steel Buildings of 1997*" Ref [9] for intermediate and special moment frames has adopted the position that:

> *"For Fully Restrained Connections, yielding must take place in the members of the frame (plastic hinge in beam, panel zone, etc.) and not in the connections.*
>
> *However, since yielding in the column is the least desirable result, the design engineer should consider designing the system such that flexural yielding occurs in the beam. For FR connections that are part of ordinary moment frames, the connecting elements may yield as long as*

0.01 radians of plastic rotation can be provided by the system"

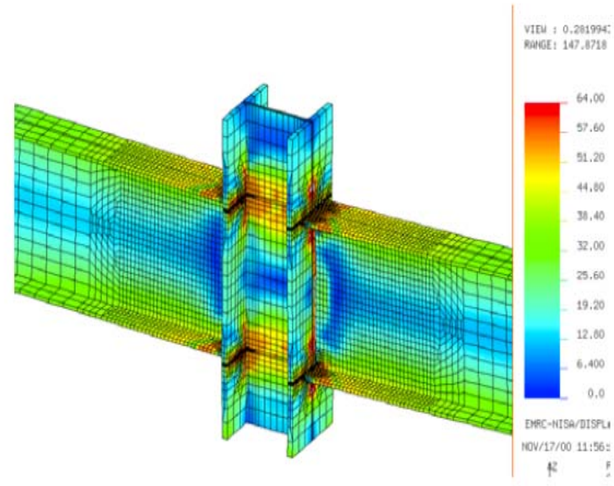
A study of the resulting stresses and strains under repeated cyclic loading of a Type 1 Joint was made as part of the study reported in Ref [6] (See Appendix). The color contour indicate the severe stress and strain concentrations at the Girder flange to column Flange intersections.

The study concluded that:

"<u>The traditional (Type 1) moment connection experienced high-order bi-directional localized plastic strain at the weld root at critical junctures between the girder and column, which is one of the causes of premature brittle fracture.</u> The strain patterns shown in Figure 16 clearly indicate the propensity for this phenomenon. The strain gradient is particularly pronounced at the mid plane of the girder near the weld and weld access hole."

The study further revealed that:

"Yield stress near the weld access hole and flange weld is exceeded early on in the loading. The strain plots near the weld access hole and flange weld show the stress reversal in the free edge of the girder flange, which is typical for traditional moment connections and a causative factor in fractures initiated from this region of the flange weld. This can potentially lead to fractures either in the flange or, far more critically from the progressive failure perspective, in the column flange. It is primarily this mode of failure that effected moment connection damage in the Northridge earthquake."



(a) Close-up of von Mises stress, ksi.

Type 1 Connection Showing Von Mises Stress Contours

3.3 Post Northridge Earthquake Damage Assessment Studies

From these studies it was clearly evident that the "*Traditional*" Type 1 pre Northridge connection details normally used and espoused by various authorities of that time have failed miserably and at joint locations that are not necessarily the worst stressed member based on post damage reanalysis of the buildings.

In the research done by *Mahin S.* Ref [3], from the University of California at Berkeley, we quote his findings:

"Comparisons of damage survey data with results of elastic analyses of the buildings (using recorded and simulated Northridge earthquake records developed for the building sites [5]) show relatively poor correlation.

Analyses suggest that the most heavily stressed joints are most likely to be damaged; however, the precise location and severity of damage was not *reliably predicted by conventional elastic dynamic analyses. The 60% most highly stressed connections in a structure (relative to their capacities) have roughly equal chance of being damaged. Areas of low computed stress were also subject to damage. Thus, analysis may not be a good way of assessing the particular joints to inspect, though it may indicate floors that should be inspected. The reasons for differences between observed and computed behavior include the effects of initial defects and poor workmanship, and the limitations of current analytical methods and models. For instance, inclusion of slabs and panel zones had an important effect.*

Most design calculations are based on an assumption that plane sections remain plane during deformation. However, review of experimental data and results of finite element analyses suggest that this is far from true, with high local bending and shear deformations being induced in beam and column flanges. This is especially pronounced when plastic shearing deformations occur in the panel zone. Results demonstrated that these panel zone deformations were often very large. In such cases, the distribution of shear stress over the depth of the beam's web is not uniform, often concentrating the majority of the shear force in the highly stressed beam flanges. Compounding this situation is the fact that actual material properties are not uniform, and vary randomly from member to member and systematically with loading direction, section size, and welding procedures. Normal member-to-member variation of material properties may result in members stronger than the connecting weld, or a column that is weaker than the supported beam. As a result, the joint may have negligible inelastic deformation capacity, regardless of workmanship." (Mahin) Ref [3]

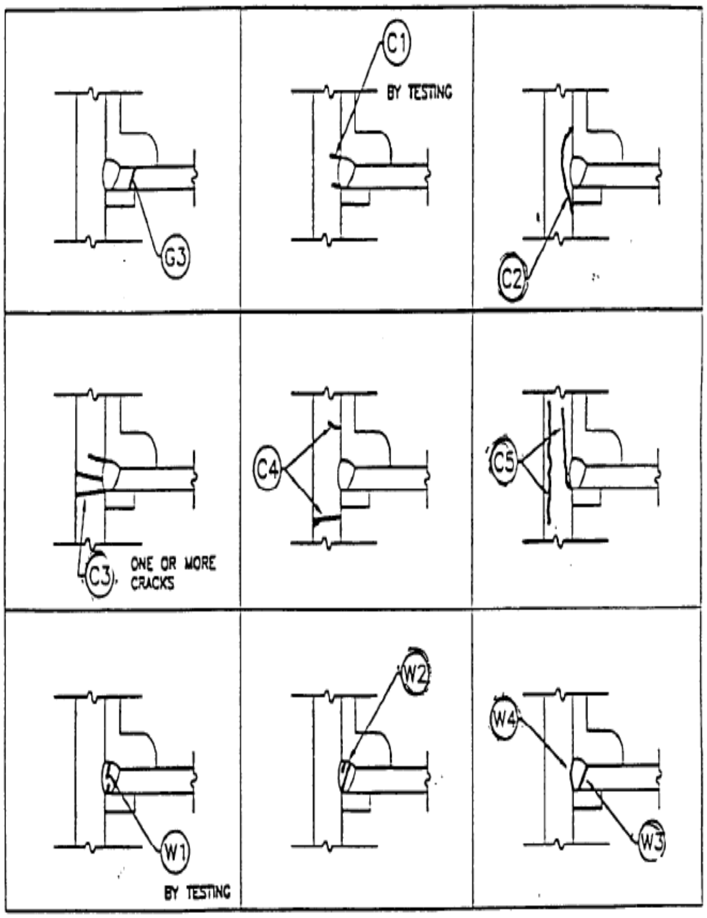
In just 15 seconds, the Northridge Earthquake invalidated historic design approaches and proved wrong the theory of integral ductile re

The Northridge earthquake caused an unexpected brittle failure on welded SMRF constructed conforming to modern building codes and standards of practice. It was proved that those welded SMRF connections did not fulfill the design intent of providing reliability and safety. Research was initiated to improve these connections.

("*The Northridge Earthquake and Welded SMRF* ") *Anon* [3].

3.4 The Northridge Earthquake and Damage to Beam Column Connections

The figure below shows the various types of damage to Joint Panel Connections sustained during the *Northridge Earthquake sustained by the Traditional Type 1 Joint Detail* after *Youssef. (The numbered arrows point to the Cracks)*



NOTE:
Numbered bubbles point to cracks

Figure 2 - WSMF Damage Types (Youssef et al.).

4.0 Conclusions

Researchers and research establishments in the United States have evaluated several candidate replacement Joint details. Full-scale load tests under cyclic loading were also conducted to determine the response of the various joint details to cyclic loading.

As a result, pre tested details have been evolved and included in the recommendations. Of this, the Type 3 joint shown subsequently has been recommended in addition to other proprietary and non-proprietary joint details.

It is suggested that Engineers who have not done so yet, consider abandoning the Type 1 Joint in favor of the Type 3 Joint in order to correct the potential problems associated with the former.

There are now available prequalified joint details, which could replace the Type 1 Joint. Tests conducted on these alternative details to replace the Type 1 connection have been made and are available in current literature *Bjorhovde R. Ref* [5] and *Houghton ref* [6].

Several details have become prequalified as replacement for Type 1 Joints in new construction.

The primary objective is to promote the "*weak beam strong column*" (WBSC) concept. This is to ensure that initial yielding will initiate at the girder a distance from the Joint and not at the more vulnerable column panel where failure would be in the brittle rather than ductile mode.

Bjorhovde R. Ref [5] made tests on such prototype joints and of these, the so-called "***Type 3 Joint***" performed very well.

For the Type 3 connections it was decided to place the continuity plates with the outside edge in line with the beam flange to cover plate interface. Figure 4 shows the details of the Revised Type 3 connection. *Bjorhovde R. Ref* [5]

4.1 Cover Plate Connections

Thus, it can be seen that the introduction of cover plates, which has the effect of making the joint strong where it is coverplated, transfers the stresses to the weaker beam section beyond the joint coverplate initiating a more ductile failure mode.

4.2 Conclusions in the Study by Bjorhovde Ref [5]

*"The tests of the "**Type 3"** connections demonstrated excellent plastic rotation and energy absorption capacities. It was also found that although cracks developed and eventually propagated through the column material, the propagation was slow and stable, with numerous crack arrests during the testing. Such was also the case for the cracks that propagated into the column k-area, demonstrating that a crack in this region will propagate in stable fashion, given appropriate connection details and fracture paths. Further, these connections used thinner cover plates and fillet welded and repositioned continuity plates. Finally, the cropping of the 413 continuity plates is important, to the effect that the ends of the welds need to be kept away from the k-area, but this observation applies to all kinds of welds and connections. In brief, fabrication and construction economies will be obtained with the Revised Type 3 connection."* Bjorhovde

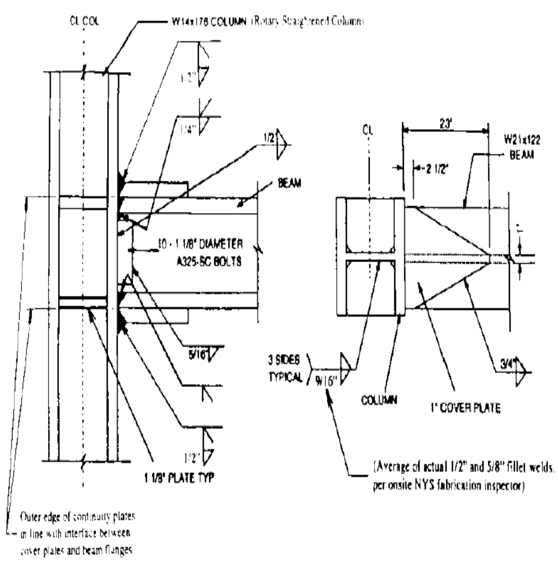
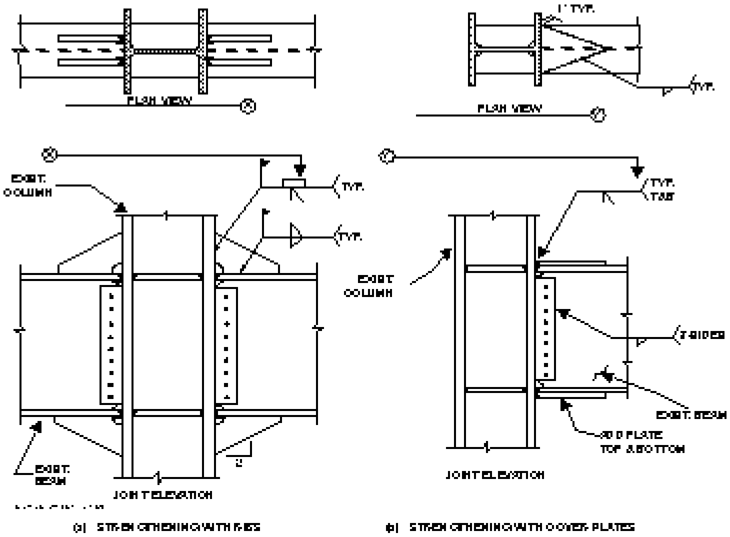


Figure 4 Revised Type 3 Connection



Alternative Joint Systems to Replace the Type 1 Joint.

4.3 Recommendations of the SAC Panel

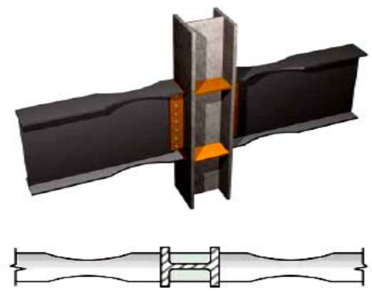
The SAC Joint Committee Ref [7] have issued recommendations for Post Northridge Earthquake Building Construction contained in *"Interim Guidelines: Evaluation, Repair, Modification and Design of Steel Moment Frames* [3] "

"The building code provisions for earthquake resistive design of Special Moment-Resisting Frames (SMRFs) assume that these structures are extremely ductile and therefore are capable of large plastic rotations at, or near to, their beam-column connections. Based on limited research, and observations of damage experienced in the Northridge Earthquake, <u>it appears that conventionally designed connection assemblies configured such that plastic deformation concentrates at the beam-column connection(referring to Type 1 Joints) are not capable of reliably withstanding large plastic rotation demands.</u> The reliability appears to decrease as the size of the connected member's increases. Other factors affecting

this reliability appear to include the quality of workmanship, joint detailing, and toughness of the base and weld metals, relative strengths of the connection elements, and the combined stresses present on these elements. Unfortunately, the quantitative relationship between these factors and connection reliability is not well defined at this time. In order to attain frames that can reliably perform in a ductile manner, these Interim Guidelines recommend that SMRF connections be configured with sufficient strength so that plastic hinges occur within the beam span and away from the face of the column. All elements of the frame, and the connection itself, should be designed with adequate strength to develop these plastic hinges. The resulting connection assemblies are somewhat complex and the factors limiting their behavior not always evident. Therefore, qualification of connection designs through prototype testing, or by reference to tests of similar connection configurations is recommended.

These procedures should also be applied to the design of Ordinary Moment-Resisting Frames (OMRFs) located in zones of higher seismicity, or for which highly reliable earthquake performance is desired, unless it can be demonstrated that the connections can resist the actual demands from a design earthquake and remain elastic. Interim Guidelines for determining if a design meets this condition are provided. Light, single-story, frame structures, the design of which is predominated by wind loads, have performed well in past earthquakes and may continue to be designed using conventional approaches, regardless of the seismic zone they are located in. Materials and workmanship are critical to frame behavior and careful specification and control of these factors is essential.

Other joint details such as the intentionally weakened beam with holes in the web, and the Reduced Beam Section (RBS), "*The Dog Bone*" and proprietary technologies such as the "*SidePlate™* " represent the other end of the spectrum. *Houghton ref [6]*

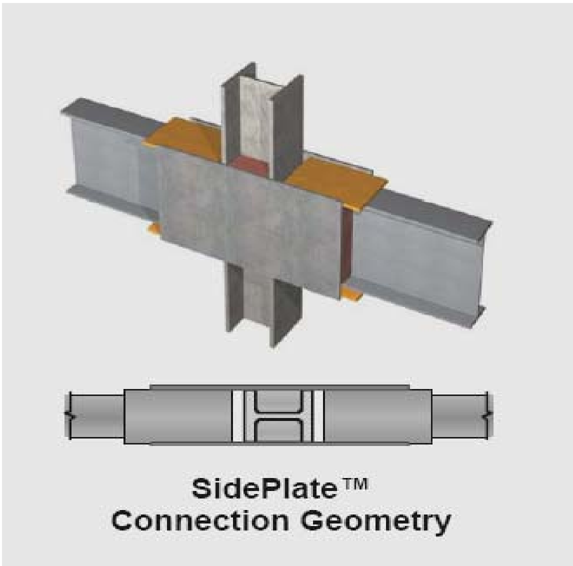


RBS
Connection Geometry

The RBS or "*Reduced Beam Section*" also known as the "*Dog Bone*" because of its shape introduces a weakening at the Beam or girder to allow it to fail in ductile mode ahead of the column.

The "*SidePlate™* " is a patented proprietary technology. The intention is to strengthen the Panel Joint with "*SidePlate™* " for the purpose of strengthening the joint and the column at the critical panel point.

Tests have shown that even with the failure of one column such as in a bomb blast, the Building will not collapse. Thus, this patented joint is now being used in construction of new US Federal Buildings.



The SAC Joint Committee also evaluated several other details one of which is the induced Plastic Hinge at the web a distance from the stiffened Girder Column joint as shown below *SAC Joint Committee ref* [7]:

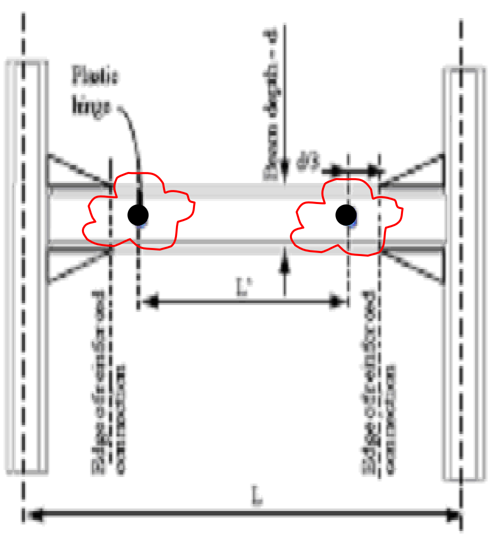


Figure 7-2 - Location of Plastic Hinge

If dead loads are not very significant, then the plastic hinges can be induced at D/3 from the end of the reinforced section. However, if gravity loads are significant then a plastic design and analysis should be undertaken to determine the actual location of plastic hinging.

5.0 Closure

It is evident from the foregoing that the Traditional Type 1 Joint Detail should not be used in Structural Details anymore and that there is a need to update, supersede, amend or rescind the details given in the ASEP Guide of 1991 pertaining to this joint detail.

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