SYNTHESIS OF DESIGN, TESTING AND ANALYSIS RESEARCH ON STEEL COLUMN BASE PLATE CONNECTIONS IN HIGH-SEISMIC ZONES

Jorge E. Grauvilardell
Department of Civil Engineering
University of Minnesota
Minneapolis, Minnesota

Daeyong Lee
Steel Structure Research Laboratory
Research Institute of Industrial Science & Technology
Hwaseong, Kyeonggi-Do
South Korea 445-813

Jerome F. Hajjar
Department of Civil Engineering
University of Minnesota
Minneapolis, Minnesota

Robert J. Dexter
Department of Civil Engineering
University of Minnesota
Minneapolis, Minnesota

Structural Engineering Report No. ST-04-02

Department of Civil Engineering
500 Pillsbury Drive SE
University of Minnesota
Minneapolis, Minnesota 55455-0116
http://www.ce.umn.edu

October 1, 2005

Abstract

Column base connections in steel structures under seismic loading are critical to successfully transferring inertia forces from the structure to the ground. Significant damage and failure of column base-plate connections has been reported in recent major earthquakes in the U.S. and Japan, emphasizing not only the importance of their function but also the lack of knowledge on their true behavior. This report summarizes conclusions regarding the behavior of several configurations of column bases according to experimental and analytical studies conducted internationally. A detailed description of the status of knowledge on topics such as the monotonic and cyclic response of column bases, progression of damage, types of failure, and design procedures, is made. An appendix also contains brief summarizes of significant prior research conducted on column bases. Particular attention has been given to the influence of the different components and parameters (i.e., dimensions, material properties, layouts, etc) on the overall behavior of the column base and the structure. This report also includes a summary of current design procedures in countries already known for their contributions to the understanding of column base behavior. A comprehensive description of the main issues not resolved in practice or in prior research studies is presented. This description addresses topics of structural design of the connection as well as its mathematical modeling. Based on this analysis, a prioritized plan for future research in the United States has been developed.

Acknowledgments

The authors thank Dr. Toko Hitaka, Prof. Masayoshi Nakashima, Prof. Subhash Goel, Mr. Richard Drake, Mr. Lanny Flynn, Mr. Timothy Fraser, and Ms. Elisa Gustafson, for their generous sharing of information and their assistance in the preparation of this report. Funding for this research was provided by the University of Minnesota.

Table of Contents

List of Figures

List of Tables

Chapter 1

Introduction

1.1. Background

Steel Moment Resisting Frames (MRFs) and Braced Frames (BFs) are both commonly used in the U.S to withstand seismic excitation. The similarity between predicted and actual behavior of these structures depends upon the accuracy of the design assumptions and the detailing of each of their structural components. One of the most important structural components of these steel structures is the column-base plate connection. However, there have been limited unified seismic design provisions for these connections in the U.S., in spite of the significant role of the column-base plate connections for the seismic performance of steel MRFs and BFs. The lack of unified provisions is particularly acute when considering the lack of information on the progression of damage in base plate connections relative to the remainder of the system that is implicit within the design of specific structural systems identified for seismic zones in ASCE 7-02 (ASCE, 2002) (e.g., special moment frames, concentrically braced frames, and eccentrically braced frames).

Recent major earthquakes have raised concern as to the need for improved understanding of the column base behavior under earthquake excitations. As reported by the Technical Council on Lifeline Earthquake Engineering (1995) and the Northridge Reconnaissance Team (1996), several column-base connections designed following previous design practices and guidelines did not perform satisfactorily during the Northridge, California earthquake of January 17, 1994. The damage to the base connections consisted mostly of excessive anchor rod elongation, unexpected early anchor rod failure, shear key failure, brittle base plate fracture, and concrete crushing (including grout crushing). Based on a statistical investigation of the damage in steel structures compiled after the 1995 Hyogo-ken Nanbu (Kobe, Japan) earthquake, Midorikawa et al. (1997) reported a relatively high incidence of damage to column-base plate connections. These facts emphasize that it is necessary to derive more reliable design methods and to develop more ductile column base connection details.

1.2. Research Objectives

To address the aforementioned needs, this research is divided into two phases. The first phase, summarized in this report, includes identifying the major engineering problems in the U.S related to column base design practice in high seismic zones and providing a comprehensive research plan to guide the direction of the second phase.

A literature investigation has been carried out to document the behavior of column bases (in experimental studies) and the response of column bases as part of multistory frames (in computational studies) under simulated earthquake forces and to identify the failure mechanisms of column bases reported from experiments. This investigation also included an analysis of the performance of column bases in steel MRFs and BFs during past earthquakes.

In addition, a survey of state-of-the-art column base design practices for both steel MRFs and BFs in high seismic regions has been carried out through contact with practitioners within the U.S., identifying major engineering and technical problems in the design and detailing of column bases in these regions. Procedures for the seismic design of column base plates from around the world are also summarized.

A synopsis of research findings is provided with a brief description of the objectives of each research, the main parameters that were analyzed, and the overall conclusions of the study. Tables with the range of parameters, material properties, and experimental failure modes of column base connections are also provided, in order to give a general overview and a practical sense of the general direction adopted in column base research.

Finally, a comprehensive analysis of design issues and a prioritized research plan for the second phase of this project have been developed, including recommendations for the major directions of future research on column base design for high seismic zones in the U.S, and proposals for the characteristics and scope of the necessary research.

1.3. Organization of the Report

This report is composed of six chapters and five appendices. Chapter 2 offers classifications of exposed and embedded column base plates. This categorization has been developed on the basis of the main characteristics (i.e., properties, behavior, failure modes) reported by worldwide studies and particular configurations obtained from the surveys. This manner of presenting the results has proven helpful for understanding what aspects of the performance of column bases have already been clarified and for highlighting areas where further research is needed.

Chapter 3 proposes additional classifications focusing on topics related to the design process. The discussion has been organized under three main headings, including:

1) Design considerations that should be accounted for when the structural analysis and consequent design are carried out;
2) Design parameters that affect the behavior of the column base with a brief explanation as to how the behavior is modified; and
3) Design procedures that are being used worldwide.

The last topic has been divided in two subsections. For the case of exposed column base plates, provisions and guidelines from the most recognized countries in this matter already exist (i.e., Japan, Europe, and the United States); consequently, a comparison between the accepted procedures from those sources can be made. However, for the case of embedded column base plate, a much more limited amount of information is available and only design procedures from Japan are reported.

Chapter 4 contains an outline and a discussion of the issues to be addressed. Issues identified after synthesizing past analytical and experimental research are included as well as those derived from the investigation of past base plate response and current design procedures.

Chapter 5 is a prioritized discussion of future research and Chapter 6 summarizes the conclusions of the research.

Appendix A includes the synopsis of the most important research to date, while Appendix B summarizes in tabular format the column base properties selected in past experiments on column base assemblages, as well as the relevant test results.

Appendix C presents the corresponding bibliography for the outlined documents. Appendix D contains a supplemental bibliography about related topics; and finally, references are incorporated in Appendix E.

Chapter 2

Classification of Column Base Plate Connections

2.1. General Classification

Column base plates may generally be classified into two groups: 1) Exposed column base plates and 2) Embedded column base plates. Even though this division is determined by the position of the base plate in relationship to the foundation element, it is considered representative of two traditionally recognized support conditions: "pinned" supports and "fixed" supports, respectively.

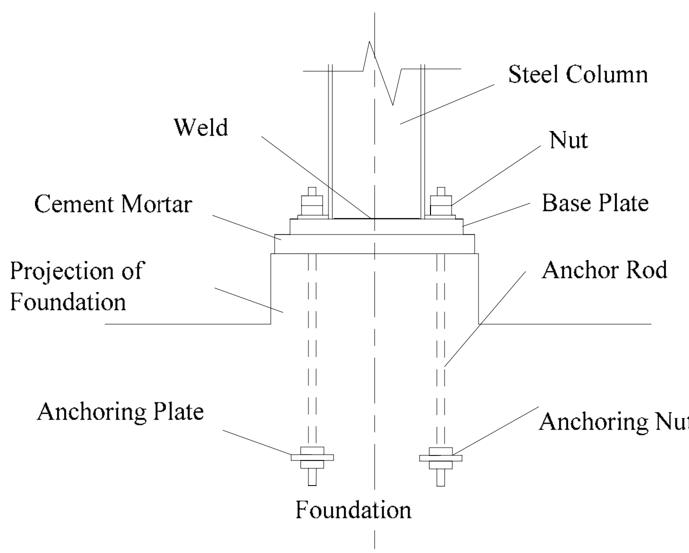


Fig. 2.1. Typical column base with exposed base plate

In the first group, in which the goal is to attain a pinned condition, a thin steel base plate welded to the end of the column is widely used to transfer, as smoothly as possible, axial compression, axial tension, and shear loads from the structure to the foundation. Exposed base plates have been particularly popular for industrial construction. Historically, two anchor rods have been used inside the portion bounded by wide-flange flanges and web to resist tension (uplift forces) and in some cases also shearing action. Frequently, the ability of these types of connections to resist moments is neglected, a fact that has been analytically and experimentally shown to be a wrong assumption in many circumstances (Galambos, 1960; Picard and Beaulieu, 1985). Proof exists that these theoretically pinned connections have failed when moments due to earthquake excitations were transferred by the base plate to a foundation not prepared for such a demand (Hitaka et al., 2003). The use of this type of column base, the configuration of which can be seen in Fig. 2.1., has not been limited to that of simple supports. The ensuing use of thicker and bigger base plates and greater and more numerous anchor rods has resulted in the full range of use of this connection from non-moment resisting to fixed conditions. This wide span of responses, which could be considered a virtue of the system, has led to failures due to unexpected behavior. Recent research (e.g., Astaneh et al., 1992; Burda and Itani, 1999; Fahmy, 1999; Lee and Goel, 2001) has confirmed that most column bases exhibit "partial-fixity". As a consequence of the actual behavior, exposed column base plates, when assumed to respond as fixed supports, proved to be able to resist the required loads only after significant deformations that were neither modeled in the structural analysis nor considered in the design. This resulted in larger than expected story drifts, greater than assumed joint connection deformations, and at times structural collapses. The consequences of these assumptions have been recorded at length in reconnaissance reports after important earthquakes (Northridge, California, 1994; Hyogo-ken Nanbu, Kobe, Japan, 1995).

The regulations of the U.S. Occupational Safety and Health Administration (OSHA) - Safety Standards for Steel Erection (OSHA, 2001), effective on January 18, 2002, require a minimum of four anchor rods in column-base plate connections. The possibility of achieving pinned behavior, in the linear range, in an exposed base plate is then less likely using such construction practices (Astaneh et al., 1992). Researchers have estimated the initial rotational stiffness in exposed column base plates experimentally, analytically, and using finite element models (Picard and Beaulieu, 1985; Sato, 1987; Melchers, 1992; Targowski et al., 1993; Wald et al., 1995). The initial stiffness of regular column base plates with two anchor rods has been reported to be as high as 50% of the corresponding theoretical fixed support stiffness, whereas for very stiff base plates, in which the rotation of the connections results primarily from the elongation of the anchor rods, a fixed condition was almost achieved. Current U.S. steel design guidelines (DeWolf and Ricker, 1990) acknowledge the capacity of exposed base plates to withstand axial forces with significant eccentricities (i.e., more than one sixth the dimension of the base plate in the direction of loading), but no guidelines are given for calculating the rotational stiffness of the connection.

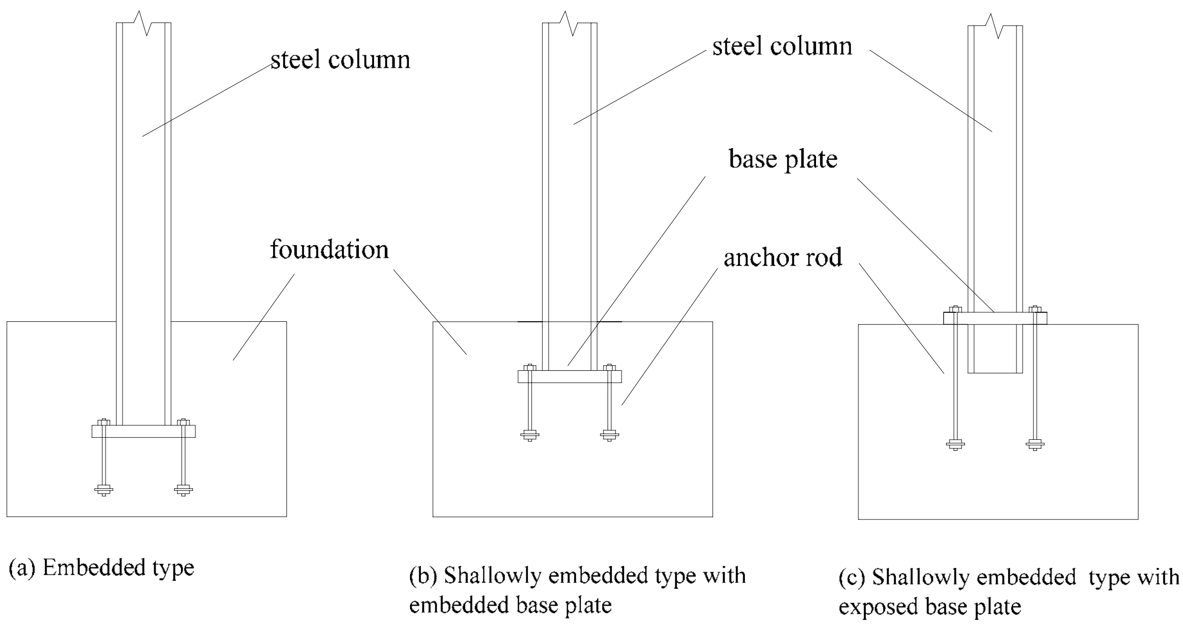


Fig. 2.2. Embedded column bases configurations

The second group includes columns embedded directly in a reinforced concrete foundation or in a grade beam. Embedded base plates have been more commonly used in office and other commercial building applications. Figure 2.2 show typical arrangements of column bases with embedded base plates. Moments, axial forces and shear are resisted mostly by the bearing of the column and the base plate against the concrete. The function of the base plate in the resistance mechanism is different, helping to increase the axial resistance of the connection. The anchor rods are designed for construction loads. They have a significant influence in the overall behavior only in the case of shallowly embedded column bases, in which the resultant mechanism is the superposition of the behavior of the embedded part of the column and the base plate-anchor rod action. Even though the stiffness of deeply embedded column bases better approximates the desired fixed condition, the more ductile behavior and easier construction of shallowly embedded column bases make these a very advantageous choice. Most of the column base research in the U.S. has concentrated on exposed column-base plate connections, even though shallowly-embedded or deeply-embedded column-base plate connections have been common for both the steel MRFs and BFs in high seismic regions. A classification of the column base connections is presented below. It has been made following the division and categorization presented in several papers by many different investigators.

2.2. Classification of Exposed Column Base Plates

2.2.1. Classification According to the Base Plate Behavior

Astaneh et al. (1992) and Fahmy (1999) have proposed a classification that many other researchers agree with. Base plates are roughly sorted according to whether the thickness is smaller, equal to, or greater than that required to form a plastic hinge in the plate. Figure 2.3. shows the three types of base plates described below and a schematic representation of their deformed shapes.

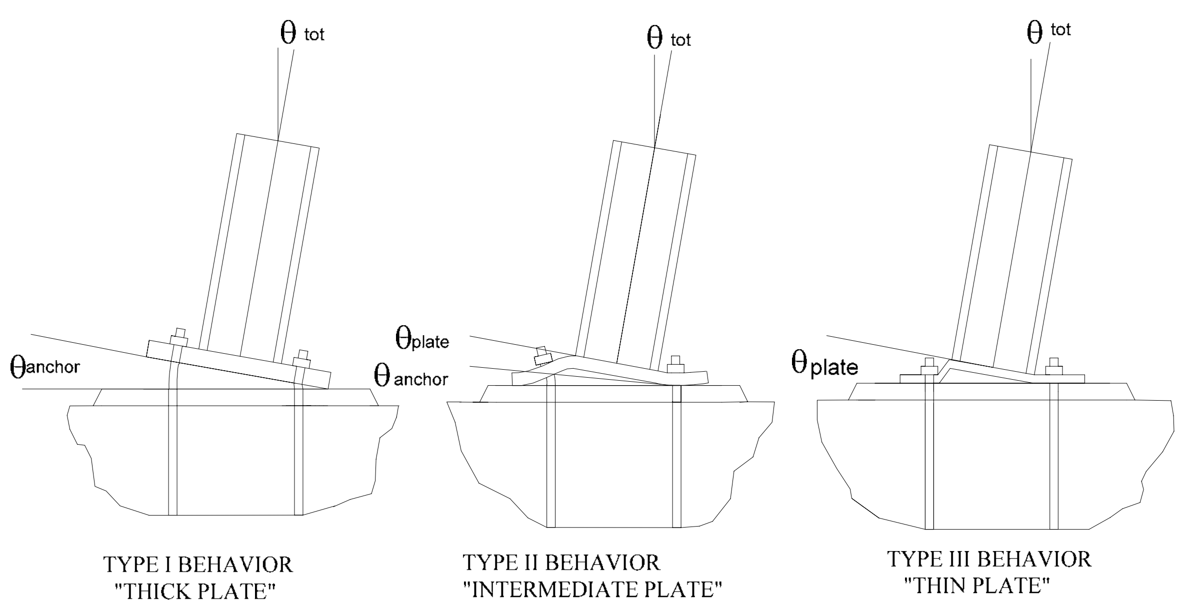


Fig. 2.3. Exposed base plate types of behavior [after Astaneh et al. (1992)]

Rigid or Thick Plates

Column base connections having thick base plates are expected to be the strongest and most rigid of the three types summarized in this classification. However, these are the ones most likely to present a non-ductile behavior due to fracture of anchor rods, often the weakest link in the design, or the development of crushing and spalling failure of the grout for large rotations (i.e., larger than 0.03 rad.). The tension forces in the anchor rods as well as concentrated compression forces in the concrete increase with the thickness of the plate. For this reason, typical detailing is recommended for the reinforcement of the foundation.

If ductile behavior of the anchor rods is desired, some particular considerations can be found in the literature. A parameter called the "yield ratio" has been defined to design anchor rods in such a way that they present ductile behavior (Sato, 1987). When subjected to tension, the threaded part of the bolt yields first. Brittle behavior occurs when the non-threaded part is not able to yield before the threaded portion fractures (Balut and Moldovan, 1997). In order to avoid this undesirable behavior, the yield ratio -- defined as the quotient between the yield strength and the tensile strength of the bolt metal – has been used as a test parameter. Ductile behavior of the anchor rods can be achieved when the area ratio between the threaded and the non-threaded cross section area of the bolt is greater than the yield ratio. The higher the area ratio, the higher the deformation the column base is able to withstand without fracture of the anchor rods. Experiments have been conducted with column tubes and stiff manufactured base plates (Sato, 1987). This high stiffness of the base plate guarantees that the rotation of the column is indeed the result only of the elongation of the anchor rods and the concrete compressive deformation. In the case of these experiments under cyclic loading, when low yield ratio anchor rods were used (i.e., on the order of 0.66), a ductile behavior with pinched hysteresis loops was obtained. No rupture was reached at the end of the test and rotations of about 0.1 rad were reported. Anchor rods with higher yield ratios (i.e., on the order of 0.94) resulted in full loops, but they ruptured at lower rotations (i.e., 0.03 rad).

Flexible or Thin Plates

Column base connections having thin base plates are characterized by flexible, ductile behavior, in which the inelasticity is concentrated in the base plate itself. Yield lines are formed along the flanges and if the base plate is thin enough, 45º yield lines can form at the corners of the base plate. The rest of the components (e.g., anchor rods, concrete foundation) remain elastic. Due to the important inelastic deformation of the base plate, the connection may act as an isolator for the structure from ground motion, helping to reduce seismic response.

Semi-rigid or Intermediate Plates.

There is some agreement among U.S. investigators that a base plate designed according to current AISC provisions (AISC-LRFD, 1999; AISC Seismic, 2002) and guidelines (DeWolf and Ricker, 1990) will have intermediate thickness and semi-rigid stiffness and strength. Some concerns have been expressed as to the possibility that base plates designed by current methods may behave more rigidly than expected, not attaining the yielding of the base plate that is sought (Lee and Goel, 2001). The failure of anchor rods in tension, which may govern, needs to be taken into account. Experimental observations have confirmed that less flexible base plates, with less bending deformation can cause damage to the grout and can result in the tension fracture of the anchor rods (Astaneh et al., 1992).

2.2.2. Classification According to the Amount of Restraint Provided

Pinned

As mentioned in the general classification, there is no exposed base plate that behaves as a pure simple connection. The importance of modeling the partial base fixity in frame analysis has been pointed out (Galambos, 1960; Picard and Beaulieu, 1985; Picard et al., 1987). Some of the documented advantages are that the buckling strength of the frame is higher, that the foundation can be designed for moments that the base plate is going to transfer, and that the overall design will result in more economical structures. Some experiments have been conducted: (1) to demonstrate that connections with two anchor rods, commonly assumed as pinned, show a stable partial restraint behavior (Picard and Beaulieu, 1985), and (2) to derive moment-rotation diagrams in order to give to the designer a formulation to use in the frame analysis (Melchers, 1992).

Alternative connection systems, considered as smart connections due to their adjusted response to different external disturbances, have been developed for both types of frames, braced and unbraced. These detailed connections in effect behave as pinned supports and have been proven to reduce the response of the structures to seismic actions. The so-called "rocking systems" with yielding base plates, one of the simplest smart connections, have been investigated in Japan in order to understand their behavior (Midorikawa et al., 2001; Midorikawa et al., 2003).

Fixed

Rigid column base plates and fixed connections are closely related. Parametric studies have been carried out to demonstrate that frames designed with detailed column base connections with rigid base plates will respond with drifts and develop moments very close to those obtained from frames with theoretical fixed supports (Fahmy, 1999). Theoretical studies for simple configurations of column base plates have been made (Salmon et al., 1957). However, this analysis shows a high initial stiffness that should be considered as an upper bound according to most recent experimental reports. In order to classify a column base connection as rigid, Wald and Jaspart (1998) have proposed limit values for the connection's initial stiffness $S_{j,ini}$ (calculated based upon Eurocode 3 Standards). This classification is based on the criterion that a partially-rigid column base connection may be classified as rigid when its inclusion in the frame analysis does not affect the ultimate resistance of the frame (i.e., the flexural buckling resistance of the column for which the partial restraint is taken into account) by more than 5% when compared with the situation in which a fully rigid column base is used. By applying this criterion, the investigators consider that a stiffness $S_{j,ini} \geq 12\ EI_c/L_c$ is the domain of rigid connections for non-sway frames. For sway frames, they considered that lateral deflection rather than the buckling resistance should be used as criterion for the classification. Using a 10% increase in lateral displacement as the limit for the initial stiffness, the domain of the rigid connections for sway frames was found to be $S_{j,ini} \geq 30\ EI_c/L_c$. Column bases with initial stiffness lower than the proposed limits are in the range of semi-rigid behavior. Eurocode 3 (CEN, 1992) uses a limiting value of $S_{j,ini} \geq 25\ EI_c/L_c$ to establish a rigid connection, and $S_{j,ini} \geq 0.5\ EI_c/L_c$ for pinned connections.

Partial Restrained

Only structures subject to gravity and moderate lateral loads (i.e., wind) may present a behavior of their column base connections that allows the typical simple classifications "fixed" or "pinned." Under severe conditions (i.e., seismic loads), the column base plate will be subjected to inelastic cycles and will act as a "semi-rigid" connection (Astaneh et al., 1992). Almost any report about exposed column base plates concludes that the exposed base plate must be modeled as a semi-rigid connection in order to more accurately represent the behavior of frames subjected to important lateral forces. In addition, emphatic recommendations have been made to include this behavior in the design of seismic loading. Some of the benefits of taking into account semi-rigid behavior in the structural analysis have been considered by Yamada and Akiyama (1997) and Kawano and Matsui (1998). These investigators have shown through analytical studies that story drift and formation of plastic hinges are more equally distributed along the height of the frame when partially restrained column bases rather than perfectly fixed ones are used. Fahmy (1999) has suggested that the initial stiffness of column bases with exposed base plate designed according to current U.S. design practice may be calculated as $S_{j,ini} \geq 2\ EI_c/L_c$. Only European and Japanese provisions define the initial stiffness of the connection and take into account its influence in the selection of design seismic forces and the corresponding behavior of the structures.

2.2.3. Classification According to the Steel Failure Mode

This classification is conceptually derived from experimental and analytical investigations. Features including strength, stiffness, and ductility of the column bases are analyzed in order to characterize each mechanism. Three regions can be recognized in the moment-rotation diagram: (1) a first region (i.e., small moments, low rotations) where the behavior is elastic; (2) a transition region where the behavior is inelastic and material hardening takes place; and (3) a softening region after the maximum moment of the connection has been reached and at the end of which the rupture of the connection occurs. These regions may also be used to characterize each mechanism. The failure mechanisms can be classified as follows (Fahmy, 1999):

Weak Column/Strong Connection

Weak column/strong connection details are characterized by the formation of a plastic hinge at the base of the steel column. The rest of the elements in the connection remain basically elastic or exhibit incipient yielding. Some of the tests carried out by Fahmy (1999) and Adany et al. (2000) performed in this way. These experiments have shown that with this behavior the post-yield deformation reaches maximum values with high strengths. Even though all the components (i.e., the base plate, anchor rods, and column) reached the yield stress, the hinge did form only in the column. The tests failed mainly by premature fracture of the welds due to the deformation of the flanges at the column base. However, specimens where sound weld details with notch toughness weld metal were used did perform satisfactorily. Based on these observations, it was reported that welds play an important role in this kind of connection. The mechanical characteristics that can be highlighted are: (1) high ductility with stable hysteresis loops, and (2) increase of the strength of the connection with an efficient use of each of its components.

Strong Column/Weak Connection

The performance of strong column/weak connection details, characterized by the inelastic deformation of one or more components of the column base assemblage, as well as the potential brittle failures that are more likely to occur (e.g., concrete crushing, anchor rod fracture) has been evaluated in several reports (DeWolf and Sarisley, 1980; Picard and Beaulieu, 1985; Thambiratnam and Paramisivam, 1986; Astaneh et al., 1992; Jaspart and Vandegans, 1998; Burda and Itani, 1999). Undersized specimens showed the behavior included in this failure mode. With the formation of two or more yield lines, the connection in general shows significant ductility, a higher reduction of the initial stiffness, and a marked drop in the post-peak behavior. These types of connections are the ones that best resemble a pinned condition when the connection behaves in the nonlinear range. Some of the characteristics that this type of column base connection may exhibit are low strength and initial stiffness, pinched hysteresis loops, but a good amount of energy dissipation. With the increase of the thickness, the strength of the connection is augmented and the ductility reduced.

Balanced Mechanism

This is an intermediate mechanism that targets achieving simultaneous and concurrent behavior from that discussed in the two classifications of connections above. The column yields at approximately the same time as one or more of the elements of the connection (i.e., the base plate or anchor rods). Thus, in this connection, not only one component is subjected to extreme deformations, but all of them undergo moderate inelastic behavior.

2.2.4. Classification According to the Concrete Failure Mode

Research carried out in Europe has indicated that the column base behavior can be divided into three failure modes according to the level of bearing concrete stresses that develop under the base plate. These patterns are shown in Fig. 2.4 and are characterized by the level of compressive axial force with respect to the ultimate bearing stress in the concrete. In order to define these patterns, an elastic plastic stress distribution is assumed in the concrete (Wald et al., 1995; Balut and Moldovan, 1997; Stamatopoulos and Ermopoulos, 1997).

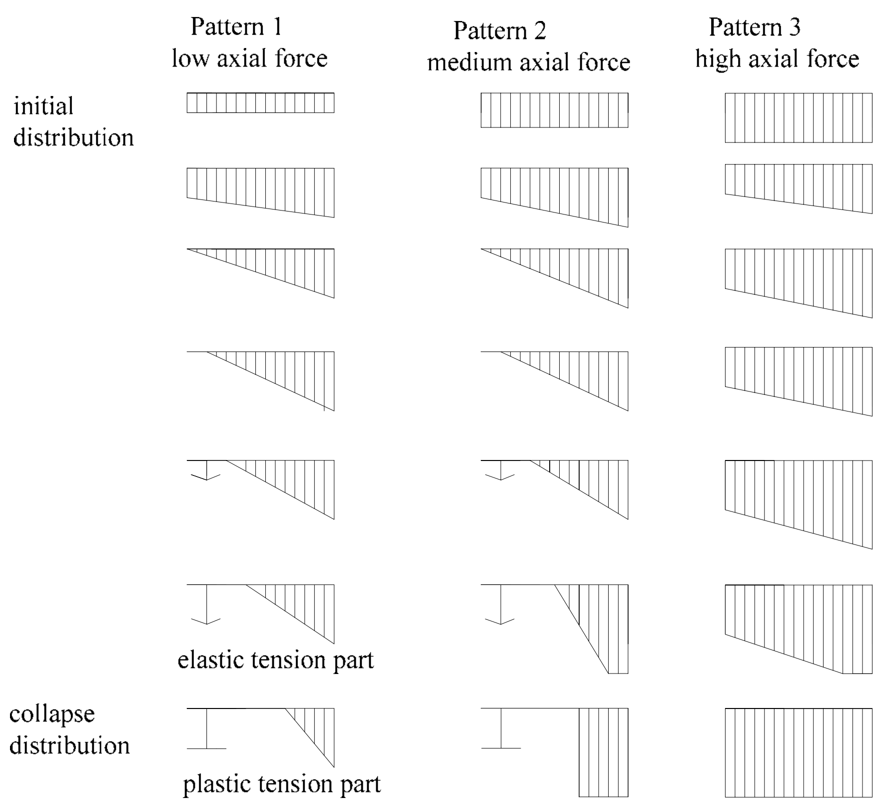


Fig. 2.4. Internal force distribution for three patterns of base plate joint in initial and collapse stages

Low Axial Loads (Pattern # 1)

For low axial loads, the bearing capacity of the concrete is never reached. The collapse occurs either when anchor rods yield or when the plastic mechanism forms in the base plate.

Medium Axial Loads (Pattern # 2)

In the case of medium axial loads, the behavior is characterized by the anchor rod reaching yielding and the concrete attaining its bearing strength.

High Axial Loads (Pattern # 3)

The failure mode for high axial loads is identified by the fact that at collapse only the concrete bearing capacity is reached.

2.2.5. Classification According to the Energy Dissipation Capacity

Fahmy (1999) provided a classification for base plates according to their energy dissipation characteristics. This classification may be an important consideration when a capacity design of the column base connection is carried out. In that case, a progression of failure should be proposed and each component designed accordingly. The following is a brief summary of this classification:

Non-dissipative Mechanisms

Non-dissipative mechanisms are failure mechanisms that do not provide outstanding energy dissipation. Some of the mechanisms reported in the literature when brittle behavior is observed are: (1) cracking of welds, (2) fracture of anchor rods (3) fracture of base plate and (4) crushing of the concrete or grouting. For mechanisms that are somewhat more ductile, excessive local buckling of the column flange may be included in this category. It is difficult to develop a stable plastic hinge with excessive local buckling. This in turn leads to a lower than expected strength capacity in the connection.

Dissipative Mechanisms

Dissipative mechanisms are able to provide stable energy dissipation. Generally speaking, the typical failure mode involves yielding of one or more connection components. This group may include 1) yielding of the base plate, 2) yielding of the anchor rod, and 3) plastification of the base of the column (i.e., forming a hinge in the column).

2.2.6. Classification According to the Type of Frame

The overall behavior of the exposed base plates and the nature of the forces acting on the column base will vary depending on the type of structure to which the column base is attached. This variation is analyzed in the next classification:

Column Bases Attached to Moment Resisting Frames

A base plate that works as part of a MRF column base will be subjected to the action of moments in addition to axial forces and shear. The literature presented in the appendices reveals that much research has focused on this kind of connection, and for that reason almost every conclusion that is mentioned for exposed base plates under the action of moments in Chapter 2 and Chapter 3 is applicable for this case. However, one of the most relevant topics that has not received attention, as will be emphasized in the discussion of issues related to exposed base plates in Chapter 4, is the combined action of moments and pull out axial forces. When gravity loads at the sides of the frames are low and the lateral forces are important, this load combination may be the most demanding for this type of column bases.

Column Bases Attached to Braced Frames

Column bases attached to braced frames have received almost no significant attention in prior research. An important set of tests have been carried out on exposed base plates subjected to axial forces only (DeWolf, 1978; Murray, 1983; Wald et al., 1994), and shear resistance has been studied when the moment was studied obtained as a result of an applied lateral load (Sato, 1987; Astaneh et al., 1992; Fahmy, 1999; Lee and Goel, 2001). However, the specific nature of this connection, e.g., the fact that in a braced frame the column base with an exposed base plate will include the interaction and force transfer from a gusset plate, has not been experientially studied. Two reports (Goldman, 1983 and Tronzo, 1983) have addressed the design of this type of connection analytically, focusing on the design of the anchor rods, shear lugs, and the gusset plate, but none of them have considered that the attachment of the gusset plate to the base plate could be a particular issue to account for (i.e., the stiffening provided by the gusset plate, or the force flow from the gusset plate to the base plate). Fig. 2.5. shows a situation in which the gusset plate is particularly large and where additional considerations could be needed (e.g., the use of 6 bolts instead of only 4 or the efficiency of the overhang portion of the base plate for transferring forces to the column base connection).

Fig. 2.5. Special detail for gusset plate and exposed base plate in a braced frame [courtesy of R. Drake, 2003]

2.3. Classification of Embedded Column Base Plates

2.3.1. Classification According to the Length of the Embedment

Deeply Embedded

In deeply embedded column base plate connections, the length of embedment of the column in the foundation or concrete grade beam is sufficient to guarantee that the behavior of the column base can be considered fixed. Recommendations on this length have been made for different column shapes. Steel tube box columns can be considered fixed when the embedment length is no less than 2D, with D being the lateral dimension of the column cross section in the plane of bending (Nakashima and Igarashi, 1986; Morino et al., 2003). Recommendations have also been given for wide-flange profiles with embedment lengths ranging from 1D to 2D (Pertold et al., 2000a, 2000b). The failure mode observed is the column "pull-out", when embedment lengths of less than 1D are used. Finite element models have been used to investigate the behavior of connections with different embedment lengths.

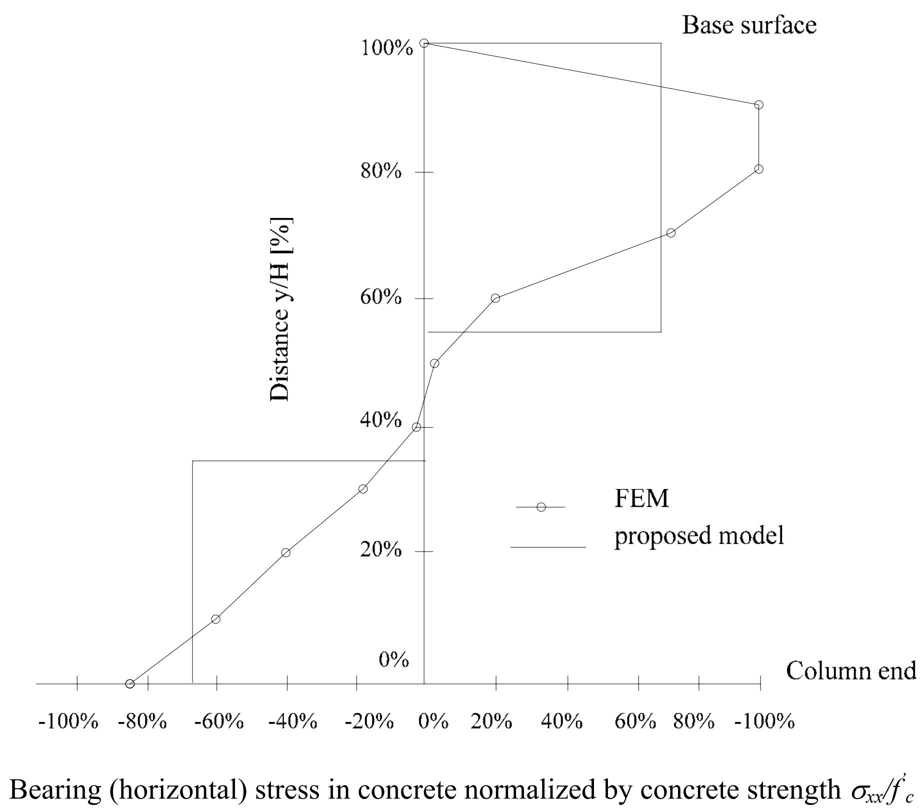


Fig. 2.6. Distribution of the bearing stress in the foundation due to a moment applied to the column. [after (Pertold et al., 2000b)]

Generally speaking, the first conclusion drawn from these analyses is that both flanges are involved in the moment resistance. The distribution of the bearing stresses along the flanges can be divided into two regions as shown in Fig. 2.6: the upper region of the embedment, where the stresses are oriented in the same direction as the lateral applied force, and the lower region, where the stresses are oriented in the opposite direction. For the case of embedment lengths greater than 2*D,* it has been found that the bearing stress due to column flexure in the concrete along the lower part of the steel column decreases due to the flexibility of the column, reaching a value of almost zero for an embedment length of 3.5*D*. On the other hand, when considering the response to axial forces, the base plate plays an important role. According to Pertold et al. (2000a, 2000b), the axial strength of the connection is the result of two mechanisms: (1) for compression, the bearing of the end of the column on the bottom of the concrete foundation, and (2) for compression or tension, the bond strength between the steel and concrete. The presence of the base plate enhances the first mechanism by providing a greater bearing area; whereas the addition of studs welded to the face of the column or reinforcing bars welded to the face of the column and anchored in the grade beam or foundation are important requirements in order to improve the second mechanism (Nakashima and Igarashi, 1986).

Shallowly Embedded

The process of constructing an embedded column to ensure full fixity has been found to have some drawbacks. One of the most important is the fact that one advantage of traditional steel construction, that the steel erection starts when the casting of the concrete foundation has been completed, is often lost (Morino et al., 2003). For that reason the shallow embedment concept was developed. The foundations are cast with the anchor rods arranged and a small box is left for placing the steel column. Once the steel column has been erected and bolted the box is filled with grout. Two procedures have been reported in the literature. In the first one the nuts as well as the base plate are embedded in this box (Nakashima, 1996). In the second, the nuts and the base plate are left outside of the foundation. The shape of the column is cut in the base plate, and the column is inserted through the plate and welded. Longer anchor rods are used and the connection, once completed, has the appearance of an exposed base plate. That means that technically only the column is embedded (Morino et al., 2003). Both systems have been investigated in Japan for steel tube box columns and concrete-filled steel tube (CFT) columns, and sketches of both were shown in Fig. 2.2. By embedding the column at a depth of one time its width using either system, substantially smaller base plates are required, and consequently fewer and smaller anchor rods are required as compared with exposed base connections. The column may be shallowly embedded in a concrete foundation, studied by Morino et al. (2003), in a grade beam, studied by Nakashima (1996), or in a reinforced concrete slab. The last two options are shown in Fig. 2.7 and Fig. 2.8, respectively.

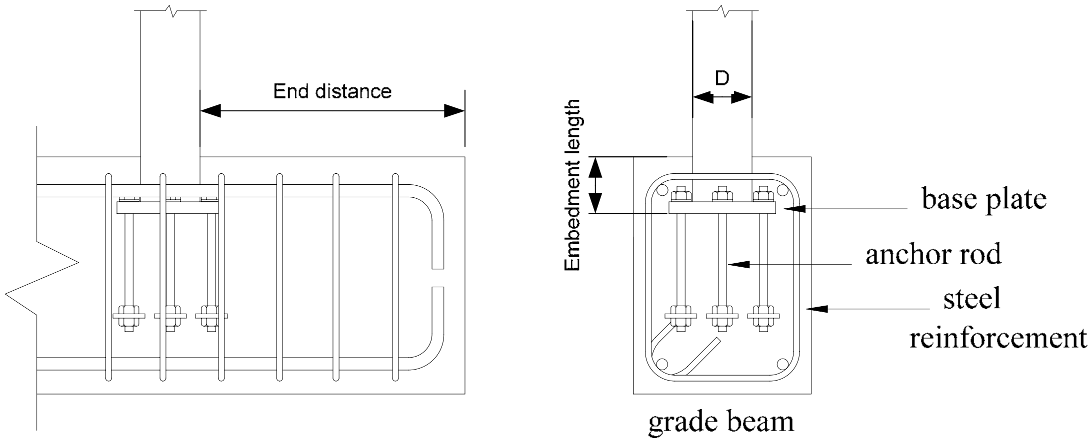


Fig. 2.7. Steel column shallowly embedded in a grade beam [after (Nakashima, 1996)]

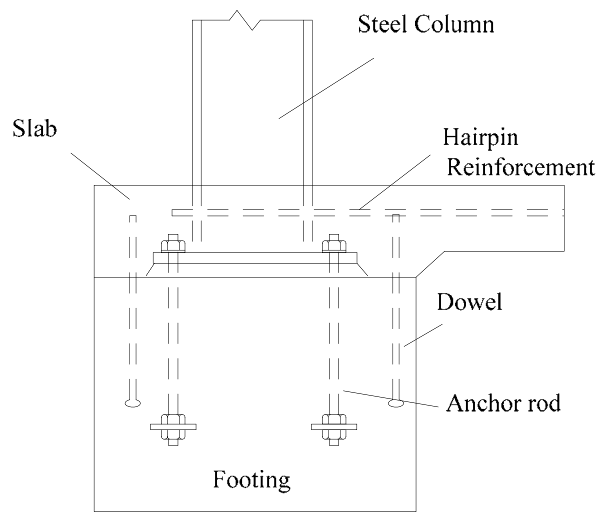


Fig. 2.8. Steel column shallowly embedded in a reinforced slab

2.3.2. Classification According to the Column Position in the Frame

Exterior Columns

This classification is included in order to highlight the effect of an additional parameter for embedded base plate connections in columns placed on the perimeter of the structure, i.e., the end distance from the column to the external side or face of the concrete foundation or grade beam. Research has shown that this end distance needs to be taken into account in the analysis of the behavior of these columns. If the end distance is small or if a careful detail of the joint has not been made, the unsupported concrete might crack when subjected to bearing forces due to prying action when the column is bent toward the external face of the foundation causing sudden strength deterioration of the column base. In addition, external columns are the most likely to be subjected to pulling forces, in which case this brittle behavior is especially undesirable. In order to minimize this effect, the minimum end distance recommended is 1.5 times the depth D of the column cross section in the plane of bending. When the end distance is less than 1.5 D, the detailing should include bars welded to the face of the columns, anchored on the interior side of the foundation and able to resist by tension the bearing stress that the concrete cannot handle (Nakashima and Igarashi, 1986).

Interior Columns

Embedded base plate connections of interior columns in the plane of bending often exhibit symmetric behavior (i.e., independent of the sign of shear or moment). This symmetric behavior is based on the fact that the concrete foundation (or grade beam) has sufficient length on both sides of the column to produce equal hysteretic responses, i.e., deformations or rotations, for both signs of moment and shear. Past research on interior columns has generally analyzed only the general parameters that affect the behavior of embedded columns (i.e., embedment length, reinforcement methods, etc).

2.3.3. Classification According to the Failure Mechanisms

With respect to a classification according to the failure mechanisms, generally speaking, the desired behavior sought for embedded column bases is that in which the column in flexure yields at its ultimate state, forming a plastic hinge at the foundation level. In other words, the embedded column base is expected to be able to transfer the full plastic moment (plus axial force and associated shear) of the steel column. This behavior results in a column base connection capable of providing full rigidity (Pertold et al., 2000a, 2000b). The failure modes reported from experiments agree in many ways with this statement and are discussed below.

2.3.3.1. Deeply embedded column base plates

Plastification of the Steel Column Base

Plastification of the steel column base can be guaranteed when the embedment length is greater than 2 times the depth *D* of the column shape, and the flexural strength of the column base is greater than 1.3 times the plastic moment strength of the column (Hitaka et al., 2003). However, external columns without adequate reinforcement may not achieve this failure mode due to cracking of the concrete.

2.3.3.2. Shallowly embedded column base plates

Plastification of the Steel Column Base

Plastification of the steel column base has been found only when thicker embedded base plates and bigger or more numerous anchor rods were used. In general, a plastic hinge in the column is not an isolated behavior, but is simultaneously accompanied with heavy cracking in the concrete. However, if a stronger base plate-anchor rod subassemblage is used, the concrete will not crack due to the flexibility of the base plate or elongation of the anchor rods, and the concrete will withstand the transfer of the full plastic moment from the column (Nakashima, 1996).

Cracking on Concrete

Concrete cracking is the controlling failure mode for the case of completely embedded base plates (i.e., when both the base plate and nuts are embedded) when weaker base plates or no anchor rods were used. The overhanging part of a base plate (i.e., the base plate planar dimensions greater than the column dimensions), when present, has been shown to cause the formation of cracks in the concrete and its consequent degradation and failure. The action of vertical reinforcement does not seem to help avoid the development of cracks. However, horizontal reinforcement did help to reduce cracks on the top of the concrete (Nakashima, 1996).

Concrete cracking was found in tests reproducing the performance of exterior columns (i.e., with asymmetrical foundations or end grade beams) with short end distances (i.e., less than $0.5D$) and small embedment lengths (i.e., less than $1.5D$). When a lateral force was applied toward the outside face of the column, the bearing stresses on the concrete, acting in the same direction as that lateral force, were found in the upper region of the embedment, and the unsupported concrete on the top of the foundation failed. When this lateral force was applied in the opposite direction, the bearing stresses on the same side were then located in the lower part of the embedment. Cracks in the concrete also developed in this lower region when no special detailing was used (Nakashima and Igarashi, 1986).

Yielding of Anchor Rods - Cracking of Welds

Once the concrete around the embedded column has crushed, yielding of the anchor rods and cracking of the welds represents the next step in the progression of damage. As mentioned earlier, these mechanisms may occur simultaneously with the failure of the concrete, and the governing failure mode is a function of the ratio between the concrete strength capacity and the strength of each mechanism (Nakashima, 1996). However, no guidelines have been given in the literature as to this relationship. Due to the rigid nature of the union between the steel column and base plate in the case of the exposed type of shallowly embedded column base connection tests, only yielding of the anchor rods (i.e., no crack of the welds) has been reported as a failure mode (Morino et al., 2003), always in conjunction with crushed concrete.

2.3.4. Classification According to Ductility

The common factor reported in the hysteretic response of column bases with embedded plates is the ductile behavior that the specimens show. When deeply embedded column base plates are used in cyclic experiments, ductile behavior with full hysteresis loops very frequently result. The reports on shallowly embedded base plate behavior will vary from pinched to full loops, depending on the relationship of the components. For example, pinched behavior results when the base plate-anchor rod subassemblage is predominant, whereas full loops are the characteristic response when the embedded contribution is dominant (Nakashima, 1996).

Similar conclusions are drawn for the case of the exposed type of shallowly embedded column bases (Morino et al., 2003).

Chapter 3

Design Considerations for Column Base Connections

3.1. Introduction

The reflection on topics and issues related to the design of column base connections may be analyzed from many points of view. For purposes of clarity, three have been considered in this report. The first point of view, the most basic one, analyzes structural design from the perspective of the technical goals desired, and thus establishes a reference point for reporting issues that are not taken into account in current provisions applied in the U.S. On the one hand, the objective is to highlight, based on the research, several design assumptions that although widely accepted are very often breached, and on the other hand to raise the awareness of the importance of some topics that are neglected in practice. The second point of view focuses on the effect, according to experimental literature, that the geometry and material properties of the column base components have on the behavior and response of the base connection. Based on this analysis, it will be possible to comment on the importance of the inclusion of some parameters in the design process of the connection. From the third point of view, the investigation summarizes a brief overview of the force transmission mechanisms and shows how the three design codes worldwide, specifically those from Japan, Europe, and the United States, approach the design of column base connections and their components. Using the aforementioned three points of view, which are based on documented information, a final discussion, in Chapter 4, highlights the need for finding answers to several issues that are not solved in practice and specific topics drawn from practical considerations and rational observations.

3.2. Investigation of Column Base Structural Design

The purpose of the structural design process is a final product in which performance closely follows the expected behavior expressed through mathematical formulations and approximations. If that performance deviates from the assumptions embedded in the analytical models, the expectation is that the actual behavior will at least be within a safe range (i.e., on the conservative side). In order to be aware of the level of safety that a structure possesses, the designer should be able to verify the integrity of the hypotheses applied. This section summarizes the associated goals of the design process for column base plates.

Resistance Capacity of the Connection

Every component of the column base (e.g., base plate, anchor rods, welds, and footings) must be able to withstand the forces that the frame imposes on them. In order to conduct a rational design, designers usually have the freedom to choose the behavior and consequent failure mode that they want for the connection. This decision allows the identification of the component or components that are expected to be the weakest link in the progression of failure and to design it or them accordingly. The remaining components will then be designed assuring higher strength in the parts one does not wish to fail in the adopted failure scenario (e.g., weld fracture, concrete cracking). In this light, seismic recommendations have been made in the U.S. that emphasize that yielding should be limited to the base plate (Astaneh et al., 1992; Burda and Itani, 1999). As such, current provisions (AISC Seismic, 2002, Section 8.5) emphasize the design for strength of each column base component, but a more formal strength capacity design of the connection is not explicitly required. As an example to illustrate the importance of a capacity design approach for column bases, it was mentioned previously that the stiffer the base plate the greater the tension forces delivered to the anchor rods. If a base plate is stiffer than assumed (e.g., because the formulation to find the thickness of the plate is conservative, or because the designer decided to use a thicker plate for other reasons), the anchor rods may yield, even though elastic behavior might have been expected. The designer needs to be cognizant of the relationship between the behaviors of the different components when sizing them and guidelines should be provided to help in the decisions.

Rigidity of the Connection

In the structural analysis of frames, theoretical support conditions are usually adopted (i.e., pinned, fixed). The reasons for this simplification in the treatment of the support condition may be found not only in the certainty that easier models can be used but also in the fact that the codes promote their use. For nonlinear response history analysis the structure shall be assumed to have a fixed base according to current provisions (FEMA 368, 2001, Section 5.7). Even though the use of realistic assumptions with regard to the stiffness and load carrying characteristics is permitted, the designer who chooses to model the structure with more accurate assumptions is penalized by having to satisfy the same drift limits, regardless of the assumed support conditions. It has been shown -analytically as well as experimentally- that the behavior of the column base is better represented when modeled as providing partial restraint (Astaneh et al., 1992; Burda and Itani, 1999; Fahmy, 1999; Lee and Goel, 2001). Many attempts with different approaches have been made to describe the linear and nonlinear behavior of the column base connection subject to monotonic and cyclic loads (Salmon et al., 1957). Finite element analysis has shown, for the case of exposed base plates, that even using an unrealistically thick base plate, the practical rigid stiffness limit cannot be achieved. In fact, some attempts have been made to provide an analytical estimate of the initial stiffness of a column base designed according to the current design practice in the U.S. (Fahmy, 1999). Many mechanical and mathematical models have been proposed for the formulation of the rotational stiffness for column bases with exposed base plates (Salmon et al., 1957; Sato, 1987; Wald et al., 1995; Fahmy, 1999). The important effect of the semi-rigidity of column base connections in the seismic response of MRFs has also been analytically quantified (Wald et al., 1995; Fahmy, 1999). Sometimes, in order to overcome these issues, embedded column bases or grade beams have been used. Their use could be considered to provide a higher safety level, in the sense that the mentioned assumptions could be attained; however, there is no experimental research about these topics in the U.S. and none of all the previously mentioned considerations have been included in design provisions so far.

In addition to responses like story drifts or moment distribution, the semi-rigid behavior of a column base also has an effect on the assessment of the effective length of the beam-columns on the first floor, in both braced and unbraced frames. Consequently, the true level of restraint imposed on the designed column base might play an important role in the frame stability since the effective length will increase as the level of restraint decreases (Galambos, 1960; Picard et al., 1987). As an example of this point it can be mentioned that while the G factor in AISC LRFD Specification (AISC, 1999) theoretically approaches infinity for a pin condition, values closer to 2, for non-sway columns, and to 5, for sway columns, have been experimentally determined from tests in exposed base plate type column bases. This finding shows that the G value for partial fixity of this kind of column bases correlates better with the proposed practical G value for fixed supports (i.e., $G = 1$; AISC, 1999) rather than the proposed practical G value for pinned supports (i.e., $G = 10$) (Picard et al., 1987).

Ductility and Energy Dissipation

Ductility may be defined as "the capability of a column base connection to withstand inelastic cyclic rotations without significant strength deterioration, stiffness degradation, or fracture of its components" (Astaneh et al., 1992). This consideration is particularly important in seismic design. The seismic design loads acting on a frame are highly dependent on the expected ductile behavior of the structure. In turn, the ductility of the structure is the result of the concurrent ductility of each of its components. Whereas quantification for the required ductility has been made clear for the behavior of joint connections and many recommendations can be found for the evaluation of this property (e.g., AISC, 1999; AISC, 2002), it is not equally clear for base plates undergoing large rotations. Only a few suggestions, in the context of exposed base plates, have been proposed for column base ductility criteria (Sato, 1987; Astaneh et al., 1992), but no equivalent work has been developed for other types of column bases (i.e., embedded base plate), and consequently no unified guideline is given to test if the expected ductility demand is met.

Even though high-energy dissipation is often associated with high ductility, a wide range of hysteretic responses results from experiments on column base assemblages. Pinched hysteresis loops as well as full loops (i.e., an important deformation still remains when the reversal cycle passes through the zero load point) can be found. Attempts have been made to correlate key parameters with the behavior sought. Some of the conclusions are presented in the next section.

3.3. Investigation of Column Base Design Parameters

The parameters involved in a column base design (i.e., geometry and material properties of the components) have a much more important effect than only the change in strength of the connection. Investigators have reported that responses like failure modes, damage progression, ductility, and stiffness will also be changed, and the need to consider these effects will be emphasized.

3.3.1. Exposed Base Plate Type of Column Bases

Thickness of the Base Plate

As stated in Chapter 2, the thickness of the base plate determines the behavior, strength and stiffness, and consequently the failure mode of the column base connection. It may be noted that despite reports about brittle failures of actual column base plates in buildings subjected to strong earthquakes (Northridge Reconnaissance Team, 1996), investigations consistently report that the thickness designed following current procedures is conservative when yielding of the base plate is expected (Fling, 1970; DeWolf, 1982; Lee and Goel, 2001). Besides the resistance, the thickness of the base plate is also associated with the ductility and hysteretic behavior of the connection (i.e., more pinched hysteresis loops are seen with thicker base plates) (Astaneh et al., 1992). Furthermore, the thickness has also been related to the distance from the edge of the base plate to the edge of the flange of the column (overhang). When the ratio of thickness to overhang distance becomes larger, the effect of the plate bending in the stress distribution becomes less significant (DeWolf, 1982).

Size of the Base Plate

The size of the base plate has a relevant and already known effect on the maximum bearing stress that the concrete foundation can reach. At present, guidelines in the U.S. recognize that the maximum bearing stress is proportional to the square root of the ratio of the concrete foundation area to the plate area (DeWolf, 1978). However, the size of the base plate has an additional effect that has been considered in research. For given loads and a constant thickness, the increase in the length of the base plate may be used to increase the distance between the face of the column and the center of the anchor. A series of tests with different base plate sizes concluded that this effect, although slight, corresponds to an increase in ductility of the connection with less degradation at the ultimate load. This was achieved due to more energy dissipation on the post-peak region (i.e., the drop of the moment-rotation curve after the maximum moment was not as sharp) based on larger flexural deformations in the base plate. The behavior in the elastic region and in the nonlinear range up to the maximum load was found to be similar in all the specimens (Burda and Itani, 1999).

Size and Number of Anchor Rods

As the diameter and the number of anchor rods increases, the initial rigidity of the column base connection as well as the connection flexural strength increase. Improvement of as much as 200% in rigidity and 100% in flexural strength have been reported (Melchers, 1992). Also, as the diameter of anchor rods varies, the ratio between the threaded and non-threaded areas consequently varies and according to Sato (1987) this fact has an effect on the ductile behavior of the column base when anchor rods are the weakest link in the failure mode.

Anchorage of Anchor Rods

Some experiments have been conducted to test the influence of the type of anchorage of the anchor rods on the final resistance of the column base. Round bars, “J” type anchors, and deformed bars, both with and without plates at the end of the bolt have been tested (Igarashi et al., 1992). The best performance (i.e., higher resistance) was obtained when deformed bars with end plates where used. The greater bond resistance offered by the rod is believed to contribute to this enhancement. For the case of base plates built on a pedestal, deformed bars helps to increase the durability of the concrete base. The other types of anchor rods yielded simultaneously with the crushing of the concrete (Igarashi et al., 1992). The minimum required embedment of anchor rods indicated in some studies is 8 times the diameter of the bolt (Salmon, 1957; Sato, 1987). The anchor rod embedment length, as shown in parametric studies, influences the rigidity of the connection (Wald et al., 1995). The contribution to the rigidity of the connection is smaller when the embedment length is greater. Finite element research has similarly shown that the connection performance is sensitive to the bolt embedment and bond length (Krishnamurthy and Thambiratnam, 1990).

Depth of the Concrete Foundation

Only a small number of experiments have shown the effect of the concrete depth on the behavior and strength of the connection. These experiments have been conducted only for the case of high axial loads with small moments. However, it is believed to be a significant variable for the determination of the ultimate strength for column base plate connections; the deeper the foundation the smaller the ultimate bearing stress of the concrete (DeWolf, 1982).

Concrete Compressive Strength

In typical base plate design calculations, a linear relationship is assumed between the concrete compressive strength (i.e., f_c') and the ultimate bearing strength of the concrete. DeWolf (1982) indicates that this relation may be a good assumption for estimating the bearing strength for concrete of up to 6000 psi. However, it has been noted (DeWolf, 1978) that for flexible plates (i.e., when the plate bending affects the ultimate load), the capacity increases in proportion to $(f_c')^{0.7}$.

Amount of Axial Load

The most important effect of the axial load on column base plate behavior is that the stiffness of the connection increases with the increase of the compressive axial load (Sato, 1987; Piccard et al., 1987: Li et al., 2000). The initial stiffness of column bases with exposed plates is directly related to the bond between the anchor rods and concrete and the union between the base plate and the concrete foundation. The beneficial effect of the axial load is found in delaying bond loss in the first case and the ability to maintain compatibility between the steel and concrete in the second case. This is one of the reasons why pretension of anchor rods is used. It has a similar effect on the overall response of the connection that the axial load has, i.e., enhancing the initial rotational rigidity of the connection and minimizing the swaying of the building against wind or minor vibrations (Igarashi, 1992; values of thirty percent of the yield strength of the anchor rods were used in the experiments). When thick base plates are used, a compressive axial load increases the flexural strength of the connection (Sato, 1987; Astaneh et al., 1992; Li et al., 2000). However, attention must be paid to the response of the concrete. The more rigid the base plate, the higher the bearing stresses (i.e., the bearing area decreases) and this situation may result in a decrease of the strength of the connection instead of the assumed increase (Wolf, 1982). In tests on rigid base plates, the detrimental effect of the action of tensile axial loads on the capacity of the column base has also been noted. Lower moment strength and rotation capacity as compared to tests with constant axial load and cyclic moments were obtained in cyclic loading experiments (i.e., tests having cyclically applied moment and axial force) where the axial load was varied with the cycles (i.e., from higher to lower compression or even reversed from compression to tension in each cycle) (Sato, 1987).

Weld detailing

The weld metal has been shown to have a critical effect on the behavior of the column base assemblage. Experimental column base connections in which the notch toughness of the weld metal was not controlled failed prematurely (Fahmy, 1999). The design of the welds is also of great importance. Astaneh et al. (1992) recommended, in addition to the use of notch toughness weld metal, to design the welds for 1.25 times the combined simultaneous effects of axial load, moment, and shear.

3.3.2. Deeply Embedded Base Plate Type Column Bases

Embedment length

The embedment length plays a fundamental role in the response of embedded column bases. The strength of the connection, as well as its stiffness, are directly related to this length. Ductile behavior is obtained when embedment lengths greater than 2D are used. Embedment lengths close to the minimum limit (i.e., 2D) yield more pinched hysteresis loops, whereas full loops can be achieved with longer embedment lengths. In the case of exterior columns, the embedment length has an additional effect. When a shorter embedment length was used, noticeable differences in strength capacity were observed when the lateral loads or moments were applied from opposite directions, and the connection showed a greater deterioration under cyclic loads for the same end distance (Nakashima and Igarashi, 1986).

Reinforcement Methods

It has been mentioned that the mechanism by which moments are transferred from the column to the foundation in deeply embedded column bases is predominantly by bearing of the faces of the column on the concrete. Even though the concrete has good capacity for withstanding these actions, some of the deformations that occur (i.e., in particular under cyclic actions) may subject the foundation to tensile stresses that the concrete is not able to handle. The consequence is a rapid degradation of the column base. The need for special reinforcement is even more important in the case of exterior and corner embedded column base plate connections, where the support provided by the external portion of the foundation is weaker compared with the internal part (i.e., provided that eccentric footings are designed). The concept of reinforcement must be understood not only for its application in maintaining the integrity of the foundation or grade beam. Its importance, in the case of embedded columns, extends to developing a capacity for maintaining the bond between the steel and the concrete (this is especially important when cyclic behavior is considered). In order to compare the effects of both types of reinforcements, it has been found that pinched hysteresis loops were obtained when only traditional reinforcement (i.e., reinforcement bent around the column) was used. Full loops were obtained with the use of studs welded to the face of the column. In particular, the use of reinforcement welded to the face of the column and anchored in the body of the concrete works in conjunction with anchor rods to achieve good restoring force characteristics under cyclic loading (Nakashima and Igarashi, 1986).

End distance

As mentioned in Chapter 2, the behavior of external column bases is affected by the end distance (i.e., the distance between the external face of an exterior column and the external edge of the foundation or grade beam). When a lateral force is applied to the column in the plane of the frame acting toward the exterior of the frame, shorter end distances result in decreased strength and a rapid deterioration of the column base flexural strength. This fact implies that special precautions need to be taken in detailing the foundation reinforcement when the dimension (i.e. end distance) is short (Nakashima and Igarashi, 1986). It was found that for end distances greater than $2D$ the cyclic behavior of exterior column base connections was identical to that of the interior column base connections.

Base Plates and Anchor Bolts

Generally speaking, it could be stated that one of the main purposes of the use of anchor rods in embedded columns is to maintain the column in position until the concrete has hardened. However, some tests carried out by Nakashima and Igarashi (1986) have shown that the presence of base plates and anchor rods, in addition to contributing to the embedment length (i.e., 2.5*D* for these tests), has a slight effect in helping to attain a stable hysteretic behavior, with hysteretic loops that are less pinched.

3.3.3. Shallowly Embedded Base Plate Type Column Bases

Embedment length

The embedment length of shallowly-embedded column base connections noticeably influences the ductile response of the connection. Embedment lengths of approximately 1*D* (i.e., one time the depth of the column cross section) have lower ductility and a marked degradation of the connection behavior compared with higher lengths (Nakashima, 1996). For that reason, it is recommended to embed the column more than one time the depth of the cross column in order to avoid "pull out" of the column and to obtain a stable hysteretic behavior. The maximum recommended embedment length, in order to consider the embedment as shallow, is 1.5*D* (where *D* is equal to the depth of the column profile), because for greater values the bearing mechanism starts to become predominant (i.e., the connection becomes deeply embedded), thus not taking advantage of the presence of the base plate and anchor rods to collaborate in the flexural resistance.

Reinforcement Method

The same general considerations already mentioned on the role of the reinforcement in deeply embedded column base plates (i.e., integrity of the foundation or grade beam and preservation of the steel concrete bond resistance) could be applied for shallowly embedded column bases. However, tests have shown that the need for a good detailing of the reinforcement is more crucial in this type of embedded connections. The formation of cracks in the concrete has a detrimental effect on the behavior of the concrete and must be resolved. For the case of grade beams, reinforced bars welded directly to the faces of the embedded part of the steel column and anchored deep in the body of the beam have shown the most effective performance in increasing strength, rigidity and energy dissipation in the tests carried out by Nakashima (1996). The anchorage provided by this type of reinforcement and the anchorage provided by the anchor rods was found to have the same effect on the final performance of the column base (i.e., flexural strength and rigidity). A less effective anchorage is obtained when the reinforcement is bent around the column and not welded to it.

Base Plate and Anchor Rod Dimensions

Since there is a shared resistance mechanism in shallowly-embedded column base connections (i.e., base plate-anchor rods and embedded column behavior), the larger the base plate dimensions the more strength and energy dissipation the connection can develop (Nakashima, 1996). Despite this statement, it has also been also found that during the post-peak behavior the strength drops more dramatically for the case of larger base plates, reaching almost the same ductility and final strength at rupture as those with smaller base plates. Concurrent with these findings, it was observed that larger base plates (i.e., with the planar dimensions exceeding the sides of the cross section of the column) caused the occurrence of horizontal cracks in the footing. All the aforementioned conclusions were obtained from tests that did not use anchor rods. The presence of anchor rods helps to improve the typical hysteretic behavior of this kind of connection, shifting the response from pinched to almost full loops. Larger anchor rods also increased the strength, rigidity, ductility and energy dissipation capacity of these connections.

3.4. Investigation of Column Base Design Procedures

Most column-base plate connections resisting large column moments in MRFs have been designed in the U.S. by referring to previous scattered journal papers, textbooks, and design guidelines. For the design of column bases in BFs in high seismic zones, insufficient research results have been provided to structural design engineers. As a possible result of not having sufficient, reliable, and comprehensive design provisions for column bases, several column-base plate connections designed following the design practices and guidelines outlined above did not perform satisfactorily during the Northridge earthquake (Northridge Reconnaissance Team, 1996).

Several analytical and experimental studies have been conducted in the U.S. to understand the complex force flow and stress distribution in the column base under large column lateral displacements in MRFs. These include DeWolf and Sarisley (1980), DeWolf (1982), Astaneh et al. (1992), Astaneh and Bergsma (1993), Burda and Itani (1999), Fahmy (1999), and Lee and Goel (2001). There are even fewer U.S. studies for the column-base plate connections in BFs, and no studies at all for any type of embedded base plate column bases. For that reason, prior to the analysis of possible approaches for the design of each general type of column base (i.e. exposed or embedded types), the conclusions about force transmission presented in the aforementioned papers are discussed.

3.4.1. Column Bases with Exposed Base Plates

Shear. The first mechanism used to transmit shear force in exposed base plates is the frictional resistance between the steel base plate and the concrete foundation. Pretension of bolts may be used to increase this resistance. When shear is combined with high moments, the contact area which provides friction may be reduced due to elongation of the anchor rods. When the frictional resistance is not sufficient to resist the design loads a variety of alternatives are possible. The addition of a shear key that takes the lateral force and transmits it by bearing on the concrete in which the tab is embedded is one option. Another possibility is to embed the base plate very shallowly. In this way the lateral face of the base plate plus the embedded part of the column will bear on the concrete. Another option is to rely on the shear capacity of the bolts. However, since the holes in the base plate are generally oversized, a gap is more likely to exist between the contour of the hole and the face of the anchor rods. Before both parts are in contact a slide must occur, leading to a decrease in the initial stiffness of the connection. Furthermore, if only some of the bolts are bearing against the side of the hole, these bolts will be overloaded unless an additional slip of the base plate may allow the distribution of the shear force among all the anchor rods present.

Compressive Axial Force. The base plate is the element used to distribute the column loads to concrete foundations. For low axial loads, the size of the base plate is almost equal to the size of the column and the distribution of the bearing stress could be considered constant if the base plate is stiff. For more slender plates or for higher axial loads, the unsupported portion between flanges and web of the column will bend. The redistribution of bearing stress will lead to a decrease of the stresses in that region and a concentration under the column shape (Stockwell, 1975; Murray, 1983).

Interaction of Axial Force and Moment. Considering a constant compression force acting on the base plate, when the bending moment is small all the plate is in contact with the concrete and the anchor rods are unloaded. For the case of large axial compressive loads, the bearing strength of the concrete governs the strength of the connection and the behavior of the footing has more influence on the overall behavior of the connection. On the other hand, for light axial loads, the response is driven by anchor rods and base plate (Melcher, 1992). As the moment increases, the bearing stress of the concrete shifts to one side of the base plate as anchor rods come under tension. For further moment increases, a nonuniform distribution of stress may result as a function of the flexural behavior of the base plate, even leading to small or zero stresses at the edge of the plates (DeWolf, 1978). The bearing stress distribution assumes a pressure bulb-like shape for flexible foundations (Krishnamurthy and Thambiratnam, 1990). Some investigators approximate this shape to a parabola (Sato, 1987), and for design purposes this curve is replaced by a constant stress distribution. As the moment continues to increase, the maximum stress will shift toward the compression edge of the base plate. This maximum bearing stress is considered to be located underneath or beyond the compression flange of the column. The transfer of this moment from the bottom of the column to the foundation is a function of the thickness of the base plate. More slender base plates will transform a portion of the moment into defomration of the base plate, transferring the rest to the anchor rods; while stiffer base plates will transfer most of the moment to the anchor rods in tension. This tension is primarily resisted by the bond resistance between the shaft of the anchor rod and the concrete. If round smooth rods are used, this resistance is rapidly exceeded and a small plate that is usually located at the embedded end of the anchor rod acts as an anchoring mechanism and the anchor rods start to elongate. For that reason it has generally been assumed in practice that this type of rods (i.e. round smooth rods) is free to elongate in tension from the beginning of the loading (Jaspart and Vandegans, 1998). On the other hand when deformed bars are used, the bond resistance is higher, and the tension force is broadly transferred to the concrete, enhancing the durability of the foundation. Once the bond resistance has vanished, the anchor rods start to elongate and the separation of the base plate from the concrete occurs. This reduces the bearing surface and shifts the resultant bearing stress outward. If on the basis of the design analysis (i.e., ductile behavior of anchor rods) anchor rods are to be free to elongate from the beginning of the loading, sleeves around the embedded part of the anchor rods may be provided to avoid the bond between steel and concrete.

U.S. Design Procedures

Earlier publications, such as Gaylord and Gaylord (1957, 1972), Salmon et al. (1957), Blodgett (1966), Soifer (1966), McGuire (1968), Maitra (1978), DeWolf and Sarisley (1980), DeWolf (1982), Ballio and Mazzolani (1983), and Thambiratnam and Paramasivam (1986), or more recent publications such as the AISC Design Guide No. 1, Column Base Plates (DeWolf and Ricker, 1990) are the most common guidelines used to design column-base plate connections resisting large column moments. For the design of column bases in BFs, insufficient past research has only provided information primarily focused on anchor rod and shear key resisting mechanisms and their capacities under large shear forces, but not for designing and detailing of column-base plate connections and attached gusset plates in these BFs. Several recommendations for the column-base plate connection design for axial column tension (or uplift), shear, or moment have been added in Part 14 of the 2001 AISC Manual of Steel Construction (AISC, 2001); and more recently, the 2002 AISC Seismic Provisions (AISC Seismic, 2002) briefly addressed seismic design criteria for column bases in Section 8.5 and its Commentary. However, these generalized provisions and recommendations have not strongly impacted current design practices for column bases, especially in high seismic zones, due to lack of detailed practical information.

Drake and Elkin (1999) presented a design procedure based on an LRFD approach. The applied factored loads (M_u and P_u) at the base of the column are resisted by bearing in concrete and tension in the anchor rod. For a given plate size, the design unknowns are the magnitude of the anchor rod ultimate force T_u and the bearing length Y. Two equilibrium equations are then used to determine the two unknowns, as follows:

$$\sum F_{vertical} = 0 \rightarrow T_u + P_u - \phi_c P_p = 0$$

$$\sum M = 0 \rightarrow \phi_c P_p \left(\frac{N}{2} - \frac{Y}{2} + f \right) - P_u (e + f) = 0$$

Where e is the eccentricity M_u/P_u, ϕ_c is the compression resistance factor = 0.60 and P_p is the ultimate force produced by the concrete block as defined in the LRFD Specification equation J9-1 and J9-2 (AISC-LRFD, 1993), and equal to

$$P_p = 0.85 f_c^{'} A_1 \sqrt{\frac{A_2}{A_1}} \quad \text{with} \quad \sqrt{\frac{A_2}{A_1}} \leq 2 \qquad \text{(LRFD J9-2)}$$

The geometric quantities N and f are defined in Fig. 3.1.

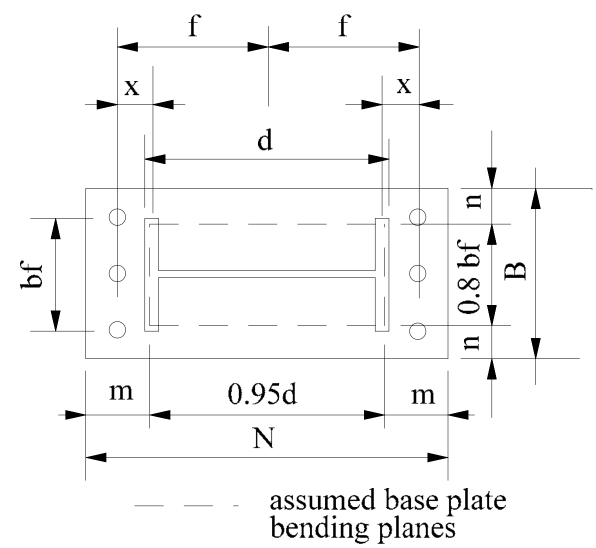


Fig. 3.1. Base plate design variables [after Drake and Elkin (1999)]

By solving those two equations, the bearing length and tension force in the rod are calculated from:

$$Y = \left(f + \frac{N}{2} \right) \pm \sqrt{\left[-q(f + \frac{N}{2}) \right]^2 - \frac{2P_u\left(f + e\right)}{q}}$$

$$\text{and } T_u = qY - P_u$$

The rods are then designed according to the LRFD Specification Section J3.2 (AISC, 1999).

$$V_{ub} \le \phi F_v A_b$$
$$T_{ub} \le \phi F_t A_b$$

Where F_v is the nominal shear strength, F_t is the nominal tensile strength, and A_b is the rod area. The base plate is then designed following flexural yielding limit states in Section F1 of the AISC (1999):

$$M_{pl} \le \phi_b M_p$$

where M_p is the nominal plastic moment. The plate thickness is then taken as the larger of:

$$t_{p(req)} \ge 2.11\sqrt{\frac{T_u x}{BF_y}}$$

$$\text{If } Y > m \quad t_{p(req)} \ge 1.49c\sqrt{\frac{P_u}{BYF_y}}$$

$$\text{If } Y < m \quad t_{p(req)} \ge 2.11\sqrt{\frac{Pu\left(m - \frac{Y}{2}\right)}{BF_y}}$$

where F_y is the yield stress of the base plate, and all the geometric quantities are shown in Fig. 3.1.

European Design Procedures

Column base design provisions published in ENV1993 Eurocode 3 (ENV, or EuroNorm Vornorm, represents a European pre-standard) (CEN, 1992) and the background of these provisions are well described in Wald et al. (1995). These provisions were developed mostly based on the COST C1 database of European research and they cover only the exposed base plate type of column base design provisions. Eurocode 3 has recently been converted to EN1993 Eurocode 3 (EN, or EuroNorm, represents a European standard) (CEN, 2003). However, Eurocode 3 provisions do not address seismic loading conditions, and Eurocode 8 which addresses seismic design does not address column base design.

The design of exposed base plate column bases is dealt with in Section 8 of Eurocode 3 (prEN1993-1-8). The design for connection strength as well as for stiffness is defined as a component based approach, in which the connection is considered to be an assembly of individual components. The components considered, the design resistances of which are used, are: (1) column web in transverse tension under the column flange (e.g., due to a brace framing in), (2) the base plate in bending under the column flange, (3) the concrete in compression under the column flange, (4) the column flange and web in compression, and (5) the anchor rods in shear and tension. In order to calculate the design compression resistance of the base plate in bending (i.e. T-stub flange) and the design bearing strength of the joint (i.e. that is function of the concrete or grout compression resistance), an equivalent T-stub in compression is defined using an effective length and effective width, as shown in Fig 3.2. Under the effective compression area (i.e. $l_{eff} * b_{eff}$) the bearing stress is assumed constant and should not exceed the design bearing strength. The value of the parameter "*c*" (called additional bearing width) is calculated as function of the thickness and the yield strength of the base plate. For the analysis on the tension side, the equivalent T-stub is designed as an end plate in bending, where a different l_{eff} is defined. This effective length is a function of the location of the anchor rods, the distances from the center of the anchor rods to the face of the column (i.e. "*m*") and to the sides of the base plate (i.e. "*e*" and "e_x").

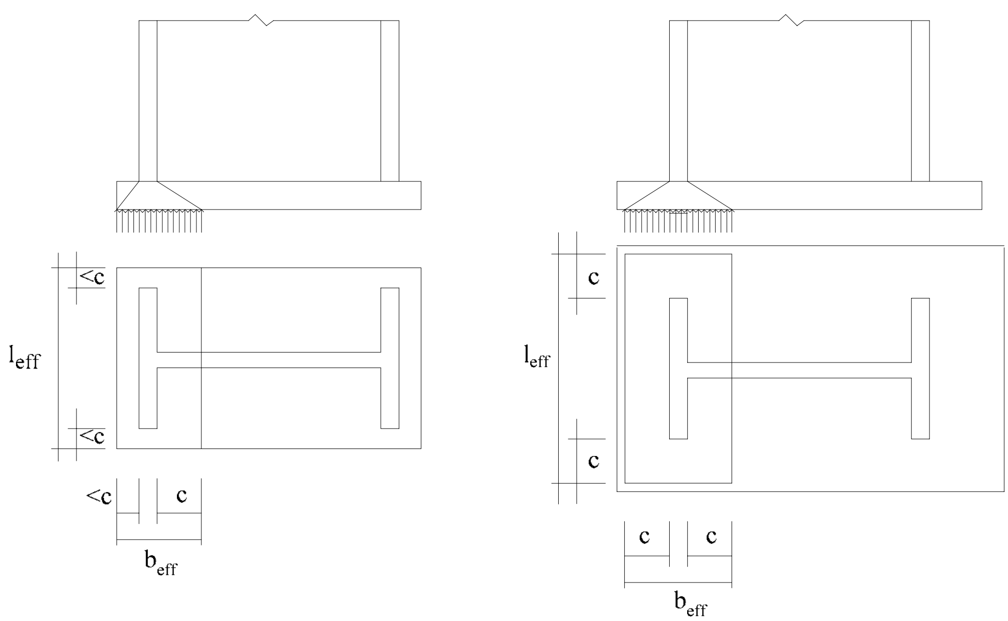
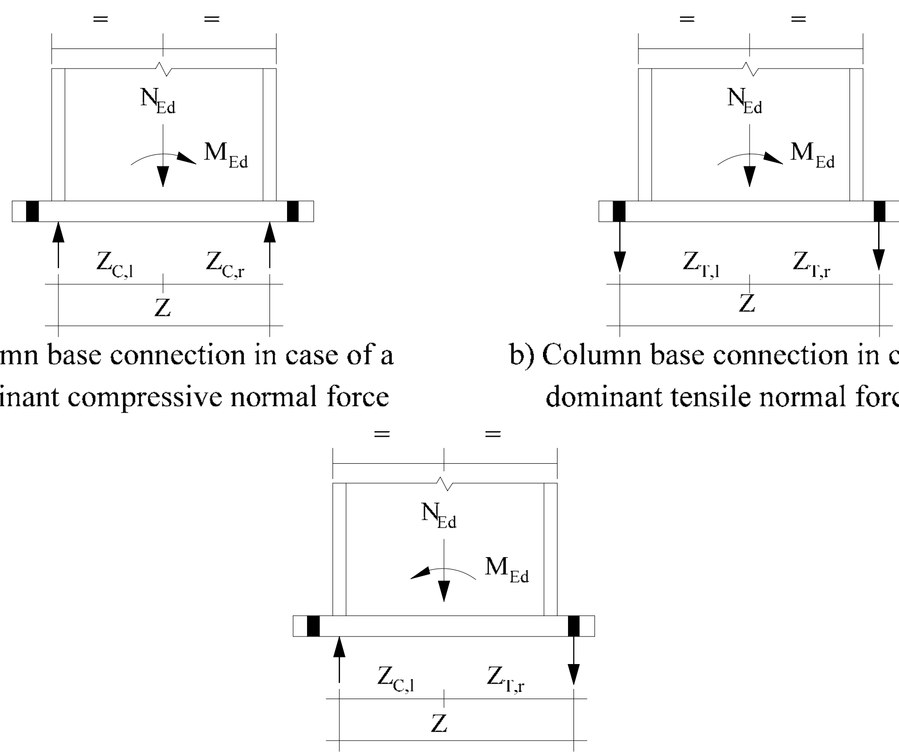


Fig. 3.2. Area of equivalent T-stub in compression for short and large projection [after EN1993 Eurocode 3 (2003)]

For the design of the anchor rods, the lever arm with which the external moments are decomposed into a couple of forces is defined by locating the tension force coincident with the centroid of the group of anchor rods and the resultant of the compression coincident with the centroid of the bearing area, as shown in Fig 3.3.



a) Column base connection in case of a dominant compressive normal force

b) Column base connection in case of a dominant tensile normal force

c) Column base connection in case of a dominant bending moment

Fig. 3.3. Force resultants in exposed base plate column bases [after EN1993 Eurocode 3 (2003)]

If the anchor rods are anchored by means of a washer plate located at the embedded end of the anchor, the total axial load is considered to be transferred by this device. If another means of anchoring is used (i.e. hooks) the anchorage distance is calculated to resist the axial force by bonding. Whichever is the smaller of the two, -the design tension resistance of the anchor rod or the design bond resistance of the concrete on the anchor rod, is taken as the resistance of the anchor. Finally, once the individual resistances of each component have been calculated, the design moment resistance of the column base is considered to be a function of them, the lever arm *"z"*, and the end distance *"e"* previously defined.

As the strength, the stiffness is also estimated –according to the same approach- through analytical simulation of mechanical model components. In this model, the base plate is idealized as a rigid bar resting on three springs. One of these springs represents the concrete stiffness and is in parallel with two other springs. These two springs in series represent the stiffness of anchor rods and the base plate in tension. The stiffness of the three springs is expressed as:

$$K_{\text{plate in tension}} = K_{15} = 0.425 \frac{l_{eff} t_p^3}{m^3}$$

$$K_{concrete} = K_{13} = \frac{E_c \sqrt{b_{eff} l_{eff}}}{1275 E_s}$$

$$K_{bolts} = K_{16} = 2 \frac{A_b}{L_{eff}}$$

The rotational stiffness is then calculated as:

$$\text{Rotational stifness} = S = \frac{E_s Z^2}{\mu \sum \frac{1}{K}}$$

where μ is a ratio of the initial rotational stiffness to the current rotational stiffness of the base plate connection.

The mechanical model considers the simulation of three collapse modes classified according to the level of axial load and moment acting on the connection.

Japanese Design Procedures

Many different types and details of column bases have also been studied in Japan. Major Japanese column base research is found in the Journal of Structural and Construction Engineering, Transactions of Architectural Institute of Japan, written in Japanese. Extensive column base research in Japan has revealed that column base design in high seismic regions is characterized by a wide variation of different parameters and that seismic behavior and performance of the column bases are highly dependent on connection details.

The estimation of the design moments M_{la} for the column base is done by modeling the rotational stiffness of the base plate connection. The formula for the column base stiffness provided by the Provisions is

$$K_{BS} = \frac{E * n_t * A_b (d_t + d_c)^2}{2 l_b}$$

Some of the symbols are shown in the Fig. 3.4.; A_b is the cross section area of the anchor rods; l_b is the embedded length of the anchor rods; and n_t is the number of anchor rods on the tension side.

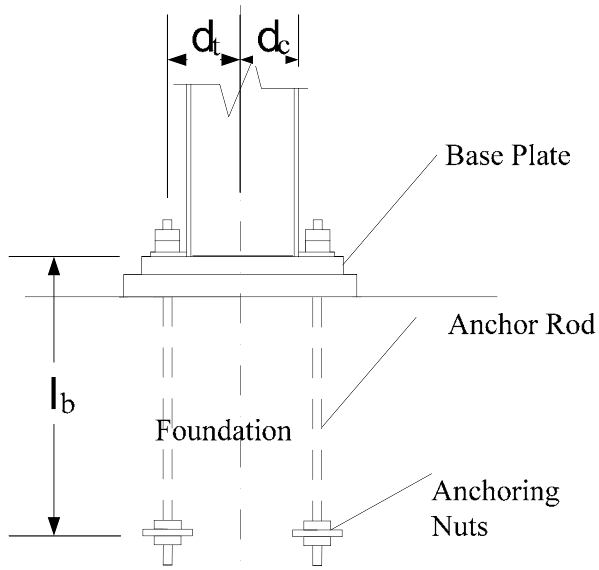


Fig. 3.4. Dimensions for an exposed base plate column base [after AIJ(2001)]

The next step is to decide which part of the column base is to yield and design it accordingly. If a plastic hinge is desired on the lower end of the column, two conditions must be satisfied: (1) the bending strength of the column base must be greater than or at least equal to 1.3 times the plastic moment of the column and (2) ductile anchor rods shall be used. If the column base is designed to yield before the column develops its plastic moment at the base, that yielding must occur due to the yielding of the anchor rods. In order to design the column base connection, its bending strength must be greater or equal to two times the design moment Ml_a. Depending on the type of anchor rods used, the bending strength of the column base is defined as follows. If no ductile anchor rods are used, then the connection must resist the demand $2Ml_a$ by means of its yielding strength M_y only. However, if ductile anchor rods are utilized, the Provisions allow using for the design the ultimate strength of the base plate connection M_u. Plastic stress distribution (i.e. $f_c=0.85f_c^{'}$) is used to calculate M_u and triangular elastic distribution for the calculation of M_y. Three cases are differentiated for the calculation of M_u, namely

a) Compression controls $N_u \geq N > N_u - T_u$

with N_u = maximum compression strength of the concrete under the base plate.

$$N_u = 0.85BDf_c^{'}$$

T_u= maximum tensile strength of anchor rods

Then $M_u = (N_u - N)d_t$

b) Tension controls $-T_u \geq N > -2T_u$

Then $M_u = (N + 2T_u)d_t$

c) Intermediate cases $N_u - T_u \geq N > -T_u$

Then $M_u = T_u d_t + \dfrac{(N + T_u)D}{2}\left(1 - \dfrac{N + T_u}{N_u}\right)$

In order to define ductile anchor rods a limit yield ratio of the steel equal to 0.75 is used (See Section 2.2.1). In other words, the anchor rod is designed in such a way that the yield strength of the unthreaded part be greater than the strength at fracture of the threaded section. Finally, the thickness of the base plate is calculated assuming the overhang of the base plate as a cantilever beam, with a uniform load applied equal to the maximum bearing stress.

3.4.2. Column Bases with Embedded Base Plates

Compressive and Tensile Axial Forces. Two mechanisms are involved in the transfer of compressive axial forces in embedded column base plates. These are the bond resistance between the faces of the column and the concrete foundation and the resistance of the reinforced concrete to the punching action on the bottom of the foundation (Pertold et al., 2000a). For the transfer of tensile axial forces, bond resistance acts in the same way it does for compressive forces, but the base plate still bearing on the concrete will subject the foundation to pull out forces. The use of shear studs welded to the face of the column helps to maintain the bond resistance. The use of vertical rebars in the foundation will transfer the pull out forces after the concrete has cracked.

Axial Loads and Moments. It has been shown in research (Pertold et al., 2000a, 2000b; Nakashima et al., 1986) that there is no noticeable interaction between axial loads and moments. For that reason the resistant mechanism for these two actions can be treated independently.

Shear and Moment. The stress distribution due to moments and shear in embedded column base connections has been investigated using finite element analysis with different embedment lengths (Pertold et al., 2000a, 2000b). The mechanism by which both types of forces are transferred is essentially the same. The exterior flange directly bears on the concrete while the interior face acts through the concrete that fills the space in between both flanges. While some investigators recognize a triangular distribution of the bearing stresses in the foundation (Nakasima et al., 1986) others identify the distribution with a trapezoidal or rectangular shape (see Fig. 2.8.) (Pertold et al., 2000b). Only Japanese provisions have addressed this topic and a brief description of their design procedure is presented below.

Japanese Design Procedures

As mentioned in the explanation of the Japanese design procedure for exposed base plates, the first step in the design of a column base connection is the estimation of the elastic rotational stiffness for the structural analysis purposes. Embedded columns with an embedment length equal or greater than 2 times the depth of the column D are assumed to be fixed, locating the rigid point at a distance equal to 1.5 times D below the concrete surface . There is no estimation of the elastic rotational stiffness proposed in the Provisions for shallowly embedded column bases (i.e. embedment length less than 2D) and particular recommendations are given about avoiding the pullout rupture of column bases

Due to the expected behavior of the connection, the column base is designed upon the nominal strength capacities of the column (i.e. flexure and axial) rather than for the demands obtained from the structural analysis. The design is divided into two parts: interior and exterior columns. For interior columns the elastic and the plastic flexural strength of the column are compared to those of the foundation. The ratio must be greater than 1.

Column: - Elastic flexural strength = ${}_cM_{sy} = Z \cdot F_y$ (where Z is the section modulus)

- Plastic flexural strength = ${}_cM_p = Z_p \cdot F_y$ (where Z_p is the plastic section modulus)

Foundation: -Elastic Flexural Strength = $M_y = \dfrac{F_{cy} \cdot B_c \cdot l \cdot d^2}{2(3l + 2d)}$ (where l is the distance from the inflexion point of the column to the level of the concrete, F_{cy} is the yielding compressive strength of the concrete taken as 2/3 of the specified compressive strength of the concrete F_c and the geometric dimensions can be found in Fig. 3.5.)

- Plastic Flexural Strength = $M_u = F_c \cdot B_c \cdot l\left[\sqrt{(2l+d)^2+d^2}-(2l+d)\right]$ (where F_c is the specified compressive strength of the concrete)

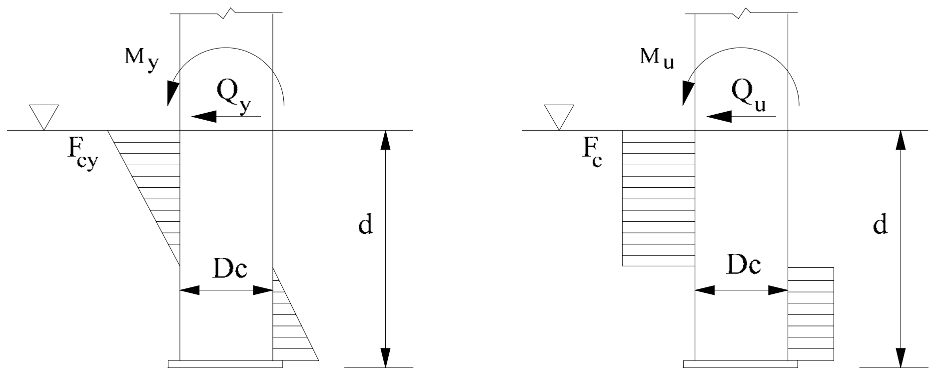


Fig. 3.5. Elastic and plastic bearing stress distributions [after AIJ (2001)]

For exterior columns, steel reinforcement must be calculated to resist the demands that the concrete, subject to tension, cannot transfer. Fig. 3.6. and Fig. 3.7 show the elastic and plastic distribution of the bearing stress and the tension force in the steel.

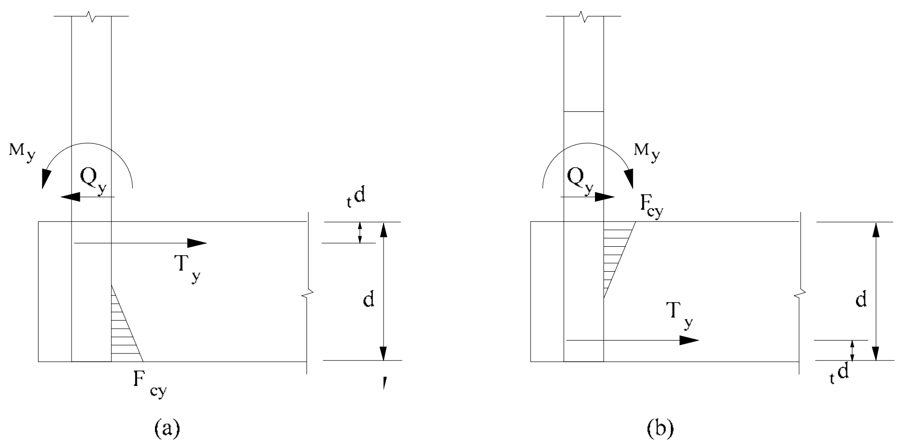


Fig. 3.6. Elastic bearing stress distribution on the foundation and tension forces on the steel Reinforcement [after AIJ (2001)]

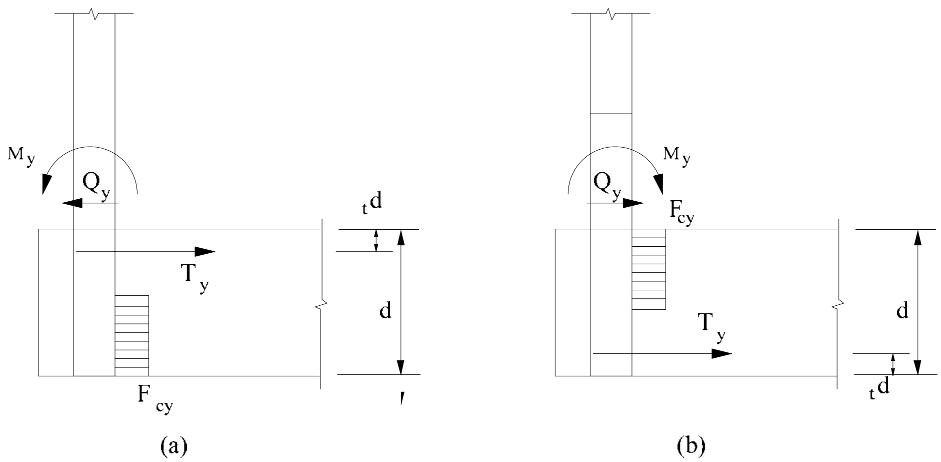


Fig. 3.7. Plastic bearing stress distribution on the foundation and tension forces on the steel reinforcement [after AIJ (2001)]

The plastic moment that the column can develop is calculated taking into account the axial loads acting on the column. The equation to calculate this plastic moment is given by:

$$ {}_{c}M_{p} = Z_{p}F_{y}\left[1-\frac{A^{2}}{(4A_{f}+A_{w})A_{w}}\left(\frac{N}{N_{y}}\right)^{2}\right] \quad \text{valid for } \left(\frac{N}{N_{y}}\right)<0.5 $$

Where A, A_f and A_w are the total, flange and web cross section areas of the column; N_y is the squash load and N is the design axial load (tension or compression); Z_p is the plastic section modulus

When the tension is on top of the foundation, the elastic tensile strength of the steel reinforcements is equal to $T_y = a_t F_{ry}$ and its plastic tensile strength is given by $T_p = a_t F_y$
The elastic flexural strength of the foundation M_y, which will be compared to the elastic flexural strength of the column ${}_cM_{sy}$, is calculated using the following expression:

$$ M_{y} = \left[T_{y} - \frac{3}{4}F_{cy}B_{c}(l+d) + \sqrt{\frac{9}{16}F_{cy}^{2}B_{c}^{2}(l+d)^{2} - \frac{3}{2}F_{cy}B_{c}T_{y}(l+{}_{t}d)}\right]\cdot l $$

When the tension is on the bottom, the expression for M_y changes as follows:

$$M_y = \left[-(\frac{3}{4} F_{cy} B_c l + T_y) + \sqrt{\frac{9}{16} F_{cy}^2 B_c^2 l^2 + \frac{3}{2} F_{cy} B_c T_y (l + d - {}_t d)} \right] \cdot l$$

The corresponding plastic flexure strength M_u with tension on top and bottom is given by:

$$M_u = F_c B_c l \left[\frac{T_y}{F_c B_c} - l - d + \sqrt{(l+d)^2 - \frac{2T_y (l + {}_t d)}{F_c B_c}} \right] \text{ tension on the top of the foundation}$$

$$M_u = -(F_c B_c l^2 + T_y l) + F_c B_c l \left[\sqrt{l^2 + \frac{2T_y (l + d - {}_t d)}{F_c B_c}} \right] \text{ tension on the bottom of the foundation}$$

Where the ratio $M_u / {}_c M_{pc}$ must be greater than 1.3.

Finally, the axial strength of the foundation is checked. For elastic design, the compressive strength capacity N_y is obtained by multiplying the area of the base plate times 2/3 the specified compressive strength of the concrete F_c. The plastic strength capacity ${}_c N_u$ uses the same procedure but applying 1.0 F_c.

The ultimate tensile strength capacity of the connection ${}_t N_u$ is carefully considered. Three quantities are compared to determine this capacity, i.e. the bearing action of the base plate's neat area (i.e. base plate area minus the column section area) on the concrete = $A_{bp} F_c$, the tensile capacity of the anchor rods = $a_{ab} F_{ab}$, and the punching shear strength T_p. The maximum of the two first parameters is compared with the last. The minimum of these two is selected as the tensile strength of the connection.

$${}_t N_u = \min\{\max(A_{bp} F_c, a_{ab} F_{ab}), T_p\}$$

The thickness of the base plate is designed following the same procedure explained for exposed base plate connections.

Chapter 4

Analysis of Issues to be Addressed

4.1. Introduction

Through the analysis presented in the previous chapters, it has been demonstrated that numerous unresolved structural issues related to the seismic as well as non-seismic behavior of column bases in the U.S. exist. Most of these topics fit into three main gaps in current U.S. seismic design provisions on column bases, applicable both to braced frames and to moment frames.

First, previous research on column bases has only started to comprehensively address all of the key issues for how to best design a column base for combined axial force (including pull-out effects as well as axial compression effects), moment, and shear.

Second, in spite of the significant role of the partial fixity (i.e., partial rigidity) of the column base in seismic behavior and performance of multi-story building structures, little has been done to synthesize prior worldwide column base research, to quantify the partial fixity for different types of column bases, and to investigate the effects of partial-fixity on the stability of columns and frames under seismic forces (Stojadinovic et al., 1998).

Third, most of the column base research in the U.S. has concentrated on exposed column-base plate connections, even though shallowly-embedded or deeply-embedded column-base plate connections have been common for both the steel MRFs and BFs in high seismic regions in the U.S. Following the damage to exposed column-base plate connections during the 1994 Northridge earthquake, U.S. engineers in high seismic regions have been developing alternative column base details to assure a fixed joint and to transfer large column moments in MRFs (and forces in the bracing member in the case of BFs) to the foundation (Cochran, 2003b). These topics leave important gaps in the understanding of the performance of these critical structural components.

This chapter summarizes the key issues in more detail for column base plate design that have not yet been studied and solidified in U.S. seismic design provisions.

4.2. Numerical Evaluation of the Partial Fixity Provided by the Column Base

A reasonable starting point for improving the treatment of the column base in design is investigating how to obtain a more accurate assessment of the amount of restraint that is provided. In seismic behavior, as addressed by Astaneh et al. (1992), the column base serves to fulfill two structural purposes. It transmits active loading from the soil to the structure and returns inertia forces from the structure back to its foundation. This phenomenon may be analyzed from the perspective of three important topics that have already been mentioned in this report. First, the seismic response is a function of the ability of the structure to dissipate the energy from dynamic actions and to tolerate large deformations without collapse. Thus, the response of a seismically designed column base should be taken into account when seismic design actions are determined. In addition, there is little benefit in taking extra care to detail ductile frame connections if the column base does not accompany that capability and collapses after a few cycles of loading. Certainty needs to be provided that the column base will behave in the expected manner. Secondly numerous analytical studies have shown that the performance of the structure changes – and sometimes noticeably- with the column base behavior (Galambos, 1960; Sato 1987; Yamada, 1997; Kawano and Matsui, 1998; Burda and Itani, 1999; Fahmy, 1999; Stojadinovic et al., 1998). Finally, the forces for which the column base itself has to be designed will vary with the rigidity. Evidence has shown hat neither the adoption of theoretical failure mechanisms nor the overdesign of each component will necessarily produce a final design on the conservative side (Hitaka et al., 2003). Following this line of thought, some of the parameters that need to be evaluated are:

Initial Elastic Stiffness

Initial elastic stiffness of the base plate is one of the first quantities that characterizes the behavior of the structure and allows a more realistic assessment of the forces transmitted to the soil under service loading. Research has also shown that providing a minimum amount of stiffness results in response that is nearly fixed and may thus be modeled as such (Picard and Beaulieu, 1985; Picard et al., 1987). It has been noted in some reports (Hitaka et al. 2003; Morino et al., 2003) that understanding the initial elastic stiffness of the column base connection is important not only to quantify the sidesway stiffness of the frame, but also to assess the strength of the critical first story (e.g., the flexural response of the first story columns).

Formulation of Moment-Rotation Characteristics

When a second-order inelastic analysis is carried out, at least two responses are sought: a) the progression in the formation of plastic hinges in the members; and b) the loading that produces instability. After advanced stages of nonlinear behavior have occurred, it is not uncommon to find plastic hinges developing at the base of the first story columns with fixed supports. Depending on the location of inelasticity in the column base connection, it is thus important to consider appropriate monotonic and cyclic nonlinear formulations to simulate this response, as the nonlinear response of the column base may not be the same as the response of a steel beam-column forming a plastic hinge (which is what may typically be modeled, e.g., in a push-over analysis).

4.3. Analytical Evaluation of the Column Base Resistant Mechanisms

The second area in which research should continue is on the subject of column design for complex combinations of required strengths or deformations that are typically seen in column bases subjected to seismic loading. For some combinations of external actions (i.e., axial compressive force plus monotonically-applied flexure and shear), more detailed experimentation has been completed, whereas for other cases (i.e., axial tension pull-out forces plus flexure and shear), the analysis has been more limited. Furthermore, the dynamic effect of these actions has been considered in scattered research for only some types of column bases and mainly for moments acting with compressive axial forces. The issues, therefore, remain unanswered, specifically are current design provisions that were developed mainly for monotonic loading adaptable to cyclic loading, and are the provisions too conservative? Based on these considerations the analysis of the problem is clarified in the following outline.

Combinations of Axial Compression, Flexure, and Shear

Chapters 2 and 3 document that research has been conducted on exposed base plates subjected either to combined cyclic axial force and flexure, or to combined cyclic axial force and shear. This decomposition is rational since the components used to resist these two groups of external actions are essentially different. With the axial compressive load playing normally a beneficial role in the enhancement of the connection strength, the strengths of the anchor rods in tension and the base plate in bending are used to transfer moments, and the friction between the base plate and the concrete foundation plus the anchor rods in shear and, at times, the use of a shear key, are used to resist shear. However, it has been shown that current design procedures in the United States (Drake and Elkin, 1999) for monotonically-loaded base plates and anchor rods in general yield conservative approximations (Famhy, 1999; Lee, 2001).

On the other hand, there has been a lack of research on embedded column bases subjected to combined loadings, particularly work in the U.S. using steel wide-flange columns. Whereas the axial compressive load has been found to have an important effect on the overall response of column bases with exposed base plates, little effect, either beneficial or detrimental, has been reported for the case of embedded column bases (neither deeply embedded nor shallowly embedded) (Nakashima and Igarashi, 1986; Nakashima, 1992) . The two internal mechanisms found to develop the axial strength capacity (i.e., the concrete-steel bond and the bearing of the base plate on the concrete above and below it) are only related to that action (i.e. compressive or tensile axial forces) and not coupled to other actions (Pertold et al., 2000a). However, for the other force components (i.e. shear and flexure), the internal mechanism to resist the external forces in embedded connections is primarily the same: the bearing resistance of the concrete surrounding the faces of the embedded column. This fact may mark a difference in the consideration of which forces will be superimposed in order to define the most demanding actions. For example, the maximum bearing stress on the face of the column flange could be the resultant of the stresses due to moment plus shear. Detailing of the reinforcement in foundations and grade beams, as well as shear connectors between the steel column and the concrete encasement, are critical. Moreover, complementary considerations need to be addressed when analyzing the resisting mechanisms, e.g., should the system overstrength factor for seismic design (AISC, 2002) be used for the design of any components in embedded column-base plate connections?

For shallowly embedded column bases, the resistance is accomplished by the simultaneous action of the concrete and the base plate-anchor rod system. As with embedded column bases, concern should be focused especially on the design of the steel reinforcement of the foundation, which will be the element responsible for ensuring ductility without significant strength degradation of the assemblage when subjected to combined loadings. The crack patterns in the foundation or grade beam should be made clear in order to provide an appropriate steel reinforcement to sustain the concrete contribution beyond its cracking. In addition, the shear connectors between the steel column and the concrete now play a significant role in the cyclic performance; these connectors may be subjected to combined cyclic shear and axial force, which can be detrimental to the shear stud ductility and strength. While significant research has been conducted on the cyclic strength of shear connectors in steel-reinforced concrete columns, composite wall systems, and composite beams (e.g., see Roeder et al., 2002; Saari et al., 2004), little has been done for the specific details being assessed for column bases, and thus adequate assessment must be made of the applicability of past work on shear connectors for column base details.

Some key questions that should thus be addressed for deeply and shallowly embedded base plate connections subjected to combined cyclic loadings include:

1. How should the interactive design strengths and interactive deformation demands of the anchor rods and base plates be calculated for embedded column base plate connections subjected to combined cyclic loadings?
2. What welding details are economical for the joint between the column and base plate in embedded column-base plate connections subjected to combined cyclic loadings?
3. What kinds of column stiffener details are recommended for embedded column-base plate connections subjected to combined cyclic loadings?

4. What are appropriate detailing requirements for the concrete foundation and concrete grade beams, particularly for size of the grade beam and steel reinforcement detailing?
5. Are the current methods for evaluating the composite action applicable to shear stud design in column bases?
6. Does the embedment length have any influence on the stud behavior?
7. If a steel grade beam is used, what is the appropriate size and connection detailing for a steel grade beam when the column base is subjected to combined loadings?
8. If a brace connection is used, including a gusset plate, does the axial force still have little effect on the performance?
9. Do the forces from the brace more quickly degrade the strength of the embedment details?
10. Do columns at the exterior of the building, with a truncated embedment on one side, need different design provisions than those on the interior?

Combinations of Axial Tension (Pull-Out), Flexure, and Shear

Combined axial tension (pull-out) plus flexure and shear potentially cause the largest demands on almost all types of column bases (Nakashima and Igarashi, 1986; Sato, 1987) . However, little attention has been given to this issue for both exposed and embedded base plates in both moment and braced frames. It is known that this kind of loading does not appear as an isolated action, but is found most in external columns during seismic loading (Hitaka et al., 2003).

For the case of exposed base plate type column bases, some components of the assemblage as well as the resistance mechanisms will be affected by the action of axial tensile forces. The shear resistance of the assemblage cannot count on the friction under the base plate from the contact surface with the concrete due to an earlier separation of both surfaces. Shear keys, a commonly used system to transfer lateral loads to the foundation, are designed as cantilever beams. If uplift occurs and the base plate is bent upwards, then the shear lug will be subjected to higher moments due to the deformation of the base plate. Another consequence is the decrease of the rotational stiffness of the connection. This reduction has been ascribed to the separation of the base plate from the concrete underneath. With less stiffness, the assemblage will undergo higher rotations. Consequently the range of deformations and stresses acting on the welds will likely increase, requiring particular consideration of their design. In the case of embedded columns in a concrete foundation, shear stud connectors may be used to offset the effects of axial tension on the column base connection. The composite action of the shear studs, welded to the column, helps to increase the pull-out resistance of the connection as well as the overall capacity to withstand several loading cycles without strength degradation (Nakashima and Igarashi, 1986). Finally, in the analysis of shallowly-embedded columns, the presence of pull-out forces has been considered the reason for defining the minimum required embedment length (i.e., 1*D*) (Pertold et al., 2000b).

4.4. Investigation of the Progression of Failure Modes

The literature on column base connections offers no unified acknowledgment of what a preferred progression of damage is in a base plate connection, or what parameters could help in the selection of the progression of damage, or how to design a column base in order to produce a specific mechanism that is sought. Capacity design principles consistent with the AISC Seismic Provisions (2002) provides one means of controlling the progression of damage in the concrete, anchor rods, steel base, steel column, steel stiffeners, and grade beam. For unbraced frames some possible options for the progression of failure are:

OPTION 1: The Base of the Column is Designed to Fail First

A column base connection, assumed as theoretically fixed or providing large partial restraint should be strong enough to allow the formation of a plastic hinge at the base of the column. It has already been discussed previously that ductility and energy dissipation can be achieved by means of this mechanism. Even though all types of column bases described in this study can be designed to provide this level of restraint, the most suitable type to be used for a weak column-strong column base connection mechanism is one in which the column base is deeply embedded. Some of the advantages of this kind of connection are:

(a) It is a straightforward connection with components that are designed to fail that include the column, followed by the shear connectors to the concrete, followed by crushing of the concrete or yielding of the steel reinforcement; typically the column base plate itself, or the stiffening components of the column section do not fail in this type of connection (Nakashima and Igarashi, 1986).

(b) The component of the column base that must maintain extra strength during failure of the column and shear connectors is the concrete foundation or grade beam; this is easily achievable due to the intrinsic strength and stiffness of reinforced concrete.

The drawbacks of this design approach have been pointed out in Chapter 2 and 3, namely:

(a) Embedded connections require pouring of the concrete at similar times to the erection of the steel superstructure, thus potentially causing conflicts on the job site.

(b) The time needed for the curing of the concrete potentially delays the steel erection.

(c) The detailing of the connection needs to be implemented more carefully in order to offset the poor performance of the concrete in tension.

Some drawbacks can be solved by using shallowly embedded base plates (i.e., drawbacks topic (a) and (b)); however, the detailing of these connections (i.e., shear studs, reinforcement bars, anchor rod layout, etc) becomes more critical. The use of column bases with an exposed base plate, although always feasible has some shortcomings that should be considered, namely:

(a) The number of components that are affected by the progressive damage that involves a weak column-strong column base connection is greater than embedded connections, including the column plastic hinge, the weld details to the base plate, the base plate, the anchor rods, and the concrete foundation.

(b) The cost of the connection may increase considerably due to the requirement that some elements must remain serviceable under ultimate loads, according to a capacity design approach.

(c) Larger rotations are more likely to occur in earlier stages of the deformation, a drawback whose consequences could be avoided if a more accurate assessment of the partial restraint of the connection is feasible.

For this type of damage progression, it is necessary to ascertain how much strength the column base must have so as to allow the column to develop its plastic hinge when subjected to combined cyclic loading. In the case of embedded base plates in particular (i.e., either deeply or shallowly embedded) geometric and strength requirements necessary to avoid degradation of the foundation (i.e., concrete strength, depth of the foundation, column embedment length, connection strength) must be developed. In addition, the recommendations must include guidelines on reinforcement detailing (i.e., reinforcement layout, anchorage, design and position of shear studs) according to the structural element to which the column is attached (i.e., footings, grade beams, or reinforced slabs) and to the different position in the frame (i.e., perimeter or internal columns). For the case of shallowly embedded base plates, it is also necessary to document the interaction of the response of the additional resisting mechanisms acting in this type of configuration (i.e., anchor rod-base plate action versus the embedded concrete resistance). The variation of the parameters involved (i.e., embedment length, size of base plate and anchor rods, reinforcement details) causes particularly different responses in this type of connection relative to achieving the weak column-strong column base connection progression of damage. For that reason, definition of the boundaries of applicability of each configuration and their consequences on the final performance of the connection should be described. In addition, although a brittle failure of the concrete usually entails a sudden strength degradation of the column base, it should be noted that for shallowly-embedded base plates this failure is more critical and should be considered in more detail.

OPTION 2: The Column Base Connection is Designed to Fail First

Exposed base plate column base connections are the only option for developing a capacity design approach based upon a strong column-weak column base connection mechanism. Generally speaking, only three components of this kind of column base connection could exhibit extensive nonlinear response:

(a) the base of the steel column, as covered in option 1 above;
(b) the base plate in flexure;
(c) the anchor rods in tension.

The yielding of the base plate provides one of the most efficient mechanisms for delivering high ductility with important amounts of energy dissipation, as presumed in one of the design procedures most used in practice (i.e., the Drake and Elkin method). The design of the base plate assumes that this element will be transforming part of the external moment into deformation energy, while the anchor rods in tension and concrete are designed to resist the full external moment, they will be able to resist forces higher than the ones that will actually be acting on them. However, as mentioned in Chapter 3, some reports pointed out that base plates designed with the Drake and Elkin method might be too stiff and not develop the desired plastic behavior at failure (Famhy, 1999; Lee and Goel, 2001) or withstand higher loads prior to yielding.

However, no rational criteria have been made available to ensure that plastification of the base plate will take place before that of the base of the column.

The use of yielding of anchor rods as the principal failure mode is been more widely studied. In Japan, for example, designing to achieve ductile failure in the anchor rods prior to failure elsewhere in the connection is encouraged. In this approach, the base plate is considered as a rigid link that transfers the full external moment into tension in the anchor rods and compression on the concrete, and the corresponding final strength prediction is relatively simple (Sato, 1987). As identified in Chapter 2, the concerns of some investigators with this solution are related to the high stresses to which the compressed concrete is subjected to and the intrinsically less ductile behavior of the anchor rods (Astaneh at al, 1992). Additional issues that should be investigated for this design approach include:

(a) Is it important to allow the anchor rods freedom to elongate as soon as loading begins?
(b) How stiff does the base plate need to be in order to transfer the full external moment without significant deformation?
(c) How should the anchor rods be designed to resist shear as well as being the weakest link in tension?
(d) What special detailing is needed to help the concrete withstand the larger compression stress it is subjected to?

OPTION 3: Combined mechanisms

The literature (Fahmy, 1999) mentions the existence of several tests in past research where a simultaneous occurrence of the aforementioned mechanism was reported. This possible combination should be considered in the present analysis.

On the other hand, for the case of braced frames, the lack of research on the behavior of column bases with gusset plates framing into the connection has already been mentioned. This implies that together with the comprehensive research plan that needs to be implemented for column bases in braced frames, the investigation of possible progression of failure modes should be included.

4.5. Investigation of the Requirements to Avoid Brittle Behavior

According to experimental findings (e.g., Sato, 1987; Burda and Itani, 1999; Fahmy, 1999; Lee and Goel, 2001), the most common source of brittle behavior in column bases may be found in a poor performance of the welds, anchor rods, or concrete. However, after the Northridge Earthquake, the Northridge Reconnaissance Team (1996) reported an additional type of brittle behavior not reproduced in experiments, namely fracture of a thick base plate. Even though premature buckling of the column flanges was found to be another possible failure mode with low energy dissipation, the use of columns with compact sections eliminates the probability of this type of failure (Fahmy, 1999). Issues related to preventing these brittle failure modes are discussed below.

Fracture of Welds

Extensive work has been completed on assessment of the fracture resistance of welds over the last decade. The AISC Seismic Provisions (2002) already have included recommendations on the use of welds with notch tough weld metal in connections in the Seismic Load Resisting System (AISC, 2002). However, according to recent investigations (Fahmy, 1999), alternative details (i.e., modified partial joint penetration welds), could be used in addition to the CVN requirement of AISC (2002) that could improve the behavior of the column base.

Fracture of Anchor Rods

Investigation is needed in order to clarify practical recommendations for avoiding the fracture of anchor rods. The analysis of the relationship between the threaded and non-threaded part of the anchor rods, as pointed out in Chapter 2 and 3, would help the development of these recommendations.

Degradation of the Concrete Support

Without adequate reinforcement, the behavior of the concrete under cyclic loading deteriorates rapidly. The development of certain cracking patterns has been shown to generate a sudden deterioration of the strength of the column base (Nakashima, 1996). However, such studies available have been conducted on box sections and the particular considerations for W shapes are needed for columns embedded in the foundation, including crushing below and above the base plate, surrounding the column, and in or surrounding the grade beam.

Fracture of the Base Plates

The fracture of the exposed base plate is a brittle behavior found after some earthquakes and reported as an unexpected and undesirable failure mode. This failure mode has not received experimental treatment and the precise reason of its occurrence is not entirely understood and could be investigated further.

4.6. Issues Not Covered in Prior U.S. Research.

Aside from the evaluation of the typical characteristics of different types of column bases, other issues related to the structural elements attached to the column base have not been covered adequately in prior research, particularly within the U.S. These issues are summarized in this section.

Use of Grade Beams or Reinforced Concrete Slabs to Strengthen the Connection

The use of grade beams has received widespread use when the goal is to transfer to the foundation axial forces and shear, with the grade beam absorbing the majority of the bending moment in the column. Reinforced concrete slabs have also been used to provide monolithic characteristics to column base connection region. Investigation of the procedures found in practice shows that both steel and reinforced concrete are used to construct grade beams.

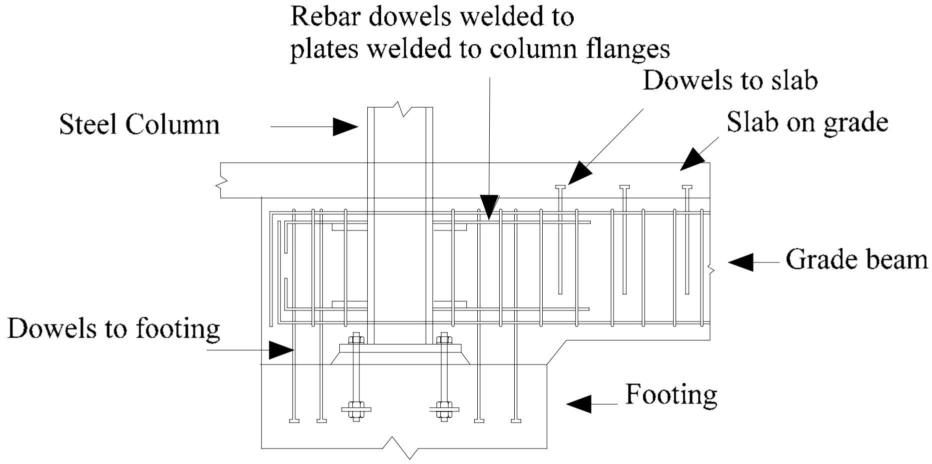


Fig. 4.1. Junction embedded base plate in a grade beam with the footing [after Cochran (2003)]

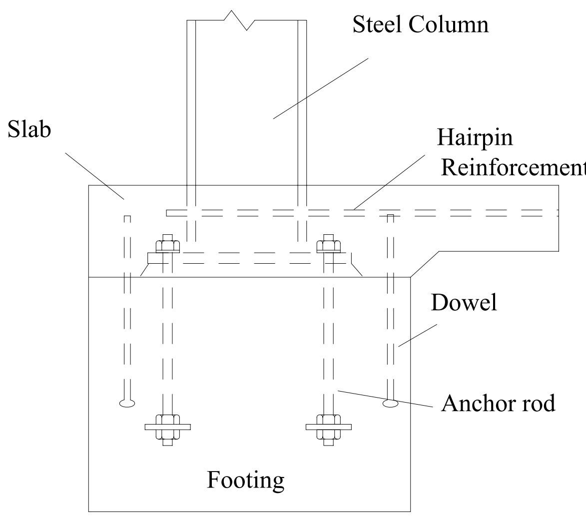


Fig. 4.2. Junction embedded base plate in a reinforced slab with the footing

Examples of some of these techniques are shown in Fig. 4.1 and Fig. 4.2. However, neither the junction of a steel grade beam with column base connections (either exposed or embedded) nor the interaction of a reinforced concrete grade beam with either a deeply- or shallowly-embedded column base plate have been verified experimentally in the U.S. for W-shape or tubular columns (for both strong and weak axis flexure), although all such configurations are used in practice. When a reinforced concrete grade beam is used, the anchorage of the connection to the foundation is usually designed by using anchor rods.

Additionally rebar dowels may be used to tie also a portion of the grade beam to the footing, as shown in Fig. 4.1. Therefore, issues to be addressed include:

(a) Are there additional considerations needed in order to model the presence of the grade beam in the structural analysis (i.e., the action of an elastic or rigid foundation underneath the beam)?
(b) What major considerations exist in designing a concrete grade beam, including the column base embedment depth, and the steel reinforcement detailing around the column base embedded in the grade beam?
(c) How large must the concrete grade beam be?
(d) What additional considerations (dimensions and reinforcement) must be given to exterior (i.e., perimeter) column-grade beam connections?

Generally speaking, the same analysis and the same questions can be used to outline the investigation of the connection between a column and a reinforced concrete slab-on-grade, although details must be investigated related to the shallow nature of the embedment for this type of configuration.

If steel grade beams are used, the issues to be addressed include:

(a) Are there additional considerations needed in order to model the presence of the grade beam in the structural analysis (i.e., the action of an elastic or rigid foundation underneath the beam)?
(b) Are the design procedures used for steel moment-resisting connections directly applicable to the design of the grade beam-base column connection?
(c) What is the proper size of steel grade beams?
(d) What are the column stiffening recommendations?

Interaction of Exposed Base Plates and Gusset Plate in Braced Frames

An important topic that has not received adequate attention in past research is the behavior of an exposed base plate to which a gusset plate is attached. Figure 4.3. shows a common connection detail where a large gusset plate forces the use of an extended base plate.

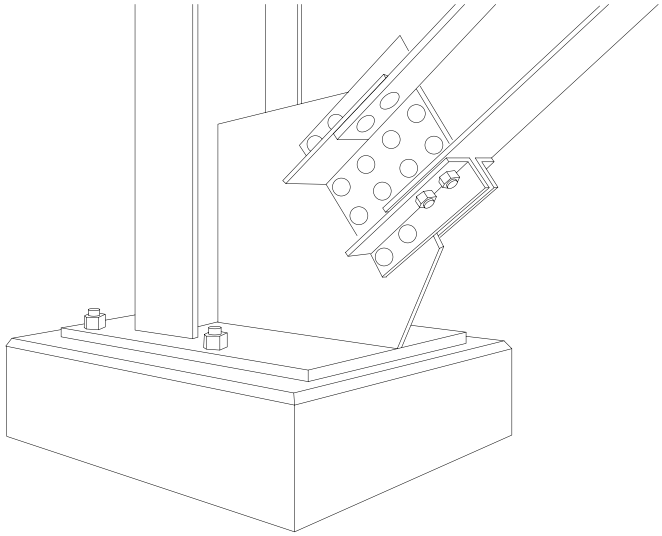


Fig. 4.3. Gusset plate-base plate interaction in a braced frame [courtesy of R.Drake, 2003]

Configurations like the one presented raise issues to be investigated, including:

(a) Are force distributions within this configuration understood adequately to assess the likely inelastic deformations of each of the components.
(b) For the extended base plate, what kinds of anchor rod arrangements are recommended and how can the anchor rod size best be designed?
(c) How should the size of an extended base plate best be determined?
(d) Does the overhang portion of the base plate (i.e., outside of the area delimited by the anchor rods) provide sufficient stiffening for the load transfer?
(e) Are conventional gusset plate design methodologies (developed mostly for gusset plates in beam-to-column joints) directly applicable for the design of gusset plates in the column bases?
(f) Where is the Working Point (WP) to be assumed for the design of the brace and gusset plate in both the exposed and embedded column-base plate connections in BFs?
(g) How much brace eccentricity from the assumed WP, causing secondary moment in the connection, can be permitted in the gusset plate design and how does this brace eccentricity influence the sizing of the gusset plate in the column base?

(h) What are the recommended gusset stiffener details for W-shape (for both the strong and weak axis cases) and tubular columns?

Behavior of Encased Column Base Connections in Braced Frames

______It is common, in commercial or institutional buildings, to find the column base connection in braced frames encased in concrete (e.g. due to the grade slab). Dowels may be used to tie this encasement to the footing. This configuration requires additional analysis due to the restraint that the concrete confers to the behavior of the connection compared with the exposed braced connection presented in the previous section. Figure 4.4. shows a probable appearance of this type of connections.

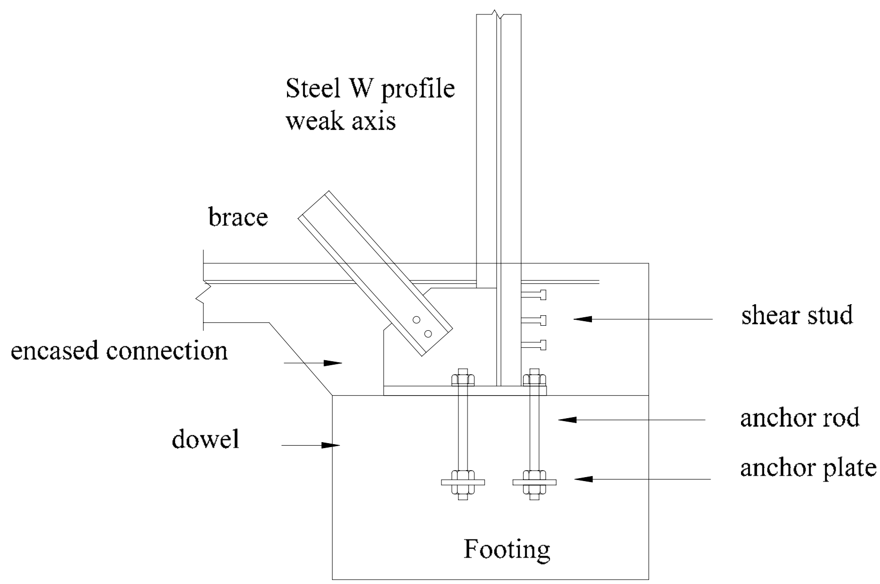


Fig. 4.4. Embedded Column base connection in a braced frame

The next chapter will summarize a general research plan for addressing the issues outlined in this chapter, prioritizing the most important issues relative to U.S. practice.

Chapter 5

Research Plan

5.1. Introduction

The discussion of the issues presented in Chapter 4 and the analysis of the information about past research contained in Appendix B are the basis for the general plan presented in this chapter. The main classification given in Chapter 2 as the representative subdivision for the two traditionally recognized support conditions (i.e., “pinned” supports and “fixed” supports) is maintained here not only for consistency but because this organization also helps to emphasize which areas need prioritized attention.

5.2. Column Bases with Embedded Base Plates

5.2.1. Research Description

This section outlines analytical and experimental studies proposed for steel wide-flange columns in which the lower portion is embedded in the concrete foundation or grade slab (see Figure 5.1 for common configurations of unbraced and braced frame column base connections). Other research around the world has included studies of embedded HSS and concrete-filled steel tube columns. Therefore it was decided to focus this proposed research on steel wide-flange columns, which are very common in U.S. building structures in seismic zones. Several configurations could be adopted for the design of this type of column base. However, as shown in the figures below, configurations commonly found in U.S. practice are recommended for study. For columns with embedded base plates, research is particularly needed in the following four areas:

- Investigation of the structural component in which the column is embedded;
- Investigation of the effect of the structural system on the response of the column base connection;
- Reproducing the position of the column in the frame;
- Simulating the loading to which the column base is subjected.

Before each of these topics is considered specifically, some details will be mentioned that are common to all the tests to be performed. According to national safety requirements, the steel column shall be welded to a steel base plate anchored in the concrete by means of four anchor rods (OSHA, 2001) that should be designed to resist at least the construction loads. All the reinforced concrete elements should be designed and detailed following all applicable ACI 318 requirements related to quality of the concrete and steel (ACI 318 Chapters 3 and 5), strength (Chapter 9), composite action (Chapter 10), seismic design (Chapter 21), footings (Chapter 15) and associated topics (ACI-318, 2002). Based on common practice, shear studs should be welded to the faces of the embedded part of the column.

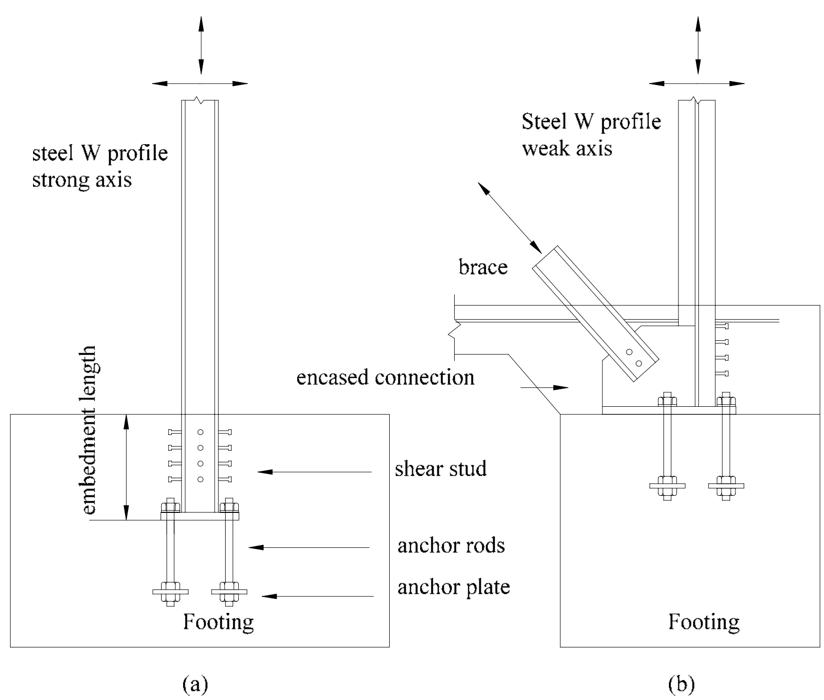


Fig. 5.1. Unbraced and braced W steel columns embedded in concrete

Concerning the structural component to which the column is attached, two general categories of embedded column bases can be differentiated: (1) columns that are embedded directly in a footing (Fig. 5.1.(a)); (2) columns that are attached to a footing and covered by a grade beam. In the former case, (i.e., Fig. 5.1.(a)), the column can be deeply or shallowly embedded in the footing (i.e., its structural response is a function of the embedment length). In the latter case, the grade beam could be made of reinforced concrete (Fig. 5.2case (a)) or of steel shape encased in concrete (Fig 5.2 case (b)). Finally the structural element that ties the grade beam-column joint to the foundation can be the anchor rods acting alone (Fig. 5.2 cases (a) or (b)) or a reinforced concrete short column (Fig. 5.2 case (c)).

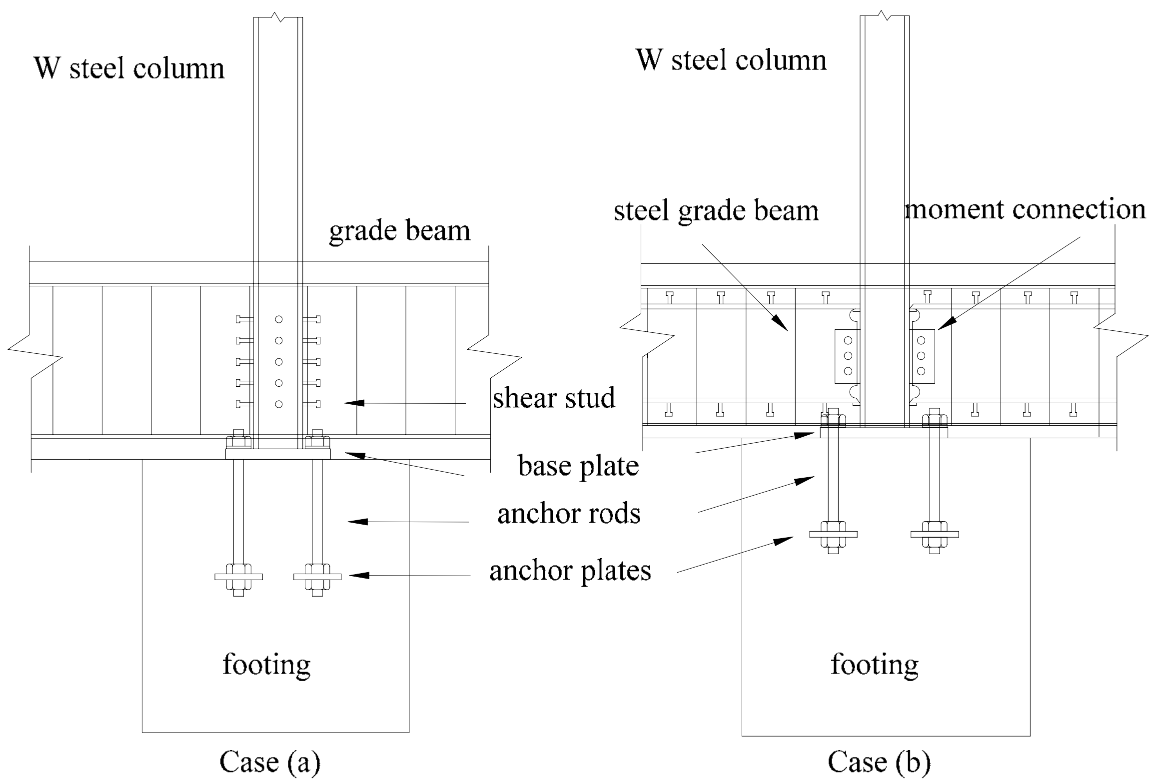


Fig. 5.2. Typical configurations of column bases with grade beams

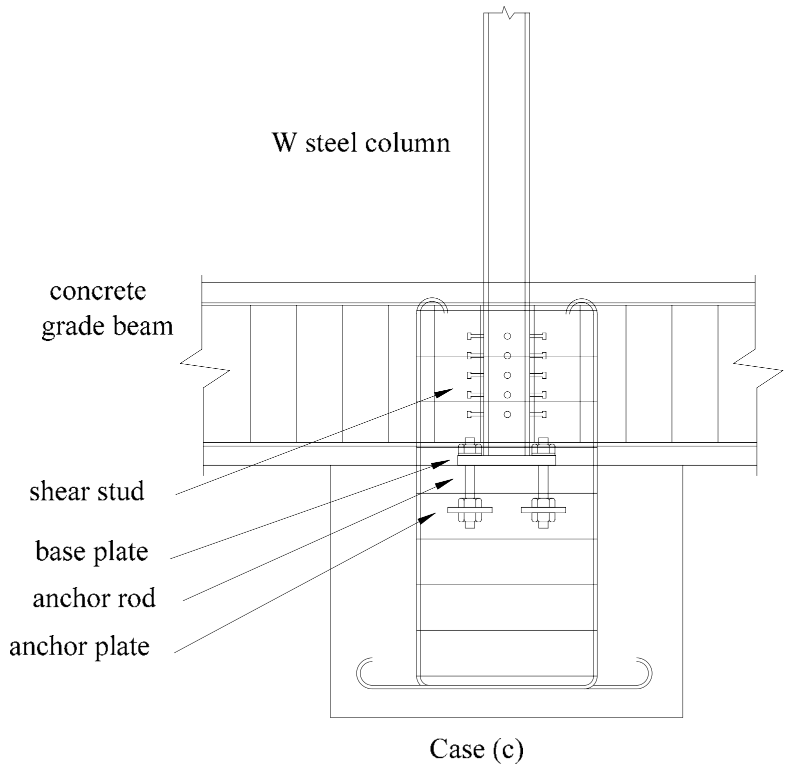


Fig. 5.2. Typical configurations of column bases with grade beams (continued)

The second issue listed above relates to the effect of the structural system on the behavior of the column base connection. While most past work on embedded column base connections has focused on columns in unbraced frames, Fig. 5.1.(b) shows a typical configuration for a braced frame where the whole connection is encased. The study should include investigation of the effect of this encasement as well as the action of the bracing and gusset plate on the performance of the whole connection.

The third aspect is the position of the column within the frame. The fact that external footings or grade beams may need additional tensile reinforcement has been emphasized in previous chapters. For that reason, besides replicating symmetric conditions (i.e., behavior that is independent of the direction of the moment, such as with interior columns), attention should be given to external columns embedded in their foundations.

Finally, the load combinations and type of loading must be considered. As mentioned in Chapters 3 and 4, the combination of shear and moment produces concurrent bearing actions on the concrete (i.e., bearing on the concrete from the faces of the embedded portion of the column), while the axial load seems to act uncoupled (i.e., the bearing is localized on the faces of the base plate). The behavior of the connection under pull-out forces as well as compressive forces should be studied. Given how little past work has been done on embedded connections based on U.S. construction practices, investigation should be directed both to monotonic as well as cyclic behavior. Concrete degradation, found to be an important parameter particularly in shallowly embedded box columns (Nakashima, 1996), needs to be studied as well, due to its influence on column base strength and stiffness degradation after some cycles of loading. Stiffening methods (i.e., shear studs) and their effects in preventing strength degradation should be analyzed.

5.2.2. Research Parameters

The parameters that should be taken into account in research on the seismic response of embedded column base plate connections to reflect current practice include:

Top Priority (in approximate order of priority)

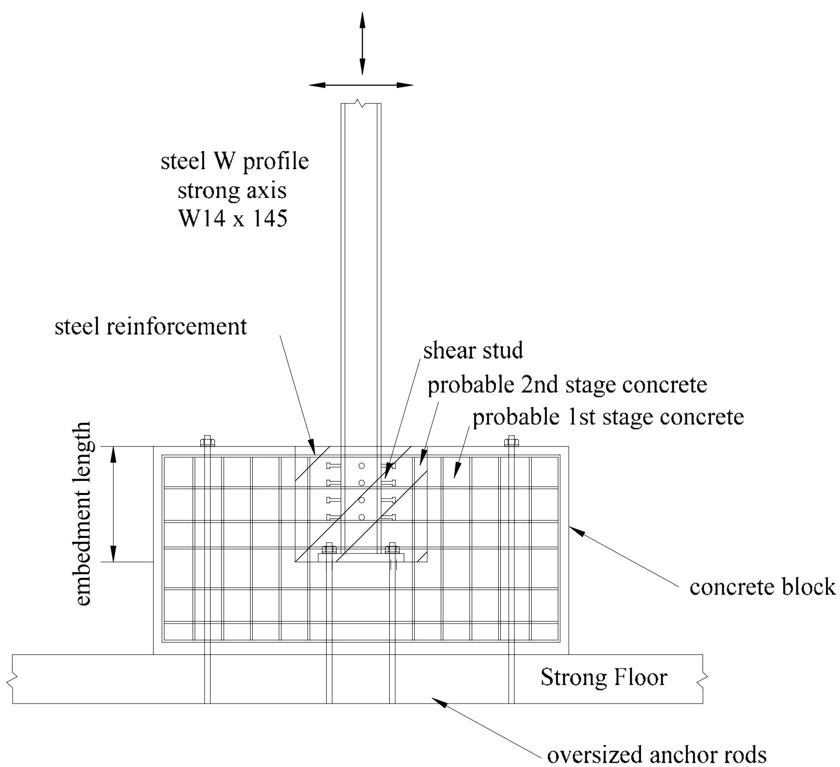
- *Strength of the concrete:* f_c'= 4 to 6 ksi (for footings). f_c'= 6 to 8 ksi (for grade beams); the strength of the concrete relative to the column strength may be important in ensuring a desired progression of failure
- *Column size:* Deep columns my pose unique transfer mechanisms relative to standard size columns
- *Embedment length:* Minimum recommended = 1*D* (*D* = depth of the steel column section, assuming strong axis flexure); maximum recommended = 3*D*.
- *Presence of grade beam*: Encased steel and reinforced concrete grade beams are common; connection details of grade beam-to-column connection (including documentation of the likely required moment strength of grade beam) also should be included in the study
- *Presence of diagonal brace, with a corresponding extension of the base plate*
- *Moment/shear ratios:* Low ratios are representative of the expected behavior in braced frames. High ratios are more characteristic of the force combinations in unbraced frames.
- *Axial loads (tensile and compressive)*
- *Welding details for base plate welds*

- *<u>Tensile steel reinforcement and shear ties in embedment and grade beam</u>*
- *<u>Composite action of shear studs</u>*
- *<u>Interior and exterior column placement</u>*

Lower Priority (i.e., less variation in these parameters is required to achieve useful results)

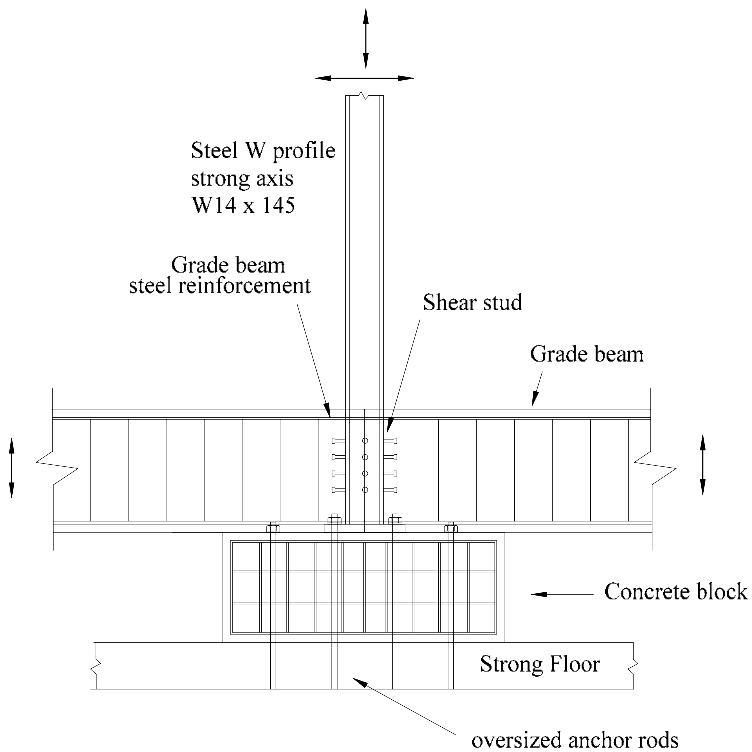
- *<u>Strength of the column steel</u>*
- *<u>Strength of the base plate steel</u>*
- *<u>Column shapes:</u>* W shapes (the research may be expanded to HSS, hollow or filled with concrete, as feasible)
- *<u>Strength of shear key</u>*

Note that investigation of the base plate and anchor rods themselves are of less value in embedded connections, as the surrounding concrete diminishes the range of stress seen in these components. Sample test configurations are shown in Fig. 5.3.



Case (a)

Fig. 5.3. Typical test specimen configurations for embedded column bases



Case (b)

Fig. 5.3. Typical test specimen configurations for embedded column bases (continued)

5.2.3. Expected Outcome

By conducting the tests detailed in the preceding sections, the research should be framed to yield the following information about embedded column base connections, in approximate order of priority:

- Formulation of the resistance mechanisms for flexure, shear and axial loads, in order to document the progression of damage (to ensure components fail in the desired order, e.g., can anchor rod yielding occur prior to significant concrete crushing) and ultimate strength of the connection and deformation for these actions.

- Quantitative classification of the connection as a function of the embedded length. This statement implies the quantitative description of the performance of the connection in the whole range of feasible embedment lengths (i.e., when the embedded behavior controls or when the combined embedded and base plate-anchor behavior controls).
- Evaluation of the contribution of the anchor rod-base plate assemblage to the total strength and stiffness of the system, particularly in connections with short embedment lengths. In particular, it is important to assess the geometric and strength requirements for fully-embedded column base connections to provide a fully restrained support with the occurrence of a hinge formation at the base of the column. In addition, a method should be established for evaluating the partial restraint provided by the connection when full fixity is not achievable (i.e., column bases with short embedment lengths in which collapse is due to failure in the concrete).
- Description of the moment-rotation responses for monotonic and cyclic loading.
- Recommendations for establishing the required moment strength in the grade beam
- Recommendations for grade beam-to-column connection details.
- Recommended column base weld details, particularly important for the cases where the anchor rod-base plate behavior contributes significantly to the total strength.
- Recommendations as to the adequacy of the ACI 318-02 requirements for the design of these types of connections in order to withstand the whole range of possible demands (e.g., interior and exterior columns, footing and grade beam, deep and shallow embedments, monotonic and cyclic loading)
- Formulation of the required forces for designing the tensile reinforcement of footings or grade beams, when the actions transferred from the column to the concrete render them necessary.
- Contribution of shear stud composite action to the connection's behavior.

- Contribution of a shear key to the behavior.

In addition to the experimental results, analytical studies are needed (e.g., theoretical derivations, finite element analyses) in order to propose how to calculate the required moments in a grade beam, knowing that the beam is not free to rotate when, due to the external forces, it is forced to bear against the soil or foundations. Additional tests, intended to confirm these analytical results, could also be conducted, in which this restrained deformation is simulated more comprehensively than the sample configurations shown in Fig. 5.3.

5.3. Column Bases with Exposed Base Plates

5.3.1. Research Description

The second group of experimental investigations to be carried out consists of assemblages in which the base plate is welded to the bottom of the column and anchored to the footing by means of embedded anchor rods. The base plate, as well as the upper part of the anchor rods with the nuts, are left exposed. This configuration is most common in the U.S. in industrial structures. Figures 5.4.(a) and (b) show that it is feasible to use this type of connection in unbraced frames as well as braced frames; the behavior of both types should be studied.

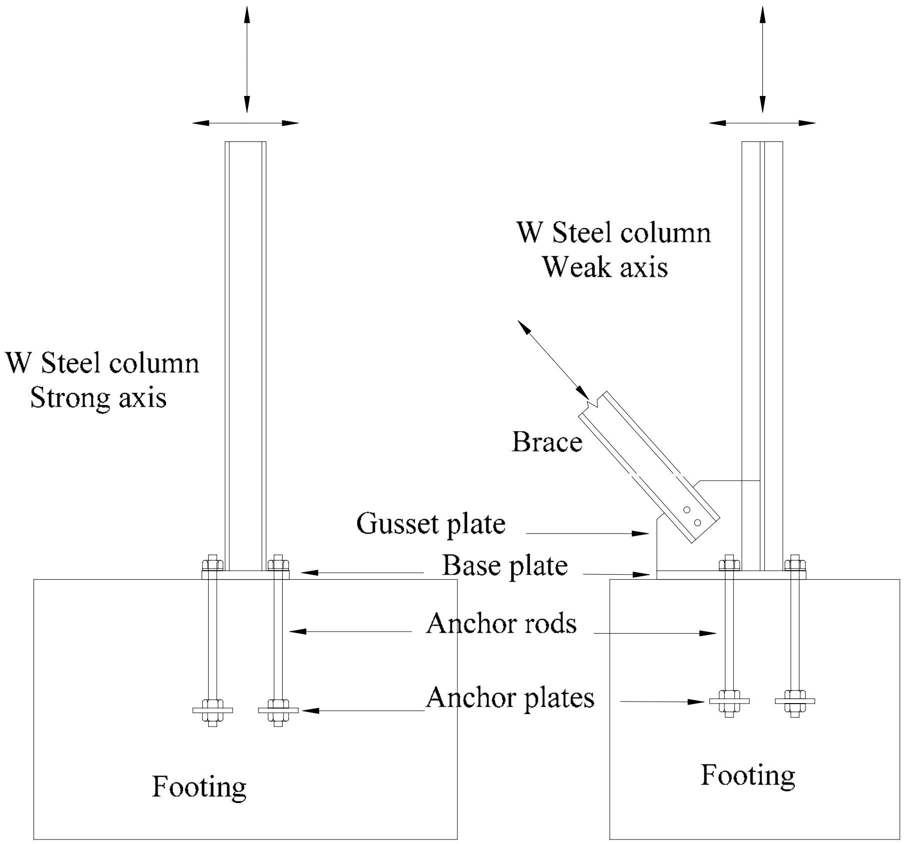


Fig. 5.4. Column bases with exposed base plates in unbraced and braced frames

In order to conduct research pertinent to U.S. practice, a minimum of four anchor rods should be used in the tests (OSHA, 2001), as well as using the ACI 318-02 (ACI, 2002) requirements for the design of the footings and steel reinforcement. While some experiments have been carried out in the U.S. on this type of assemblage with W-shape columns, as well as research on H–shapes in Europe and Japan (as tabulated in Appendix B and discussed in the prior chapters), the scope of the investigation should be broadened as follows:

- Investigation of the structural system to which the column base is attached (unbraced and braced frames);
- Description of the recommended progressions of failure and the requirements to achieve these progressions;
- Investigation of the partial restraint provided by the column connection;
- Emulating the type of loading to which the column base is subjected.

It has been stated in previous chapters that prior research has focused on the behavior of exposed base plate column bases in unbraced frames, particularly on documentation of the ultimate strength limit states and associated design procedures for a combination of axial force, flexure, and shear. However, there still are areas in which the investigation on column bases in unbraced frames needs to expand to include alternative configurations. Figure 5.5. shows a common case in seismic zones where a steel grade beam is added to the structural system acting in such a way that allows the use of an exposed base plate. Additionally, the testing of column bases in braced frames presents another area in which almost no work has been carried out. For these reasons, future research should focus on developing design procedures that accurately represent the behavior of the connection (i.e. expected failure modes, anticipated progression of damage, predicted partial restraint and ultimate strength, etc), for both unbraced as well as braced frames.

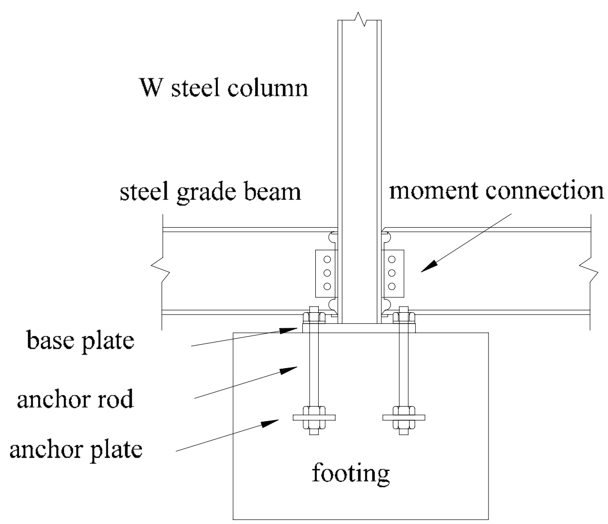


Fig. 5.5. Unbraced frame column base connection with steel grade beam and exposed base plate.

The second topic mentioned in the above listing intends to highlight the fact that a majority of the research effort should focus on the analysis of the progression of damage and ultimate failure modes. Appendix B shows that of a total of nineteen specimens tested in the U.S., thirteen failed due to fracture of the welds. This has been shown to be a brittle, and not a desirable, failure mode. Even though some tests were deliberately conceived to show the inadequacy of common weld details, and a number of them showed fracture after yielding of some of the column base components (i.e., the base plate), brittle failures are not desired for this type of connection. Additionally, despite the fact that several reports (as shown in Chapter 2 and Chapter 3) have pointed out that the use of thicker base plates leads to an increase in strength, ductility, and rotational stiffness of the connection, which are desirable features in high seismic areas; and that thicker base plates are more likely to produce a weak anchor rod failure mode (i.e., a ductile failure mode when ductile anchor rods are used), almost no research has been carried out to study this behavior and make appropriate recommendations for their use.

The third aspect of the investigation of exposed base plate performance aims to address the development of a methodology for accurately assessing the calculation of the forces the connection will be subjected to and reliably reproducing the behavior of the structure (i.e., member forces, sway displacements) through establishing a more accurate assessment of the stiffness of exposed column base connections.

Finally, as stated in the Commentary, Section 8.5 of the 2002 AISC Seismic Provisions (AISC, 2002) the emphasis of future research should be on the investigation of structures under cyclic loading where seismic conditions are simulated and larger prototypes are tested.

5.3.2. Research Parameters

The parameters that shall be varied or taken into account are described in the next outline:

Top Priority (in approximate order of priority)

- *Strength of the concrete:* f_c'= 4 to 6 ksi (for footings).
- *Strength of the anchor rod steel*
- *Welding details for base plate welds*
- *Size of the base plate*
- *Presence of diagonal brace, with a corresponding extension of the base plate and extended layout of anchor rods (i.e., use of six anchor rods)*
- *Column size:* Deep columns may pose unique transfer mechanisms relative to standard size columns
- *Moment/shear ratios:* Low ratios are representative of the expected behavior in braced frames, and lead to necessary further investigation of shear transfer in

these types of base connections (e.g., design of shear keys, etc.). High ratios are more characteristic of the force combinations in unbraced frames.

- *Axial loads (tensile and compressive)*

Lower Priority (i.e., less variation in these parameters is required to achieve useful results)

- *Strength of the column steel*
- *Strength of the base plate steel*
- *Column shapes:* W shapes (the research may be expanded to HSS, hollow or filled with concrete, as feasible)
- *Strength of shear key*

A sample test configuration for a braced frame connection is shown in Fig. 5.6.

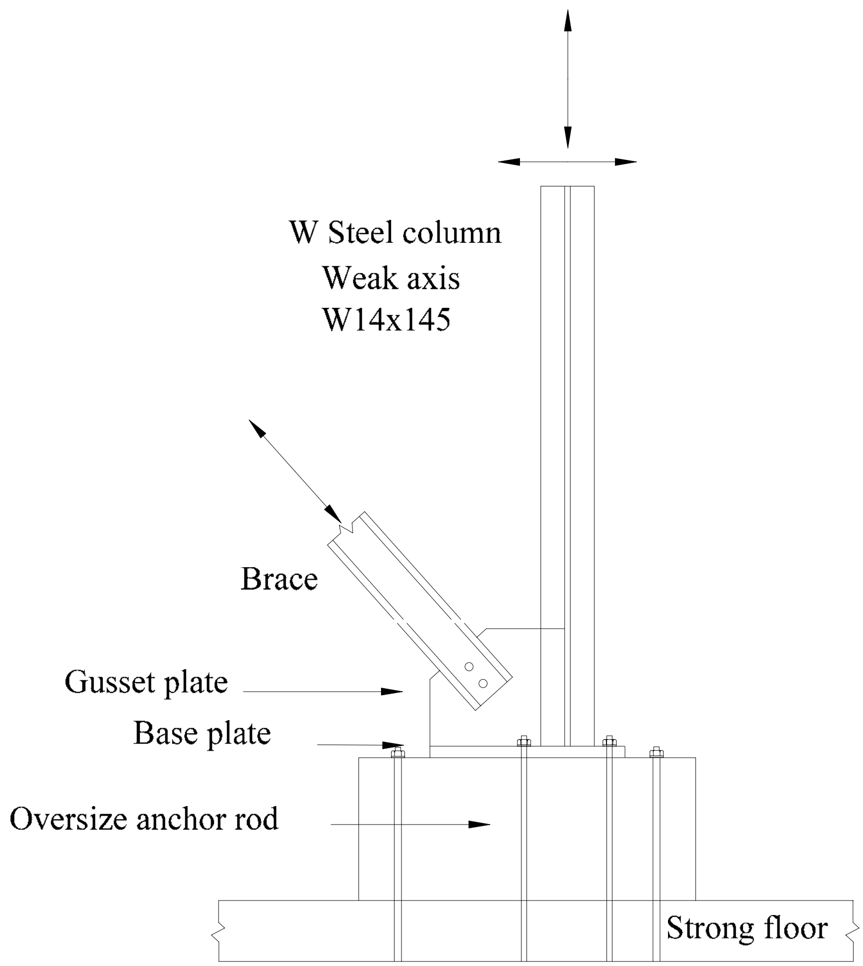


Fig. 5.6. Typical test specimen for braced frame column base connection

5.3.3. Expected Outcome

By conducting the tests detailed in the preceding sections, the research should be framed to yield the following information about exposed column base connections, in approximate order of priority:

- Development of design procedures that consider alternative progressions of damage for the assessment of the strength of the connection, ensuring the occurrence of desired ultimate failure modes. These design procedures and assessments of the likely progression of damage should be provided for both unbraced and braced.
- Assessment of the stiffness and ductility of a column base with exposed base plates in unbraced frames.
- Design recommendations about welding details, stiffening methods, and feasible assembly configurations for connections that will perform in seismic areas.

Besides the expected experimental results mentioned, analytical studies are needed (e.g., theoretical derivations, finite element analyses) in order to propose how to calculate the required moments in a grade beam, knowing that the beam is not free to rotate when, due to the external forces, it is forced to bear against the soil or foundations. Additional tests, intended to confirm these analytical results, could also be conducted, in which this restrained deformation is simulated.

5.4. Final considerations.

The preceding sections have analyzed potential research topics in a general prioritized fashion (i.e., embedded column base behavior analysis is deemed to be a higher priority than the exposed base plate case, and parameters and issues are prioritized within each section). The manner of presentation aims to offer a simplified and clear overview of all the aspects to be included. However, not all the topics of embedded connections should be considered as having precedence over all of those for exposed base plates. In view of that, a final and more detailed outline is presented herein, with the intention of specifying the research path that should be followed. This list is presented based on an assessment of the current state-of-the-art of research on column bases and on prevalent design practices in the U.S. In order of importance, these topics are:

(1) Design procedures and seismic recommendations for deeply- and shallowly-embedded base plate column bases in unbraced frames under monotonic and cyclic load combinations.

(2) Design procedures and seismic recommendations for exposed base plate column bases in braced frames under monotonic and cyclic load combinations.

(3) Design procedures and seismic recommendations for exposed base plate column bases in unbraced frames under monotonic and cyclic load combinations (with particular emphasis on uplift forces).

(4) Progression of damage and feasible failure modes in column bases with deeply-embedded base plates in unbraced frames.

(5) Progression of damage and feasible failure modes in column bases with deeply-embedded base plates in braced frames.

(6) Progression of damage and feasible failure modes in column bases with exposed and shallowly-embedded base plates in unbraced frames.

(7) Progression of damage and feasible failure modes in column bases with exposed and shallowly-embedded base plates in braced frames.

(8) Evaluation of partial fixity in column bases with exposed or shallowly-embedded bases plates in unbraced frames under monotonic and cyclic load combinations.

(9) Evaluation of partial fixity in column bases with deeply-embedded bases plates in unbraced frames under monotonic and cyclic load combinations.

(10) Design procedures and seismic recommendations for deeply-embedded column bases in braced frames under monotonic and cyclic load combinations.

Chapter 6

Conclusions

Extensive column base research has contributed to understanding the significant parameters that govern the varied responses of different types and arrangements of column bases under representative loading conditions. A number of investigators worldwide have made substantial contributions and at the same time have highlighted the nature of the research that is left to be done. As has been highlighted throughout this report, some of the issues are critical and their clarification is urgent; others are intended to refine the prediction of the overall behavior of structure or the assessment of damage progression in column base connections.

Chapter 2 has presented a classification of the different types of column bases. From the exposed base plate type, in which the base plate is one of the most important links in the force transfer mechanism, to the deeply embedded type, in which the base plate plays practically no role, all the range of feasible configurations have been considered, such as:

(1) A column base connection with a thick exposed base plate that acts as a rigid link, transferring the external moments directly to the anchor rods that can be designed to act as a ductile component.

(2) A flexible exposed base plate that dissipates significant energy in bending deformation, thus reducing the forces to which the anchor rods are subject.

(3) A shallowly embedded base plate that improves its strength and stiffness by adding the bearing resistance of the concrete in which the base plate and a portion of the column is embedded.

(4) A deeply embedded base plate in which the strength of the connection is the result of the bearing resistance of the concrete, and its high stiffness is the consequence of the ability of the concrete to fix the column, allowing smaller rotations.

Additionally, classifications of the column bases in terms of the grade of restraint provided, the ability to dissipate energy, the failure mechanisms observed, and the structure or structural element in which the column bases act completed the summary of performance.

Chapter 3 has focused on structural design topics, highlighting the properties that a column base should satisfy (i.e., strength, stiffness, and ductility) as well as the effects, already recognized in many experiments, that the main parameters in the column base design (i.e., size of the base plate, size and layout of anchor rods, material properties, embedment length, steel reinforcement, weld detailing) have on the final response of the connection. In order to broaden the overview and compare with U.S. practice, current design procedures from Japan and Europe have been described, showing how their specifications approach some of the most relevant issues in column base design (e.g., initial stiffness, stress distributions, behavior assumptions).

Chapter 4 has summarized the main concerns related to understanding the behavior and design of the components of the whole spectrum of column bases. It is critical to design these components knowing the progression of damage that will be expected as the loading increases. It is also vital to provide engineers with the tools for designing and accurately evaluating the alternative configurations that are used to emulate the theoretical support conditions assumed in the structural analysis (e.g., deeply embedded column base to reproduce fixed conditions). It is important for seismic design to improve the evaluation of the contribution of the partial restraint offered by the column base to the final response of the building. It is also pertinent to evaluate the feasibility of alternative column base design procedures that may produce equivalent structural responses with more economic and reliable configurations (e.g., rigid exposed base plates with ductile anchor rods).

Lastly, Chapter 5 has provided a prioritized research plan to guide the study of the more critical problems and unresolved issues found in current U.S. seismic practice on column base connections, namely:

(1) The design of embedded column bases.

(2) The additional topics needed to thoroughly evaluate the behavior of column bases with exposed base plates.

(3) The design of grade beams.

(4) The design of braced frame column base connections.

It is hoped that the breakdown analysis and further recommendations carried out in this report may help to clarify that future research in the U.S. needs to take place and that the topics that need to be investigated will generate a better understanding of the behavior of the column base connection, a more accurate assessment of its effect, and a greater confidence in the final behavior of the structure consistent with the assumptions made in the design procedures.

Appendix A: Synopses of Column Base Experimental and Analytical Studies

A.1. Exposed Type Column Base in Unbraced Frames

Salmon, C. G., Schenker, L., and Johnston, B. G. (1957)

Introduction

The authors conducted a theoretical study to investigate moment-rotation characteristics of column anchorages that are commonly found in light industrial buildings. Rectangular and circular base plates were selected for this study.

Theoretical Study

Methods of determining the moment-rotation characteristics of column anchorages were presented. Upper and lower bounds for maximum resisting moment and maximum rotation, respectively, were developed. Horizontal shear effect on a simple anchorage was also investigated and was found to have little effect on the ultimate resisting moment but somewhat more effect on the maximum rotation. Formulation of the complete moment-rotation curve was developed in five stages and the use of the formulae was illustrated in appendixes. In the conclusion, the authors noted that the procedure developed might be a first approximation until experimental data became available.

Voce, G. J. (1958)

Introduction

The author questioned the assumption of a uniform bearing stress distribution when some restraints exist at stanchion bases. It was noted that a moment should exist at the bottom of the column in order to keep the base plate flat and the column shaft vertical. Simplified equations were proposed for the calculation of moment and shear forces for designing the stanchion shaft. An example was also provided to show how the maximum combined stress could be calculated.

Fling R. S. (1970)

Introduction

When load is applied with a wide flange column, the area between the column flanges is often subjected to bending. In this paper, a minimum column base plate thickness for that case was provided on the basis of an analytical study.

Theoretical Study

For lightly loaded columns, the required plate dimensions are approximately equal to or smaller than the overall cross-sectional dimensions of the column. In these cases, the design of the base plate thickness is usually governed by the bending in the region between the column flanges. The author proposed the use of yield line theory and provided an equation that could calculate the minimum thickness of the base plate. He assumed that plate bending was elastic and used a safety factor of 2.0. In a discussion, Gogate (1970) noted that Fling neglected the favorable influence of the anchor rods. In reply, Fling stated it would be difficult to account for the anchor rods due to the variance in actual base plate designs.

Sandhu, B. S. (1973)

Introduction

Based on the procedures given on page 3-95 of the AISC Manual, 7th Edition, the author developed an alignment chart for designing base plates of axially loaded columns. Two examples were provided to illustrate the use of the alignment chart.

Stockwell, F. W. (1975)

Introduction

The author developed design aids for the rapid selection of base plates. Those design aids are included in the AISC Design Guide No. 1 – Column Base Plates. Two concrete compressive strengths (i.e., 3 ksi and 4 ksi) were considered for the design. The author also proposed a design procedure for lightly loaded columns. The method proposed assumed that only the H-shaped portion of the base plate under the column was effective. He noted that the assumed width of the strip was a matter of engineering judgment and should be highly dependent on column size, base plate overhang, bearing stress, etc.

Bird, W. R. (1976)

Introduction

A graphic approach to design was presented in this paper, providing rapid and accurate solutions for the design of steel bearing plates that could be expanded to cover all column sections and loading conditions. Two examples were provided to illustrate the application of the charts to the sizing of the base plates.

DeWolf, J. T. (1978)

Introduction

In this work, both experimental and analytical studies were performed. A simple empirical design approach was developed on the basis of the experimental results. The test results were also compared with the allowable bearing stresses estimated from the AISC Specification.

Experimental Study

This paper presented results of 19 tests of axially loaded base plates. Six sets of tests were conducted using A_c/A_b ratios of 1.0, 2.0, and 4.0. Three thicknesses were chosen for each set. Test No. 2 was a repeat of Test No. 1 with a different concrete strength. Unreinforced concrete cubes were used as pedestals, so additional confinement could be considered when the pedestal was reinforced. The typical failure mode was an inverted cone failure of the concrete under the base plate.

Theoretical Study

In order to predict the average bearing stress at failure, a simple empirical equation was developed. This equation was a function of: (1) concrete strength, (2) ratio of concrete to base plate area, and (3) ratio of the distance between the edge of the loading plate and the plate thickness. The experimental study indicated that the AISC Specification was conservative. The author concluded that a larger safety factor was desirable for the design of base plates than for beams and columns. It was also noted that the proposed method should not be used for lightly loaded W-shape columns because the effects of base plate bending between the flanges are significant for those cases.

Maitra, N. (1978)

Introduction

In this paper, the author provided a graphical aid for the design of base plates subject to moments. A triangular shape was assumed for the bearing stress distribution under the base plate. The design aid is included in the AISC Design Guide No. 1 (DeWolf and Ricker, 1990).

DeWolf, J. T. and Sarisley E. F. (1980)

Introduction

An experimental investigation was performed to review the design practice for steel column base plates subjected to axial loads and moments. Comparisons between the experimental outputs and the estimations from two different design methods (i.e., the working stress method and the ultimate strength method) were made.

Experimental Study

A total of 16 specimens were tested. The variables in the test series were (1) anchor rod size, (2) base plate thickness, and (3) load eccentricity. In all the tests, the ratio of the concrete area to that of the base plate was 2.0. The concrete was not reinforced. A TS 4 x 4 x 0.5 in. tube of ASTM A618 steel, with a yield stress of 50 ksi, was used for the column section. Fillet welds were used for all connections. The anchor rods were cut from standard hot-rolled A36 bar and the nuts were hand-tightened prior to testing.

Typical failure modes were: (1) crushing of the concrete; (2) yielding of the anchor rod; or (3) formation of a plastic hinge in the base plate. When failure was governed by yielding of the anchor rods, yielding occurred within the concrete, not in the threaded part. Three major conclusions were obtained from the study:

(1) When the anchor rod strength is relatively large, the distance between the anchor rod on the tension side and the bearing zone on the compression side is small.

(2) Effect of the concrete confinement should be included in the design.

(3) Increasing the base plate thickness can lead to decrease of the connection capacity due to large bearing stresses under the base plate and consequent premature concrete (grout) crushing.

DeWolf, J. T. (1982)

Introduction

The author discussed studies on column base design practice, focusing on how well design approaches modeled the actual behavior and suggesting areas in need of modification. Axially loaded base plates were discussed first. To begin, the basic design approach was outlined and then the effects of the variables were discussed. These variables included concrete compressive strength, ratio of concrete area to plate area, plate thickness, base plate yield stress, and concrete pedestal depth. For the axial load plus moment cases, the effect of one additional variable (i.e., anchor rod force) was added.

Results indicated that consideration should be given to some additional variables and that the designer should be cognizant of the limitations of the existing design procedures. Based on previous research and discussions within the paper, several conclusions were drawn.

(1) For pedestals with a depth greater than the lesser of the horizontal dimensions, reinforcement should be provided for both, the axially loaded case and the axial load plus moment case. This reinforcement should consist of longitudinal bars and stirrups or ties, and the first stirrup should be placed below the base plate.

(2) The author recommended using the ultimate strength method for column base design, especially for the axial load plus moment case.

(3) For the ultimate strength method, 1.0 could be conservatively used for the ratio of concrete area to plate area.

(4) For the ultimate strength method, the base plate thickness should be designed using the full plastic moment capacity of the plate.

Murray, T. M. (1983)

Introduction

The author conducted a numerical and experimental study to investigate the effective bearing area under the base plate. He also conducted another set of analytical and experimental studies in order to examine the base plate behavior and capacity under the uplift loading.

Computational Study (For Gravity Loading)

An elastic finite element analysis was performed. Springs were used to connect the plate to a rigid foundation, and they were then disconnected when uplift occurred. A separation of approximately 28% of the contact area between the base plate and the foundation was observed.

Experimental Study (For Gravity Loading)

Two gravity load tests were conducted to verify the finite element analysis results. The column and base plate sections were fabricated from A572 Gr.50 steel. Specimen BP1 had a 6 x 8 x 3/8 in. base plate and Specimen BP2 an 8 x 12 x 3/8 in. base plate. Reinforced concrete pedestals were cast and a layer of expansive grout was placed between the plate and concrete to provide a uniform bearing surface. Two ¾ in. diameter A307 anchor rods were cast in the specimens. Reasonable agreement was found between the measured and predicted deflections and separations. Based on the numerical and experimental study, the author proposed that Stockwell's approach should be used. He also defined an effective bearing portion of the base plate under the column.

Theoretical Study (For Uplift Loading)

Based on a yield line analysis, a design approach for base plates subject to uplift was developed. One design example was provided for this case.

Experimental Study (For Uplift Loading)

Tests were conducted on four base plate specimens, each one consisting of two column/base plate sections, separated by a 1 in. grout and bolted with two 1 in. A36 threaded studs. The reference column consisted of a short column section and a 1 in. thick base plate. In all tests, the maximum load applied was approximately equal to the predicted ultimate load from the yield line analysis.

Akiyama, H., Kurosawa, M., Wakuni, N., and Nishimura, I. (1984)

Introduction

A total of 25 exposed-type column base connection specimens were tested under reversal cyclic loads. These specimens consisted of 1-HAP type, 2-HAB type, 3-HBB type, 2-TB type, 2-SB type, 6-HC type, 4-TC type, and 5-ST type specimen configurations. The HAP and HAB series consisted of anchor blocks at the end of anchor rods. The HBB series were attached to the concrete foundation using deformed round anchor rods. The TB series were made of circular tube columns. The SB series were fixed to the concrete foundation with only two anchor rods and assumed as a pinned condition. The HC and TC series were seated on a concrete riser. Lastly, the ST series consisted of nuts (ST-1) or anchor block of 100 x 100 x 26 mm (ST-2).

Also, equations were developed for the design of column base connections under column moments and axial loads. Ultimate strength and deformability of the exposed-type column base connections were examined in various combinations of base plate, anchor rods, and concrete strength. Empirical formulas for strength, stiffness, and ultimate deformability of the column base connection were derived and applicability of these equations was verified by comparison with the experimental outputs. In the conclusions, the authors also proposed two conditions that can prevent undesirable connection failure modes (i.e., failure in the concrete foundation and pull out of the anchor rods).

Picard, A. and Beaulieu, D. (1985)

Introduction

An experimental study was conducted to quantitatively determine the influence of the column axial load on the flexibility factors for bending about the principal axes of column base connections with two or four anchor rods. Comparisons were made between the experimental moment-rotation curves and the derived corresponding theoretical curves.

Experimental Study

The authors tested fifteen specimens to investigate the behavior of column-base plate connections under various loading conditions and to determine the influence of column axial loads on the connection rigidity. Three different column sections (i.e., M, W, and HSS) and two different anchor rod layouts (i.e., 2-rod type and 4-rod type) were selected as main experimental parameters. Two different loading conditions were applied: (1) lateral loads only and (2) axial loads with different eccentricities.

The tests were performed on 1220 mm long column stubs. Specified yield strengths of column wide-flange sections and hollow sections were 300 and 350 MPa, respectively. The dimensions of the base plate tested in this experimental study were 130 x 140 x 11 mm for the M 100 x 19 column section (with 2 anchor rods) and 190 x 300 x 29 mm for both the W 150 x 37 and HSS 152.4 x 152.4 x 12.7 column sections (with 4 anchor rods). Specified yield strength of the steel base plate was 250 MPa. Smooth round bars of 19 mm diameter, threaded at both ends, were used as anchor rods for all specimens. The anchorage was designed to develop the tensile capacity of the anchor rod steel. Cement grout was placed between the base plate and the concrete footing (except for specimen 3F). The grout thickness was approximately 20 mm.

The test results indicated that the axial loads applied to the column significantly increase the rotational stiffness of the connection, and this rotational restraint in the connection is sufficient to affect frame responses. The authors insisted that consideration of the partial rotational restraint offered by the so-called pinned column base connections might lead to a significant decrease in lateral displacements and P-Δ effects in a given structure. The results also showed that the method of analysis used to determine the ultimate moment capacity of the base connection in this study was conservative.

Thambiratnam, D. P. and Paramasivam, P. (1986)

Introduction

Experiments were conducted to study the behavior of base plates under the action of axial loads and moments by eccentric loading on the column. Test results were compared with the predictions from the Working Stress Method.

Experimental Study

In a total of 16 tests, the behavior of 12 distinct specimens was studied in depth. Some of the tests were repeated to check consistency and symmetry of strains with respect to the X-X axis. Columns were made up of a 4 x 4 x 1/2 in. box section. Main variables in the test series were the eccentricity of the loading and the base plate thickness. Three sets of tests were conducted. Three thicknesses (7/8 in., 3/4 in., and 5/8 in.) were chosen for each set. Loads were applied at eccentricities of 1 in., 3 in., 5 in., and 7 in.

Dimensions of the concrete block were 12 x 12 x 11 in. and the concrete blocks had a concrete compressive strength f'_c of 7 ksi. Reinforcement was not used. In order to effectively distribute the bearing force transferred from the base plate, a layer of 3/8 in. grout was placed between the base plate and concrete. Anchor rods that were 17 in. long, 3/4 in. diameter, with both ends threaded, were cast in the concrete. Each anchor rod was hand-tightened prior to the testing. The anchor rod was intentionally over-designed so that yielding of the anchor rod would be avoided.

At the lowest eccentricity, cracking of the concrete controlled the failure, while at other eccentricities the primary failure mode was yielding of the base plate. The test results showed that the safety factor of the specimens that failed by yielding of the base plate, ranged from 1.09 to 1.89.

Except at the lowest eccentricity, the concrete block did not reach its limit state at primary failure of the specimens. It was also observed that the critical section of the base plate was the lateral section under the foot of the column face on the side of the load. On the basis of the results, the authors noted that flexible base plates, loaded at high eccentricities, could fail from yielding of the plate and might not always behave as predicted by contemporary design practice. They also confirmed one of the conclusions from DeWolf and Sarisley (1980): relatively thick base plates behave like rigid plates causing larger bearing stresses and premature grout crushing.

Picard, A. and Beaulieu, D. (1987)

Introduction

An experimental study was conducted to determine the value of the rigidity ratio at the column base (G_L). It was found that the flexural stiffness of an exposed-type column base connection, which was generally considered as pinned condition, had a very beneficial effect on column stability and frame response. To illustrate the advantages of the actual column base restraints in analysis and design a design example was provided.

Experimental Study

In order to investigate the effects on the connection rigidity ratio (G_L) of typical column base connections fixed to the footing by two or four anchor rods, a total of fourteen specimens were tested under combined axial loads and bending moments.

The tests were performed on 2000 mm long columns. Specified yield strength was 300 MPa for column flange sections and steel base plates, and 350 MPa for hollow sections. The specimens were attached to a steel pedestal or reinforced concrete footing by means of two or four anchor rods. 19 mm round bars, threaded at both ends, were used as anchor rods for all the specimens. Cement grout approximately 25 mm thick was used between the base plate and steel pedestal. The axial load was first applied to the column using a vertical hydraulic jack and kept constant during the test. Then a horizontal load was applied at its midpoint. From the moment-rotation curves plotted, it can be seen that the slope of the curves increased with the column axial loads. Consequently the flexibility factor, which is the inverse of the slope, and the rigidity ratio, decreased.

It has been generally admitted that the type of connections described above behave like a hinge and a $G_L = 10.0$ is recommended for the calculation of the column effective length. The test results however indicated that for weak axis buckling, a conservative value of the rigidity ratio would be $G_{LY} = 0.50$ and for strong axis buckling $G_{LX} = 1.50$ for the case of non-sway frames. The effective length of the column could therefore be reduced. A design example provided in this paper showed that the calculated column strength could be increased by up to 30% due to the change in the rigidity ratio.

Sato, K. (1987)

Introduction

A series of experimental and analytical studies were performed to investigate ultimate capacities of several column base connection components and effects of column base hysteresis characteristics on the seismic behavior of superstructures in moment frames. Two preliminary experimental studies were conducted to examine the concrete bearing strength under the base plate and the behavior of tension-induced anchor rods under the selected cyclic loads. Another experimental investigation was also conducted with full-scale column base specimens to verify the results from the two preliminary experimental studies and to evaluate the fixity of the connection. Finally, the behavior of multi-story frame buildings and the effect of partial fixity of the column base were investigated.

Experimental Study

It has been known from past experiments that the bearing stress under the base plate has a rectangular shape when the connection reaches its ultimate state. An experimental study was performed in order to investigate the bearing capacity of concrete. Two variables were selected for this study, distance between the edge of the base plate and the edge of the concrete and bearing width. A total of 12 specimens were tested. Based on experimental observation, an empirical formula relating that distance and the bearing concrete stress was proposed.

The main objective of the second experimental test was to investigate the effects of tension-induced anchor rods on the initial fixity of a column base connection. Eight anchor rod assembly tests were performed. When the tensile force T was applied to the anchor rod at the base plate level, an additional tensile force ΔT was generated. A formula was provided in this study for the calculation of ΔT.

The full-scale column base specimens were tested under combined axial, shear, and bending moments. The concrete pedestal was designed to be adequately reinforced for force transfer even under the ultimate state of the connection. The column was also selected so as to not yield at the ultimate state. The main experimental parameters were base plate dimensions, column axial load, and yield ratio of anchor rod. Based on an evaluation of the connection fixity against rotation, the author proposed a column base rotational stiffness equation. Evaluation formulas for the ultimate moment were also developed for five different bearing stress conditions on the compression side.

Lastly, the author performed a dynamic frame analysis to investigate the effect of column base hysteresis characteristics on the behavior of superstructures in moment frames. For this study, three different frame models with 2, 4, and 6 stories were configured and analyzed loading them with El Centro (NS) and Taft (EW) earthquake data. For the column base fixity against rotation, the proposed analysis model was used. From the results of the analyses, several design considerations were made for the aseismic design of the column base connections. In the conclusion, the author noted that further investigation should be made to solve severe excitation problems in frames with a short fundamental period under specific earthquake waves.

Hon, K. K. and Melchers, R. E. (1988)

Introduction

The authors presented experimental findings for pinned column base connection behavior under axial and bending moment. The results demonstrated the important role of base plate thickness and rod size in moment resistance and stiffness of the connection, and the insignificant role of the grout layer and packing.

Experimental Study

A total of 26 full-size connections were tested to understand the behavior of pinned base plates (with 2-anchor rods) under axial and bending moments and to investigate the main factors which influence that behavior. The test program included three different column sizes (460 UB, 310 UC, and 310 UB), base plate thicknesses of 12 mm, 16 mm, 20 mm, 25 mm, and 30 mm, and rod sizes of M20 and M24. The concrete foundation was reinforced and clamped to the laboratory strong floor.

For relatively thinner base plates, overall connection behavior was ductile due to the formation of yield lines in the base plate. Three different base plate yielding patterns were observed with this experimental study. For thicker base plates, overall connection behavior tended to be governed by the anchor rod characteristics and the failure mode was somewhat brittle. No failure of the column-to-base plate fillet welds was observed under the axial forces and moments applied.

Several major findings in this experimental study were summarized:

(1) Larger base plate planar dimensions led to a slight increase in connection stiffness and some mild increase in moment capacity, due to the longer resisting moment arm under the base plate.

(2) The increase of the anchor rod size led to the increase of the connection stiffness.

(3) An increase in the column size resulted in an increase of the bending stiffness and the moment capacity of the connection, provided that the base plate was the critical element in the resisting mechanism.

(4) When loaded with small moments, the base plate deformation did lead to relatively small prying forces, but as loading continued to increase, the base plate commenced to lift off the grout and thus the prying forces were diminished. At large moments, prying forces did not exist due to separation between the base plate and the grout in the region of the anchor rod line.

Sato, K. and Kamagata, S. (1988)

Introduction

Through his previous experiments, the first author investigated the effects of base plate size, column axial load, and yield ratio of the anchor rod on hysteretic column base connection behavior, finding an inelastic column base hysteretic curve with low energy dissipation capacity until yielding of the anchor rods occurred. Based on these experimental results, some practical design formulae were proposed to evaluate fixity and ultimate strength of the column base connections.

In this paper, the moment-rotation relationship was idealized as a tri-linear curve, with two inflection points defined 1) by the separation (debonding) of the anchor rod and, 2) by the yielding of the threaded part of the anchor rod. For design applications, the authors proposed a simplified formula for evaluating column base fixity. In order to investigate the validity of the proposed formula, a dynamic frame analysis was conducted. Seismic response analyses were performed on three different frame models (2, 4, and 6 stories). The seismic waves used in this analysis were El Centro (NS) and Taft (EW). The results showed that seismic response of the two storey model could be significantly affected by the hysteresis of the column base.

Penserini, P. and Colson, A. (1989)

Introduction

A mathematical model, determining the ultimate limit strength (ULS) of a column base connection, was developed based on the limit analysis method. Comparison with experimental results was made for "fixed" and "pinned" connection cases. The estimated ULS curves are in good agreement with the selected experimental failure points.

Ricker, D. T. (1989)

Introduction

Many practical aspects for the design of column-base plate connections are covered in this paper. The contents form the second part of the AISC Design Guide No. 1.

Thambiratnam, D. P. and Krishnamurthy, N. (1989)

Introduction

A three-dimensional finite element analysis was carried out on exposed-type column base plate connections under column axial loads and moments. A special purpose analysis program, FEABOC (Finite Element Analysis of Bolted Connections), was used for this numerical parametric study. Twelve distinctive specimens were analyzed, and the effect of the base plate thickness and eccentricity of the loads on global connection responses was investigated. This study was focused on strain distributions in the base plate.

Lifting of the base plate is one of the phenomena that can be easily seen from the experimental studies. After the bond between the anchor rod and the concrete is broken, lifting of the base plate will certainly cause localized pressure distributions under the base plate on the compression side. The FEABOC program was able to provide a good simulation of this case. The authors noted that classical analytical methods were unable to bring out these effects.

Ahmed, S. and Kreps, R. R. (1990)

Introduction

The author noted that the new approach suggested in the AISC Manual, 9th Edition, was sometimes overly conservative and even inconsistent. Considering the width to length ratios of usual column sections, the authors suggested a moment coefficient of 0.022 so that the maximum moment in the base plate is 0.022 x f_p x d^2 kip-in./in., where d is the depth of the column.

Krishnamurthy, N. and Thambiratnam, D. P. (1990)

Introduction

The authors studied steel base plates under vertical eccentric loads. Effects of different plate thicknesses and load eccentricities were investigated using a two-dimensional linear elastic finite element analysis tool. Because of the type of analysis, failure was defined as first yield in the connection. Computed results were evaluated with the limited experimental data available. No strong agreement was observed between the computed and measured results, but the authors indicated, however, that the behavior patterns predicted by the finite element tool and those observed in the tests were qualitatively similar.

From the analytical study, the authors noted that the actual behavior of steel base plates was considerably different from the assumed rigid plate behavior implied in the design practice. This paper also concluded that the finite element method could be a powerful tool for the study of column base plates under various loading conditions and for the development of more realistic design methods.

Thornton, W. A. (1990A)

Introduction

Two analytical models for the design of base plate thickness were formulated. The estimations were compared with a method suggested by Ahmed and Kreps (1990) and the method of the AISC Manual, 8th Edition.

Theoretical Study

A design equation (Model 1) for the determination of the base plate thickness was presented. This equation was derived on the basis of a yield line method, assuming the three supported edges (i.e., column flanges and column web along the side of length *d*) as completely fixed, and with a safety factor of 2.0. Another equation (Model 2) was also derived based on a different assumption. The plate was completely fixed to the column web along the side of length *d* but simply supported along the sides of half of the column flange. Based on comparisons with the method of Ahmed and Kreps (1990), and the method of the AISC Manual, 8th Edition, the second model was recommended for use to replace the AISC Manual, 9th Edition, for design of base plates under axial loads.

Thornton, W. A. (1990B)

Introduction

An analytical study was conducted to maximize the benefit of the Murray-Stockwell method (Murray, 1983) for the lightly loaded plate, to define the boundary between lightly and heavily loaded plates, and to combine the two approaches based on different bearing stress distributions (i.e., H-shaped distribution and uniform distribution).

Theoretical Study

The Murray-Stockwell method posed two problems. First, the boundary between lightly and heavily loaded plates was not defined. Second, the Murray-Stockwell method assumes a peak pressure of F_p over an H-shaped region whereas the conventional assumed a uniform pressure over the entire contact area. These two approaches were combined and a new variable λ was newly introduced to limit the lightly loaded base plate.

Robertson, A. P. (1991)

Introduction

Base fixity effects on portal frame behavior were studied experimentally. This paper described the experimental facility and the test procedures and results for the column base fixity studies conducted.

The centroidal line geometry of the tested frame was: span 12 m, eave height 4 m, frame spacing 4 m, roof pitch 10 degrees. Facility overall frame length (seven frames) was 24 m. This frame, profiled steel clad portal building, was intensively instrumented with strain gauges, displacement transducers, and wind pressure sensors. Special column bases that enabled the frames to be connected to the concrete footing pad with varying degrees of rotational restraint were designed and installed.

Three different column base fixities (pinned, intermediate, and fixed) were considered. The portal frame was tested under different static loading arrangements. The experimental results (bending moment distribution in the frame and column displacements) for each of the three base connection test cases were compared with theoretical results for each of three selected rotational restraint conditions. The rotational stiffnesses selected for this study were 150 kNm/rad for pinned bases, 1500 kNm/rad for intermediate bases, and 3300 kNm/rad for the fixed bases.

The study showed that the base fixities provided by typical two and four rod connections are significant, particularly with regard to sway displacements. It also showed that partial base fixity could be adequately modeled by a simple rotational spring.

Astaneh, A., Bergsma, G., and Shen, J. H. (1992)

Introduction

An experimental and analytical study was performed to generate information on actual cyclic behavior and ductility of column base connections and to develop design recommendations to achieve better seismic performance. The main objectives of this study were to generate experimental data on the cyclic behavior of one of the most common types of column bases, to use experimental data to develop more realistic column base design models, and to investigate seismic design recommendations.

Experimental Study

The authors tested six column-base plate connection assemblages consisting of W-shape column sections to study their cyclic behavior under axial loads and cyclic lateral displacements. The columns were W6x25, A36 steel. All base plates were 9 x 12 in. with thicknesses of 1/4 in., 1/2 in., and 3/4 in. Two identical plate specimens for each thickness were tested with different axial loads. Four ASTM A307 anchor rods bent *90°* at the bottom were used to connect the base plate to the grout and concrete foundation. Only two variables in this experimental study were considered: 1) the amount of axial load; and 2) the thickness of the base plate.

The experimental results including the moment-rotation response of the base plates, the behavior of the anchor rods, the effects of column axial load on the cyclic connection behavior, and the connection behavior against shear force were discussed. Based on the six tests carried out under combined column axial and moment loads, the following general observations were made:

(1) Larger moment resistance of base plates developed with larger column axial loads.

(2) The moment-rotation behavior was more stable under lighter axial loads.

(3) Yielding in the base plate could be a good source of energy dissipation for ductile connection behavior.

(4) Tension forces in the anchor rods increased significantly with the increase of the base plate thickness.

Nine limit states in the column base connections were also discussed: 1) plastic hinge formation in the column; 2) yielding failure of the base plate; 3) compression failure of grout in bearing; 4) tension fracture of anchor rods; 5) failure of grout under combined compression and shear; 6) bearing failure of concrete; 7) failure of column base welds; 8) failure of the foundation; and 9) failure of soil or piles supporting the foundation.

Based on the analyses of the test results and the experience with actual performance of base plates during previous earthquakes, several seismic design recommendations were formulated. The authors classified the base plate behavior into one of three categories, i.e., thin, intermediate, and thick base plate. For each category, they suggested a simplified base plate behavior model.

Several conclusions were drawn from the experimental and analytical studies:

(1) In order to ensure that welds connecting the column to the base plate are not the weakest link, they should have the capacity to resist 1.25 times the combined simultaneous effects of axial load, bending, and shear developed in the column or the base plate, whichever is smaller.

(2) Tension fracture in the anchor rods should be avoided.

(3) Use of the ultimate strength design procedure was recommended for the design of column base connections due to inelasticity in the base plates.

(4) The base plates that are usually assumed as "pinned joint" can develop significant moment resistance. Hence, it was recommended to model base plates as semi-rigid connections and not as "pinned" or "fixed" conditions.

Igarashi, S., Kadoya, H., Nakashima, S., and Suzuki, M. (1992)

Introduction

Mechanical behavior of exposed type column base connections were investigated under repeated bending and shearing forces applied to the top of the column with zero axial forces. Four different anchorage methods were tested: specimen No.1) hooked anchor rod with round steel; specimen No. 2) anchorage plate placed at the bottom of the round steel; specimen No. 3) same as in (2), plus 30% pretension in the anchor rod; and specimen No. 4) threaded deformed bar with anchorage plate.

Based on the experimental investigation, the authors noted that the type of anchorage selected can significantly affect the rigidity and durability of the exposed type column base connections when concrete foundations with a riser are used. The experimental results showed that deformed bars (specimen No. 4) were most effective in enhancing the durability of the connection.

Melchers, R. E. (1992)

Introduction

The author presented the results of 10 reversal cyclic load tests on "pinned" column base connections (with two or four anchor rods inside the column flanges). This paper also presented a simple mathematical model for the prediction of the elastic stiffness of the connection. Comparisons were made between the test results and the estimations from the proposed stiffness model.

Experimental Study

10 specimens were tested under cyclic column bending without axial load. All the specimens consisted of 200 UB 25 column stubs and 300 x 200 mm base plates. Effects of three different base plate thicknesses and two different anchor rod sizes were primarily investigated in the experimental program. The test results showed that the connection moment capacity was governed by base plate yielding. Comparison between tests with similar base plate thickness also indicated that an increase of the anchor rod diameter can increase the rotational stiffness of the connection.

Theoretical Study

In order to predict elastic stiffness of the column base connection, a simple mathematical model was developed. Rod extension, base plate deformation, and prying effects were assumed as major contributors to the elastic connection response. The contributions of the extension of the base plate beyond the column flange and the deformation in the column stub between the point of rotation measurement and the base plate were ignored. The proposed model was verified by comparison with the reported test results in this paper as well as with earlier tests done by Hon and Melchers (1988) on larger column sections and thicker base plates.

Sputo, T. (1993)

Introduction

A design procedure for determining base plate thickness under a round pipe column section was developed. Allowable Stress Design criteria were primarily targeted for their application. An example was provided to illustrate the design procedure.

Theoretical Study

The required thickness for a square base plate supporting a round pipe column was derived on the basis of a yield line theory. Three different cases were considered for this study: 1) plate bending within the area enclosed by the column; 2) bending of the base plate outside the column; and 3) lightly loaded base plate. For each case, an equation for the calculation of the base plate thickness was provided.

Targowski, R., Lamblin, D., and Guerlement, G. (1993)

Introduction

An experimental and numerical study was conducted to identify the limit state of an unstiffened base plate connection for various column sections, to investigate associated base plate yield line distribution, and to analyze the unilateral contact between the base plate and the concrete foundation.

Experimental Study

A total of twelve connection assemblages consisting of six different column sections (i.e., square, rectangular, circular, channel, and two different I-sections) and two different base plate thicknesses (i.e., 6 mm and 10 mm) were prepared and tested under pure bending of the column. Each connection assemblage was connected to the concrete foundation using four anchor rods. No grout layer was cast between the base plate and the concrete footing. A rectangular 400 x 300 mm base plate was selected, fillet welded to the end of column stub with a continuous 6 mm weld throat.

When external loading was initiated and the base plate began to bend, some regions of the lower surface of the plate lifted up and lost contact with the foundation. With the load increase, base plate yielding was observed. At the next stage of loading, the continuous bending and lifting up of the base plate caused a change of load distribution in the system. Based on the experimental results, two base plate yield mechanisms were suggested and limit loads for the base plate were theoretically calculated. The authors noted that the kinematical theorem of limit analysis could provide a powerful and simple method for obtaining an upper bound of the limit bending moment applied to the column that produces failure of the base plate.

Finite Element Analysis

A finite element analysis was conducted to investigate the nonlinear behavior of base plates under pure column bending and to predict the contact force distribution between the base plate and the concrete foundation. Only the case of square column sections was studied. For the finite element model, the concrete foundation was assumed as a rigid plate and the anchor rod was simplified as a special nonlinear one-dimensional element. Perfect adhesion was also assumed between the base plate and the rod head. The friction between the base plate and the foundation was neglected.

Based on observation of the stress distribution under the base plate, unexpected transverse deformation of the base plate on the compression side was noted. This analytical study also found non-uniform prying force distribution on the tension side. The authors concluded that to investigate this problem more precisely it was necessary to change the finite element model, especially in the contact region between the base plate and anchor rods.

Chhabra, S. (1994)

Introduction

Design tables were presented to compare the base plate dimensions provided by the AISC Manual, 8th and 9th Editions. The tables were generated only for W14 column sections, A36 steel base plates and a 4.0 ksi concrete strength. A study of these tables showed that, in the lower column size range, the 9th Edition method generally produced lighter base plates than the 8th Edition. The author noted, however, that the 9th Edition led to the same design as the 8th Edition in the case of higher loads.

Melchers, R. E. and Maas, G. (1994)

Introduction

The authors reported results obtained from in situ observations of the behavior of a steel portal frame under winter wind conditions. This portal frame was a symmetric structure of 30.5 m span, 5.13 m eave height, and consisting of 11 frames spaced 6.1 m. The recorded wind (gust) speed was 2 m/s and it was corrected to the standard 10 m gradient height.

The strain gauge readings were converted to moments for each instrumented column and rafter section. As expected, the base plate and rod did not show visually observable deformations under the applied wind load conditions. This revealed that all the deformations remained in the elastic range. The degree of base fixity was estimated by relating the observed change in moment values to the applied loading and then comparing these with the theoretically calculated results. The stiffness of the column base was reasonably well predicted using the algorithm outlined in Appendix A.

The observed base stiffnesses were consistent with the theoretically predicted values. The results obtained confirm that column bases in real portal frames, nominally assumed as "pinned" joints, could have significant rotational stiffness. Since the frame remained elastic during the test, the base stiffness of the selected portal frame could have been modeled as a linear rotational spring in a computer analysis. The experiments and analytical study also indicated that, at low loads, "pinned" base plates essentially act as "fixed" connections due to the presence of the structure's weight.

Wald, F., Simek, I., and Seifert, J. (1994)

Introduction

An experimental program for the study of column base connection behavior under various loading conditions was briefly summarized. Three different sets of experimental studies were performed. In order to study the stiffness and resistance that develops in column base connections, full-scale connection specimens consisting of two or four anchor rods were tested. Component tests were performed with test setup simulating the resisting mechanism on the tension and compression sides to investigate the contribution of each connection element to the global connection response.

Wald, F., Sokol, Z., and Steenhuis, M. (1995)

Introduction

This paper proposed a stiffness column base design model. Based on the component method, it was derived to be compatible with stress design methodology according to Eurocode 3, Annex L, and with the beam-to-column connection stiffness prediction included in Annex J. Calculation of the stiffness coefficient for each connection component was presented, and values that should be considered for the stiffness calculation were tabulated.

The proposed design model was able to calculate a moment-rotation curve for a given column base connection under constant axial forces. In order to verify the accuracy of the proposed prediction model, the analytical estimations were compared with published experimental observations. A parametric study was also conducted to investigate the influence of main design variables on the column base connection behavior under combined column axial loads and bending moments. These variables included embedded length of rods, base plate thickness, concrete quality, and column axial force.

Ermopoulos, J. Ch. and Stamatopoulos, G. N. (1996a)

Introduction

The authors proposed an analytical procedure that could develop M- φ curve diagram of the column base connections under combined axial and moment loads. In addition, a simple formula was developed that was adequately accurate to the M- φ curves obtained by the analytical procedure.

Analytical Study

Based on the level of concrete bearing stresses under the base plate, column-base plate connections were classified into three types and an analytical model for each connection type was developed. The main parameters included in the three analytical models were: (1) the size and thickness of the base plate, 2) the size, length, and location of anchor rods, 3) material properties of the connection elements, 4) the geometry of the concrete foundation, and 5) the amount of axial loads.

The authors developed a generalized analytical procedure, which led to the relation between moment M and rotation φ of the connection for a selected combination of axial load and moment. Keeping the value of axial load N constant and increasing the value of M, additional points of the M-φ are obtained. In order to verify the accuracy of their models, the analytical estimations were compared with the results of the tests conducted by Astaneh et al. (1992).

For easy application of this analytical procedure, a simple nonlinear formula to predict the M-φ curves was proposed. The authors insisted that this formula, which describes a better approximation of the degree of fixity in the column base connection, could be easily introduced in the equilibrium system of any frame.

Ermopoulos, J. Ch. and Stamatopoulos, G. N. (1996b)

Introduction

The authors expanded their previous study (Ermopoulos and Stamatopoulos, 1996a) to cyclic loading cases. M-φ curves for a given cyclic sequence of loading were plotted and the accuracy of the results was verified based on comparisons with existing experimental results.

Analytical Study

An analytical procedure that could show the cyclic nonlinear responses of column base connections was proposed. In order to determine the hysteretic loop of each loading cycle, a formula accounting for the horizontal hysteretic effect was proposed. By using the new formula, a final relation for the determination of the connection behavior under cyclic loading was formulated.

In order to determine the M-φ curve for a given cyclic loading, the M-φ curve for the case of monotonic loading should be calculated first. From this curve, the characteristic quantities, M_o and φ_o, and the curve fitting coefficient, α, are easily obtained. By plugging these values into the final equation, the M-φ curve for the given cyclic loading can be plotted. To verify the accuracy of the results, the estimations from this analysis procedure were compared with existing experimental outputs, including Astaneh et al. (1992). The authors concluded that the proposed equations should be further verified for broader design applications.

Surtees, J. O. and Yeap, S. H. (1996)

Introduction

The authors provided useful design charts for preliminary sizing of members in pitched roof portal frames. They also provided an independent means of checking simple plastic collapse strength derived from computer analysis. Those design aids allowed for the effect of partial column base restraint and enabled the direct determination of the minimum length of eave haunch. Three examples of their use were offered in this paper. However, the computer-independent design charts are applicable only to pinned-base portal frames.

Balut, N. and Moldovan, A. (1997)

Introduction

The authors discussed the contribution of concrete, column base, and anchoring rods on the column base connection behavior. Particularly for the column base, two general factors that could significantly affect the connection deformation capacity in the post-elastic range (i.e. instability and strain hardening effects) were discussed in depth. The discussions were expanded to several desirable aspects of behavior of column base–anchor rod assemblies and concrete–column base–anchor rod assemblies. A computer program named ROMB 96 (Rotation and Moment at the column Base), which was developed based on the above approach, was introduced at the end of this paper and illustrated with two examples.

In the conclusion, the authors noted that the column base connection should be capable of enough post-elastic deformability if a plastic hinge at the column base was expected. Another major conclusion of their study was that a weaker link in the connection should be located at the column base rather than in the anchor rods.

Stamatopoulos, G. N. and Ermopoulos, J. Ch. (1997)

Introduction

This paper presented a procedure for the development of M-P interaction curves. This methodology was developed on the basis of three proposed failure modes according to the level of bearing stresses under the base plate on the compression side. The authors also included a typical procedure for the analysis of a structure consisting of semi-rigid column base connections.

Analytical Study

The authors studied the ultimate behavior of exposed-type column base connections under combined axial loads and moments and proposed three failure modes for the connection. For each failure mode and corresponding bearing stress diagram, unknown design variables were solved and the solutions were generalized. The parameters considered in this study were size and thickness of the plate; size, length and location of the anchor rods; material properties and the amount of column axial load.

A procedure for the determination of M-P interaction curves was also described in this paper. The whole procedure with the three different modes was illustrated in a flowchart form. For various configurations of column bases, M-P curves were obtained. In order to verify the accuracy of the proposed method, the results were compared with DeWolf and Sarisley (1980), Thambiratnam and Paramasivam (1986) and other analytical results.

In conclusion, the authors summarized the typical procedure for the analysis of the structure, including semi-rigid column base connections, and provided a design example to illustrate how a sway frame with a semi-rigid column base could be analyzed.

Wald, F. and Sokol, Z. (1997)

Introduction

A column base pre-design model for convenient design practice was developed and presented in this paper. Highlights of the pre-design model were:

Column depth / base plate thickness (h_c / t) ≤ 25

Base plate thickness / anchor rod diameter (t / d) ≤ 2

Base plate thickness / distance of the anchor rod to the column flange (t / e_a) ≤ 0.7

Initial stiffness of the connection = E z^2 t / 20

This paper focused on a comparison between the pre-design model and the component method for the estimation of the bending stiffness of column base connections. The component method was developed from the Eurocode 3-Annex J stiffness model based on component stiffness prediction of beam-to-column connections. The use was extended to the column base connection design by introduction of the normal force. The authors introduced several connection component stiffnesses, based on the component deformability prediction, and these values were tabulated for different connection response patterns.

In conclusion, it was noted that the prediction of column base stiffness by the component method is a practical method with good accuracy for frame analysis. The authors also concluded that the error of pre-designed stiffnesses with respect to the values calculated by the component method after the final detailing was acceptable.

Akiyama, H., Yamada, S., Takahashi, M., Katsura, D., Kimura, K., and Yahata, S. (1998)

Introduction

Full-scale shaking table tests of exposed-type column bases were carried out to verify major experimental observations from previous static tests and to examine the applicability of column base connections with weak (thin) base plates. Two base plate specimens consisting of different thicknesses (60 mm and 30 mm) were tested. 760 x 760 mm SM 490 base plates were connected to a rigid bottom using 20-SS400 (M33) anchor rods. A relatively strong column was selected so that most plastic yields and deformations could be concentrated on the other components of the connection.

The stronger (thicker) base plate specimen showed slip-type connection responses because its plastic deformation was caused by monotonic tensile deformation of the anchor rods. This characteristic response was also observed in previous static tests on exposed-type column base connections and this dynamic experiment study did not change those observations. The test results of the first specimen proved the applicability of static experiments for the prediction of the dynamic response of exposed type column bases. On the other hand, the hysteresis of the weaker (thinner) base plate specimen showed spindle shape connection responses due to plastic deformation of the base plate under repeated bending. The experimental results clearly showed that the energy absorbing capacity of a column base with a weak (thin) base plate could be greater than the capacity of a weak anchor rod type column base.

Ermopoulos, J. Ch. and Michaltsos, G. T. (1998)

Introduction

A new methodology, leading to an analytical model to describe the non-linear stress distribution under base plates, was proposed in this paper. Both elastic and elasto-plastic behaviors of the connection were covered by this analytical model. Three different connection behavior models (rigid, semi-rigid, and flexible) were also proposed and a computer program was written to calculate unknown design variables (i.e., the maximum compressive stress on the concrete, the width of the compressive area, the tension force in the anchor rods, and the plastified part of the stress distribution diagram).

Analytical Study

The author classified the column base connections according to three different types; rigid (type D1), semi-rigid (type D2), and flexible (type D3). The corresponding stress distribution under the base plate for each particular type had the general form shown in the Astaneh (1992). Bearing stress distribution curves were proposed for the application of equilibrium equations for the design of base plates.

The deformation, d_i, consisting of the depth of the bearing stress distribution, had to be determined in order to establish the necessary equilibrium equations of the elastic system. From equilibrium and compatibility equations, the length of the compressed area under the base plate could be calculated. For the plastic analysis, it was assumed that the maximum compressive stress σ_c is equal to the yield stress of concrete σ_f. The plastic length d_p was calculated with an iterative procedure and the stresses out of the length d_p were multiplied by the magnification factor (σ_n / σ_f), where σ_n is the ideal max σ_c obtained from the elastic analysis.

Based on results of a given design example, the authors concluded that the proposed stress distribution diagram provides a more accurate analysis which is closer to the actual connection behavior, instead of the uniformly distributed bearing stress assumption.

Jaspart, J. P. and Vandegans, D. (1998)

Introduction

A mechanical spring model which could predict the moment-rotation response of column base connections was briefly introduced. To develop this model, the component method described in Annex J of Eurocode 3 was used and extended. The proposed model was then compared with experimental laboratory tests.

Experimental Study

Twelve I-shaped columns column-base plate connections were tested under combined non-cyclic loads (i.e., axial loads plus moments). Two anchor rod layouts (i.e., 2-rod type and 4-rod type) were used for the test specimens, and the effects of three main parameters (i.e., axial load, base plate thickness, and number of anchor rods) were studied. The same loading history was applied to all the tests, with full compressive axial force on the column applied first and later a progressive column bending moment applied until collapse.

The steel profile HE160B, with S355 steel grade, was used as column member. The base plates were welded to the column with 6 mm fillet welds. Two different base plate thicknesses (i.e., 15mm and 30 mm) were tested in this experimental study. The steel grade of the base plate was S235. The difference in grades between column and base plate was intended to force the collapse in the base plates and not in the column. For the anchor rods, M20 10-9 were used. A grout layer was placed between the base plate and the concrete block so as to ensure good contact between them.

Three typical failure modes were observed in this experiment; failure of anchor rods, yielding of the base plate, and crushing of the concrete. The experimental study showed that the bond stresses between anchor rod and concrete foundation could vanish quickly at the beginning of loading. They also found that the contact zone on the compression side changes with the increase of the axial load.

Mechanical Model

Based on their experimental observations and using the component method described in Annex J of Eurocode 3, the authors developed a mechanical model that could estimate the moment-rotation responses of a column base connection. In this model, each component of the column base was represented by means of component springs. Each of these springs was characterized by its own deformability curve. Comparisons were made between the proposed mechanical model and the 12 experimental test results.

Wald, F. and Jaspart, J. (1998)

Introduction

An overview of the recent progress of the ECCS / COST C1 ad-hoc working group for the development of column base connection design methodologies was presented in this paper. Influence of the rotational behavior of column bases on the structural response of non-sway and sway frames was first discussed and a specific classification for the rotational stiffness of the connection (rigid, semi-rigid, and pinned) was suggested. In order to demonstrate the benefit which could result from a refined design approach based on the actual rotational characteristics of the column bases, an industrial portal frame was chosen as an example. Column base connections with two rods and four rods were considered in this example.

The authors listed several components that should be taken into account for the development of a more realistic column base design methodology. Those were the concrete block in compression, base plate bending in the compression zone, base plate bending in the tension zone, anchor rods in tension, anchor rods in shear, and column web and flange in compression. In conclusion, the authors assured that design rules for the characterization of column bases should be available through the activities of an ECCS / COST C1 ad-hoc working group. They also noted that their research effort would be done in accordance with the component method used in Eurocode 3 - revised Annex J for beam-to-column joints and beam splices.

Burda, J. J. and Itani, A. M. (1999)

Introduction

An experimental and analytical study was carried out to investigate the cyclic behavior of column-base plate assemblies and to examine the static and dynamic responses of a steel moment resisting frame with different base plate fixities. Based on these investigations, a design procedure that limits the inelastic activity in the base plate was recommended.

Experimental Study

Six one-half scale column-base plate assemblies were tested under constant gravity load and lateral cyclic column deformations. A W8x48 profile was chosen to represent the one-half scale model of the selected prototype column. Two base plate sizes, 14.5 x 14.5 in. and 19.5 x 19.5 in, and three different base plate thicknesses, 0.75 in., 1 in., and 1.25 in., were selected for each base plate. The thickness of the base plate was determined by utilizing the proposed limit state procedure to ensure that the plastic moment capacity of the base plate is less than that of the column. ASTM A193 Gr. B7 threaded rods with a diameter of 1-1/2 in. and yield stress of 105 ksi were used to fix the base plate to the concrete footing.

The test results showed that the column-base plate assemblies were able to dissipate significant amounts of energy by base plate bending. Thinner base plates showed more ductile behavior than thicker base plates. In addition, these tests showed the vulnerability of the welds between the column and the base plate. The primary failure mode of the tested base plates was fracture of the weld between the column and base plate.

Analytical Study

In order to understand the stress distribution profile beneath the base plate, nonlinear finite element analyses were conducted. Six finite element models were created to replicate each of the column-base plate assemblies used in the experimental study. From this study it was observed that the compressive stress distribution concentrated underneath the column flange and varied significantly depending on the base plate thickness.

Drain-3DX program was also utilized to investigate the nonlinear static and linear dynamic responses of a four-story building. For the dynamic time history analysis, three different earthquake records were used: El Centro (1940), Loma Prieta (1989), and Northridge (1994). A total of five moment-resisting frame models were developed, two with fixed and pinned base restraints and three with realistic rotational spring restraints at the base. It was shown that the column base fixity has a significant effect on the design of building frames.

At the end of this paper, a design procedure was proposed for steel base plates subject to both axial load and bending moment. Through experimental observation and analytical studies, the applicability of this design procedure was verified. An example of a base plate design using the proposed guidelines was also presented, and the newly designed connection geometries were compared with results from other existing design procedures.

Drake, R. and Elkin, S. (1999)

Introduction

The authors suggested a design procedure that adopted an equivalent rectangular bearing stress block, instead of a triangular shape, for the application of the LRFD approach. Two design examples were provided to illustrate the design procedures.

Theoretical Study

This paper presented a design approach that uses the factored load directly in a method consistent with the equations of static equilibrium and the 1993 AISC LRFD Specification. Four different loading cases were considered for the design of base plate thickness. Those loading cases were axial load-no moment-no uplift (Case A), small moment without uplift (Case B), maximum moment without uplift (Case C), and moment with uplift (Case D). General solutions were provided for each case. Two limit states were considered for the design of the base plate, i.e., concrete bearing limit state and base plate flexural yielding limit state. In addition, yield line theory was used for the analysis of bending in the base plate in the area between the column flanges.

Fahmy, M. (1999)

Introduction

An analytical and experimental study was conducted to investigate the force flow in the column base connection under combined moment, shear and axial force, to identify the failure modes of a column base connection, and to develop seismic design recommendations for column base connections bending about their strong axis. These studies were focused on typical exposed column base connections. Both analytical and experimental research phases were extended into parametric studies with various combinations of independent parameters. These parameters were base plate, anchor rods, axial load magnitude, column size, and concrete foundation properties.

Three conceptually different connection mechanisms were defined. The first mechanism was characterized by a plastic hinge formation in the column (weak column/strong connection). The second mechanism was distinguished by simultaneous yielding of the column and one or more elements of the connection (balanced mechanism). The third mechanism was distinguished by inelastic deformation of one or more elements: anchor rods, concrete foundation, and/or base plate (strong column/ weak connection). A connection strength ratio Γ was introduced in this dissertation to help designers define the desired failure mode of the column base connection.

Analytical Study

The analytical study was divided into three parts: (1) development of a finite element model; (2) investigation of the global and local mechanical characteristics of the connection; and (3) steel moment-resisting frame analysis. Especially for the frame analysis, a typical three-story four-bay steel moment-resisting frame was designed according to the U.S. building codes applicable in Los Angeles, California. Push-over analysis was used to compare the strength and stiffness of different building models as well as to study plastic hinge sequence and deformation mechanisms of the building. A series of time-history analyses were also carried out to evaluate the response of buildings with respect to the maximum values of the inter-story drifts. Semi-rigid column base connections were used to evaluate the effects of semi-rigidity on nonlinear static and dynamic response of steel moment-resisting frames.

Based on intensive nonlinear frame analysis, the following conclusions were obtained:

(1) Use of realistic semi-rigid column base models could reduce the stiffness of the frame by up to 10 % compared to the fixed support model.

(2) The frames with realistic semi-rigid column bases satisfied the 1994 UBC drift requirements.

(3) The frames responded predominantly in their first vibration mode.

Experimental Study

Three exposed-type column base specimens were tested under cyclic lateral column displacements. The specimens were designed and detailed for a hypothetical steel frame building with W10x77 columns and a story height of 14 ft. These three specimens consisted of 20x20x2 ¾ in. A36 base plates. The base plate of the first specimen was anchored using six 1 ¼ in. ASTM A354 Gr. BD anchor rods, while the other two specimens were anchored using four 2.0 in. anchor rods. The only difference between the second and third specimens was the type of weld metal used to connect the column to the base plate (i.e. E70T-7 (NR311) and E70TG-K2 (NR311Ni)).

The experimental results showed that the weak column / strong connection mechanism can maximize the post-yield deformation and can be considered as desirable connection behavior under earthquake excitations. The weld metal was shown to have a great effect on the connection behavior.

Kontoleon, M. J., Mistakidis, E. S., Baniotopoulos, C. C., and Panagiotopoulos, P. D (1999)

Introduction

A numerical parametric investigation was conducted using a two-dimensional finite element plane stress model developed in this study. This model contained all essential features that could characterize the base plate separation problems under large moment loads. Material yielding, contact interface slip, and interface interaction were taken into account in this model. The static behavior of the discrete model and the iterative scheme for the solution of the problem were also detailed in this paper.

The connection studied in the numerical analysis consisted of an RHS 120/200/10 steel column connected to a concrete block through a steel plate and six M20-5.6 anchor rods. Three different base plate thicknesses (20, 25, and 30 mm) and six different column axial loads (0, 100, 200, 300, 400, and 500 kN) were considered as critical parameters. At each increment, a lateral column displacement of 2 mm was applied to the connection. From the numerical parametric study, the authors found that the stiffness (thickness) of the base plate was a significant parameter, affecting the development of prying action at the active contact areas of the base plate on its tension side.

Adany, S., Calado, L., and Dunai, L. (2000)

Introduction

An experimental study was conducted to investigate the cyclic behavior of end-plate type connections and their failure modes. The general behavior of four major column base components was discussed and four different modes of connection failure were defined. The test specimens were designed to achieve the defined failure modes. Based on the experimental moment-rotation relationship, the cyclic characteristics of each specimen were investigated, and the cyclic performance of each was analyzed.

Experimental Study

Five end-plate type joint specimens were tested under lateral cyclic loads. These specimens were mounted on an H-shaped steel base element, simulating the concrete foundation, and fixed with short anchor rods. Due to deterioration of the nut thread in one of the five specimens (CB1) this test was repeated with the rods and nuts replaced.

A practically rigid base made of an H-shaped column section with four anchor rods was selected for this study. The main geometrical dimensions of the specimens were tabulated. In order to connect the column to the base plate, butt welds were used to minimize the risk of weld failure during the test.

Through the experimental study, the moment-rotation responses of the specimens were investigated and the effects of rod elongation and slip between the base plate and base element on cyclic connection behavior were discussed. Furthermore, four characteristic joint parameters (full ductility ratio, resistance ratio, rigidity ratio, and absorbed energy ratio) were calculated and the results examined.

From the experimental observation, the authors noted that the cracks in the weld between the column section and the base plate could significantly degrade the moment resistance and energy absorption capacity of the connection. Rod pre-tensioning was noted to have had no significant effect on the cyclic connection behavior. The authors concluded that the most advantageous connection performance could be obtained if the deformations were concentrated in the column section, forming a plastic hinge.

Adany, S. and Dunai, L. (2000)

Introduction

Two monotonic loading analytical studies were completed on steel-to-steel and steel-to-concrete T-stubs in parallel with experimental testing. In order to determine the stiffness of different T-stubs with snug-tightened and pre-tensioned anchor rods, an experimental study was performed. A total of 16 T-stub specimens were tested in a standard arrangement. For the analytical study, only six specimens were numerically modeled and the analyses were done for snug-tightened cases. The axial load-displacement relationships were calculated for each case. A good agreement between the experimental and numerical stiffness values was observed within the linear axial load-displacement range. In the non-linear range, however, the difference was bigger between the experimental and numerical secant stiffness values. A comparison was also made between the experimental and numerical analyses on base plate T-stubs in tension force. The measured and calculated curves showed very similar responses even though a noticeable difference was found on the initial part of the curves.

The numerical analysis model was then applied for a cyclic parametric study of T-stubs with different anchor rod / base plate stiffness ratios. The main purpose of this parametric study was to examine the rod-end plate interaction and its effect on the cyclic behavior of T-stubs. Four specimens (TS1, TS2, TS3, and TS4) were prepared and numerically analyzed. TS1 corresponded to the case of strong anchor rods that remained elastic during the whole applied loading history. TS4 consisted of a stiff plate that remained elastic under the loads. Based on the analysis results, two possible T-stub behaviors (strong rod and weak rod cases) were further discussed.

From the above monotonic and cyclic study on T-stubs, the authors concluded that application of the proposed numerical model could be extended to the analyses of complex end-plate type joints under various loading conditions.

Kontoleon, M. J. and Baniotopoulos, C. C. (2000)

Introduction

This paper presented an iterative numerical method for solving frictional contact between a loaded elastic body and a rigid obstacle. The method developed was based on the theoretical results of non-smooth mechanics and intended to effectively simulate the frictional unilateral contact problem that arises on a steel base plate connection. The proposed method was illustrated by means of a numerical example of a two-dimensional finite element model of a column base connection.

Li, T., Sakai, J., and Matsui, C. (2000)

Introduction

In order to better understand the behavior of steel-concrete composite column bases under combined axial and cyclic lateral loads and to provide suitable formulas for the evaluation of flexural strength and rotational stiffness of the connections, an experimental study was conducted. The experimental responses were compared with three different flexural strength models developed from previous Japanese column base researches.

Experimental study

Seven specimens consisting of different column types (concrete filled steel tube, empty steel tube, rectangular section, and circular section) were tested. Test setup was composed of concrete filled steel tube column or steel tube column, reinforced footing beam, high base unit of cast steel designed by Hitachi Metal Corporation, and 4 high strength anchor rods. The test specimens were subjected to cyclic lateral loads at the column top under a constant axial load. The yield flexural strength of the column base was defined as the flexural moment when the threaded portions of the tensile anchor rods first reached their yield stress. The ultimate flexural strength of the column base was defined as the flexural moment when the non-threaded portions of the tensile anchor rods first reached their yield stress.

From the experimental results, it was confirmed that the rotational stiffness and flexural strength of the concrete filled tube column base calculated according to the formulas given by High Base Manual (Hitachi Metal Corporation, 1986) coincided with the test results in the case of lower column axial loads. For the specimens with higher axial loads, however, the formulas underestimated their rotation stiffness and flexural strength. The comparison also indicated that several modifications should be taken into consideration in the column base design formulas for the application to the concrete filled tube column bases, especially for the column bases under high axial loads.

Several conclusions from this experimental study are summarized below:

(1) The ultimate flexural strength of specimens under high axial force could be almost twice of that of specimens under lower axial force.

(2) The different column types (concrete filled steel tube, empty steel tube, rectangular section, or circular section) tested in this experimental study did not influence the behavior of the composite column bases.

(3) The rotational stiffness and ultimate strength of the specimens increased with higher axial loads or stronger anchor rods.

(4) The rotational stiffness of the composite column bases can be well estimated by the Equation 2 proposed by Kato et al. (1984).

Wald, F., Bouguin, V., Sokol, Z., and Muzeau, J. (2000)

Introduction

The application of the component method to the design of column bases supporting RHS columns was presented in this paper. For this application, several possible failure modes of T-stubs were explained and the effective length of T-stubs in tension was analytically derived. This analytical model was verified through the comparison with a finite element simulation. Other possible models for connection resistance and rotational stiffness under combined column axial loads and bending moments were also presented in this paper.

In earlier work, one of the authors proposed a column base design model in his thesis and this model was compared with the published moment-rotation experimental outputs. The results exhibited a good agreement of the prediction model to the existing tests. The authors concluded that the Eurocode 3 procedure based on Annex J and Annex L procedures could be used to determine the resistance and stiffness of column bases supporting RHS columns.

Lee, D. and Goel, S. C. (2001)

Introduction

An analytical and experimental study was conducted to evaluate the Drake and Elkin's design method (the D&E method, 1999) and to develop a more reliable design method, if available for the weak axis bending case. Two typical exposed-type column-base plate connections were selected to evaluate the D&E method numerically and experimentally.

In this report, a new column-base plate connection design method was also developed based on a proposed new design force profile and the concept of relative strength ratios among the connection elements. Also, two limit states of column-base plate connections were defined to prevent undesirable connection failures and to maximize the cyclic connection ductility.

Numerical Parametric Study

A parametric study was conducted using the D&E method for the design of exposed-type column-base plate connections bending about the weak axis, to investigate the effects of the relative strength ratio among the connection elements (i.e., column, base plate, and anchor rods) on the connection behavior under large column lateral displacements. For this numerical study, a finite element analysis model that could effectively simulate force transfer at major contact interfaces in the connection was developed. A total of 43 three-dimensional finite element meshes with different base plate thicknesses, anchor rod sizes (stiffnesses), and grout compressive strengths were configured and analyzed. The study revealed several possible limitations in the basic assumptions of the D&E method and the subsequent design calculations, and concurrently pointed to several important design considerations needed to develop rational and reliable design methods.

Experimental Study

Four exposed-type column-base plate connection sub-assemblages (two for 6-rod connection and two for 4-rod connection) were tested under the SAC Phase II loading history in the direction of the column weak axis. W12x96 Gr. 50 columns were connected to 20x20x2.25 in. A36 base plate using PJP groove welds, and fixed to the foundation with two different anchor rod sizes.

The global cyclic performance of each test specimen and the behavior of major connection elements under large column lateral displacements were analyzed. The effects of different numbers of anchor rods, different relative strength between base plate and anchor rods, and different filler metal and welding detail on the cyclic ductility of the connection were also examined. Only one of the four specimens completed the entire loading history without significant strength degradation and formed a plastic hinge at the bottom of the column. The other three specimens showed a limited ductility in the connection.

New Design Method and Two Limit States

A new design force profile that more realistically describes the actual force distribution in the column-base plate connection was developed on the basis of the numerical parametric study and experimental observation. With this profile and the concept of relative strength ratios among the connection elements, a new design methodology was proposed.

In order to prevent undesirable connection failures and to maximize the cyclic connection ductility, two limit states of the column-base plate connection were also defined in this report. The first limit state was defined on the tension side based on the local stresses around the anchor rods, and the second on the compression side based on the proposed effective bearing areas on the grout.

In the proposed design method the use of flexible base plates is considered desirable. However, since thinner base plates might cause increase of local stress concentration in the anchor rods and could result in undesirable anchor rod failure, a minimum thickness of the base plate is provided to be determined with two different approaches (i.e., numerical and theoretical).

The second limit state of the connection is defined on the compression side in order to provide the required minimum compressive strength of the grout. Based on the numerical analysis results and the experimental observations, effective areas for the bearing forces were proposed to calculate the nominal bearing strength of the grout.

Midorikawa, M., Azuhata, T., Ishihara, T., and Wada, A. (2001)

Introduction

When weak base plates yield during a strong earthquake, a rocking vibration can be caused in structural frames. Nonlinear time history analyses of the rocking systems with weak base plates were examined in this paper based on comparison with the analysis results of the simple rocking systems and the fixed base systems. One bay, five-story reduced-scale frame models were configured and analyzed. The earthquake motions used for the analyses were 1940 El Centro NS and 1995 Kobe NS.

From the analysis, it was observed that story shear forces of the base plate yielding systems were reduced as much as those of the simple rocking systems. In addition, the roof displacement and axial force of the first story column were less than those of the simple rocking system. The authors concluded that the rocking system with weak base plates could be effective to reduce seismic responses of the building.

Miyasaka, H., Arai, S., Uchiyama, M., Yamada, T., and Hashimoto, A. (2001)

Introduction

In order to investigate the behavior of anchor rods and base plates in exposed-type column base connections, an experimental study was conducted. Tensile test of four SS490 anchor rods with different diameters was performed first to examine the nonlinear behavior after yielding.

For the study of the base plate behavior and the interaction between the base plate and anchor rods, 4-point bending tests were executed using full-scale mirror specimens. A total of 8 specimens were configured and two specimens tested at the same time under column bending without axial loads. Main experimental parameters were anchor rod location, anchor rod length, and base plate thickness. All columns were made of SN490B 400 x 400 x 32 tube section and five different thicknesses (20, 30, 40, 50, 60 mm) of the SN490B base plate were selected for this study.

An effort was made to relate the deflection of the base plates to the tensile force of the anchor rods under large column moments. Based on the experimental observations, the authors proposed analytical equations which could predict ultimate strength of the exposed-type column base connection and its rotational rigidity. The authors showed that the relative dimension (size) ratio between the base plate and the anchor rod is one of the most important factors affecting the nonlinear connection behavior. It was also confirmed that the exact evaluation of the anchor rod tensile characteristics could be a key feature for estimating more accurately the connection behavior above the elastic range.

Somiya, Y., Fukuchi, Y., and Chin, B. (2002)

Introduction

This paper presented an experimental observation on exposed-type column base connections under column axial and horizontal forces. The connection detail included a tube column, a square base plate, and four anchor rods. Two different set of tests were performed. The main experimental parameter of the first test group considered four different initial column axial forces. A total of five specimens were tested up to 6/100 rad. lateral cyclic deformation of the column tip. Another seven specimens were prepared and tested within the second test group. The main experimental parameters of the second set were the rate of change of the column axial force, the column tube thickness, and the base plate thickness.

In this study, the authors focused on non-linear behavior and ultimate strength of the exposed-type column base connections with variable column axial forces. From experimental observation, the significance of the axial force variation and the un-symmetric connection global responses were noted. The authors also proposed several simplified equations to estimate the ultimate strength of the connection and its rotational stiffness.

Tamai, H. (2003)

Introduction

The author proposed a one-dimensional (cantilever) numerical analysis model that could estimate resisting capacity of exposed-type column bases under bending moments and variable column axial forces. Applicability and availability of the numerical model was confirmed through cyclic loading tests on exposed-type column base specimens. Influence of variable axial force on the rotational stiffness and ultimate moment capacity of the column base were also investigated.

Comparisons between the analytical estimation and the experimental outputs yielded the following several conclusions:

(1) The exposed-type column base tested showed approximately 50% of the elastic rotational stiffness of the fixed column base.

(2) The variation of the column axial force can significantly affect the rotational stiffness and the bending moment capacity of the exposed-type column bases.

(3) The increase of the column tensile axial force resulted in concentration of plastic deformation in anchor rods and the deterioration of the bending moment capacity in the connection.

(4) The proposed analytical method satisfactorily simulated the hysteresis characteristics of the slip-type connection that resulted from the variation of the rotational stiffness and bending moment capacity. This variation results from the change of the column axial loads and from the plastic deformation of the anchor rod.

Cowie, K., Hyland, C., and Mago, N. (2004)

Introduction

An analytical investigation was carried out in order to develop a design procedure to calculate the bearing stress on the concrete under an exposed column base plate. The design procedure, named "Lapping Strip Method" by the authors, is based on the T-stub model used in the annex L, Eurocode 3 (1993). According to a FEA model developed by the authors, there are two regions in an H-section (i.e. where the web and the flanges meet), where a higher concentration of bearing stresses is observed. The rigid area concept and the T-stub method is then modified to account for these increased stresses in those regions. The flange and the web are considered independently and then the bearing pressures are superimposed in the lapping zones to produce the required bearing capacity which is set equal to the concrete bearing capacity. The effective cantilever length "c" of the equivalent rigid area (see Eurocode 3 design procedure in Section 3.4.1. and Fig. 3.2. for notations) is defined using the elastic capacity of the base plate. Formulations are provided to show how the rigid bearing areas are calculated for H-sections, channel sections and rectangular and circular hollow sections. Finally, the Finite Element Analysis results are provided.

A.2 Embedded-Type Column Base in Unbraced Frames

Nakashima, S. and Igarashi, S. (1986)

Introduction

In order to examine the bearing mechanism in embedded-type column base connections and to investigate the effects of various reinforcing methods on the connection strength, stiffness, and hysteretic characteristics, a total of 93 embedded-type column base specimens were tested. All the specimens consisted of steel tube section with or without concrete filling. Only column shear forces and moments developed from the cyclic column lateral displacements were applied in this experimental study.

Three main variables were selected for the study of the bearing mechanism and these were (1) embedment length, (2) column width to thickness ratio, and (3) concrete filling. For the study of the effects of reinforcing methods, the main variables were: (4) size and thickness of the base plate, (5) spacing between the anchor rods, (6) hoops, (7) ribs, and (8) welded reinforcements.

The authors compared force-displacement (rotation) responses of the specimens and the major observations were summarized as follows:

(1) The load-displacement curves showed behavior that exhibited slip of the column base relative to the concrete when the embedment length was short.

(2) The presence of anchor rods or base plates had little effect on the load-displacement curves, when the embedment length was twice the tubular depth.

(3) The thickness of the base plate and the reinforcing hoop did not significantly affect the cyclic connection responses, while the concrete strength mildly changed the hysteretic characteristics in the connection.

(4) The presence of welded reinforcements, welded rib or concrete filling slightly increased the stiffness of the connection, with more stable softening responses of the hysteresis loops compared to the loop without any reinforcements.

(5) In the case of unreinforced details, the column embedment length needed for the formation of a plastic hinge in the steel column at the maximum load was influenced by the column width to thickness ratio, the existence of concrete filling, the yield strength of the column, and the compressive strength of the concrete.

Suzuki, T. and Nakashima, S. (1986)

Introduction

Embedded-type column base connections consisting of H-shaped column cross sections (H-125 x 125 x 6.5 x 9) were tested under cyclic horizontal loading. For this experimental study, a total of 38 column base specimens were encased inside of the concrete riser with various reinforcing details. Two different test series were carried out. "A" series tests (18 specimens) included both the 2 rod and 4 rod connections. The main experimental parameters for the "A" series were height and section of the main reinforcement in the concrete riser and steel ratio of the hoop. The study focused on force transfer and deformation characteristics of the main reinforcement under large connection deformations and development of an equation for the connection ultimate strength. "B" series tests included 2 rod connections only. The main experimental parameters for the "B" series were height of the main reinforcement, steel ratio of the hoop, use of stud dowels, reinforcing details around the top of the concrete riser, and section area of the concrete riser. This study focused on concrete shear failure and the effects of shear reinforcement.

In order to estimate the ultimate moment and shear capacities of embedded column base connections under column horizontal forces, several equations were derived and presented in this paper. Three basic assumptions were used for the development of these equations:

(1) Bond stress between the steel column and concrete was ignorable.

(2) Bearing stress under the base plate on the compression side is ignorable.

(3) Base plates have enough strength to resist the forces transferred from the other connection elements.

The experimental results were compared with the estimations from the equations introduced in this paper. From the direct comparison, it was found that the estimated values underestimated the experimental results and the experimental to estimated ratios averaged 1.3 for the 2-rod connections and 1.7 for the 4-rod connections. Other general conclusions from this experimental study were:

(1) A significant strength degradation of the connection can develop after yielding in the shear reinforcement (hoop).

(2) The elastic stiffness of the connection could be highly affected by the height of the main reinforcement and the section area of the concrete riser, but was less affected by the steel ratio of the main reinforcement and the shear reinforcement and reinforcing details selected for this experimental study.

(3) Reinforcement around the top of the concrete riser increased the shear resistance in the connection and thus resulted in increase of the ultimate strength and plastic deformation capacity of the connection.

Nakashima, S. and Igarashi, S. (1987)

Introduction

In order to study the behavior of steel tubular exterior columns embedded in a steel reinforced footing a total of 29 embedded-type column base specimens were tested. The investigation concentrated on the effects of the embedment length, end distance (i.e., the distance on the unsupported side) and various reinforcing methods on the connection strength, stiffness, and hysteretic characteristics. The specimens were subjected to axial and shear forces and moments developed from the cyclic column lateral displacements.

Two main variables were selected for the study of the resistance mechanism: (1) embedment length and (2) end distance. For the study of the effects of reinforcing methods, the main variables were: (1) ring anchor bars (2) welded anchor bars and ribs for wall reinforcement, and (3) welded shear studs.

The authors compared force-displacement (rotation) responses of the specimens and the major observations were summarized as follows:

(1) The load-displacement curves showed slip type behavior when the embedment length was short. Additionally, for shorter embedment lengths, when the load was applied toward the unsupported end, lower connection strength was obtained.

(2) For end distances greater than 2.00 times the depth of the column D, the hysteretic behavior of the column base was similar for both directions of the applied lateral load. Consequently, adequate reinforcement is needed for end distances lesser than 1.00 D.

(3) The presence of welded reinforcements, welded rib, or shear studs provided the strength needed for the connection to show similar hysteretic behavior for both directions of the applied lateral load.

Nakashima, S. (1992)

Introduction

Two different sets of experimental studies were conducted to investigate the effects of column base encasement on the cyclic connection performance. The hysteresis loop of each specimen was studied and an expression for estimating the ultimate strength of the encased column base connections was provided. Empirical values and calculated values were compared. The ratio of the empirical value to the calculated value was approximately 1.2, indicating that the empirical value was on the conservative side.

Experimental Study

The first set of the experimental study tested a total of ten column base specimens. Four different column base connection types (pinned, fixed, exposed, and encased) were selected for the study, but this paper mainly discussed mechanical properties of the encased column bases. A pair of specimens formed a 1-story 1-span rigid frame approximately ¼ scale. Dimensions and material properties of the connection elements were tabulated, including tubular column and rectangular base plates.

The hysteretic performance, stress distribution, and height of the column inflection point of a 1-story 1-span rigid frame under lateral loading were investigated and the results were discussed. From the experimental study, the author concluded that the column base encasement into the reinforced concrete was very effective in reinforcing or repairing the connections damaged due to the application of lateral forces. He also found that the height of the inflection point decreases as the horizontal force increases, and this could be affected by the height and the section of the encasement. In addition, it was noted that the mechanical properties of the column base connection could be changed depending on the type of reinforcement adopted.

A second set of encased column bases were tested under lateral cyclic forces. Test results of only fourteen representative specimens out of the total thirty were presented in this paper. Main parameters investigated in this study were presence/absence of anchor rods, amount of steel reinforcement for shear reinforcement, height and section of the encasement, presence/absence of main reinforcing hooks, and column axial force/yield load ratio. Prior to the application of the horizontal force, axial pressures corresponding to 0, 1/6, 1/3 and 2/3 times the yield load of the column section were applied.

From the second experimental study, the author concluded that if the height of the encasement is approximately 3 times the outside dimension of the steel tube, its rigidity would exceed that of the fixed column base case. He also noted that vertical reinforcing and shear reinforcing steel would not affect the initial rigidity of the connection but could affect the maximum strength and the strength beyond the expected maximum strength.

Nakashima, S. (1996)

Introduction

Shallowly embedded column base connections were studied in this paper. Base plate sizes, anchor rod layouts, and methods of reinforcing the embedded portions of the column bases were the experimental parameters. This study focused on investigating how the mechanical properties of the column bases could be influenced by those parameters.

Experimental Study

A total of 20 shallowly embedded and 2 exposed column base specimens, scaled to 1/3 – 1/4 of the full size, were tested under lateral cyclic loads with no column axial loads. Two different embedment depth were considered, L/D = 1.0 and 1.5, where L is the embedment depth and D is the side dimension of the steel tube. Six different anchor rod layouts and six different base plate dimensions were selected as main experimental parameters. All columns were made of a cold-formed square steel tube of 100 x 100 x 6 mm. The author included detailed layouts of anchor rods, base plates, and reinforcements.

From the experiments, several observations were made:

(1) The hysteretic loops for the exposed column base specimens were characterized by slips while those for the embedded column bases had bulges near the zero displacement.

(2) The exposed column bases were superior to the embedded column bases in connection rigidity but the embedded column bases showed a higher maximum strength.

(3) The mechanical properties of embedded column bases with L/D = 1.0 were substantially affected by the bottom end detailing of steel columns.

(4) Direct anchorage of steel column bases was found to be as effective as anchorage by means of anchor rods

(5) The presence of base plates and anchor rods significantly affected the strength and ductility of the shallowly embedded column bases.

(6) Anchor bars directly connected to the steel columns were highly effective for both connection performance and maximum strength, whereas hoops and slab reinforcing bars were less effective.

Based on general experimental observations, the author concluded that by shallowly embedding the column bases in concrete footing, enough rigidity, strength, and ductility in the connection can be achieved with proper joint detailing. He also noted that the dimension of the embedded base plates can be reduced compared to the exposed column bases.

Kallolil, J. J., Chakrabarti, S. K., and Mishra, R. C. (1998)

Introduction

An experimental study was performed to investigate the behavior of a shallowly embedded base plate-anchor assembly under several typical loading conditions. Comparisons were made between the experimental results and the estimations from the concrete beam analogy method for the design of the shallowly embedded base plates.

Experimental Study

The authors tested three full-scale specimens under eccentric vertical and horizontal loads. This experiment primarily observed the strains in anchor and base plate and the deformation of the base plate.

Three identical reinforced concrete supports, the external dimensions of which were of 2 x 0.42 x 0.6 m, were cast with an embedded base plate–anchor assembly. Each specimen consisted of 16 mm thick mild steel plate with a planar dimension of 250 x 400 mm. These plates were fixed to the reinforced concrete support with six 12 mm diameter mild steel J anchors welded to the plate. For the column of Specimen I, a box section with outer dimensions 200 x 100 mm was fabricated using four rolled steel channels. The other two specimens were made of rolled I-sections. These three specimens were tested with vertical load eccentricities of 400, 300, and 200 mm, respectively. While the vertical loads were applied gradually in small increments until failure of the specimen, the horizontal loads were kept within a proportion of 0.2 times the vertical loads.

Throughout the experiment, it was observed that the ultimate failure in the connection was triggered mainly by the yielding of the anchors and embedded base plates before the concrete bearing pressure reached its critical value. Consequently, the authors noted that the experimental values of the ultimate load and moment were lower than the values obtained from the concrete beam analogy method. The authors concluded, however, that the concrete beam analogy method could generally be acceptable for the design of the embedded base plate.

Pertold, J., Xiao, R. Y., and Wald, F. (2000a)

Introduction

Experimental and numerical parametric studies were carried out with two different sets of experimental tests analyzed first. The specimens were specifically designed so that the bond and punching resistance of the embedded column base could be quantified. Numerical parametric analysis using a finite element analysis tool was also conducted to investigate behavior of the column base connections under various load and column embedment conditions. The numerical modeling developed in this paper provided a basis for the design model that was introduced in part II of this paper (Pertold, Xiao, and Wald, 2000B).

Experimental Study

The authors conducted an experimental study to measure the resistance of the embedded column base under vertical compression loads. Two sets of specimens were designed and constructed for this experiment. The first set of specimens was designed to investigate the bond strength between the embedded steel column and concrete base. The second set of specimens was designed to measure the resistance of the lower part of the concrete block against the column punching force.

An HEB 100 column section was used for all the specimens. The design strength of the steel column used was S235. The vertical load was applied through the column center line. Each loading step was applied after the progressive column deflection due to the previous loading step stopped.

Three specimens were tested in the first set. Base plates were not added to the bottom of the column in these specimens. A void was created at the bottom of the column so that the column could slide down to the bottom of the concrete foundation. The depth of the concrete block was 500 mm and the length of the column embedment 400 mm. The experimental results showed that a very high vertical force could be supported by the bond between the column flange and the concrete base. The average initial stiffness for the three tests was 2165 N/mm. After the elastic response range, the stiffness decreased by approximately 50%. When the maximum capacity was reached, there was a sudden decrease in the bond strength, leaving almost zero of residual strength.

Another three specimens were tested to measure the resistance of the concrete base against the column punching force. To exclude the bonding effect from the interface between the column flanges and concrete base, the surface of the column was greased. The column embedment length was 180 mm. The depth of the concrete base was 300 mm for the first two specimens and 400 mm for the third specimen. The initial stiffness in the punching test was similar to the values obtained from the previous bond strength tests. The reduction in stiffness of the column base connection was very small until failure occurred, indicating a brittle connection failure. The failure mode for the three specimens was punching of the concrete base. The authors concluded that the punching resistance of the embedded concrete base could be less efficient in resisting the vertical column axial force as compared with the bond resistance.

Numerical Modeling

Three different sets of numerical parametric studies were conducted to investigate the behavior of column bases and their components under various embedment conditions.

1) For the study of the mechanism of horizontal shear transfer between column flanges and concrete, two finite element models were configured and analyzed: the first had concrete filling between the column flanges and the second had a void between the flanges. Comparing the stresses and maximum resistances from these two analysis results, it was concluded that the internal part of the column flanges could participate in the horizontal stress transfer by nearly 50%.

2) In order to investigate the effects of the embedment of column and base plates on the resistance of the column to axial compression loads, two other finite element models were studied. The first model checked the behavior of an embedded base plate and the second model studied the behavior of a base plate on the concrete surface. The embedded base plate had 16% higher plastic load capacity than the base plate on the concrete surface. Based on this, the authors recommended using the existing non-embedded base plate design methods for the design of embedded base plate cases.

3) Numerical models with three different column embedment lengths, 1 b_c, 2 b_c, and 3.5 b_c (where b_c is the width of column cross section) were analyzed to understand the influence of embedment length on stress distribution inside the concrete base. By integrating the horizontal stress distributions and simplifying the results, the authors suggested a rectangular bearing stress distribution pattern. In this proposed stress profile, the value of horizontal stress was truncated to 67% of f_{ck}.

Pertold, J., Xiao, R. Y., and Wald, F. (2000b)

Introduction

A design model for embedded column base connections under combined loading conditions was proposed in this paper. Firstly, methods for the calculation of the moment, shear, and vertical resistance were introduced. The proposed method focused on improvement of shear and vertical resistance calculation. As a result, the length of embedded column needed to transfer the given design forces decreased.

Comparing with existing experimental and numerical results, the proposed design model was found to be accurate and satisfactory. After extensive validation, it was concluded that this model was safe and relatively simple for a column embedment greater than 100 mm. Finally, the authors emphasized that the proposed design model incorporated the vertical resistance of the column base, which hasn't been included in current design codes.

Hitaka, T., Suita, K., and Kato, M. (2003)

Introduction

The authors introduced a brief history of steel column base design in Japan and described three different types of column base connections popularly used by Japanese engineers. This paper also presented current Building Center of Japan (BCJ) design procedure for the exposed-type column base and embedded column connections. The last chapter of this paper briefly introduced a recent experimental research on CFT column bases in Japan (Morino et al., 2003)

In Japan, three types of column base connections are used: 1) exposed-type, 2) encased base plate, and 3) embedded column. Unfortunately, extensive damage to the column base connections was reported after the 1995 Kobe earthquake. The typical connection failure modes were anchor rod elongation or fracture, large base plate plastification, and fracture at welds between the column and base plate. Efforts have been made after the event to investigate proper seismic design force into the column base connection. The results were reflected in the BCJ design procedure.

In their expansion of the discussion about the BCJ design procedures, the authors introduced a two-step design procedure. For the explanation, an equation for the calculation of the rotational stiffness was described and the expected behavior from several connection components was detailed. The authors also discussed other design and detailing issues related to the exposed-type column base connection and the embedded column connection. It was noted in this paper that the "ductile" anchor rods (yield ratio of no more than 0.75) were specified in the Japanese Industrial Standard in 2002 although they are not yet readily available.

Morino, S., Kawaguchi, J., Tsuji, A., and Kadoya, H. (2003)

Introduction

In order to investigate strength and stiffness of a CFT semi-embedded column base connection, an experimental and analytical study was conducted. Monotonic and cyclic loading tests were carried out for the study of the connection resisting mechanism under column lateral displacements without axial loads. Equations that could estimate the connection strength and stiffness were developed and these equations were verified through comparison with the experimental outputs.

Experimental Study

A total of 8 shallowly-embedded type column base specimens were tested. Four specimens were tested under monotonic column lateral displacements and four specimens under cyclic column lateral displacements. The main experimental parameter was the embedded depth of the CFT column in the concrete foundation. A 200 x 200 x 9 mm cold -rolled square tube made of BCR295 steel (Japanese Standard) was used for the CFT column. The column tube passed through the base plate, and the boundary was fillet welded. The anchor rods were designed to yield when the column end moment reached the full-plastic capacity.

Through the experiments, four different types of connection failure modes were observed. These were (1) anchor rod yield only, (2) anchor rod yield and concrete cracking, (3) anchor rod and column yield (with local buckling) and concrete cracking, and (4) column yield only. It was observed that the global load-rotation responses under cyclic loading were similar to those under monotonic loading even though slight differences appeared after the measured maximum strengths. The authors noted that local buckling of the column tube did not affect the global connection behavior much.

Analytical Study

The failure modes of the connection were further investigated, and a theoretical model which could estimate the maximum connection strength was developed. Three connection resisting mechanisms were considered, i.e., anchor rod-base plate interaction, prying action of the embedded portion of the CFT column, and tension of vertical reinforcing bars in the concrete foundation. Based on the deformations of anchor rods and base plate, and accounting for the concrete bearing stress caused by the base plate and the embedded column tube, two initial connection stiffness models (one for the shallowly embedded case and one for the deeply embedded case) were also developed. Through comparison with the experimental results, it was shown that the maximum strength and initial stiffness of each specimen were fairly well estimated by the proposed equations.

A.3. Exposed-Type Column Base in Braced Frames

Goldman, C. (1983)

Introduction

The author discussed common design problems related to column bases subjected to shear and uplift forces only. These forces could be the result of braced systems resisting horizontal and overturning action originated by winds and earthquakes. General solutions that had been used successfully by the author in these situations were discussed in this paper.

Shear capacity of anchor rods and shear lugs was discussed first, and an example was provided to illustrate how to design the anchor rod, the base plate, and the shear lug. An alternative solution was also illustrated for the case of a column base depressed into a concrete pocket. Following these two examples several general notes were made, but no conclusions were provided in this paper. Instead, the author proposed general design criteria for anchor rods and shear lugs.

Tronzo, T. M. (1984)

Introduction

Design of column bases carrying axial compression load only or axial plus shear loads were discussed in general. The contents included anchor rod embedment length, shear lug design stress, anchor rod design, gusset plate analysis, and stresses due to heavy welds. Two design examples (one for the axial load and one for axial plus shear load case) were provided to illustrate the design procedures given by the author. The second example covered the design of a shear plate, connected to the base plate, and shear lug. In the conclusion, the author noted that there was no absolutely correct method for designing large column bases.

A.4. Embedded-Type Column Base in Braced Frames

Kohzu, I., Kaneta, K., Fujii, A., Fujii, K., and Kida, T. (1991)

<u>*Introduction*</u>

The authors conducted an experimental study to investigate the behavior and ductility of shallowly embedded-type steel column-to-footing connections subjected to cyclic bending moments as well as column axial and diagonal brace forces. Four specimens (Type E, F, G, and H) with H-shaped column sections and six anchor rods were prepared and tested in this experimental study. Type E and Type F Specimens were designed to simulate typical column base connections in moment frames and Type G and Type H Specimens were designed to simulate a column base connection in braced frames. Loading conditions were decided from the dynamic analysis of the model frame.

The experimental study focused on investigating the effects on the cyclic performance and the ultimate capacity of the connection of the concrete cover and the reinforcement over the base plate. The connections embedded about 0.6 times the steel column depth exhibited ductile cyclic performance. This experimental study also showed that the stiffness of the column-to-footing connection could be significantly increased by the concrete cover and the reinforcement. Based on the observation, the authors noted that if the amount of the reinforcement of the column stub was properly determined, the shear and moment capacity of the connection could be significantly increased. This was so even after the punching shear failure of the concrete cover above the base plate. In the last part of this study the authors proposed an analytical model which could estimate the ultimate strength and rigidity of the connection under complex loading conditions. The analytical estimations, when compared with the experimental outputs, showed a good agreement of the results.

A.5. Seismic Analysis and Design of Column Bases

Yamada, S. and Akiyama, H. (1997)

Introduction

The effects of column base rigidity on earthquake resistance of multi-story steel frames were investigated. 9-story steel frames were configured for this study and El Centro (NS) and El Centro (EW) earthquake records were used for the time history analysis. Column base rigidity was defined as the ratio between rotational stiffness of the column and rotational stiffness of the connection. Five different column base rigidities (i.e., h, 2.0, 1.0, 0.5, 0.0) were selected for this analytical study. Two major observations were summarized:

(1) When the rigidity of the column base becomes smaller, concentration of damage on the lower end of the column of the first story decreases and, instead, damage in upper beams and panels increases.

(2) Decreasing column base rigidity could improve the ultimate earthquake resistance of the steel moment frame.

Yamada, S., Akiyama, H., and Sadamoto, M. (1997)

Introduction

A study was conducted to investigate the influence of the non-linear behavior of a column base with slip-type hysteresis characteristics on the ultimate earthquake resistance of steel moment frames. The authors carried out a series of inelastic response analysis of multi-story steel frames consisting of exposed-type column bases with weak anchor rods.

From the analytical results, the following conclusions were drawn:

(1) The exposed-type column base with weak anchor rods could mitigate possible damage at the bottom of the 1st story columns and damage concentration at the 1st story beams and panel zones.

(2) A value of 0.03 rad. was reasonable for the limit rotational deformation (story drift) of the exposed-type column base in mid-rise steel moment frames.

(3) The intensity of damage concentration at the structural elements of the 1st story in the frames with exposed type column bases was much less than that of frames with pinned or fixed type column bases.

(4) The ultimate earthquake resistance of frames with exposed type column bases was similar to that of frames with pinned type column bases.

(5) If the frame columns were ductile, ultimate earthquake resistance of the frames with exposed type column bases were similar to or a little better than that of the frames with fixed column bases. If the frame columns were not ductile, ultimate earthquake resistance of the frames with exposed type column bases was much better than that of the frames with fixed column bases.

(6) The energy absorbing capacity of the exposed type column bases was much smaller than that of other structural elements.

Kawano, A. and Matsui, C. (1998)

Introduction

A numerical response analysis was conducted to study inelastic behavior of weak-beam steel frames with different column base restoring force characteristics, i.e., slip-type and degrading stiffness-type. The analytical study was carried out with 1-story and 5-story frame models.

From this study, the authors noted no significant differences on the frame responses between the two types of column base restoring force characteristics. The authors observed that the two restoring force characteristics selected did not significantly change the maximum rotation of the column base connection, maximum plastic ratio, and accumulated plastic deformations. The authors also noted that the response of the frame under earthquake excitation could be more directly affected by plastic deformation of the column base connection and upper story frame elements than by the column base restoring force characteristics (i.e., type of the connection).

Stojadinovic, B., Spacone, E., Goel, S. C., and Kwon, M. (1998)

Introduction

Two typical 3-story steel moment-resisting frame buildings consisting of different column base semi-rigidities were analyzed. Push-over analyses were used to compare the strength and stiffness of the frame models as well as to study the plastic hinge formation sequence and the deformation mechanism of the frames. A series of time-history analyses was also conducted to evaluate the effects of different semi-rigid column base models on the seismic response of the frames.

Analytical Study

This paper presented the results of a parametric study that proposed to evaluate the consequences of using semi-rigid column base models for steel moment-resisting frames. For this study, two typical three-story steel moment-resisting frame buildings designed according to U.S. Codes were modeled and analyzed using SNAP-2D and FEAP. The column bases were modeled as rotational springs with a varying degree of stiffness and strength.

Eight frame models, each with a different column base semi-rigidity, were generated for each building. Based on the computed fundamental periods, the UBC-94 base shear coefficients and the equivalent lateral force distribution for each frame model were calculated. A comparison of the fundamental periods showed that the use of realistic semi-rigid column base models reduces the stiffness of the frames by approximately 10 – 16 %, compared to the fixed support models.

Time history analysis was also performed with one semi-rigid column base model and both the fixed and pinned column base models. This evaluation provided useful information on how the maximum values of inter-story drift, roof displacement, and rotation demands on the plastic hinge formation depended on the degree of column base semi-rigidity.

From the observation of the analytical study, it was found that buildings with realistic semi-rigid column base models behave in a manner similar to buildings with fixed supports. The authors also observed that a reduction of column base stiffness and strength could result in an increase of rotational demand on the first floor beams during the push-over analyses, and this was confirmed by the results of the time-history analyses. In the conclusion, the authors noted that unintended reduction of column base stiffness, often due to poor workmanship, might cause an increase of rotation demands on the first story connections and might result in a soft-story mechanism.

Hasegawa, T. (2000)

Introduction

In order to investigate the seismic response of steel frames consisting of exposed-type column bases, a time history analysis using a fish-bone shaped frame model was carried out. Three different buildings were selected for this study with 3-story, 6-story, and 9-story structures. For the ground motions, El Centro NS component, JMA Kobe NS component, and Hachinohe EW component were applied with intensities for the input earthquake of 150 cm/sec and 300 cm/sec.

Based on the results of the analysis, the author proposed a required plastic rotation capacity of exposed-type column bases against the applied strong earthquake ground motions. In order to evaluate the applicability of the proposed plastic rotation capacity, new time history response analyses were performed with two steel frames (e.g., a 4-bay and 3-story frame) that consisted of different member sizes and column base dimensions. No discussions and conclusions were stated in this paper regarding the results from this analysis.

Yamada, S. (2000)

Introduction

A series of inelastic response analyses of 9-story steel moment resisting frames were carried out. Six weak beam type 9-story moment resisting frames and five weak panel type 9-story moment resisting frames were prepared for this analytical study, and responses from the actions of the NS component of the 1940 El Centro earthquake record and the EW component of the 1968 Hachinohe record were calculated. Three different types of column bases (exposed type, fixed type, and pinned type) were considered but the study was mostly focused on the case of multi-story steel moment resisting frames with exposed type column bases, especially with weak anchor rods. The hysteresis characteristics of the frame members and column bases applied in the analyses were referred to previous experimental results.

Several conclusions were obtained from this analytical study:

(1) The intensity of damage concentration at the structural element of the 1st story in the frames with exposed type column bases was much smaller than that of the frames with pinned column bases or with fixed column bases.

(2) Ultimate earthquake resistance of the frames with exposed type column bases was similar to that of the frames with pinned column bases.

(3) Ultimate earthquake resistance of the frames with exposed type column bases was much better than that of the frames with fixed column bases.

(4) Absorbed energy by the exposed type column bases was much smaller than that of other structural elements.

Michaltsos, G. T. and Ermopoulos, J. C. (2001)

Introduction

The authors conducted an analytical study to investigate the dynamic behavior of a column base connection simulated by a cantilever. The support was considered to be elastically restrained against rotation and horizontal displacements by two linear springs, one rotational and one translational. A lateral distributed dynamic loading along the cantilever height and earthquake ground motions were considered for the external excitations.

For the proposed analysis model, a computer program was developed and a numerical analysis conducted. From the analysis, the following three facts were observed: (1) increase of the load P corresponded to a decrease of ω_1 up to 30%; (2) increase of the concentrated mass M corresponded to a decrease of ω_1 up to 400%; and (3) increase of the rotational inertia J corresponded to a decrease of ω_1 up to 700%. In the conclusion, it was noted that the proposed analysis model should be verified using finite element analysis.

Arlekar, J. N. and Murty, C. V. R (2002)

Introduction

This paper first presented P-V-M interaction curves generated for different levels of axial loads using the hysteretic steel stress-strain curves. Secondly, the authors proposed a truss analogy model for the design of column base connections. Finally, a general procedure for the capacity design of the column base connections was provided.

Theoretical Study

Normalized P-V-M interaction curves were developed. Appropriate strength factors were applied to the moment capacity to account for several uncertainties. It was found that the worst case for column base connection design would be when P=0 based on the observation that the generated P-M-V interaction was symmetric about P=0 plane. During earthquake generated shaking, the column might go through the load level P=0. The authors noted that, in such cases, the column base connections should be designed for the moment and shear capacity of the column corresponding to P=0.

In order to better present the flow of forces near the column base connections, a new truss analogy model was proposed. Based on finite element analysis, two different trusses were considered for the design calculation: one for the normal force in the column flange and the other for the shear. The authors introduced a comprehensive design procedure based on the capacity design concept for the design of column base connections. A connection configuration consisting of outer flange cover plates and vertical rib plates was used for the explanation of the design procedure.

Appendix B: Table of Experimental Parameters

Table B.1. Exposed Type Column Base in Unbraced Frames

Article	No. of Test	Column Type and Size	Column Grade (ksi)	Base Plate Area, *L* x *W* (inch)	Base Plate Thickness, t_p (inch)
DeWolf (1978)	19 (Three sets of tests) Test No. 2 was a repeat of Test No. 1	Short loading plate instead of column 1.52 x 1.52 in. 2.08 x 2.08in.	N.A.	4.88 x 4.88 5.81 x 5.81 6.56 x 6.56 7.81 x 7.81 9.25 x 9.25	0.625 0.750 0.875 1.000 1.125
DeWolf & Sarisley (1980)	16	TS4x4x0.5	A618 Gr. 50 (nominal)	9 x 7	0.754 0.884 0.990
Murray (1983) Column Compression	2	Web 7.5 x 0.1799 11.5 x 0.25 in. Flange 6 x 0.25 in.	A572 Gr. 50	8 x 6 12 x 8	3/8
Murray (1983) Column Tension	4	Web thickness 0.183 – 0.316 Flange 6 x 0.25 8 x 0.315	A 572 Gr. 50	8 x 6 10 x 8 12 x 6 12 x 8	0.364 – 0.377
Akiyama et al. (1984)	25	2 H-9.84x9.84x0.35 x0.55 H-9.84x9.84x0.35 x0.55	N.A.	13.78 x 13.78 16.14 x 16.14 19.29 x 19.29 Circular Dia. 14.17	0.47- 1.26 - 1.57 - 2.36
Picard & Beaulieu (1985)	15	M3.94x0.75 W5.91x1.46 HSS6.0x6.0 x0.5	43.51 (flange sections) 50.76 (hollow sections)	5.51 x 5.12 11.81 x 7.48	0.43 - 1.14
Thambirat-nam & Para-masivam (1986)	12	4 x 4 x 1/2 in. Box section	N.A.	9 x 7	7/8 - 3/4 - 5/8

Base Plate Grade (ksi)	Concrete Foundation Area, *L* x *W* (inch)	Concrete Foundation Depth, *D* (inch)	Strength of Concrete, f'_c (ksi)	No. of Anchor Rods	Anchor Rod Diameter (inch)
27.4 – 44.3 (tested)	6.88 x 6.88 8.25 x 8.25 9.25 x 9.25 9.75 x 9.75 11.00 x 11.00 13.12 x 13.12	6.88 8.25 9.25 9.75 11.00 13.12	2.72 – 3.52	N.A.	N.A.
33.7 – 38.8 (tested)	12 x 10.5	Total depth of concrete block is 24 in. (mirror type test specimen)	2.98 – 3.38	1 (outside of column)	0.465 0.748 0.995
A 572 Gr. 50	14 x 12 18 x 14	18	Grout (7day) 4.7 Foundation (7day) 1.4-1.7 (BP1) 3.7-3.8 (BP2)	2 (inside of column flanges)	3/4
A 572 Gr. 50	N.A.	N.A.	N.A.	2 (inside of column flanges)	1
N.A.	N.A.	35.43 (for HC and TC series) 31.50 (for all the other series)	1.56 – 4.16	2- 4- 6-8-10	0.79- 0.87- 1.42
36.26 (nominal)	50 x 24	18.1	3.28 4.39	2, 4	0.75
35 (tested	12 x 12	11	7	1	0.75

Anchor Rod Grade (ksi)	Loading Type	Main Test Parameters	Typical Failure Mode(s)	Results Reported
N.A.	- Axial only	- Aconc/Arod - Base plate thickness	- Concrete cube cracking	- Test failure load - Comparison between test output and analysis - A simple empirical design approach
A36	- Axial plus moment (from eccentricity)	- Axial load eccentricity - Anchor rod diameter - plate thickness	- Concrete crushing - Anchor rod yielding - Base plate yielding	- Test failure load - P-M interaction curve - Comparison between test output and estimations from two different design approach(WSD and USD)
A307	- Axial only	- Area of column - Column web thickness	- Grout crushing (BP1) - Out of test setup capacity (BP2)	- Base plate penetration into grout - Effective bearing area
A36	- Axial tension only	- Base plate area - Column dimension	- Base plate yielding	- Base plate deflection - Comparison between test output and estimations from yield line theory
42.24– 64.15	- Axial plus moment (cyclic)	- End detail and depth of anchor rod - Shape of column and base plate	- Concrete crushing - Anchor rod pull-out	- Moment-rotation - Anchor rod deformation - Equations for connection elastic stiffness, ultimate capacity, and maximum inelastic deformation capacity
44.67 (tested)	- Bending moment only - Axial plus moment (from eccentricity)	- Column section - No. of anchor rods	- grout crushing - rupture of anchor rod	- Moment-rotation - Experimental flexibility factor at column base - Comparison between theoretical and experimental flexibility factor
35 (tested)	- Axial plus moment (from eccentricity)	- Base plate thickness - Eccentricity of axial load	- Concrete block failure - Base plate yielding - Anchor rod yielding	- Longitudinal strain variation in base plate - P-M interaction curve - Comparison between test results and predictions from the Working Stress Method

Article	No. of Test	Column Type and Size	Column Grade (ksi)	Base Plate Area, *L* x *W* (inch)	Base Plate Thickness, t_p (inch)
Picard & Beaulieu (1985)	14	W5.91x1.18 W7.87x1.42 HSS4.0x4.0 x1/4	43.51 (flange sections) 50.76 (hollow sections)	9.84 x 8.86 12.99 x 10.83 12.8x 12.8 12.8 x 9.84 13.78 x 13.78 15.75 x 13.78	0.71- 0.87- 1- 1.02- 1.06-1.14
Sato (1987)	6	أ11.81x 11.81 (for B-3-4) أ7.87 x 7.87 (for the other specimens)	N.A.	21.65 x 21.65 (for B-3-4) 14.9 x 14.9 (for the other specimens)	2.40 (for B-3-4) 1.26 (for the other specimens)
Hon & Melchers (1988)	26	460UB (18.1 in) 310UC (12.2 in) 310UB (12.2 in)	Flange 37.71 – 42.79 Web 43.08 – 48.88	19.68x 11.81 19.68 x 15.75 23.62 x 11.81	0.47- 0.63- 0.79- 0.98- 1.18
Astaneh et al. (1992)	6	W6x25	A36	12 x 9	1/4 - 1/2- 3/4
Igarashi et al. (1992)	4	أ7.87x7.87x0.35	59.47	14.96 x 14.96	1.42
Melchers (1992)	10	200UB25 (7.87 x 0.98)	N.A.	11.81 x 7.87	0.24- 0.39- 0.47
Targowski et al. (1993)	12	Square Rectangular Circular Channel I-section A I-section B	N.A.	15.75 x 11.81	0.24- 0.39

Base Plate Grade (ksi)	Concrete Foundation Area, *L* x *W* (inch)	Concrete Foundation Depth, *D* (inch)	Strength of Concrete, f'_c (ksi)	No. of Anchor Rods	Anchor Rod Diameter (inch)
43.51	50 x 24 Steel pedestals were also used to fix column base	18.11	Grout 5.95 (at the time of test)	2, 4	0.75
N.A.	29.53 x 29.53 (for B-3-4) 22.83 x 22.83 (for the other specimens)	51.18 (for B-3-4) 31.50 (for the other specimens)	3.23 (for B-2-OS) 4.41 (for the other specimens)	4	1.18- 1.65
34.81 – 56.56	Length 26.77	35.04	4.86	2	0.79- 0.94
N.A.	24 x 24	12	3	4	0.75
55.12	20.47 x 20.47 (concrete riser area)	39.37	4.06 (concrete riser) 7.76 (grout)	4	1.18
43.51, 77.31, 83.11	Length 15.75	23.62	N.A.	2-4 (inside of column flanges)	0.47- 0.63
39.89 (0.24 in.) 45.11 (0.39 in.)	39.37 x 39.37	39.37 (including steel beam pedestal)	N.A.	4	0.94 (0.63 at threaded part)

Anchor Rod Grade (ksi)	Loading Type	Main Test Parameters	Typical Failure Mode(s)	Results Reported
N.A.	- Axial plus moment	- Shape of column - Base plate area and tp - No. of anchor rods	- Column buckling in the direction of weak axis	- Moment-rotation - Experimental flexibility factor - Variation of rigidity ratio with column axial load
41.24 (for B-2-OS) 81.07 (for the other specimens)	- Axial plus moment (cyclic)	- Size of base plate - Column axial load - Yield ratio of anchor rod	- Anchor rod breakage - Concrete failure - Anchor rod yielding	- Moment-rotation - Equation of column fixity - Evaluation equation for connection ultimate bending moment
73.24 – 100.51	- Axial plus moment (eccentricity)	- Base plate thickness - Anchor rod size	- Anchor rod failure - Base plate yield mechanism	- Moment-rotation - Effects of each connection element on connection behavior
A307	- Axial plus moment (cyclic)	- Base plate thickness - Column axial load	- Column and plate yielding - Rod and weld fracture - Grout crushing	- Moment-rotation - Behavior of anchor rod - Effects of column axial and shear forces - Development of seismic design recommendation
52.21	- Moment (cyclic)	- Type of anchor rod	- Concrete riser and grout cracking and crushing - Anchor rod yielding	- Force-displacement - Effects of anchor rod type on column base durability
28.28- 50.62- 51.34	- Moment (cyclic)	- Base plate thickness - No. and size of anchor rod - Anchor rod yield strength	- Base plate yielding - Anchor rod yielding	- Moment-rotation - Estimation of initial (elastic) stiffness
N.A.	- Moment	- Different column section - Base plate thickness	- Base plate yielding - Anchor rod elongation	- Base plate deformation and yielding - Discussion on bi-axial bending of base plate - Comparison with finite element analysis

Article	No. of Test	Column Type and Size	Column Grade (ksi)	Base Plate Area, *L* x *W* (inch)	Base Plate Thickness, t_p (inch)
Wald et al. (1994)	14	H100 (3.94 in.) H160 (6.3 in.)	N.A.	8.66 x 8.66 11.81 x 8.66	0.39- 0.47-0.78
Akiyama et al. (1998) Shaking	2	أ19.69x19.69 x0.87	70.83	29.92 x 29.92	1.18- 2.36
Jaspart & Vandegans (1998)	12	HE160B (6.3 in.)	S355	13.39 x 8.66 8.66 x 8.66	0.59-1.18
Burda & Itani (1999)	6	W8x48	A36	14.5 x 14.5 19.5 x 19.5	0.75- 1.0-1.25
Fahmy (1999) Michigan Strong Axis	3	W10x77	A572 Gr. 50	20 x 20	2.75
Adany et al. (2000)	5	HEA200 (7.87 in.)	N.A.	14.57 x 7.87	0.47- 0.63- 0.98
Li et al. (2000)	7	Square and Circular CFT9.84x9.84 (t=0.47)	N.A.	16.93 x 16.93	2.16 (cast steel base plate)

Base Plate Grade (ksi)	Concrete Foundation Area, *L* x *W* (inch)	Concrete Foundation Depth, *D* (inch)	Strength of Concrete, f'_c (ksi)	No. of Anchor Rods	Anchor Rod Diameter (inch)
N.A.	21.65 x 21.65	19.68	N.A.	0, 2, 4	0.94
45.23– 47.65	N.A.	N.A.	N.A.	20	1.30
S235	47.24 x 23.62	23.62	N.A.	2 (inside of column flange) 4	0.79
37.6 – 47.3	72 x 72	24	52- 57	4	1.5
A36	74 x 74	36	6	4, 6	2.0 (for 6 anchor rods) 1.25 (for 4 anchor
N.A. S235 was assumed for preliminary calculations	N.A.	N.A.	N.A.	4	N.A.
N.A.	70.87 x 23.62	27.56	3.44– 4.26	4	1.18

Anchor Rod Grade (ksi)	Loading Type	Main Test Parameters	Typical Failure Mode(s)	Results Reported
N.A.	- Axial - Axial plus moment -Moment	- Base plate thickness	N.A.	- Deformation of base plate along the neutral axis
41.67	- Moment (transferred from shaking table)	- Base plate thickness	- Anchor rod elongation - Base plate yielding	- Moment-rotation - Comparison between thin and thick base pate cases
M20 10-9	- Axial plus moment	- Base plate thickness - No. of anchor rods	- Failure of anchor rod and concrete - Yielding of base plate and column	- Moment-rotation - Development of spring model - Comparison with experimental results
105 (A193 Gr. B7)	- Axial plus moment (cyclic)	- Base plate area - Base plate thickness	- Fracture of the weld between column and base plate	- Force-displacement - Development of a design methodology - Time history frame analysis with different column base fixity
130 (nominal) (A354 Gr. BD)	- Moment (cyclic)	- No. of anchor rods - Weld material	- Fracture of the weld between column and base plate	- Force-displacement - Push-over and time history analysis for a model frame - Classification of failure mechanism and introduction of a design approach
N.A.	- Moment (cyclic)	- End-plate thickness - Anchor bolt pre-tensioning	- Base plate yielding - Anchor rod yielding - Column local buckling	- Moment-rotation - Ductility ratio - Resistance ratio - Rigidity ratio - Absorbed energy ratio
83.54 (SD490& H-AB490) 48.73 (SS490)	- Axial plus moment (cyclic)	- Column section - Concrete filling - Anchor rod strength	- Anchor rod yielding - Buckling of steel tube	- P-M interaction curve - Moment-rotation - Verification of the existing rotational stiffness equation

Article	No. of Test	Column Type and Size	Column Grade (ksi)	Base Plate Area, *L* x *W* (inch)	Base Plate Thickness, t_p (inch)
Lee & Goel (2001) (2001)	4	W12x96	49.2 – 51.5 (tested)	20 x 20	2.25
Miyasaka et al. (2001)	8	أ15.75x15.75 x1.26	SN490B	23.62 x 23.62 25.20 x 25.20 27.56 x 27.56 29.92 x 29.92	0.79- 1.18-1.57- 1.99- 2.36
Somiya et al. (2002)	5 (test group 1) 7 (test group 2)	أ7.87x7.87x0.24 أ7.87x7.87x0.35 أ7.87x7.87x0.47	N.A.	15.75 x 15.75	1.42- 1.53- 1.57- 1.77

Base Plate Grade (ksi)	Concrete Foundation Area, *L* x *W* (inch)	Concrete Foundation Depth, *D* (inch)	Strength of Concrete, f'_c (ksi)	No. of Anchor Rods	Anchor Rod Diameter (inch)
A36	74 x 74	36	6	4- 6	2.0 (for 6 anchor rods) 1.25 (for 4 anchor rods)
48.30 – 50.18 (SN490B)	N.A.	Total concrete block depth is 40 cm (mirror type test specimen)	N.A.	4	N.A.
37.42– 62.57	23.62 x 23.62	31.50	N.A.	4	0.87

Anchor Rod Grade (ksi)	Loading Type	Main Test Parameters	Typical Failure Mode(s)	Results Reported
130 (nominal) (A354 Gr. BD)	- Moment (cyclic)	- No. of anchor rods - Weld material	- Fracture of the weld between column and base plate	- Numerical and experimental evaluation of the D&E method (Drake and Elkin, 1999) - New design approach based on relative strength ratio among the connection elements
SS490	- Moment	- Base plate thickness - Location of anchor rod	- Base plate deformation and yielding	- Moment-rotation - Base plate out-of-plane deformation - Evaluation of connection ultimate strength and rotational rigidity
N.A.	- Axial and moment	- Different initial axial load and load rate - Plate and tube thickness	- Base plate yielding - Anchor rod yielding	- Force-displacement with different initial axial load - Simplified equations for connection ultimate strength and rotational stiffness

Table B.2. Embedded Type Column Base in Unbraced Frames

Article	No. of Test	Column Type and Size	Column Grade (ksi)	Base Plate Area, *L* x *W* (inch)	Base Plate Thickness, t_p (inch)
Nakashima & Igarashi (1986)	93	أ5.9 x 5.9 x 0.24 x 0.17 x 0.09 أ3.94 x 3.94 x 0.24 x 0.17 x 0.09	STKR41 46.02-57.1	6.30 x 6.30 9.84 x 6.30 11.22 x 6.30	0.23-0.47- 0.75
Suzuki & Nakashima (1986)	38	H125x125x6.5x9	SS41 54.6-55.3-58.7	7.9 x 10.2 6.3 x 6.3 5.5 x 5.5	0.75 0.75 0.75
Nakashima & Igarashi (1987)	29	أ3.94 x 3.94 x 0.24	STKR41 57.9-57.3	5.12 x 5.12	0.47- 0.75
Nakashima (1992)	10 30	أ3.94 x 3.94 x 0.24 أ5.91 x 5.91 x 0.24	57.9 STKR40 50.5-56.3	5.12 x 5.12 7.48 x 7.48 N.A.	0.47- 0.63 N.A.
Nakashima (1996)	23	أ3.94 x 3.94 x 0.24	STKR40 52.7-64.8	3.94 x 3.94 4.33 x 4.33 5.12 x 5.12 7.48 x 7.48	0.47- 0.63
Kallolil et al. (1998)	3	4 C 3.94 x 1.97 I 7.88 x 1.97	N.A.	15.75 x 9.84	0.63
Pertold et al. (2000)	3 (bond) 3 (punching)	HEB100 (H 3.94 in.)	S235 34.08	N.A.	N.A.
Morino et al. (2003)	8	أ7.88 x 7.88 x 0.35	BCR295 45.3-54.6	-	1.26

Base Plate Grade (ksi)	Concrete Foundation Area, *L* x *W* (inch)	Concrete Foundation Depth, *D* (inch)	Strength of Concrete, f'_c (ksi)	No. of Anchor Rods	Anchor Rod Diameter (inch)
SS41 51.0 – 56.4	17.32 (footing width)	15.75	3.20-3.73	4	0.47
SS41 54.11	11.81 x 14.96 (pedestal) 11.81 x 13.78 (pedestal)	18.70 18.70	2.5-2.6-2.8	2-4	0.63
SS41 46.2 – 48.4	11.81 (footing width)	15.75	4.28	4	0.47
41.2 N.A.	11.81 (beam width) 11.81 x 11.81	15.75 17.72 (3D)	3.36 (beam) 3.66 (Encasement) 3.16-3.77 (Encasement)	4-8 4	0.47 0.63
SS40 38.4-50.76	11.81 (beam width) 11.81 x 15.75	15.75	3.67	4-8-12	0.39-0.47-0.63
-	78.74 x 16.53	23.62	6.45 (cube)	6	0.47
-	19.68 x 19.68	19.68, 11.81	4.87	N.A.	N.A.
-	62.99 x 23.62	37.40	4.23-5.38	4	0.47- 1.06 - 1.18

Anchor Rod Grade (ksi)	Loading Type	Main Test Parameters	Typical Failure Mode(s)	Results Reported
SS41 56.0	- Bending and shear force	- Embedment length - D/t ratio - Concrete filling - Base plate size - Reinforc. methods - Anchor rods.	- Base of column yield - Local Buckling - Concrete bearing - Yielding of web - Yielding of rein-forcement bars	- Moment-rotation curves - Concrete cracking patterns
SS41 60.2	Bending and shear force (Cyclic)	- Embedment length - Base plate size - Number of anchor rods - Reinforc. methods	- Concrete bearing	- Moment-rotation curves - Concrete cracking patterns - Stress and Strain distribution in pedestal
SS41 40.80 -44.38	- Axial , bending and shear force - Pulling force	- Embedment length - End distance - Loading method - Effect of concrete filling - Reinforc. methods - Anchor rods.	- Base of column yield - Local Buckling - Concrete bearing - Column web yield - Reinforcement bars yield	- Strain distribution in Steel column - Strain distribution in Hoop reinforcement - Bearing Stress distribution at top of footing
47.42	- Axial , bending and shear force - Axial only	- Supports conditions - - - - - - - - - - - - - - - Anchor rods -shear reinforcem. -Encasement size -Axial force ratio - Bar hooks	- - - - - - - - - - - - - - - - Concrete encasement cracking	- Horizontal Force, story deflection angle curve - Inflection point height ratios - Q load deflections curves - Envelope curves
46.41-60.91	- Bending	- Base plate area - Anchor rod layout - Reinforc. methods	-	- Moment-rotation - Envelope curves of M/Mp-R/Rp relations
-	- Axial plus moment (cyclic)	- End detail and depth of anchor rod - Shape of column and base plate	-	- Plate Deformations - Anchor stress variation
N.A.	- Axial load	- Embedment length	- Bond - Punching shear	-Load-displacements relations -Numerical modeling of the test
43.80-46.41	- Monotonic and Cyclic horizontal load	- Embedment length	- Anchor rods yield - Anchor rods and column yielding - Anchor rods yield and concrete cracks - Concrete cracking	- H/H_{cu}-R relations

Appendix C: Bibliography of Experimental and Analytical Research on Column Bases

Adany, S., Calado, L., and Dunai, L. (2000). "Experimental Studies on Cyclic Behavior Modes of Base-Plate Connections," *Proceedings of the Third International Conference on the Behavior of Steel Structures in Seismic Areas (STESSA 2000)*, Montreal, Canada, 97-104.

Adany, S. and Dunai, L. (2000). "Monotonic and cyclic loading analysis of bolted T-stubs," *Proceedings of the Third International Conference on the Behavior of Steel Structures in Seismic Areas (STESSA 2000)*, Montreal, Canada, pp 89-96.

Ahmed, S. and Kreps, R.R. (1990). "Inconsistencies in Column Base Plate Design in the New AISC ASD Manual," *Engineering Journal*, AISC, Vol. 27, No. 3, Third Quarter, pp. 106-107.

Akiyama, H., Kurosawa, M., Wakuni, N., and Nishinura, I. (1984). "Strength and Deformation of Exposed Type of Steel Column Bases," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 342, pp. 46-54.

Akiyama, H., Yamada, S., Takahashi, M., Katsura, D., Kumura, K., and Yahata, S. (1998). "Full Scale Shaking Test of the Exposed-Type Column Bases," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 514, pp. 185-192.

Arlekar, J. N. and Murty, C. V. (2002). "P-V-M Interaction Curves for Seismic Design of Steel Column Base Connections," *Engineering Journal*, AISC, Vol. 39, Third Quarter, pp. 154-165.

Astaneh, A., Bergsma, G., and Shen J. H. (1992). "Behavior and Design of Base Plates for Gravity, Wind and Seismic Loads," *Proceedings of the National Steel Construction Conference*, Las Vegas, Nevada, AISC, Chicago, Illinois.

Balut, N. and Moldovan, A. (1997). "A Model for the Behavior of Column Base Connections," *Proceedings of the Second Conference STESSA*, Kyoto, Japan.

Bird, W. R. (1976). "Rapid Selection of Column Base Plates," *Engineering Journal*, AISC, Volume 13, No. 2, Second Quarter, pp. 43-47.

Burda, J. J. and Itani, A. M. (1999). "Studies of Seismic Behavior of Steel Base Plates," *Report No. CCEER 99-7, Center for Civil Engineering Earthquake Research, Dept. of Civil Engineering, Univ. of Nevada, Reno, Nevada.*

Chhabra S. J. (1994). "Column Base Plate Design Table," *Engineering Journal*, AISC, Vol. 31, No. 1, First Quarter, pp. 12-20.

Cowie, K., Hyland, C., and Mago, N. (2004). "Column Base Plate Design-Lapping Strip Method", *Proceedings of the 2004 Pacific Structural Steel Conference,* Long Beach, California.

DeWolf, J. T. (1978). "Axially Loaded Column Base Plates," *Journal of the Structural Division,* ASCE, Vol. 104, No. ST5, May, pp. 781-794.

DeWolf, J.T. and Sarisley, E. F. (1980). "Column Base Plates with Axial Loads and Moments," *Journal of the Structural Division*, ASCE, Vol. 106, No. ST11, November, pp. 2167-2184.

DeWolf, J. T. (1982). "Column Base Plates," *Structural Engineering Practice*, Vol. 1, No. 1, pp. 39-51.

Drake, R. M. and Elkin, S. J. (1999). "Beam-Column Base Plate Design - LRFD Method," *Engineering Journal*, AISC, Vol. 36, No. 1, First Quarter, pp. 29-38.

Ermopoulos, J. and Stamatopoulos, G. (1996a). "Mathematical Modeling of Column Base Plate Connections," *Journal of Constructional Steel Research*, Vol. 36, No. 2, pp. 79-100.

Ermopoulos, J. and Stamatopoulos, G. (1996b). "Analytical Modeling of Column Base Plates under Cyclic Loading," *Journal of Constructional Steel Research*, Vol. 40, No. 3, pp. 225-238.

Ermopoulos, J. and Michaltsos, G. T. (1998). "Analytical Modeling of Stress Distribution under Column Base Plates," *Journal of Constructional Steel Research*, Vol. 46, No. 1-3, Paper No. 136

Fahmy, M. (1999). "Seismic Behavior of Moment-resisting Steel Column Bases," *Ph.D. Dissertation,* Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, Michigan.

Fling R. S. (1970). "Design of Steel Bearing Plates," *Engineering Journal*, AISC, Vol. 7, April, pp. 37-40.

Goldman, C. (1983). "Design of Column Base Plates and Anchor Bolts for Uplift and Shear," *Structural Engineering Practice,* Vol. 2, No. 2, pp. 103-115.

Hasegawa, T. (2000). "Seismic Response Behavior Steel Rigid Frames Having Exposed-Type Column Base," *Proceedings of the U.S.-Japan Workshop on Seismic Fracture Issues in Steel Structures*, San Francisco, California.

Hitaka, T., Suitaa K., and Kato M. (2003). "CFT Column Base Design and Practice in Japan," *Proceedings of the International Workshop on Steel and Concrete Composite Construction (IWSCCC-2003)*, Report No. NCREE-03-026, National Center for Research in Earthquake Engineering, Taipei, Taiwan, October 8-9, 2003, National Center for Research in Earthquake Engineering, Taipei, Taiwan, pp. 35-45.

Hon, K.K. and Melchers, R.E. (1988). "Experimental Behavior of Steel Column Bases" *Journal of Constructional Steel Research*, Vol. 9, Paper No. 143, pp. 35-50.

Igarashi, S., Kadoya, H., Nakashima, S., and Suzuki, M. (1992). "Behavior of Exposed-Type Fixed Column Base Connected to Riser Foundation," *Proceedings of the Tenth World Conference on Earthquake Engineering*, Madrid, Spain.

Jaspart, J. P. and Vandegans, D. (1998). "Application of Component Method to Column Bases," *Journal of Constructional Steel Research*, Vol. 48, pp. 89-106.

Kallolil, J. J., Chakrabarti, S. K., and Mishra, R. C. (1998). "Experimental Investigation of Embedded Steel Plates in Reinforced Concrete Structures," *Engineering Structures*, Vol. 20, No. 1-2, pp. 105-112.

Kawano, A. and Matsui, C. (1998). "On the Effect of Restoring Force Characteristics of Column-Base on Inelastic Response Behavior of Weak-Beam Steel Frame under Earthquake Ground Motion," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 507, pp. 139-146.

Kohzu, I., Kaneta, K., Fujii, A., Fujii, K., and Kida, T. (1991). "Experimental Study on Very Shallowly Embedded Type Steel Column-to-Footing Connections," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 421, pp. 59-68.

Kontoleon, M. J., Mistakidis, E.S., Baniotopoulos, C. C., and Panagiotopoulos, P. D. (1999). "Parametric Analysis of the Structural Response of Steel Base Connections," *Computers and Structures*, Vol. 71, pp. 87-103.

Kontoleon, M. J., and Baniotopoulos, C. C., (2000). "Computational Aspects on the Frictional Unilateral Contact Problem Arising on Steel Base Plate Connections," *Computers and Structures*, Vol. 78, pp. 303-309.

Krishnamurthy, N. and Thambiratnam, D. P. (1990). "Finite Element Analysis of Column Base Plates," *Computers and Structures*, Vol. 34, No. 2, pp. 215-223.

Lee, D. (2001), "Seismic Behavior of Column-Base Plate Connections Bending about Weak Axis," *Doctoral Dissertation*, Dept. of Civil and Environmental Engineering, Univ. of Michigan, Ann Arbor, Michigan.

Li, T., Sakai, J., and Matsui, C (2000). "Seismic Behavior of Steel-Concrete Composite Column Bases," *Proceedings of the 12th World Conference on Earthquake Engineering*, Paper 1072.

Maitra, N. (1978). "Graphical Aid for Design of Base Plate Subjected to Moment," *Engineering Journal*, AISC, Vol. 15, No. 2, Second Quarter, pp. 50-53.

Melchers, R. E. (1992). "Column-Base Response under Applied Moment," *Journal of Constructional Steel Research*, Vol. 23, pp. 127-143.

Melchers, R. E. and Maas, G. (1994). "Column Base Restraint Effect for a Steel Portal Frame," *The Structural Engineer*, Vol. 72, Nos. 4/15, February, pp. 61-67.

Michaltsos, G. T. and Ermopoulos, J. C. (2001). "Dynamic Response of Column Bases," *Engineering Structures*, Vol. 23, pp. 58-71.

Midorikawa, M., Azuhata, T., Ishihara, T., and Wada, A. (2001). "Effects of Base Plate Yielding on Seismic Response of Rocking Structural Systems," *Proceedings of U.S.-Japan Cooperative Research on Urban Earthquake Disaster Mitigation,* Seattle, Washington

Miyasaka, H., Arai, S., Uchiyama, M., Yamada, T., and Hashimoto, A. (2001). "Elasto-Plastic Behavior of Structural Elements Consist in Exposure Fixed-Type Steel Column Base. Part I – Behavior to Bending Moment," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No 550, December 2001, pp.167-174.

Morino, S., Kawaguchi, J., Tsuji, A., and Kadoya, H. (2003). "Strength and Stiffness of CFT Semi-embedded Type Column Base," *Proceedings of ASSCCA 2003 Conference,* Sydney, Australia, A. A. Balkema, Sydney, Australia.

Murray, T. M. (1983). "Design of Lightly Loaded Steel Column Base Plates," *Engineering Journal,* AISC, Vol. 20, No. 4, Fourth Quarter, pp. 143-152.

Nakashima, S. and Igarashi, S. (1986). "Behavior of Steel Square Tubular Column Bases Embedded in Concrete Footings under Bending Moment and Shearing Force: Part 1 – Test Program and Load-Displacement relations," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 366, pp. 106-118.

Nakashima, S. and Igarashi, S. (1987). "Behavior of Steel Square Tubular Column Bases of Corner Columns Embedded in Concrete Footings," *Proceedings of International Conference on Steel and Aluminum Structures,* Cardiff, July 1987, pp.376-385.

Nakashima, S. (1992). "Mechanical Characteristics of Steel Column-Base Connections Repaired by Concrete Encasement," *Proceedings of the Tenth World Conference on Earthquake Engineering*, Madrid, Spain.

Nakashima, S. (1996). "Response of Steel Column Bases Embedded Shallowly into Foundation Beams," *Proceedings of the Eleventh World Conference on Earthquake Engineering*, Acapulco, Mexico.

Picard, A. and Beaulieu, D. (1985). "Behavior of a Simple Column Base Connection," *Canadian Journal of Civil Engineering*, Vol. 12, pp. 126-136.

Picard, A., Beaulieu, D., and Perusse, B. (1987). "Rotational Restraint of a Simple Column Base Connection," *Canadian Journal of Civil Engineering*, Vol. 14, pp. 49-57.

Penserini, P. and Colson, A. (1989). "Ultimate Limit Strength of Column-Base Connections," *Journal of Constructional Steel Research*, Vol. 14, pp. 301-320.

Pertold J., Xiao, R. Y., and Wald, F. (2000a). "Embedded Steel Column Bases – I. Experiments and Numerical Simulation," *Journal of Constructional Steel Research*, Vol. 56, pp. 253-270.

Pertold J., Xiao, R. Y., and Wald, F. (2000b). "Embedded Steel Column Bases – II. Design Model Proposal," *Journal of Constructional Steel Research*, Vol. 56, pp. 271-286.

Ricker, D. T. (1989). "Some Practical Aspects of column Base Selection," *Engineering Journal*, AISC, Vol. 26, No. 3, Third Quarter, pp. 81-89.

Robertson, A. P. (1991). "A Study of Base Fixity Effects on Portal Frame Behaviour," *The Structural Engineer,* Vol. 69, No. 2/22, January 1991, pp. 17-24.

Salmon, C. G., Shaker, L., and Johnston, B. G. (1957). "Moment-Rotation Characteristics of Column Anchorages," *Transactions*, ASCE, Vol. 122, pp. 132-154.

Sandhog, B. S., (1973). "Steel Column Base Plate Design," *Engineering Journal*, AISC, Vol. 10, No. 4, Fourth Quarter, pp. 48-51.

Sato, K. (1987). "A Research on the Aseismic Behavior of Steel Column Base for Evaluating Its Strength Capacity and Fixity," *Report No. 69*, Kajima Institute of Construction Technology, Tokyo, Japan.

Sato, K. and Kamagata, S. (1988). "The Aseismic Behavior of Steel Column-Base," *Proceedings of the Ninth World Conference on Earthquake Engineering*, Tokyo, Japan.

Somiya, Y., Fukuchi, Y., and Chin, B. (2002). "Experimental Study on Elasto-Plastic Behavior and Strength Estimation of Exposed-Type Column Base with Variable Axial Force," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 562, pp. 137-143.

Sputo, T. (1993). "Design of Pipe Column Base Plates under Gravity Load," *Engineering Journal*, AISC, Vol. 30, No. 2, Second Quarter, pp. 41-43.

Stamatopoulos, G. and Ermopoulos, J. (1997). "Interaction Curves for Column Base-Plate Connections," *Journal of Constructional Steel Research*, Vol. 44, No. 1-2, pp. 69-89.

Stockwell, F. W. Jr. (1975). "Preliminary Base Plate Selection," *Engineering Journal*, AISC, Vol. 12, No. 3, Third Quarter, pp. 92-99.

Stojadinovic, B., Spacone, E., Goel, S. C., and Kwon, M. (1998). "Influence of Semi-Rigid Column Base Models on the Response of Steel MRF Buildings," *Proceedings of the Sixth U.S. National Conference on Earthquake Engineering*, Seattle, Washington, EERI, Oakland, California.

Surtees, J. O. and Yeap, S. H. (1996). "Load-strength charts for pitched roof, haunched steel portal frames with partial base restraint," *The Structural Engineer,* Volume 74/No 1/2, January 1996, pp. 9-13.

Suzuki, T. and Nakashima, S. (1986). "An Experimental Study on Steel Column Bases Consolidated with Reinforced Concrete Stud: Part 1 – Tests on Steel Column Base under

Bending Moment and Shearing Force," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 368, pp. 37-48.

Tamai, H (2003). "Loading test on Exposed-Type Column Base Subjected to Cyclic Bending Moment and Variable Axial Force," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 567, pp. 149-156.

Targowski, R., Lamblin, D., and Guerlement, G. (1993). "Base plate Column Connection under Bending: Experimental and Numerical Study," *Journal of Constructional Steel Research*, Vol. 27, pp. 37-54.

Thambiratnam, D. P. and Paramasivam, P. (1986). "Base Plates under Axial Loads and Moments," *Journal of Structural Engineering*, ASCE, Vol. 112, No. 5, pp. 1166-1181.

Thambiratnam, D. P. and Krishnamurthy N. (1989). "Computer Analysis of Column Base Plates," *Computers and Structures,* Vol. 33, No. 3, pp. 839-850.

Thornton, W. A. (1990a). "Design of Small Base Plates for Wide Flange Columns," *Engineering Journal*, AISC, Vol. 27, No. 3, Third Quarter, pp. 108-110.

Thornton, W. A. (1990b), "Design of Base Plates for Wide Flange Columns- A Concatenation of Methods," *Engineering Journal*, AISC, Vol. 27, No. 4, Fourth Quarter, pp. 173-174.

Tronzo, T. M. (1984). "Design of Heavy Steel Column Bases: Handling Forces- Some Practical Procedures," *Structural Engineering Practice,* Vol. 2, No. 4, pp. 279-300.

Voce, G.J. (1958).). "Restraint at Stanchion Bases," *The Structural Engineer*, April 1958, pp. 136-137.

Wald, F., Simek, I., and Seifert, J. (1994). "The Tests of the Column-Base Components," *Proceedings of the First International Conference STESSA*, Timisoara, Romania.

Wald, F., Sokol, Z., and Steenhuis, M. (1995). "Proposal of the Stiffness Design Model of the Column Bases," *Proceedings of the Third International Workshop on Connections in Steel Structures*, Trento, Italy.

Wald, F. and Sokol, Z. (1997). "Model of the Column-Base Stiffness," *Proceedings of the Second International Conference STESSA*, Kyoto, Japan.

Wald, F. and Jaspart, J. (1998). "Stiffness Design of Column Bases," *Journal of Constructional Steel Research*, Vol. 46, Nos. 1-3, Paper No. 135

Wald, F., Bouguin, V., Sokol, Z., and Muzeau, J. (2000). "Effective Length of T-Stub of RHS Column Base Plates," *Proceedings of Connections in Steel Structures IV*, Roanoke, Virginia.

Yamada, S. and Akiyama, H. (1997). "Influence of the Rigidity of Column Bases on the Ultimate Earthquake Resistance of Multi-Story Steel Moment Frames," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 496, pp. 113.

Yamada, S., Akiyama, H., and Sadamoto, M. (1997). "Influence of the Elasto-Plastic Behavior of Column Bases with Slip-Type Hysteresis Characteristics on the Ultimate Earthquake Resistance of Multi-Story Steel Moment Frames," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 502, pp. 141-147.

Yamada, S. (2000). "Influence of the Elasto-Plastic Behavior of Column Bases on the Ultimate Earthquake Resistance of Multi-Story Steel Moment Frames," *Proceedings of the Twelfth World Conference on Earthquake Engineering*, Auckland, New Zealand, January 30-February 4, 2000.

Appendix D: Supplemental Bibliography

American Concrete Institute (ACI) (1991), "State-of-the Art Report on Anchorage to Concrete," *(ACI, 355.1R-91)*, ACI, Farmington Hill, Michigan.

American Concrete Institute (ACI) (2001), "Code Requirements for Nuclear Safety Related Concrete Structures" *(ACI, 349-01)*, ACI, Farmington Hill, Michigan.

Astaneh, A. and Bergsma, G. (1993), "Cyclic Behavior and Seismic Design of Steel Base Plates," *Proceedings of Structures Congress*, ASCE, Vol. 1, pp. 409-414.

Balio, G. and Mazzolani, F. M. (1983), "Theory and Design of Steel Structures, 1st Ed", Chapman and Hall, New York, New York, pp. 257-264.

Fahmy, M., Stojadinovic, B., Goel, S. C., and Sokol, T. (1998), "Load Path and Deformation Mechanism of Moment Resisting Steel Column Bases," *Proceedings of the Sixth U.S. National Conference on Earthquake Engineering*, EERI, Seattle, Washington.

Fahmy, M., Stojadinovic, B., Goel, S. C., and Sokol, T. (1998), "Seismic Behavior of Moment Resisting Steel Column Bases," *Proceedings of the 11th European Conference on Earthquake Engineering,* Balkema, Rotterdam.

Fahmy, M., Stojadinovic, B., and Goel, S. C., (1999), "Analytical and Experimental Studies on the Seismic Response of Steel Column Bases," *Proceedings of the 8th Canadian Conference on Earthquake Engineering,* Vancouver, Canada.

Fisher, J. M., "Structural Details in Industrial Buildings," *Engineering Journal, AISC,* Third Quarter 1981, pp. 83-89.

Kawano, A., Matsui, C., and Takaki, Y. (1998). "Inelastic Response Behavior of Multi-Story Frame Considered Restoring Force Characteristics of Column-Base under Ground Motion," *Proceedings of the Tenth Earthquake Engineering Symposium*, Vol. 2, Paper No. G1-12, pp. 2347-2352, Tokyo, Japan.

Kawano, H, Kutani, K., and Masuda, K. (2003), "Experimental Studies on the Shear Resistance of Anchor Bolts in Exposed Type of Steel Column Bases," *Journal of Structural and Construction Engineering*, Transactions of AIJ, No. 567, May, 2003, pp. 141-148. (in Japanese)

Lee, D., Goel, S. C., and Stojadinovic, B. (2000), "Seismic Behavior of Column-Base Plate Connections about Weak Axis Bending," *Proceedings of the U.S.-Japan Workshop on Seismic Fracture Issues in Steel Structures*, San Francisco, California.

Lee, D. and Goel, S. C. (2001). "Seismic Behavior of Column-Base Plate Connections Bending about Weak Axis," *Report No. UMCEE 01-09*, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, Michigan.

Lee, D., Goel, S. C., and Stojadinovic, B. (2001), "Analytical Evaluation of Seismic Behavior of Column-Base Plate Connections under Weak Axis Bending in Moment Resisting Frames," *Proceedings of the First International Conference on Steel & Composite Structures*, Pusan, Korea.

Lee, D., Goel, S. C., and Stojadinovic, B. (2002), "Relative Strength Effects on Seismic Behavior of Column-Base Plate Connections under Weak Axis Bending," *Proceedings of the Seventh U.S. National Conference on Earthquake Engineering*, EERI, Boston, Massachusetts.

Melchers, R. E. (1987). "Modelling of Column-Base Behavior," *Proceedings of the First Conference on Connections in Steel Structures*, Cachan, France.

Midorikawa, M., Hasegawa, T., Mukai, A., Nishiyama, I., Fukuta, T., and Yamanouchi, H. (1997). "Damage Investigation of Steel Buildings in Specific Areas Observed from the 1995 Hyogoken-Nanbu Earthquake," *Proceedings of the U.S.-Japan Joint Workshop on Brittle Fracture of Steel Building Subject to Earthquakes*, San Francisco, California.

Nakashima, S, (1998). "Mechanical Characteristics of Exposed Portions of Anchor Bolts in Steel Column Bases under Combined Tension and Shear," *Journal of Constructional Steel Research,* 1998, Vol. 46, No.1-3, Paper No.277.

Ohi, K., Tanaka, H., and Takanashi, K. (1981). "Ultimate Strength of Steel Column Bases," *Bulletin No. 14, Earthquake Resistant Structure Research Center*, Tokyo, Japan.

Technical Council on Lifeline Earthquake Engineering (1995). *Northridge Earthquake – Lifeline Performance and Post-Earthquake Response*, Monograph No. 8, ASCE, New York, New York.

Wald, F. (1995). "Column-Bases, 1st ed.," *CVUT*, Praha, Czech Republic.

Appendix E: References[1]

American Concrete Institute (ACI) (2002), "Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)," *ACI*, Farmington Hill, Michigan.

American Institute of Steel Construction (AISC) (1999). "Load and Resistance Factor Design Specification for Structural Steel Buildings," *AISC*, Chicago, Illinois.

American Institute of Steel Construction (AISC) (2001). "Load and Resistance Factor Design Manual of Steel Construction," *3rd Ed., AISC*, Chicago, Illinois.

American Institute of Steel Construction (AISC) (2002). "Seismic Provisions for Structural Steel Buildings," *AISC*, Chicago, Illinois.

American Society of Civil Engineers (ASCE) (2002). "Minimum Design Loads for Buildings and Other Structures," *Publication No. ASCE 7-02, ASCE*, Reston, Virginia.

Architectural Institute of Japan (AIJ)(2001). "Recommendations for Design of Connections in Steel Structures", Tokyo (in Japanese)

Astaneh, A. and Bergsma, G. (1993). "Cyclic Behavior and Seismic Design of Steel Base Plates," *Proceedings of the ASCE Structures Congress 1993*, Vol. 1, ASCE, New York, New York, pp. 409-414.

Ballio, G. and Mazzolani, F. M. (1983). "Theory and Design of Steel Structures, 1st Ed," Chapman and Hall, New York, New York, pp. 257-264.

Blodgett, O. W. (1966). "Design of Welded Structures," *The James F. Lincoln Arc Welding Foundation,* Cleveland, Ohio, pp. 3.3-1 – 3.3-32.

Cochran, M.L. (2003a). "Seismic Design and Steel Connection Detailing," *Proceedings of North American Steel Construction Conference*, AISC, Baltimore, Maryland.

Cochran, M. L. (2003b). Personal communication to Daeyong Lee, NASCC 2003, Baltimore, Maryland.

Comité Européen de Normalisation (CEN) (1992). "ENV1993 Eurocode 3; Design of Steel Structures," Brussels, Belgium.

Comité Européen de Normalisation (CEN) (2003). "EN1993 Eurocode 3; Design of Steel Structures," Brussels, Belgium.

DeWolf, J. T., and Ricker, D. T. (1990). "Column Base Plates," *Steel Design Guide Series No. 1, AISC,* Chicago, Illinois.

Federal Emergency Management Agency (FEMA) (2000). "NEHRP Recommended Provisions for seismic Regulations for New Buildings and Other Structures," *Building Seismic Safety Council,* Washington, D.C.

Galambos, T.V. (1960). "Influence of Partial Base Fixity on Frame Stability," *Journal of the Structural Division,* May 1960.

Gaylord, E. H. and Gaylord, C. N. (1957). "Design of Steel Structures, 1st ed.," McGraw-Hill, New York, New York, pp. 304-315.

Gaylord, E. H. and Gaylord, C. N. (1972). "Design of Steel Structures, 2nd ed.," McGraw-Hill, New York, New York, pp. 479-485.

[1] When not listed in Appendices C or D.

McGuire, W. (1968). "Steel Structures, 1st ed.," Prentice-Hall, Inc., Englewood Cliffs, New Jersey, pp. 987-1004.

Northridge Reconnaissance Team (1996). "Earthquake Spectra, Northridge Earthquake of January 17, 1994," *Reconnaissance Report,* Supplement C-2, Volume 11, pp. 25-47.

Occupational Safety and Health Administration (OSHA) (2001). "Safety Standards for Steel Erection," *29 CFR Part 1926 and Subpart R of 29 CFR Part 1926, RIN No. 1218-AA65*, OSHA, U.S. Department of Labor.

Roeder, C. W., Chmielowski, R., and Brown, C. B. (1999). "Shear Connector Requirements for Embedded Steel Sections," *Journal of Structural Engineering*, ASCE, Vol. 125, No. 2, February 1999, pp. 142-151.

Saari, W., Hajjar, J. F., Schultz, A. E., and Shield, C. K. (2004). "Behavior of Shear Studs in Steel Frames with Reinforced Concrete Infill Walls," *Journal of Constructional Steel Research,* in press.

Soifer, H. (1966). "Design of Base Plates and Anchor Bolts with Simple Assumptions," *Civil Engineering*, ASCE, Vol. 36, No. 4, pp. 63.