IS YOUR STRUCTURE SUITABLY BRACED?

IS YOUR STRUCTURE SUITABLY BRACED?

1993 Conference

April 6-7, 1993
Milwaukee, Wisconsin

ORGANIZED BY:

Structural Stability Research Council
American Institute of Steel Construction
American Iron & Steel Institute
Metal Building Manufacturers Association

CO-SPONSORED BY:

National Center for Earthquake Engineering Research

EDITED BY:

James M. Ricles, Associate Director
Diana Walsh, Secretarial Assistant

ISBN 1-879749-55-6

STRUCTURAL STABILITY RESEARCH COUNCIL
Lehigh University
Fritz Engineering Laboratory
13 E. Packer Avenue
Bethlehem, PA 18015-3191 U.S.A.
PHONE (215) 758-3522
FAX (215) 758-4522

Printed in the United States of America

FOREWORD

"Is Your Structure Suitably Braced?"

A day and a half conference concerned with the bracing requirements for metal structures was held in Milwaukee, Wisconsin on April 6 and 7. This conference was an expansion of the half day theme session that has been a traditional part of the Annual Technical Session of the Structural Stability Research Council. The conference was conceived as a method of bringing the concerns, research and design information on the difficult topic of bracing design for metal structures to a wider professional audience than normally attends SSRC meetings. In this respect the conference was a great success.

There were over 170 persons who attended the conference, representing ten different countries. A large group of attendees were practitioners from Wisconsin and Northern Illinois. These were the type of engineers that the conference was intended to attract. They certainly received some good information on considerations in the design of bracing for metal structures.

The twenty papers in these Proceedings were presented in five sessions: two dealing with beams and the others with columns, building systems and frames. Special recognition is deserving to the keynote speakers for these sessions: *Joe Yura, Ted Galambos, Russ Bridge, Dick Kaehler and Bill Baker.* The knowledge of this impressive group of experts was put to test in the final session of the conference where practical questions from the audience were presented to them in a panel discussion, which is also summarized in these proceedings. Recognition and thanks is also due to the other contributing authors and co-authors for their efforts in preparing papers and making presentations at the conference.

At noon on Wednesday there was a conference luncheon. The speaker at the luncheon was Mike Tylk, a consultant from the Chicago area, who presented some thought-provoking situations in an entertaining manner under the title, *"The Dreaded Friday Afternoon Phone Call"*. This presentation also appears in the Proceedings. Thanks, Mike, for your time and effort.

The SSRC was assisted in this conference by four co-sponsors:

American Institute of Steel Construction
American Iron & Steel Institute
Metal Building Manufacturers Association
National Center for Earthquake Engineering Research

A special note of thanks is due to five local sponsors who contributed financially to support the conference:

CH2M Hill
Computerized Structural Design
Graef Anhalt Schloemer Associates
Howard Needles Tammen & Bergendoff
Society of Iron & Steel Fabricators of Wisconsin

The person who deserves the most recognition for this conference is Jerry Iffland, Chairman of the SSRC Finance Committee and President of Iffland Kavanagh Waterbury, P.C. in New York City. The whole concept of condensing the traditional SSRC meeting and expanding the theme session to a full conference was Jerry's. He also had the idea for the eye catching publicity brochure and his firm put forth the energy and expense in a large mailing. He was also the person who initially suggested Milwaukee as the meeting site. Thanks!

There were many people involved behind the scenes both before and during the conference to make it a successful event. I thank the active participants on the planning committee: Clarence Miller and Ramulu Vinnakota from the SSRC, Nestor Iwankiw of AISC, Gill Harris and Don Johnson from MBMA. The SSRC staff put forth considerable extra effort in planning and arranging for this conference, in addition to the requirements for the preceding SSRC meeting. For this we recognize: Lynn Beedle, Director; Jim Ricles, Associate Director; Lesleigh Federinic, Administrative Secretary; and Diana Walsh, Secretarial Assistant. Also, thanks to the students from Marquette University and the University of Wisconsin-Milwaukee for their efficient assistance during the conference.

Hopefully these Proceedings will become an important part of structural engineering literature. They contain some excellent information on the design of bracing systems for a variety of structural applications. Careful reading will also reveal alternate ways of considering a topic and some areas of controversy or question. These will undoubtedly become future topics of consideration by the Structural Stability Research Council.

D. R. Sherman

Donald R. Sherman
Chairman

Milwaukee, Wisconsin
April, 1993

TABLE OF CONTENTS

SESSION 4 BUILDING SYSTEMS

SESSION 5 FRAMES

THE DREADED FRIDAY AFTERNOON PHONE CALL

MICHAEL J. TYLK
TYLK, GUSTAFSON AND ASSOCIATES, INC.

The dreaded Friday afternoon phone call usually goes something like this "Mike, we've got a problem." Now these calls usually do not occur on Monday mornings or on Wednesdays or as a matter of course when any other engineers are in the office. No, they usually occur when you are alone in the office late on a Friday afternoon when every one else has gone home.

Leslie Robertson got a dreaded Friday afternoon phone call a few weeks ago. "Leslie, we have a problem over here at the World Trade Center, how soon can you be here?"

I'm going to talk today about three dreaded Friday afternoon phone calls which all happened to me on Fridays.

The first dreaded Friday afternoon phone call came in the winter of 1979, I had been in business by myself for about three years and had designed a typical strip shopping center which had opened in October of that year. It had been snowing all day. Matter of fact, it had been snowing since late the night before. There was at least 18 inches of new snow on the ground and there had been 6 to 8 inches on the ground before this storm had started. Dave (not his real name) the architect for the project had called me to say "Mike, your roof is collapsing" I replied what do you mean, <u>my</u> roof is collapsing.

You will notice that when there is a problem, the pronouns get changed. Not, "the roof" or "my roof" or even "our roof", but "your roof". Dave went on to say that "my welds were failing on my bar joists" and "How soon can you get there". My heart was in my throat as you can imagine. I told him I would leave right away and be there as soon as I could.

Now at this time I'd like to fill you in on two other phone calls that I had gotten from Dave earlier in the day. The first one was late morning, "Mike, Joe the building manager out at the Commons is concerned about the amount of snow on the roof and wants to know if it is Okay to put a 2500 pound bobcat up there to push the snow off". I replied "is he nuts?" "the roof might hold it, but it will tear up the built up tar and gravel roof. "NO" I told him, "No bobcat, the roof was designed to meet the building code and it would be okay. Secretly, I wished that I had selected one bar joist size larger, but I eluded nothing but confidence that the roof was okay.

The second call from Dave that day came just after lunch. Joe wasn't happy about leaving the snow alone on the roof when Dave called him back to tell him "No bobcat" So Dave suggested that they get a snow blower up there and blow the snow off. He was real proud of himself, he had told Joe that if he started in the middle of the roof he could blow the snow both ways to the edge and eventually off. He was calling me to confirm what he had told Joe. "Dave" I said, "the roof is 100 feet by 700 feet, we have three 33'-4 bays in the 100 foot direction, if you start in the middle you'd be lucky to blow the snow 15 to 20 feet and eventually you will have twice as much snow in the outside bays. Then it may collapse." I again re-emphasized that the roof design was adequate and not to worry. Then later I got that dreaded Friday afternoon phone call. So with my heart in my throat I left for the building. Because of the weather it took me about an hour and a half to get there and all along the way I went over the design of the building in my head. Did I make a mistake? Would this end my career? I relaxed a little bit when I turned the corner and first saw the building. It was still standing. I headed over to the area where there was a crowd of people. I identified myself and went inside the building and this is what I found.

The roof construction was typical bar joists with horizontal bridging bearing on rolled steel beams or girders. However at this location a concrete block wall had been built separating a maintenance area from a beauty shop. The wall had been built tight to the roof deck and the bridging angles were mortared in tight. In the maintenance area a sprinkler pipe had been installed between the joist and the wall. In order to get the pipe in, the sprinkler fitter had cut the lower bridging angle and later welded it back. Now with two feet of snow on the roof, the bar joist were deflecting as can be expected. However the concrete block wall prevented the closest bar joist and the bridging angles to deflect and eventually the weld where the sprinkler fitter had replaced the bridging angle snapped.

Evidently the noise was like a rifle shot with a subsequent thud. The ladies in the beauty shop on the other side of the wall were alarmed and called the maintenance man who found the broken weld and called the owner, who in turn called the architect, who then called me. This was somewhat like the game you play at parties where you whisper something into a persons ear at one end of a line and they in turn repeat it to the next person and so on. By the time it gets to the end of the line it doesn't resemble anything like what you told the first person. One broken weld on a bridging angle turned into the welds breaking and the roof collapsing.

I instructed them to cut holes around the bridging angles in the wall to allow for deflection and weld the bridging angle back together. The hole in the wall around the bridging angle also had to be stuffed with fireproofing material.

I spent the next hour walking around inside and outside of the building to check if everything else was okay. It was. I was very tempted to chastise the owner and architect for putting me through some anxious moments. The hour and a half drive to the building was the worst. However I bit my tongue, smiled, shook hands and left.

The second dreaded Friday afternoon phone call came in October of 1981. George Wright, my late partner, was in the hospital having had a mild heart attack. George had been working on an erection procedure for a bridge over the Chicago Sanitary and Ship Canal. Our client, a steel erector called late on a Friday afternoon and said "Mike, first thing in the morning I am going to drive my Manitowoc 3900 on to a barge and I wonder if I'm going to tip it over."

It sounded like a simple problem of statics. Just like a footing with an eccentric load. Is the resultant in or out of the Kern? I asked the client for the weights and sizes of the crane and barge and told him I'd call him back in a little while. I then started to work out my little statics problem.

All of a sudden I realized that I didn't know what I was doing. A Masters Degree in Architectural Engineering from the University of Illinois, ten years at Skidmore, Owings and Merrill and five years in business for myself and suddenly here was a problem that I had never encountered before. The worse part of it was that it sounded like such a simple problem that I told the client that I'd call him back in a little while. Trying not to think of what might happen, I started searching through our library looking for a text book that could help me and solve this simple little problem. After striking out with Gaylord & Gaylor, Roark, Timosheko, and Seely & Smith, I tried Peck Hanson and Thornburn, Wayne Tèng and Terzaghi and Peck. Nothing. Now these books had never failed me before. By the time I set down the last Hool and Kinne book we had in the office I was in a definite panic. I was about to lose one of my partners best clients.

Then I realized that George must have solved this problem dozens of times, having been with American Bridge for 15 years. I then went to what he referred to as his bible, American Bridges' "Construction Engineering Handbook" and under the section "Major Equipment" found a section called "Boats and Barges", and some sample problems under the heading of "List of Floating Craft". I was home free, I got out my calculator and worked the sample problems and then worked out my problem.

It would work, the barge was heavy enough that the crane could drive on to the end of it with out it tipping over. So I called our client back apologizing that a "little while" had taken two and a half hours. After I told him that the crane would not tip over the barge when he drove it on the next morning, he replied "good but just in case I am going to lash the tug boat to the other end of the barge." I didn't know he had a tug boat!

The last dreaded Friday afternoon phone call that I am going to talk about today came in June of 1991. A three story condominium building with parking in the basement was located approximately one mile from our office. We had done some minor consulting work on other buildings in the area and knew the local real estate agent and a local maintenance contractor real well. The maintenance contractor had been called by a woman tenant on the third floor of the building. "One of my doors doesn't close anymore" was her complaint. So Joel, the maintenance contractor, sent over one of his guys who planed down the bottom of the door. Two weeks later the tenant called Joel back to complain again that her door didn't close anymore. This time Joel himself went back and planed the door down again. Now one week later she called again "Her door didn't close". Now Joel thought that the first time maybe his guy didn't do a good job, but the second time he knew he had done it right himself so he called the real estate agent who managed the building for the Condominium Association. Joel told Jack the Real Estate agent that there was something seriously wrong and recommended that we be called in to investigate. Jack called us. I had left for home already, but Ed Rahe, one of our Associate Partners had made the mistake of being the last one to leave the office. He got the dreaded Friday afternoon phone call. After being there only a few minutes, Ed called one of our other Associate Partners, George Marrs, who lived nearby and explained the problem. George replied "call some ironworkers" and "tell them to bring some steel columns and steel plates."

This is what they found; A steel pipe column in the parking garage in the basement was leaning approximately 5" in its 8 foot height. There was severe buckling of the column near the floor. The W16 beam it was supporting had deformed into a parallelogram having been welded on the top to weld plates in the precast slab it was supporting.

A decision was quickly reached to evacuate the building until temporary repairs could be made. The ironworkers arrived and using some W6 columns and 3/4 inch steel plates installed a temporary fix. One W6 column was placed on each side of the pipe column and 3/4 inch steel plates were welded into the web of the W16 beam. This work was completed by 3 o'clock Saturday morning and by Saturday afternoon, after we determined that the building was not going to collapse, the tenants were allowed back into the building.

And now as Paul Harvey would say, "This is the rest of the story"

The building was only about 12 years old. The steel pipe columns had spray on fireproofing on them, and a light gage metal cover surrounded the fireproofing. The sheet metal cover kept the fireproofing from being knocked off by tenants brushing past them after parking their cars. Also the basement flooded periodically. Not a lot of water, six to eight inches at the most. But the fireproofing acted like a wick and the cover prevented it from drying out. This combined with the road salts coming off the cars had caused the column to severely corrode. Within 12 years the loss of section was in some places in excess of 3/16 of an inch, and the column which only had a wall thickness of .28 inches started to collapse upon itself. On this particular column the loss of section was not symmetrical and thus the column leaned. But on others we found the column uniformly collapsed like an accordion until there was enough resistance to hold the load.

After the temporary fix, we inspected the entire building. Some columns had compressed up to 2 inches. We then designed a shoring system that allowed us to remove all of the distressed columns and beams, jack the building back up, and replace all of the steel beams and columns. For replacement we used double extra strong columns which have a wall thickness of .864 inches.

The lesson learned is leave early on Friday afternoons, and let some one else get the dreaded Friday afternoon phone call.

FUNDAMENTALS OF BEAM BRACING

Joseph A. Yura
University of Texas at Austin

Introduction

The purpose of this paper is to provide a fairly comprehensive view of the subject of beam stability bracing. Factors that affect bracing requirements will be discussed and design methods proposed which are illustrated by design examples. The design examples emphasize simplicity. Before going into specific topics related to beam bracing, some important concepts developed for column bracing by Winter (1960) will be presented because these concepts will be extended to beams later.

For a perfectly straight column with a midheight brace stiffness β_L, the relationship between P_{cr} and β_L is shown in Fig. 1 (Timoshenko, 1961). The column buckles between brace points at full or ideal bracing; in this case the ideal brace stiffness $\beta_i = 2\ P_e/\ L_b$ where $P_e = \pi^2\ EI/L_b^2$. Any brace with a stiffness up to the ideal value will significantly increase the column buckling load. Winter (1960) showed that effective braces require not only adequate stiffness but also sufficient strength. The strength requirement is directly related to the magnitude of the initial out-of-straightness of the member to be braced.

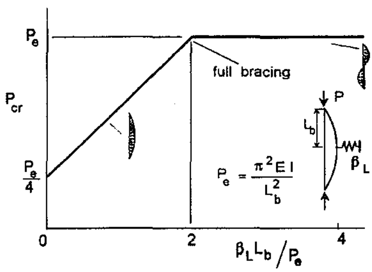


Fig. 1 Effect of Brace Stiffness

For a column with an initial out-of-straightness (half sine curve) with a displacement Δ_o at midheight and a midheight brace stiffness equal to the ideal value, the heavy solid line in Fig. 2(a) shows the relationship between Δ_T and P. For P = 0, $\Delta_T = \Delta_o$. When P increases and approaches the buckling load, π^2 EI/L_b^2, the total deflection Δ_T becomes very large. For example, when the applied load is within 5% of the buckling load, $\Delta_T = 20\Delta_o$. If a brace stiffness twice the value of the ideal stiffness is used, much smaller deflections occur. When the load just reaches the buckling load, the $\Delta_T = 2\Delta_o$. For $\beta_L = 3\beta_i$ and $P = P_e$, $\Delta_T = 1.5\Delta_o$. The brace force, F_{br}, is equal to $(\Delta_T - \Delta_o)\beta_L$ and is directly related to the magnitude of the initial imperfection. If a member is fairly straight, the brace force will be small. Conversely, members with large initial out-of-straightness will require larger braces. If the brace stiffness is equal to the ideal value, then the brace force gets very large as the buckling load is approached because Δ_T gets very large as

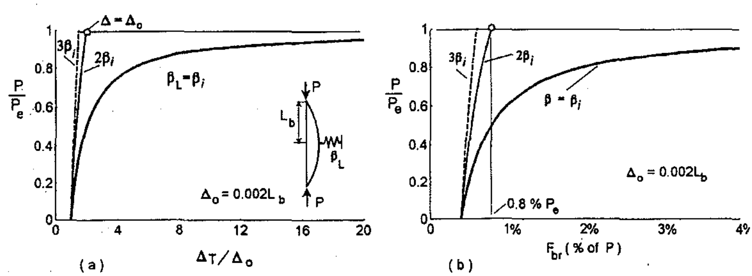


Fig. 2 Braced Column with Initial Out-of-Straightness

shown in Fig. 2(a). For example, at $P = 0.95P_{cr}$ and $\Delta_o = L_b / 500$, the brace force is 7.6% of P_e which is off the scale of the graph. Theoretically the brace force will be infinity when the buckling load is reached if the ideal brace stiffness is used. Thus, a brace system will not be satisfactory if the theoretical ideal stiffness is provided because the brace forces get too large. If the brace stiffness is overdesigned, as represented by the $\beta_L = 2\beta_i$ and $3\beta_i$ curves in Fig. 2(b), then the brace forces will be more reasonable. For a brace stiffness twice the ideal value and a $\Delta_o = L_b / 500$, the brace force is only 0.8%P_e at $P = P_e$, not infinity as in the ideal brace stiffness case. For a brace stiffness ten times the ideal value, the brace force will reduce even further to 0.44%. The brace force cannot be less than 0.004P corresponding to $\Delta = 0$ (an infinitely stiff brace) for $\Delta_o = L_b / 500$. For columns Yura (1971) showed that the brace force could conservatively be taken as 0.008 of the column load. This force is based on a brace stiffness at least twice the ideal value and an initial out-of-straightness of $L_b / 500$.

Published bracing requirements for beams usually only consider the effect of brace stiffness because perfectly straight beams are considered. Such solutions should not be used directly in design. Similarly, design rules based on strength considerations only, such as a 2% rule, can result in inadequate bracing systems. Both strength and stiffness of the brace system must be checked.

Beam Bracing Systems

Beam bracing is a much more complicated topic compared to column bracing. This is due mainly to the fact that most column buckling involves primarily bending whereas beam buckling involves both flexure and torsion. An effective beam brace resists twist of the cross section. In general bracing may be divided into two main categories, lateral and torsional bracing as illustrated in Fig. 3. Lateral bracing restrains lateral displacement as its name implies. The effectiveness of a lateral brace is related to the degree that twist of the cross section is restrained. For a simply supported beam subjected to uniform moment, the center of twist is located at a point outside the tension flange; the top flange moves laterally much more than the bottom flange. Therefore, a lateral brace restricts twist best when it is located at the top flange. Lateral bracing attached at the bottom flange of a simply supported beam is almost totally ineffective. A torsional brace can be differentiated from a lateral brace in that twist of the cross section is restrained directly, as in the case of twin beams with a cross frame or diaphragm between the members. The cross frame location, while able to displace laterally, is still considered a brace point because twist is prevented. Some systems such as concrete slabs can act both as lateral and torsional braces. Bracing that controls both lateral movement and twist is more effective than lateral or torsional braces acting alone (Tong and Chen, 1988; Yura, 1992). However, since bracing requirements are so minimal, it is more practical to develop separate design recommendations for these two types of systems.

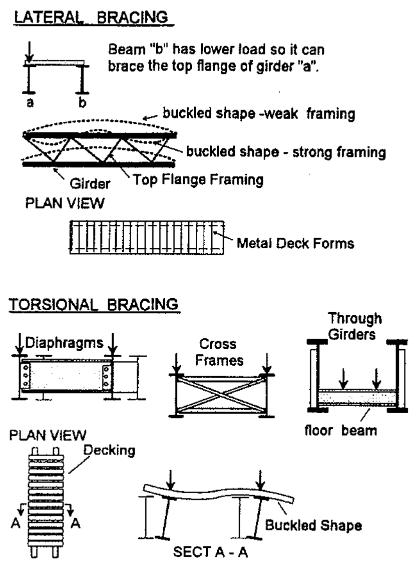


Fig. 3 Types of Beam Bracing

Lateral bracing can be divided into four categories: relative, discrete, continuous and lean-on. A relative brace system controls the relative lateral movement between two points along the span of the

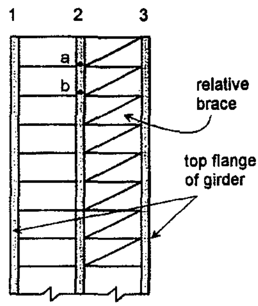


Fig. 4 Relative Bracing

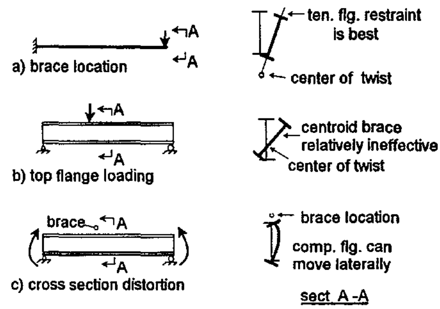


Fig. 5 Factors That Affect Brace Effectiveness

girder. The top flange horizontal truss system shown in Fig. 4 is an example of a relative brace system. The system relies on the fact that if the girders buckle laterally, points **a** and **b** would move different amounts. Since the diagonal brace prevents points a and b from moving different amounts, lateral buckling cannot occur except between the brace points. Typically, if a perpendicular cut anywhere along the span length passes through one of the bracing members, the brace system is a relative type. Discrete systems can be represented by individual lateral springs along the span length. Temporary guy cables attached to the top flange of a girder during erection would be a discrete bracing system. A lean-on system relies on the lateral buckling strength of lightly loaded adjacent girders to laterally support a more heavily loaded girder when all the girders are horizontally tied together. In a lean-on system all girders must buckle simultaneously. In continuous bracing systems, there is no "unbraced" length. In this paper only relative and discrete systems that provide full bracing will be considered. Design recommendations for lean-on systems and continuous lateral bracing are given elsewhere (Yura, 1992,1993). Torsional brace systems can be discrete or continuous as shown in Fig. 3. Both types are considered herein.

Some of the factors that affect brace design are shown in Fig. 5. A lateral brace should be attached where it best offsets the twist. For a cantilever beam in (a), the best location is the top tension flange , not the compression flange. Top flange loading reduces the effectiveness of a top flange brace because such loading causes the center of twist to shift toward the top flange as shown in (b). Larger lateral braces are required for top flange loading. If cross members provide bracing above the top flange, case (c),the compression flange can still deflect laterally if cross-section is not prevented by stiffeners. In the following sections the effect of loading conditions, load location. brace location and cross-section distortion on brace requirements will be presented. All the cases considered were solved using the elastic finite element program BASP (Akay, 1977; Choo, 1987) which considers local and lateral-torsional buckling including cross-section distortion. The BASP program will handle many types of restraints including lateral and torsional braces at any node point along the span along with transverse and longitudinal stiffeners. The solutions and the design recommendations presented are consistent with the work of others: Kirby and Nethercot (1979), Linder and Schmidt (1982), Medland (1980), Milner (1977), Nakamura (1981, 1988), Nethercot (1989), Taylor and Ojalvo (1966), Tong and Chen (1988), Trahair and Nethercot (1982), Wakabayashi (1983), and Wang and Nethercot (1989).

Lateral Bracing of Beams

Behavior. The uniform moment condition is the basic case for lateral buckling of beams. If a lateral brace is placed at the midspan of such a beam, the effect of different brace sizes (stiffness) is illustrated by the BASP solutions for a W16x26 section 20 ft long in Fig. 6. For a brace attached to the top (compression) flange, the beam buckling capacity initially increases almost linearly as the brace stiffness increases. If the brace stiffness is less than 1.6 k/in., the beam buckles in a shape resembling a half sine curve. Even though there is lateral movement at the brace point, the load increase can be more than three times the unbraced case. The ideal brace stiffness required to force the beam to buckle between lateral supports is 1.6 k/in. in this example. Any brace stiffness greater than this value does not increase the beam buckling capacity and the buckled shape is a full sine curve. When the brace is attached at the top flange, there is no cross section distortion. No stiffener is required at the brace point.

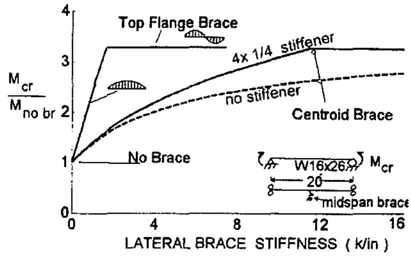


Fig. 6 Effect of Lateral Brace Location

A lateral brace placed at the centroid of the cross section requires an ideal stiffness of 11.4 k/in. if a 4 x 1/4 stiffener is attached at midspan and 53.7 k/in. (off scale) if no stiffener is used. Substantially more bracing is required for the no stiffener case because of web distortion at the brace point. The centroid bracing system is less efficient than the top flange brace because the centroid brace force causes the center of twist to move above the bottom flange and closer to the brace point which is undesirable for lateral bracing.

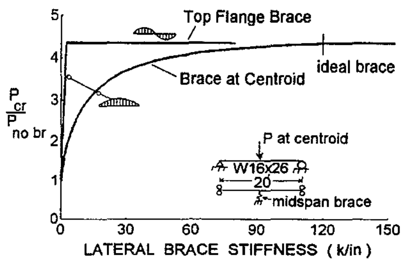


Fig. 7 Midspan Load at Centroid

For the case of a beam with a concentrated centroid load at midspan, shown in Fig. 7, the moment varies along the length. The ideal centroid brace (110 k/in.) is 44 times larger than the ideal top flange brace (2.5 k/in.). For both brace locations cross section distortion had a minor effect (<3%). The maximum beam moment at midspan when the beam buckles between the braces is 1.80 times greater than the uniform moment case which is close to the C_b factor = 1.75 given in specifications (AISC, AASHTO). This higher buckling moment is the main reason why the ideal top flange brace requirement is 1.56 times greater (2.49 vs. 1.6 k/in.) than the uniform moment case.

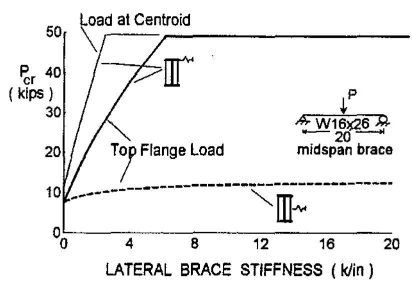


Fig. 8 Effect of Load and Brace Position

Figure 8 shows the effects of load and brace position on the buckling strength of laterally braced beams. If the load is at the top flange, the effectiveness of a top flange brace is greatly reduced. For example, for a brace stiffness of 2.5 k/in., the beam would buckle between the ends and the midspan brace at a centroid load close to 50 kips. If the load is at the top flange, the beam will buckle at a load of 28 kips. For top flange loading, the ideal top flange brace would have to be increased to 6.2 k/in. to force buckling between the braces. The load position effect must be considered in the brace design requirements. This effect is even more important if the lateral brace is attached at the centroid. The results shown in Fig. 8 indicate that a centroid brace is almost totally ineffective for top flange loading. This is not due to cross section distortion since a stiffener was used at the brace point. The top flange loading causes the center of twist at buckling to shift to a position close to mid-depth for most practical unbraced lengths, as shown in Fig. 5. Since there is virtually no lateral displacement near the centroid for top flange loading, a lateral brace at the centroid will not brace the beam. Because of cross-section distortion and top flange loading effects, lateral braces at the centroid are not recommended. Lateral braces must be placed near the top flange of simply supported and overhanging spans. Design recommendations will be developed only for the top flange lateral bracing situation. Torsional bracing near the centroid or even the bottom flange can be effective as discussed later.

The load position effect discussed above assumes that the load remains vertical during buckling and passes through the plane of the web. In the laboratory, a top flange loading condition is achieved by loading through a knife edge at the middle of the flange. In structures the load is applied to the beams through secondary members or the slab itself. Loading through the deck can provide a beneficial "tipping" effect illustrated in Fig. 9. As the beam tries to buckle, the contact point shifts from mid-flange to the flange tip resulting in a restoring torque which increases the buckling capacity. Unfortunately, cross- section distortion severely limits the beneficial effects of tipping. Linder (1982, in German) has developed a solution for the tipping effect which considers the flange-web distortion. The test data (Linder,1982; Raju, 1992)indicates that a cross member merely resting (not positively attached) on the top flange can significantly increase the lateral buckling capacity. The tipping solution is sensitive to the initial shape of the cross section and location of the load point on the flange. Because of these difficulties, it is recommended that the tipping effect not be considered in design.

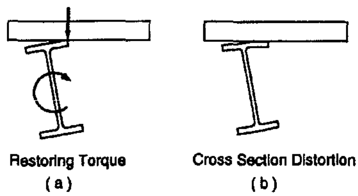


Fig. 9 Tipping Effect

When a beam is bent in double curvature the compression flange switches from the top flange to the bottom flange at the inflection point. Beams with compression in both the top and bottom flanges along the span have more severe bracing requirements than beams with compression on just one side as illustrated by the comparison of the cases given in Fig. 10. The solid lines are BASP solutions for a 20 ft long W16x26 beam subjected to equal but opposite end moments and with lateral bracing at the midspan inflection point. For no bracing the buckling moment is 1350 in-k. A brace attached to one flange is ineffective for reverse curvature because twist at midspan is not prevented. If lateral bracing is attached to both flanges, the buckling moment increases nonlinearly as the brace stiffness increases to 24 k/in, the ideal value shown by the black dot. Greater brace stiffness has no effect because buckling occurs between the brace points. The ideal brace stiffness for a beam with a concentrated midspan load is 2.6 k/in at M_{cr} = 2920 in-k as shown by the dashed lines. For the two load cases the moment diagrams between brace points are similar, maximum moment at one end and zero moment at the other end. In design a C_b = 1.75 is used for these cases which corresponds to an expected maximum moment of 2810 in-k. The double curvature case reached a maximum moment 25% higher because of warping restraint at midspan provided by the adjacent tension flange. In the concentrated load case no such restraint is available since the compression flanges of both unbraced segments are adjacent to each other. On the other hand, the brace stiffness at each flange must be 9.2 times the ideal value of the concentrated load case to achieve the 25 % increase. Since warping restraint is usually ignored in design M_{cr} = 2810 in-k is the maximum

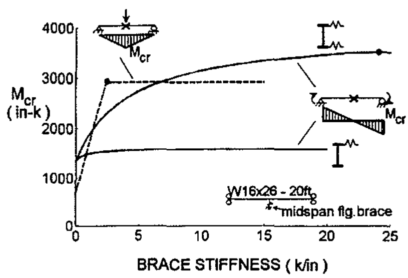


Fig. 10 Beams with Inflection Points

design moment. At this moment level the double curvature case requires a brace stiffness of 5.6 k/in which is about twice that required for the concentrated load case. The results in Fig. 10 show that not only is it incorrect to assume that an inflection point is a brace point but also that bracing requirements for beams with inflection points are greater than cases of single curvature. For other cases of double curvature such as uniformly loaded beams with end restraint (moments), the observations are similar.

Up to this point only beams with a single midspan lateral brace have been discussed. The bracing effect of a beam with multiple braces is shown in Fig. 11. The response of a beam with three equally spaced braces is shown by the solid line. When the lateral brace stiffness, β_L, is less than 0.14 k/in., the beam will buckle in a single wave. In this region a small increase in brace stiffness greatly increases the buckling load. For $0.14 < \beta_L < 1.14$, the buckled shape switches to two waves and the relative effectiveness of the lateral brace is reduced. For $1.4 < \beta_L < 2.75$, the bucked shape is three waves. The ideal brace stiffness is 2.75 k/in. at which the unbraced length can be considered 10 ft. For the 20 ft span with a single brace at midspan discussed previously which is shown by the dashed line, a brace stiffness of only 1.6 k/in. was required to reduce the unbraced length to 10 ft. Thus the number of lateral braces along the span affects the brace requirements. A similar behavior has been derived for columns (Timoshenko and Gere, 1961) where changing from one brace to three braces required an increase in ideal column brace stiffness of 1.71, which is the same as that shown in Fig. 9 for beams, 2.75/1.6 = 1.72.

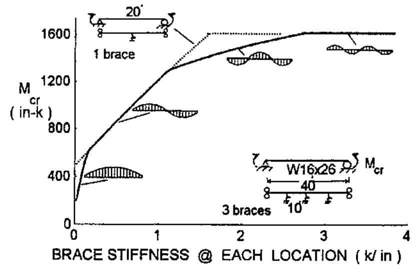


Fig. 11 Multiple Lateral Bracing

Yura and Phillips (1992) report the results of a test program on the lateral and torsional bracing of beams for comparison with the theoretical studies presented above. Some typical test results show good correlation with the BASP theory in Fig. 12. Since the theoretical results were found to be reliable, significant variables from the theory were included in the development of the design recommendations given in the following section. In summary, moment gradient, brace location, load location, brace stiffness and number of braces affect the buckling strength of laterally braced beams. The effect of cross section distortion can be effectively eliminated by placing the lateral brace near the top flange.

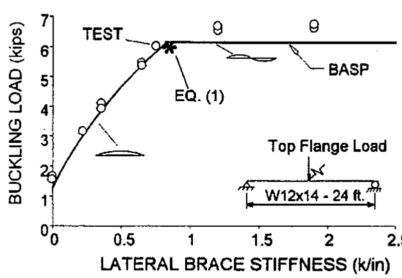


Fig. 12 Lateral Bracing Tests

Lateral Brace Design. In the previous section it was shown that the buckling load increases as the brace stiffness increases until full bracing causes the beam to buckle between braces. In many instances the relationship between bracing stiffness and buckling load is nonlinear as evidenced by the response shown in Fig. 11 for multiple braces. A general design equation has been developed for braced beams which is gives good correlation with exact solutions for the entire range of zero bracing to full bracing (Yura, 1992b). That braced beam equation is applicable to both continuous and discrete bracing systems, but it is fairly complicated. In most design situations full bracing is assumed or desired, that is, buckling between the brace points is assumed . For full bracing a simpler design alternative based on Winter's approach was developed (Yura, 1992b) and is presented below.

For elastic beams under uniform moment the Winter ideal lateral brace stiffness required to force buckling between the braces is $\beta_i = \#P_f / L_b$ where $P_f = \pi^2 EI_{yc} / L^2_b$, I_{yc} is the out-of-plane moment of inertia of the compression flange which is $I_y/2$ for doubly symmetric cross sections, and # is a coefficient depending on the number of braces n within the span, as given in Table 1(Winter, 1960) or approximated by $\# = 4 - (2/n)$. The C_b factor given in design specifications for nonuniform moment diagrams can be used to estimate the increased brace requirements for other loading cases. For example, for a simply supported beam with a load and brace at midspan shown in Fig. 7, the full bracing stiffness required is 1.56 times greater than the uniform moment case. The $C_b = 1.75$ for this loading case provides a conservative estimate of the increase. An additional modifying factor $C_d = 1 + (M_S / M_L)^2$ is required when there are inflection points along the span (double curvature), where M_S and M_L are the maximum moments causing compression in the top and bottom flanges as shown in Fig. 13. The moment ratio must be equal to or less than one, so C_d varies between 1 and 2. In double curvature cases lateral braces must be attached to both flanges. Top flange loading increases the brace requirements even when bracing is provided at the load point. The magnitude of the increase is affected by the number of braces along the span as given by the modifying factor $C_L = 1 + (1.2/n)$. For one brace $C_L = 2.2$; for many braces top flange loading has no effect on brace requirements, i.e. $C_d = 1.0$.

Table 1. Brace Coefficient

Number of Braces	Brace Coef.
1	2
2	3
3	3.41
4	3.63
Many	4.0

In summary, a modified Winter's ideal bracing stiffness can defined as follows,

$$\beta_i^* = \frac{\# C_b P_f}{L_b} C_L C_d \qquad (1)$$

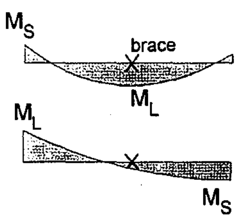


Fig. 13 Double Curvature

For the W12x14 beams laterally braced at midspan shown in Fig. 12, $L_b = 144$ in., $\# = 2$, $C_b = 1.75$, $C_L = 1 + 1.2/1 = 2.2$, and $P_f = \pi^2 (29000)(2.32/2)/(144)^2 = 16.01$ kips, $\beta_i^* = 0.856$ k/in. which is shown by the * in Fig. 12. Equation (1) compares very favorably with the test results and with the theoretical BASP results. For design the ideal stiffness given by Eq. (1) must be doubled for beams with initial out-of-straightness so brace forces can be maintained at reasonable levels as discussed earlier. The brace force requirement for beams follows directly from the column $F_{br} = 0.008P$ for discrete braces given earlier. The column load P is replaced with the equivalent compressive beam flange force, either $(C_b P_f)$ or M_f/h, where M_f is the maximum beam moment and h is the distance between flange centroids. The M_f/h estimate of the flange force is applicable for both the elastic and inelastic regions. For relative bracing the force requirement is one half the discrete value. The lateral brace design recommendations which follow are based on an initial out-of-straightness of adjacent brace points of $L_b/500$. The combined

LATERAL BRACING DESIGN REQUIREMENTS

Stiffness: $\beta_L^* = 2\,\#\,(C_b P_f)\, C_L\, C_d / L_b \quad or \quad 2\,\#\,(M_f / h)\, C_L\, C_D / L_b$ (2)

where $\#$ = $4 - (2/n)$ or the coefficient in Table 1 for discrete bracing; = 1.0 for relative bracing

$C_b P_f$ = $C_b \pi^2 E I_{yc} / L_b^2$; or = (M_f / h) where M_f is the maximum beam moment

C_L = $1 + (1.2/n)$ for top flange loading; = 1.0 for other loading

C_d = $1 + (M_S / M_L)^2$ for double curvature; = 1.0 for single curvature

n = number of braces

Strength: Discrete bracing: $F_{br} = 0.008\, C_L\, C_d\, M_f / h$ (3)

Relative bracing: $F_{br} = 0.004\, C_L\, C_d\, M_f / h$ (4)

values of # and $C_L$vary between 4.0 and 4.8 for all values of n so Eq. (2) can be conservatively simplified for all situations to $\beta_L^* = 10\ M_f / h$ for single curvature and $\beta_L^* = 20\ M_f / h$ for double curvature.

Some adjustments to the design requirements are necessary to account for the different design code methodologies, i.e. allowable stress design, load factor design. etc.. In AASHTO-LFD and AISC-LRFD, M_f is the factored moment; in Allowable Stress Design, M_f is based on service loads. The C_bP_f form of Eq (2) can be used directly for all specifications because it is based on geometric properties of the beam, i.e., $\beta_L \geq \beta_L^*$ where β_L is the brace stiffness provided. The brace strength requirements, Eqs. (3) and (4), can also be used directly since the design strengths or resistances given in each code are consistent with the appropriate factored or service loads. Only the M_f / h form of Eq. (2) which relies on the applied load level used in the structural analysis must be altered as follows:

AISC-LRFD:	$\beta_L \geq \beta_L^* / \phi$	where $\phi = 0.75$ is suggested
AISC-ASD:	$\beta_L \geq 2\,\beta_L^*$	where 2 is a safety factor
AASHTO-LFD:	$\beta_L \geq \beta_L^*$	no change

The discrete and relative lateral bracing requirements are illustrated in the following two design examples.

Lateral Brace Design Examples. Two different lateral bracing systems are used to stabilize five composite steel plate girders during bridge construction; a discrete system in Example 1 and a relative bracing in Example 2. The AASHTO- Load Factor Design Specification is used. Each brace shown dashed in Example 1 controls the lateral movement of one point along the span, whereas the diagonals in the top flange truss system shown in Example 2 control the relative lateral displacement of two adjacent points. Relative systems require 1/2 the brace force and from 1/2 to 1/4 of the stiffness for discrete systems. In both examples, a tension type structural system was used but the bracing formulas are also applicable to compression systems such as K-braces. In Example 1 the full bracing requirements for strength and stiffness given by Eqs. (2) and (3) are based on each brace stabilizing five girders. Since the moment diagram gives compression in one flange, C_d for double curvature is not considered.

In both examples, stiffness controls the brace area, not the strength requirement. In Example 1 the stiffness criterion required a brace area 3.7 times greater than the strength formula. Even if the brace was designed for 2% of the compression flange force (a commonly used bracing rule), the brace system would be inadequate. It is important to recognize that <u>both</u> stiffness and strength must be adequate for a satisfactory bracing system.

Torsional Bracing of Beams

Examples of torsional bracing systems were shown in Fig. 3. Twist can be prevented by attaching a deck to the top flange of a simply supported beam, by floor beams attached near the bottom tension flange of through girders or by diaphragms located near the centroid of the stringer. Twist can also be restrained by cross frames that prevent the relative movement of the top and bottom flanges. The effectiveness of torsional braces attached at different locations on the cross section will be presented.

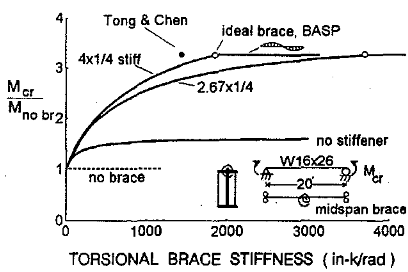
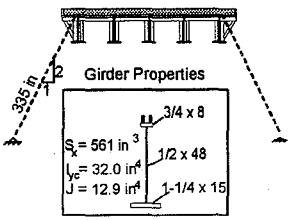


Fig. 14 Torsional Brace at Midspan

Behavior. The BASP solution for a simply supported beam with a top flange torsional brace attached at midspan is shown in Fig. 14. The buckling strength - brace stiffness relationships are non-linear and quite different from the top flange lateral bracing linear response given in Fig. 6 for the same beam and loading. For top flange lateral bracing a stiffener has no effect. A torsional brace can only increase the buckling capacity about fifty percent above the unbraced case if no stiffener is used. Local cross-section distortion at midspan reduces the brace effectiveness. If a web stiffener is used with the torsional brace attached to the compression flange, then the buckling strength will increase until buckling occurs between the braces at 3.3 times the no-brace case. The ideal or full bracing requires a stiffness of 1580 in-k/radian for a 4 x 1/4 stiffener and 3700 in-k/radian for a 2.67 x 1/4 stiffener. Tong and Chen (1988) developed a closed form solution for ideal torsional brace stiffness neglecting cross-section distortion that is given by the solid dot at 1450 in-k/radian in Fig. 14. The difference between the Tong solution and the BASP results is due to web distortion . Their solution would require a 6 x 3/8 stiffener to reach the maximum buckling load. If the Tong ideal stiffness (1450 in-k/radian) is used with a 2.67 x 1/4 stiffener, the buckling load is reduced by 14%; no stiffener gives a 51% reduction.

LATERAL BRACING - DESIGN EXAMPLE 1



Span = 80 ft. ; 10 in. concrete slab

5 girders @ 8 ft spacing, A36 steel

Design a lateral bracing system to stabilize the girders during the deck pour. Use the external tension system shown. The form supports transmit some load to the bottom flange so assume centroid loading.

Use Load Factor Design for the construction condition - L.F. = 1.3

Loads: Steel girder: $A = 48.75\ \text{in.}^2$, wt = 165 lb/ft

Conc. slab: $8' \times \frac{10}{12} \times 150\ \text{lb/ft}^3 = \underline{1000\ \text{lb/ft}}$

$w = 1165\ \text{lb/ft} = 1.165\ \text{k/ft}$

$$M = \frac{1}{8} w L^2 \times \text{L.F.} = \frac{1}{8}(1.165)(80)^2 1.3 = 1211\ \text{k-ft}$$

$$M_y = 36\,(561)/12 = 1682\ \text{k-ft} > 1211\ \text{k-ft}$$

Try 4 lateral braces @ 16-ft spacing

Check lateral buckling - center 16-ft is most critical (AASHTO 10-102c)

$$M = 91 \times 10^6 (1.0) \frac{32.0}{16 \times 12} \sqrt{0.772 \frac{12.9}{32.0} + 9.87 \left(\frac{50}{16 \times 12}\right)^2}$$

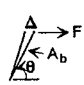
$$= 15020000\ \text{lb-in} = 1251\ \text{k-ft} > 1211\ \text{k-ft}$$ <u>4 braces required</u>

<u>Brace Design:</u> Use the full bracing formula - discrete system - See Eq 2 & 3

$$P_f = \frac{\pi^2 (29000)(32.0)}{(16 \times 12)^2} = 248\ \text{kips}; \quad \# = 4 - \frac{2}{4} = 3.5; \quad C_b = 1.0; \quad C_L = 1.0$$

$$\beta_L^* = 2\,\frac{3.5\,(248)(1.0)(1.0)}{16 \times 12} = 9.04\ \text{k/in. for ea. girder} = 45.2\ \text{k/in. for 5 girders} = F/\Delta$$

$$\text{Brace stiffness} = \cos^2\theta \left(\frac{AE}{L}\right)_b = \frac{A_b\,(29000)}{(\sqrt{5}\,)^2\, 335} = 45.2\ \text{k/in.}$$



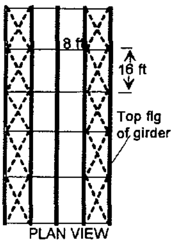
$$\boxed{A_b = 2.61\ \text{in.}^2} \longleftarrow \text{CONTROLS}$$

Brace Strength: (A36 steel)

$$F_{br} = 0.008\,\underbrace{(5)}_{\text{five girders}}\,(1211 \times 12 / 49.0) = 11.86\ \text{k}$$

$$A_b F_y = 11.37 / \cos\theta; \quad A_b = \frac{11.86\sqrt{5}}{36} = \underline{0.74\ \text{in.}^2}$$

LATERAL BRACING - DESIGN EXAMPLE 2



Same as Example 1 except the bracing system is a relative system - a top flange horizontal truss. Each truss stabilizes 2-1/2 girders. The unbraced length of the girder flange is 16 ft which was checked in Example 1.

Brace stiffness:

$P_f = 248$ kips

$C_b = 1.0$

$C_L = 1.0$

$$\beta^*_L = \frac{2 P_f C_b C_L}{L_b}$$

$$= \frac{2(248)\,1.0\,(1.0)}{16 \times 12} = 2.58 \text{ k/in for ea. girder}$$

$$= 6.45 \text{ k/in for 2 1/2 girders}$$

$$\cos^2\theta \left(\frac{AE}{L}\right)_b = \left(\frac{1}{\sqrt{5}}\right)^2 \frac{A_b\,(29000)}{(8 \times 12 \times \sqrt{5})} = 6.45 \text{ k/in}; \quad \underline{A_b = 0.239 \text{ in}^2} \longleftarrow$$

Brace strength: $F_{br} = 0.004\,(2\ 1/2)\,\dfrac{(1211 \times 12)}{49} = 2.97$ kips

$$A_b F_y = 2.97\sqrt{5} \quad ; \quad A_b = \frac{2.97\sqrt{5}}{36} = \underline{0.184 \text{ in}^2}$$

Stiffness controls the brace size; 9/16 ϕ OK $A = 0.248 \text{ in}^2$

Figure 15 shows that torsional bracing on the tension flange (dashed line) is just as effective as compression flange bracing (solid line), even with no stiffener. If the beam has no stiffeners, splitting bracing equally between the two flanges gives a greater capacity than placing all the bracing on just one flange. The dot-dash curve is the solution if web distortion is prevented by transverse stiffeners. The distortion does not have to be gross to affect strength, as shown in Fig. 16 for a total torsional brace stiffness of 3000 in-k/radian. If the W16x26 section has transverse stiffeners, the buckled cross section at midspan has no distortion as shown by the heavy solid lines and M_{cr}= 1582 in-k. If no stiffeners are used, the buckling load drops to 1133 in-k, a 28% decrease, yet there is only slight distortion as shown by the dashed shape. The overall angle of twist for the braced beam is much smaller than the twist in the unbraced case (dot-dash curve).

The effect of load position on torsionally braced beams is not very significant, as shown in Fig. 17. The difference in load between the two curves for top flange and centroid loading for braced beams is almost equal to the difference in strength

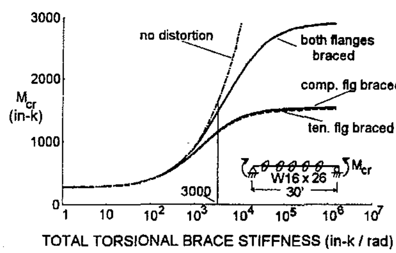


Fig. 15 Effect of Torsional Brace Location

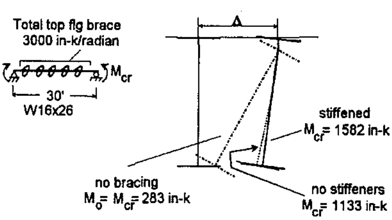


Fig. 16 Effect of Cross Section Distortion

for the unbraced beams (zero brace stiffness). The ideal brace stiffness for top flange loading is 18% greater than for centroid loading. This behavior is different from that shown in Fig. 8 for lateral bracing where the top flange loading ideal brace is 2.5 times that for centroid loading.

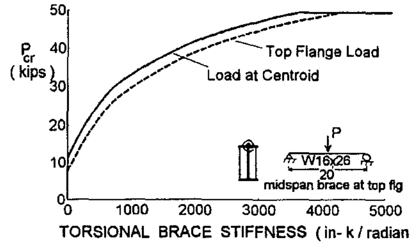


Fig. 17 Effect of Load Position

Figure 18 summarizes the behavior of a 40-ft span with three equal torsional braces spaced 10-ft apart. The beam was stiffened at each brace point to control the distortion. The response is non-linear and follows the pattern discussed earlier for a single brace. For brace stiffness less than 1400 in-k/radian, the stringer buckled into a single wave. Only in the stiffness range of 1400-1600 in-k/radian did multi-wave buckled shapes appear. The ideal brace stiffness at each location was slightly greater than 1600 in-k/radian. This behavior is very different from the multiple lateral bracing case for the same beam shown in Fig. 11. For multiple lateral bracing the beam buckled into two waves when the moment reached 600 in-k and then into three waves at M_{cr} = 1280 in-k. For torsional bracing, the single wave controlled up to M_{cr} = 1520 in-k. Since the maximum moment of 1600 corresponds to buckling between the braces, it can be assumed, for design purposes, that torsionally braced beams buckle in a single wave until the brace stiffness is sufficient to force buckling between the braces. The figure also shows that a single torsional brace at midspan of a 20-ft span (unbraced length = 10 ft) requires about the same ideal brace stiffness as three braces spaced at 10 ft. In the lateral brace case the three brace system requires 1.7 times the ideal stiffness of the single brace system, as shown in Fig. 11.

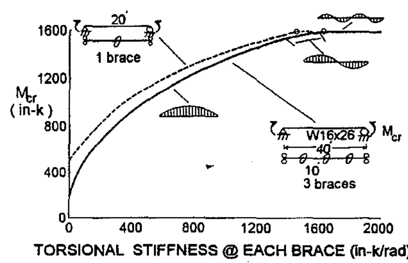


Fig. 18 Multiple Torsional Braces

Tests have been conducted on torsionally braced beams with various stiffener details which are presented elsewhere (Yura, 1992). The tests show good agreement with the Basp solutions.

Buckling Strength of Torsionally Braced Beams. Taylor and Ojalvo (1973) give the following exact equation for the critical moment of a doubly symmetric beam under uniform moment with continuous torsional bracing

$$M_{cr} = \sqrt{M_o^2 + \bar{\beta}_b E I_y} \qquad (5)$$

where M_o is the buckling capacity of the unbraced beam and $\bar{\beta}_b$ = attached torsional brace stiffness (k-in/rad per in. length). Equation (5), which assumes no cross section distortion, is shown by the dot-dash line in Fig. 19. The solid lines are BASP results for a W16x26 section with no stiffeners and spans of 10 ft, 20 ft, and 30 ft under uniform moment with braces attached to the compression flange. Cross-section distortion

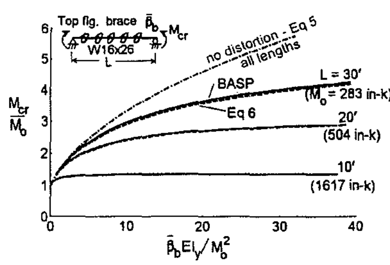


Fig. 19 Approximate Buckling Formula

causes the poor correlation between Eq. (5) and the BASP results. Milner (1977) showed that cross-section distortion could be handled by using an effective brace stiffness, β_T, which has been expanded (Yura, 1992) to include the effect of stiffeners and other factors as follows,

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g} \qquad (6)$$

where β_b is the stiffness of the attached brace, β_{sec} is the cross-section web stiffness and β_g is the girder system stiffness. The effective brace stiffness is less than the smallest of β_b, β_{sec} or β_g.

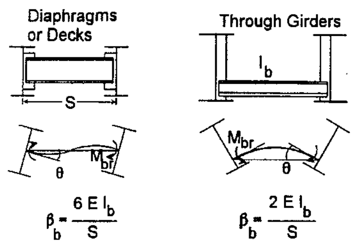
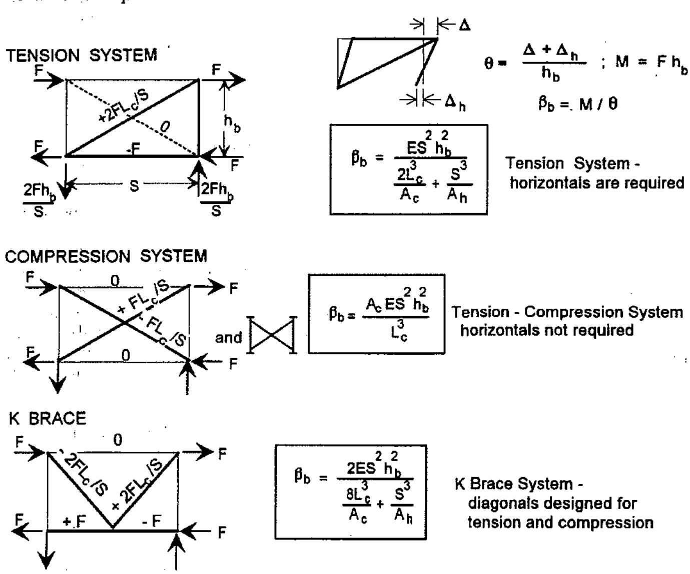


Fig. 20 Torsional Bracing Stiffness

The β_b of some common torsional brace systems are given in Figs. 20 and 21. Systems comprised of diaphragms, slabs, and floor systems for through girders in Fig. 20 assume that the connection between the girder and the brace can support a bracing moment M_{br}. If partially restrained connections are used, their flexibility should also be included in Eq. (5). Elastic truss analyses were used to derive the stiffness of the cross frame systems shown in Fig. 21. If the diagonals of a X-system are designed for tension only, then horizontal members are required in the system. In the K-brace system a top horizontal is not required.



A_h = area of horizontal members
A_c = area of diagonal members
E = modulus of elasticity
L_c = length of diagonal members
S = spacing of girders
h_b = height of the cross frame

Fig. 21 Stiffness Formulas for Cross Frames

In crossframes and diaphragms the brace moments M_{br} are reacted by vertical forces on the main girders as shown in Fig. 22. These forces increase some main girder moments and decreases others. The effect is greater for the twin girder system **B** compared to the interconnected system **A**. The vertical couple causes a differential displacement in adjacent girders which reduces the torsional stiffness of the cross frame system. For a brace only at midspan in a twin girder system the contribution of the inplane girder flexibility to the brace system stiffness is

$$\beta_g = \frac{12\, S^2\, EI_x}{L^3} \qquad (7)$$

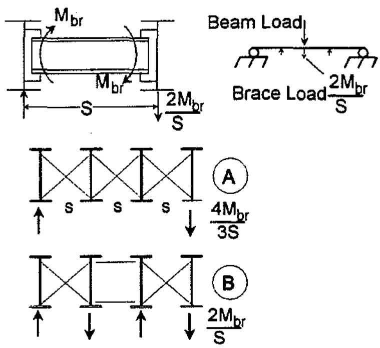


Fig. 22 Beam Load from Braces

where I_x is the strong axis moment of inertia of one girder and L is the span length. As the number of girders increase, the effect of girder stiffness will be less significant. In multi-girder systems, the factor 12 in Eq. 7 can be conservatively changed to 24 $(n_g - 1)^2/n_g$ where n_g is the number of girders. For example, in a six-girder system, the factor becomes 100 or more than eight times the twin girder value of 12. Helwig (1993) has shown that for twin girders the strong axis stiffness factor β_g is significant and Eq. (7) can be used even when there is more than one brace along the span.

Cross-section distortion can be approximated by considering the flexibility of the web, including full depth stiffeners if any, as follows:

$$\beta_{sec} = 3.3\,\frac{E}{h}\left[\frac{(N + 1.5h)\, t_w^3}{12} + \frac{t_s\, b_s^3}{12}\right] \qquad (8)$$

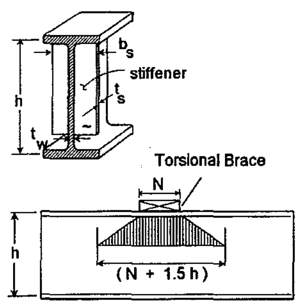


Fig. 23 Effective Web Width

where t_w = thickness of web, h = depth of web, t_s = thickness of stiffener, b_s = width of stiffener, and N = contact length of the torsional brace as shown in Fig. 23. For continuous bracing use an effective net width of 1 in. instead of (N + 1.5h) in β_{sec} and $\bar{\beta}_b$ in place of β_b to get $\bar{\beta}_T$. The dashed lines in Fig. 19 based on Eqs. (5) and (6) show good agreement with the BASP theoretical solutions . For the 10 ft and 20 ft spans, BASP and Eq. (6) are almost identical. Other cases with discrete braces and different size stiffeners also show good agreement.

In general, stiffeners or connection details such as clip angles, can be used to control distortion. For decks and through girders, the stiffener must be attached to the flange that is braced. Diaphragms are usually W shapes or channel sections connected to the web of the stringer or girders through clip angles, shear tabs or stiffeners. When full depth stiffeners or connection details are used to control distortion,the stiffener size to give the desired stiffness can be determined from Eq. (8). For partial depth stiffening illustrated in Fig. 24, the stiffness of the various sections of the web can be evaluated separately, then combined as follows:

$$\beta_i = \frac{3.3E}{h_i}\left(\frac{h}{h_i}\right)^2\left(\frac{(N+1.5h_i)t_w^3}{12} + \frac{t_s\, b_s^3}{12}\right) \qquad (9)$$

where $h_i = h_c$, h_s, or h_t and

$$\frac{1}{\beta_{sec}} = \frac{1}{\beta_c} + \frac{1}{\beta_s} + \frac{1}{\beta_t} \qquad (10)$$

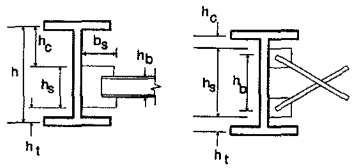


Fig. 24 Partially Stiffened Webs

The portion of the web within h_b can be considered infinitely stiff. For rolled sections, if the diaphragm connection extends over at least one-half the beam depth, then cross-section distortion will not be significant because the webs are fairly stocky compared to built-up sections. The depth of the diaphragm, h_s, can be less than one-half the girder depth as long as it provides the necessary stiffness to reach the required moment. Cross frames without web stiffeners should have a depth h_s of at least 3/4 of the beam depth to minimize distortion. The location of a diaphragm or cross frame on the cross section is not very important; it does not have to be located close to the compression flange. The stiffeners or connection angles do not have to be welded to the flanges when diaphragms are used. For cross frames, β_s, should be taken as infinity; only h_t and h_c will affect distortion. If stiffeners are required for flange connected torsional braces on rolled beams, they should extend at least 3/4 depth to be fully effective.

Equation (5) was developed for doubly-symmetric sections. The torsional bracing effect for singly-symmetric sections can be approximated by replacing I_y in Eqs. (5) with I_{eff} defined as follows:

$$I_{eff} = I_{yc} + \frac{t}{c} I_{yt} \qquad (11)$$

where I_{yc} and I_{yt} are the lateral moment of inertia of the compression flange and tension flange respectively, and c and t are the distances from the neutral bending axis to the centroid of the compression and tension flanges respectively, as shown in Fig. 25(a). For a doubly symmetric section c = t and Eq. (11) reduces to I_y. A comparison between BASP solutions and Eqs. (5) and (11) for three different girders with torsional braces is shown in Fig. 25(b). The curves for a W16x26 show very good agreement. In the other two cases, one of the flanges of the W16x26 section was increased to 10x1/2. In one case the small flange is in tension and in the other case, the compression flange is the smallest. In all cases Eq. (11) is in good agreement with the theoretical buckling load given by BASP.

Equation (5) shows that the buckling load increases without limit as the continuous torsional brace stiffness increases. When enough bracing is provided, yielding will control the beam strength so M_{cr} can not exceed M_y, the yield or plastic strength of the section. It was found that Eq. (5) for continuous bracing could be adapted for discrete torsional braces by summing the stiffness of each brace along the span and dividing by the beam length to get an equivalent continuous brace stiffness. In this case M_{cr}

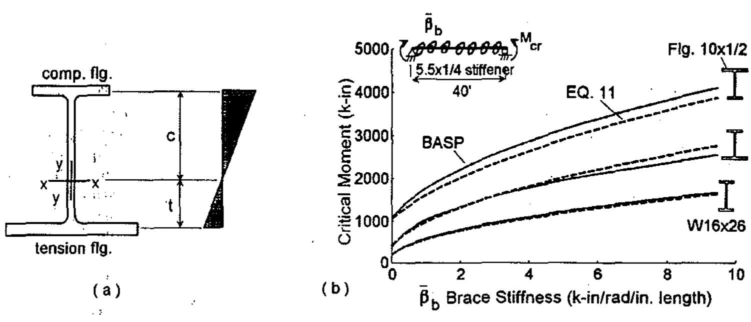


Fig. 25 Singly Symmetric Girders

will be limited to M_s, the moment corresponding to buckling between the brace points. By adjusting Eq. (5) for top flange loading and other loading conditions, the following general formula can be used for the buckling strength of torsionally braced beams :

$$M_{cr} = \sqrt{C_{bu}^2 M_o^2 + \frac{C_{bb}^2 \overline{\beta}_T E I_{eff}}{C_T}} \leq M_y \text{ or } M_s \tag{12}$$

where C_{bu} and C_{bb} are the two limiting C_b factors corresponding to an unbraced beam (very weak braces) and an effectively braced beam (buckling between the braces); C_T is a top flange loading modification factor; $C_T = 1.2$ for top flange loading and $C_T = 1.0$ for centroid loading; and $\overline{\beta}_T$ is the equivalent effective continuous torsional brace (in-k/radian/in. length) from Eq.(6). The following two cases illustrate the accuracy of Eq. (12).

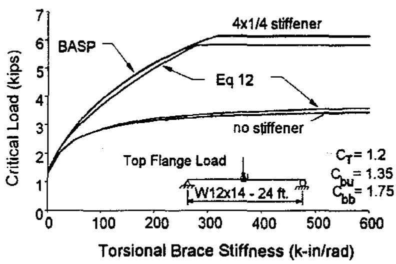


Fig. 26 Effect of Stiffener

For the case of a single torsional brace at midspan shown in Fig. 26, C_{bu}= 1.35 for a concentrated load at the midspan of an unbraced beam (Galambos, 1988). Usually designers conservatively use C_b= 1.0 for this case. For the beam assumed braced at midspan, C_{bb}= 1.75 for a straight line moment diagram with zero moment at one end of the unbraced length. These two values of C_b are used with any value of brace torsional stiffness in Eq. (12). For accuracy at small values of brace stiffness the unbraced buckling capacity $C_{bu}M_o$ should also consider top flange loading effects. Equation (12) shows excellent agreement with the BASP theory. With no stiffener, β_{sec} from Eq. (8) is 114 in-k/radian, so the effective brace stiffness β_T from Eq. (6) cannot be greater than 114 regardless of the brace stiffness magnitude at midspan. Equations (6), (8) and (12) predict the buckling very accurately for all values of attached bracing, even at very low values of bracing stiffness. A 4 x 1/4 stiffener increased β_{sec} from 114 to 11000 in-k/radian. This makes the effective brace stiffness very close to the applied stiffness, β_b. With a 4 x 1/4 stiffener, the effective stiffness is 138 in-k/radian if the attached brace stiffness is 140 in-k/radian. The bracing equations can be used to determine the required stiffener size to reduce the effect of distortion to some tolerance level, say 5%.

Figure 27 shows the correlation between the approximate buckling strength, Eq. (12) and the exact BASP solution for the case of a concentrated midspan load at the centroid with three equally spaced braces along the span. Stiffeners at the three brace points prevent cross-section distortion so $\overline{\beta}_T = 3\beta_b/288$ in.. Two horizontal cutoffs for Eq. (12) corresponding to the theoretical moment at buckling between the braces are shown. The K = 1.0 limit assumes that the critical unbraced length, which is adjacent to the midspan load, is not restrained by the more lightly loaded end spans. To account for the effect of the end span restraint, an effective length factor K = 0.88 was calculated using the procedure given in the SSRC Guide (Galambos, 1988). Figure 27 shows that it is impractical to rely on side span end restraint in determining the buckling load between braces. An infinitely stiff brace is required to reach a moment corresponding to K = 0.88. If a K factor of 1 is used in the buckling strength formula, the

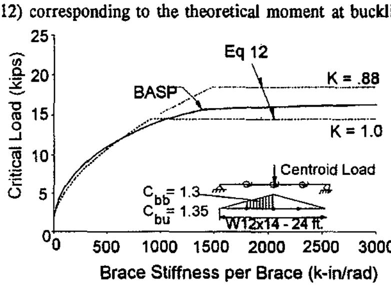


Fig. 27 Multiple Discrete Braces

comparison between Eq. (12) and the BASP solution is good. Equation (12) should not be used with K factors less than 1.0; the results will be unconservative at moments approaching the full bracing case. Similar results were obtained for laterally braced beams (Yura, 1992).

Torsional Brace Design. There are two basic torsional bracing systems shown in Figs. 20 and 21: bending members represented by diaphragms, decks or floor beams; and trusses for the cross frames. The two systems can be correlated by noting that $M_{br} = F_{br}\, h_b$, where h_b is the depth of the cross frame. The term "brace forces" used hereinafter refers to both M_{br} and F_{br}. Equation (12) gives the relationship between brace stiffness and M_{cr} for an ideally straight beam. For beams with an initial twist, θ_o, it is assumed that the brace design requirements are affected in a similar manner as that developed for lateral bracing of beams with initial out-of-straightness. The required brace stiffness $\overline{\beta}_T^*$, which must be at least twice the ideal stiffness to keep brace forces small, can be obtained by rearranging Eq. (12)

$$\overline{\beta}_T^* = 2\left(M_{cr}^2 - C_{bu}^2 M_o^2\right)\frac{C_T}{C_{bb}^2 E I_{eff}} \tag{13}$$

For discrete braces $\beta_T^* = \overline{\beta}_T^* L/n$. The brace force $M_{br} = \beta_T^*\, \theta_o$. An initial twist $\theta_o = 1°$ (0.0175 radians) is recommended. For a 14-in deep section this assumed initial twist corresponds to a 0.25 in. relative displacement between the top and bottom flanges. Equation (13) can be conservatively simplified by neglecting the $C_{bu}M_o$ term which will be small compared to M_{cr} at full bracing and by taking the maximum C_T, which is 1.2 for top flange loading. The simplified stiffness and brace force requirements are given in the following summary.

TORSIONAL BRACING DESIGN REQUIREMENTS

Stiffness:

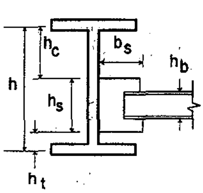
$$\beta_T^* = \overline{\beta}_T^* L/n = 2.4 L M_f^2 / (n E I_{eff} C_{bb}^2) \tag{14}$$

Strength:

$$M_{br} = F_{br}\, h_b = 0.04 L M_f^2 / (n E I_{eff} C_{bb}^2) \tag{15}$$

where M_f = maximum beam moment
I_{eff} = $I_{yc} + (t/c)\, I_{yt}$; = I_y for doubly symmetric sections (see Fig. 25)
C_{bb} = moment diagram modification factor for the full bracing condition
L = span length
n = number of braces along the span

The available effective stiffness of the brace system β_T is calculated as follows:



$$\frac{1}{\beta_T} = \frac{1}{\beta_c} + \frac{1}{\beta_s} + \frac{1}{\beta_t} + \frac{1}{\beta_b} + \frac{1}{\beta_g} \tag{16}$$

$$\beta_c,\, \beta_s,\, \beta_t = \frac{3.3E}{h_i}\left(\frac{h}{h_i}\right)^2\left(\frac{(N+1.5h_i)t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \tag{17}$$

where $h_i = h_c, h_s$, or h_t ; N = bearing length (see Fig. 23)
β_b = stiffness of attached brace (see Figs. 20 and 21);
$\beta_g = 24(n_g - 1)^2 S^2 EI_x / (L^3 n_g)$ (18)
where n_g is the number of interconnected girders (see Fig. 22)

The torsional brace stiffness requirement, Eq. (14), must be adjusted for the different design specifications as discussed earlier for the lateral brace requirements:

AISC-LRFD:	$\beta_T \geq \beta_T^* / \phi$	where $\phi = 0.75$ is suggested
AISC-ASD:	$\beta_T \geq 2\,\beta_T^*$	where 2 is a safety factor
AASHTO-LFD:	$\beta_T \geq \beta_T^*$	no change

Torsional Brace Design Examples. In Example 3 a diaphragm torsional bracing system is designed by the AASHTO-LFD specification to stabilize the five steel girders during construction as described in Examples 1 and 2 for lateral bracing. The strength criterion, Eq. 15, is initially assumed to control the size of the diaphragm. A C10×15.3 is sufficient to brace the girders. Both yielding and buckling of the diaphragm are checked. The stiffness of the C10×15.3 section, 195,500 in-k/radian, is much greater than required but the connection to the web of the girder and the in-plane girder flexibility also affect the stiffness. In this example, the in-plane girder stiffness is very large and its affect on the brace system stiffness is only 2%. In most practical designs, except for twin girders, this effect can be ignored. If a full depth connection stiffener is used, a 3/8 × 3-1/2 in. section is required. The weld design between the channel and the stiffener, which is not shown, must transmit the bracing moment of 293 in-k.

The 40-in. deep cross frame design in Example 4 required a brace force of 7.13 kips from Eq. (15). The factored girder moment of 1211 k-ft. gives an approximate compression force in the girder of 1211 × 12/49 = 296 kips. Thus, the brace force is 2.5% of the equivalent girder force in this case. The framing details provide sufficient stiffness. The 3-in. unstiffened web at the top and bottom flanges was small enough to keep β_{sec} well above the required value. For illustration purposes, a 30-in. deep cross frame attached near the compression flange is also considered. In this case, the cross frame itself provides a large stiffness, but the 14-in. unstiffened web is too flexible. Cross-section distortion reduces the system stiffness to 16,900 in.-k/radian, which is less than the required value. If this same cross frame was placed at the girder midheight, the two 7-in. unstiffened web zones top and bottom would be stiff enough to satisfy the brace requirements. For a fixed depth of cross frame, attachment at the mid-depth provides more effective brace stiffness than attachment close to either flange

Closing Remarks and Limitations

Two general structural systems are available for bracing beams, lateral systems and torsional systems. Torsional bracing is less sensitive than lateral bracing to conditions such as top flange loading, brace location, and number of braces, but more affected by cross-section distortion. The bracing recommendations can be used in the inelastic buckling range up to M_p if the M_f form of the lateral brace stiffness equation is used (Ales, 1993).

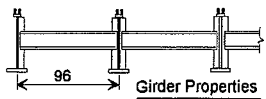
The recommendations do not address the bracing requirements for moment redistribution or ductility in seismic design. The bracing formulations will be accurate for design situations in which the buckling strength does not rely on effective lengths less than one. Lateral restraint provided by lightly loaded side spans should, in general, not be considered because the brace requirements would be much larger than the recommendations herein. Also, laboratory observations in the author's experience (usually unplanned failures of test setups) show that brace forces can be very large when local flange or web buckling occurs prior to lateral instability. After local buckling the cross section is unsymmetric and vertical loads develop very significant out of plane load components. The bracing recommendations do not address such situations.

References

Akay, H.U., Johnson, C.P., and Will, K.M., 1977, "Lateral and Local Buckling of Beams and Frames," *Journal of the Structural Division*, ASCE, ST9, September, pp. 1821-1832.

Ales, J. M. and Yura, J. A., 1993, "Bracing Design for Inelastic Structures", Structural Stability Research Council Conference-Is Your Structure Suitably Braced?, April 6-7, Milwaukee, WI..

TORSIONAL BRACING - DESIGN EXAMPLE 3



Girder Properties

$h = 49.0$, $c = 30.85$, $t = 18.15$ in

$I_x = 17500$, $I_{yc} = 32.0$, $I_{yt} = 352$ in^4

Same as Example 1 except use the diaphragm system shown. In Ex. 1, four lateral braces were required which gave $M_r = 1251$ k-ft, just 3% greater than the required mom. Since torsional braces will impose some additional vertical forces on the girders as shown in Fig. 22, probably five braces should be used. However, for comparison with Examples 1 & 2, a four-brace system will be designed. $M_{req'd} = 1211$ k-ft (see Ex. 1)

Eq. (11): $I_{eff} = 32 + \frac{18.15}{30.85} 352 = 239 \text{ in}^4$

Eq. (15) $M_{br} = \frac{0.04\,(80 \times 12)\,(1211 \times 12)^2}{4\,(29000)\,239\,(1.0)^2} = 293$ in-k

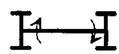
A36 Steel: Req'd $S_x = 293/36 = \underline{8.14 \text{ in}^3}$

Try C10 x 15.39 -

$I_x = 67.4 \text{ in}^4$, $S_x = 13.5 \text{ in}^3$, $t_f = 0.436$ in , $b = 2.60$ in , $J = 0.21 \text{ in}^4$

Check lateral buckling of the diaphragm

$I_{yc} = (2.60)^3\,(0.436) / 12 = 0.639$ in

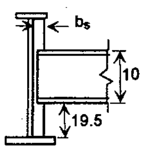


$C_b = 2.3$

$M_r = 91 \times 10^6 (2.3)\ \frac{0.639}{96} \sqrt{\frac{.772\,(0.21)}{0.639} + \left(\frac{\pi\, 10}{96}\right)^2}$ (AASHTO 10-102c)

$= 837000$ lb-in $= 837$ in-k $> M_y = 13.5(36) = 486 > 293$ in-k , OK

Check stiffness: Eq. (14); $\beta_{T\,req'd} = \frac{2.4(80 \times 12)\,(1211 \times 12)^2}{4(29000)\,239\,\,(1.0)^2} = 17550$ in-k/radian



The stiffness of the diaphragms on the exterior girders is $6EI_{br}/S$. Since there are diaphragms on both sides of each interior girder, the stiffness is $2 \times 6EI_{br}/S$. The average stiffness available to each girder is $(2 \times 6 + 3 \times 12)/5 = 9.6\ EI_{br}/S$.

$\beta_b = 9.6\,(29000)\,67.4/96 = 195500$ in-k/radian

Girder : Eq. (18) $\beta_g = \frac{24\,(5\text{-}1)^2\,29000\,(96)^2\,17500}{5\,(80 \times 12)^3} = 406000$ in-k/radian

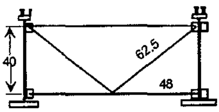
Distortion : From Fig 23 and Eq. (17) determine the required stiffener size , $t_s = 3/8$ in

$$\beta_s = \frac{3.3\,(29000)}{19.5}\left(\frac{49}{19.5}\right)^2\left(\frac{1.5 \times 19.5 \times .5^3}{12} + \frac{.375\, b_s^3}{12}\right) \quad (1)$$

From Eq. (16) $\frac{1}{17550} = \frac{1}{195500} + \frac{1}{406000} + \frac{2}{\beta_s}$; $\beta_s = 40500$ in-k/radian (2)

Equating (1) and (2) gives $b_s = 3.17$ in. - <u>Use 3/8 x 3-1/2 stiffener</u>

TORSIONAL BRACING - DESIGN EXAMPLE 4



Same as Example 3, but use cross frames. Make all member sizes the same. A K-frame system will be considered using double angle members welded to connection gusset plates. Member lengths are shown in inches. Use four crossframes. See Examples 1 and 3 for section properties. Use A36 steel.

Assume brace strength criterion controls - Eq. (15)

$$F_{br}\,(40) = \frac{0.04\,(80 \times 12)\,(1211 \times 12)^2}{4\,(29000)\,239\,(1.0)^2} = 293 \text{ in-k} \quad ; \quad \underline{F_{br} = 7.31 \text{ kips}}$$

From Fig. 21 : Max force = diagonal force = $\frac{2F_{br}L_c}{S} = \frac{2(7.31)\,62.5}{96} = 9.52$ kips - comp

The AASHTO Load Factor method does not give a strength formula for compression members so the formulation in Allowable Stress Design will be used. Convert to ASD by dividing the member force by the 1.3 load factor to get an equivalent service load force.

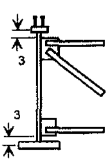
Diagonal Force (ASD) = 9.52/1.3 = 7.3 kips

<u>Try 2L - 2 1/2 x 2 1/2 x 1/4</u> $\quad r_x = .769$ in. , $A = 2.38$ in.^{2}

$\ell / r = 62.5 / .769 = 81.2$; $F_a = 16980 - .53\,(81.2)^2 = 13490$ psi $= 13.5$ ksi

$P_{allow} = 13.49\,(2.38) = 32.1$ kips > 7.3 kips *OK*

Check brace stiffness:



Eq. (14) ; $\beta_{T\,req'd} = 17550$ in-k/radian - see Example 3

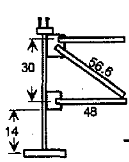
Fig. 21 : $\quad \beta_b = \frac{2(29000)\,(96)^2\,(40)^2\,(2.38)}{8\,(62.5)^3 + (96)^3} = 717000$ in-k/radian

Girder : $\quad \beta_g = 406000$ in-k/radian - see Example 3

$$\beta_c = \beta_t = \frac{3.3\,(29000)}{3.0}\left(\frac{49}{3.0}\right)^2\left(\frac{1.5\,(3.0)\,(.5)^3}{12}\right) = 399000 \text{ in-k/rad}$$

Eq. (16) : $\frac{1}{\beta_T} = \frac{1}{717000} + \frac{1}{406000} + \frac{2}{399000}$; $\underline{\beta_T = 113000 > 17550}$ in-k/rad *OK*

Evaluate the cross frame shown below



$$\beta_b = \frac{2(29000)\,(96)^2\,(30)^2\,(2.38)}{8\,(56.6)^3 + (96)^3} = 490000 \text{ in-k/radian}$$

$$\beta_t = \frac{3.3\,(29000)}{14.0}\left(\frac{49}{14.0}\right)^2\left(\frac{1.5\,(14.0)\,(.5)^3}{12}\right) = 18300 \text{ in-k/rad}$$

$\frac{1}{\beta_T} = \frac{1}{490000} + \frac{1}{406000} + \frac{1}{18300}$; $\underline{\beta_T = 16900 < 17550}$ in-k/rad *NG*

Choo, K.M., 1987, , "Buckling Program BASP for Use on a Microcomputer", Thesis presented to The University of Texas at Austin, May

Galambos, T.V., Ed., 1988, Structural Stability Research Council, *Guide to Stability Design Criteria for Metal Structures*, 4th Edition, New York: John Wiley & Sons, Inc.

Helwig, T.A., Yura, J.A. and Frank K.H., 1993, "Bracing Forces in Diaphragms and Cross Frames," SSRC Conference-Is Your Structure Suitably Braced?, April 6-7, Milwaukee, WI..

Kirby, P.A. and Nethercot, D.A., 1979, *Design for Structural Stability*, Wiley.

Linder, J., and Schmidt, J.S., 1982, "Biegedrillknicken von I-Trägern unter Berücksichtigung wirklichkeitsnaher Lasteinleitung," *Der Stahlbau*, 51, H.9, S. 257-263.

Milner, H.R., 1977, "Design of Simple Supported Beams Braced Against Twisting on the Tension Flange," *Civil Engineering Transactions, Institute of Engineers, Australia*, CE 20(1), pp. 37-42.

Medland, I.C., 1980, "Buckling of Interbraced Beam Systems", *Engineering Structures*, 2, April, p 90-96

Nakamura, T. and Wakabayashi, M.(1981), 'Lateral Buckling of Beams Braced by Purlins", *Inelastic Instability of Steel Structures and Structural Elements*, U.S. Japan Seminar, Fujita-Galambos, ed,

Nakamura, T. (1988), "Strength and Deformability of H-Shaped Steel Beams and Lateral Bracing Requirements", *J. Constructional Steel Research*, 9, p 217-228

Nethercot, D. A., (1989), "The Design of Bracing Systems for Plate Girder Bridges", *Structural Eng / Earthquake Eng.*, Vol. 6, No. 1, 59s-68s,Review, (Proc JSCE, No 404 / I-11), April.

Raju, S., Webb, S. and Yura, J.,1992, "Bracing Effects of Bridge Decks", *Proceedings*, 9th Annual International Bridge Conference, Pittsburgh, June 15, 9 p.

Taylor, A.C., and Ojalvo, M., 1966, "Torsional Restraint of Lateral Buckling," *Journal of the Structural Division*, ASCE, ST2, April, pp. 115-129.

Timoshenko, S., and Gere, J., 1961, *Theory of Elastic Stability*, New York: McGraw-Hill .

Tong, G.S., and Chen, S.H., 1988, "Buckling of Laterally and Torsionally Braced Beams," *Journal Construction Steel Research*, 11. p 41-55

Trahair, N.S., and Nethercot, D.A., 1982, "Bracing Requirements in Thin-Walled Structures," Chapter 3, *Developments in Thin-Walled Structures - Vol. 2*, Rhodes and Walker - Ed., Elsevier, pp. 93-129.

Wakabayashi, M. and Nakamura, T.,(1983),"Buckling of Laterally Braced Beams", *Eng. Struct*, 5, April

Wang, Y. C. and Nethercot, D. A., 1989, "Ultimate Strength Analysis of Three-Dimensional Braced I-Beams", *Proceedings, Institution of Civil Engineers*, London,Part 2, 87, March, p 87-112

Winter, G., 1980, "Lateral Bracing of Columns and Beams," *ASCE Transactions*,Vol. 125, pp. 809-825.

Yura, J.A., 1971, "Design of Bracing for Columns, Beams and Frames," *AISC National Engineering Conference*, Cleveland.

Yura, J.A., and Phillips, B.,1992, "Bracing Requirements for Elastic Steel Beams," Report No. 1239-1, Center for Transportation Research, University of Texas at Austin, May ,73 p.

Yura, J.A., Phillips, B., Raju, S. and Webb, S.,1992, "Bracing of Steel Beams in Bridges," Report No. 1239-4F, Center for Transportation Research, University of Texas at Austin, October, 80 p.

Yura, J. A., 1993, "Lean-On Bracing Systems", Structural Stability Research Council Conference-Is Your Structure Suitably Braced?, April 6-7, Milwaukee, WI..

BRACING OF COLD-FORMED CHANNELS NOT ATTACHED TO DECK OR SHEETING

by Kenneth T. Kavanagh[1] and Duane S. Ellifritt[2]

INTRODUCTION

The theme of this conference is, "Is Your Structure Suitably Braced?" Everyone would agree that bracing is necessary for stability of both members and structures, but how much is enough? And if a little is good, is a lot better? This paper explores the possibility that **too much bracing** may **not** be helpful and may, is some cases, cause the designer to believe a structural member is stronger than it really is. Tests on cold-formed channels at the University of Florida and the University of Western Australia confirmed that current design specifications for lateral-torsional buckling of channels not attached to sheeting may be unconservative for short, discretely braced spans.

BACKGROUND

Research on flexural strength of discretely braced channels and zees not attached to sheeting was conducted at the University of Florida in 1991 (1). The purpose of this work was to see if it was really necessary to brace such members at the quarter-points, as the AISI Specification (2) required, or if it was acceptable to use fewer braces and a reduced strength, based on lateral-torsional buckling formulas in the Specification.

One surprising result in the Florida tests was that the more bracing used in a given span length, the less conservative the AISI lateral buckling equations become. It was concluded that this apparent anomaly came about because lateral-torsional buckling was no longer the failure mode. The accepted design formulas were <u>predicting a failure that never took place</u>, because some other limit state occurred first.

That other limit state was "distortional buckling" of the flange and lip that usually occurred at a braced point because the section was not permitted to twist there and additional compression force built up in the lip. A summary of the results of all 23 tests at Florida and their corresponding brace configurations is shown in Table 1. It can be seen that the failure predicted by lateral-torsional buckling for 1/4- and 1/3-point bracing is generally higher than the actual test failure load.

---

1 Senior Lecturer, Civil Engineering, University of Western Australia, Nedlands, 6009 Western Australia

2 Crom Professor of Structural Design, University of Florida, Gainesville, Florida

Table 1. UNIVERSITY OF FLORIDA TEST RESULTS

Group	Test #*	M_t	M_n $C_b = 1$	M_n Other C_b	M_t/M_n $C_b = 1$	M_t/M_n Other C_b	a/L	Failure Modes**
I	C14U	44.43	44.12		101%		1.00	A
	C14M	110.73	115.17	124.82	96%	89%	0.50	B
	C14T	112.48	135.08		83%		0.33	C
	C14Q	152.65	142.00		108%		0.25	C
	C14F	143.88	150.86		95%		0.00	C
II	C12U	72.14	60.44		119%		1.00	A
	C12M	171.98	153.23	163.32	112%	105%	0.50	B
III	Z16U	17.54	11.31		155%		1.00	A
	Z16M	47.18	43.04	54.11	110%	87%	0.50	B
	Z16T	55.17	71.86		77%		0.33	D
	Z16Q	65.31	80.20		81%		0.25	C
	Z16F	111.53	90.33		123%		0.00	E
IV	Z13U	30.89	18.57		166%		1.00	A
V	C14TU	51.60	42.43		122%		1.00	A
	C14TT	105.24	136.41		77%		0.33	C
	C14TB	141.42	136.41		104%		0.33	C
	C14MT	123.00	117.19	132.25	105%	93%	0.50	C
	C14MB	120.84	117.19	132.25	103%	91%	0.50	C
VI	Z16TU	18.30	11.16		164%		1.00	A
	Z16TT	48.72	78.84		62%		0.33	E
	Z16TB	62.76	78.84		80%		0.33	B
	Z16MT	45.97	42.34	71.92	109%	64%	0.50	B
	Z16MB	70.27	42.34	71.92	166%	98%	0.50	B

* Test Designation:

C 14 U

Section type	Gage	Bracing Condition
C or Z	12	U = Unbraced T = Third-point F = Fully braced
	13	M = Mid-point Q = Quarter point
	14	TU = Third-point loading, unbraced
	16	TT = Third-point loading, top flange, braced
		TB = Third-point loading, top and bottom flanges braced
		MT = Mid-point loading, top flange braced
		MB = Mid-point loading, top and bottom flange braced.

M_t = Failure moment from test, in -k
M_n = Nominal Calculated Moment, in -k (AISI, Sec. C3.1.2a)

** Modes:
A = Web buckled at midspan
B = Flange buckled at midspan
C = Stiffener/flange/web buckled at load point
D = Web buckled in midspan - stiffeners buckled in end spans
E = Stiffener buckled in midspan

This research ended without answering the question of how to best predict this type of distortional buckling. Additional tests were performed at the University of Western Australia to provide answers to this question.

BEHAVIOR OF BRACED CHANNELS

When a channel section is loaded in the plane of the web, or through the flange, there exists a primary torsion which produces twist. The equations of equilibrium couple the lateral motion and twist, so that the torsion is accompanied by weak axis bending. The section, therefore, develops axial stress due to a combination of major axis bending, minor axis bending, and warping (twist). Additional local stresses are added to the lip by concentrated loads or lateral brace forces.

For the case of strong axis bending only, the stress state is independent of the brace configuration. For twist and weak axis bending, however, the stress state is a function of the brace configuration. Fig. 1a shows the rotation pattern which results from a central point load and third-point bracing.

The outward deflection of the compression flange at the center span relieves the lip of compression, and failure occurs near the web. Fig. 1b shows the rotation pattern for third point loading and mid-point bracing. The reverse curvature at center span increases the stress at the lip, and failure occurs at the flange/lip junction.

Since the total stress is a combination of bending and torsion, the ratio of warping stresses to bending stresses exerts a strong influence on the failure of the cross section. At very short spans, or low span-depth ratios, the bending moment becomes small and the torsional moment increases. At long spans, or high span-to-depth ratio, the bending moment is large, and the torsional moment decreases. Thus, the AISI formulation would appear to be best suited to the design of long beams, or those in which the torsional stresses are small in relation to the bending stresses.

EXPERIMENTAL PROGRAM

Tests at the University of Florida (1) revealed that unbraced or mid-point braced members consistently fell above the AISI prediction curve, while most of the third- or quarter-point braced members fell below the prediction curve.

Ten similar tests were conducted at the Civil Engineering Laboratory of the University of Western Australia with the loading applied in the plane of the web at the neutral axis. The cross-section, shown in Fig. 3 was the same for all tests. The material was AS1397, with a nominal yield strength of 500MPa, and a measured yield strength of 550 MPa. Variables in the test program included span length, load position, and brace location, as summarized in Fig. 2.

Channels were tested in a back-to-back configuration, with strain gauges at the point of expected failure. Equal deflection of the two channels was provided by a rigid loading frame extending well below the two specimens. The cross-section and loading frame is shown in Fig. 3.

ANALYTICAL MODELLING

Classical second order theory for lateral-torsional buckling can be derived from energy considerations (3) using a simple sine wave for the case of an unbraced span with constant moment. The governing equations:

$$\begin{vmatrix} M_{xx} & 0 \\ 0 & M_{xx} \end{vmatrix} \begin{Bmatrix} u \\ \theta \end{Bmatrix} + \begin{vmatrix} EI_{yy}\pi^4/L^4 & 0 \\ 0 & GJ\pi^2/L^2 + EI_{ww}\pi^4/L^4 \end{vmatrix} \begin{Bmatrix} u \\ \theta \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \qquad \text{Eq. 1}$$

yield the exact solution for buckling under a constant moment. Torsional moment on the cross-section enters the equations on the right hand side:

$$\begin{vmatrix} M_{xx} & 0 \\ 0 & M_{xx} \end{vmatrix} \begin{Bmatrix} u \\ \theta \end{Bmatrix} + \begin{vmatrix} EI_{yy}\pi^4/L^4 & 0 \\ 0 & GJ\pi^2/L^2 + EI_{ww}\pi^4/L^4 \end{vmatrix} \begin{Bmatrix} u \\ \theta \end{Bmatrix} = \begin{Bmatrix} 0 \\ M_t \end{Bmatrix} \qquad \text{Eq. 2}$$

The presence of vertical bending, M_{xx}, couples the rotation, (θ), to the lateral bending, (u), so that torsion results in a combination of lateral motion and twist. The simple energy equations can be expanded to a finite element formulation through the use of piecewise cubic equations in (u) and (θ), so that beams with varying moment, varying torque, and arbitrary boundary conditions are accurately modelled.(4)

Coupling between (u) and (θ) results in a combination of three stress patterns in the cross-section, as shown in Fig. 4.

In addition to the global bending stresses, local distortional stresses can result from the application of point loads, either at brace points or at points of concentrated load. A simple model for the distortional stress can be based upon the concentrated shear flow in the lip, using a sine wave approximation for the distortional shape.(5) The local model is shown in Fig. 5.

The distortional model (above) can also be used to calculate the distortional buckling of the cross-section. A typical buckling curve is shown in Fig. 6 for a uniform stress on the compression flange.

The computer model in this paper has been based on the elastic distortional buckling values derived from a series of flange stress distributions. One approximate formula was derived for the long wave flange mode (Fig. 6), and a second was based on the classical buckling solutions for a rectangular plate in bending plus compression.(6)

Flange Buckling $\sigma_{cr}(\text{MPa}) = 820 - 900\ (\alpha) + 400\ (\alpha)^2$

Web Buckling $k = 4 + 5.6\ (\alpha^2 - 2\alpha + 1) - .6\ (\alpha^3 - 3\alpha + 2)$

$$\sigma_{cr} = k\pi^2 E/12/(1-\nu^2)/(b/t)^2$$

where: α = stress ratio

Distortional stresses are amplified by the flange buckling stress, since the distortional bending wave length and the buckling wave length have similar longitudinal dimensions.

$$\sigma_d = \sigma_d/(1 - [\sigma_x + \sigma_y + \sigma_w]/\sigma_{cr})$$

where σ_d = distortional lip stress in Figure 5

$\sigma_x, \sigma_y, \sigma_w$ = stress distributions in Figure 4

The AISI provisions for stiffened flanges include the calculation of an effective flange width, which is dependent on the maximum flange stress, and the flange stress distribution.

$$\lambda = 1.052/\ k \cdot w/t \cdot\ f/E$$

$$\rho = (1 - .22/\lambda)/\lambda$$

$$b = w, \text{ if } \lambda \leq .673$$

$$b = \rho w, \text{ if } \lambda > .673$$

where terms are defined in the AISI Specification, Section B 2.1. Since the stiffened plate represents post-buckling behavior, it is treated separately from the elastic distortional buckling and the elastic web buckling in the computer model. The coupled equations, Eq. (2), are solved incrementally. At each step, the flange stress and stress distribution are calculated. The effective width is calculated, and the revised section properties are determined. The process is continued until the maximum stress exceeds one of: a) the yield stress, b) the distortional buckling stress, or c) the web buckling stress.

EXPERIMENTAL AND ANALYTICAL RESULTS

Experimental results are compared with the AISI Specification provisions and with the computer model in Figs. 7 and 8. The lateral torsional buckling curves (full and effective section) are based upon a 1.46 moment multiplier in the central brace case, and a 1.41 moment multiplier in the third point brace case.* Both multipliers were derived from a finite element lateral torsional buckling analysis, which includes the effects of moment distribution across the entire span and the location of braces in the span. Fig. 7 shows the comparison curves and test values for third point loading and mid-point bracing (Tests 1,2,9,10 in Fig. 2b). Fig 8 shows the same curves for the case of third point bracing and mid point loading (Tests 3,4,5 in Fig. 2b). In both figures, the short unbraced span corresponds to a short total beam span, and the test results fall below the AISI curves.

Tabulated results of the test program are given in Table 2:

Table 2
ANALYTICAL AND EXPERIMENTAL
RESULTS

Test	a/L	M_{test} kN-m	M_{AISI} kN-m	$M_{analysi}$ kN-m	Ratio M_{test}/M_{AISI}	Ratio M_{test}/M_{anal}
1	.5	3.95	3.16	3.6	1.25	1.09
2	.5	4.06	3.16	3.6	1.28	1.13
3	.33	5.07	5.00	4.8	1.01	1.06
4	.33	5.47	5.00	4.8	1.09	1.14
5	.43	5.18	5.88	4.7	0.88	1.10
6	.25	5.12	5.48	5.2	0.93	0.98
7	1.0	3.06	3.62	3.3	0.85	0.93
8	1.0	2.94	3.62	3.3	0.81	0.91
9	.5	5.08	5.78	3.8	0.88	1.34
10	.5	5.30	5.78	3.8	0.92	1.39

A comparison of test 5 with tests 3 and 4 shows that the AISI predicts an increase in load due to shorter unbraced length, while the analysis predicts a decrease in load due to proportionally larger torsion. The test results are approximately unchanged, or are slightly decreasing. A comparison of test 9 with tests 1 and 2 shows a dramatic increase in the AISI predicted failure, with only moderate increases in both the test results and the analytical model. The underestimation of failure in the analytical model is believed to be due to the use of an elastic buckling model for the web (with no allowance for post-buckling strength). It is interesting that the analytical model predicts a fully effective section at failure (due to a non-uniform stress distribution in the flange), while the AISI Specification significantly reduces the effective flange width (due to a uniform distribution).

* The moment multiplier applies to a constant moment on simply supported unbraced span.

In Fig. 7, the computer model exceeds the lateral torsional buckling load for long unbraced lengths, which reflects the failure criterion used in the model. The first buckle mode of a centrally braced specimen is anti-symmetric, with zero stress at center span. Since failure is based on cross-sectional stresses at center span, the section passes over the first mode, and fails in the second (symmetric) mode. Experimental specimens also failed in a symmetric pattern, which may reflect the presence of some weak axis moment resistance at the central brace point.

SUMMARY AND CONCLUSIONS

The AISI Specification for lateral-torsional buckling of unsheeted members has been shown to be unconservative for members in which the torsional stresses are high in relation to bending stresses. Tests and analytical results indicate that the stresses and failure patterns are strongly dependent upon the brace configuration, and not simply upon the unbraced length. An analytical solution is presented that predicts conservative results for these so-called "short" spans.

REFERENCES

1. Ellifritt, D. S., Sputo, T., and Haynes, J., "Flexural Strength and Deflections of Discretely Braced Channel and Zee Sections", Proceedings, 11th International Specialty Conference on Cold-Formed Steel Design, U. of Missouri, Rolla, October 1992, pp. 109-129.

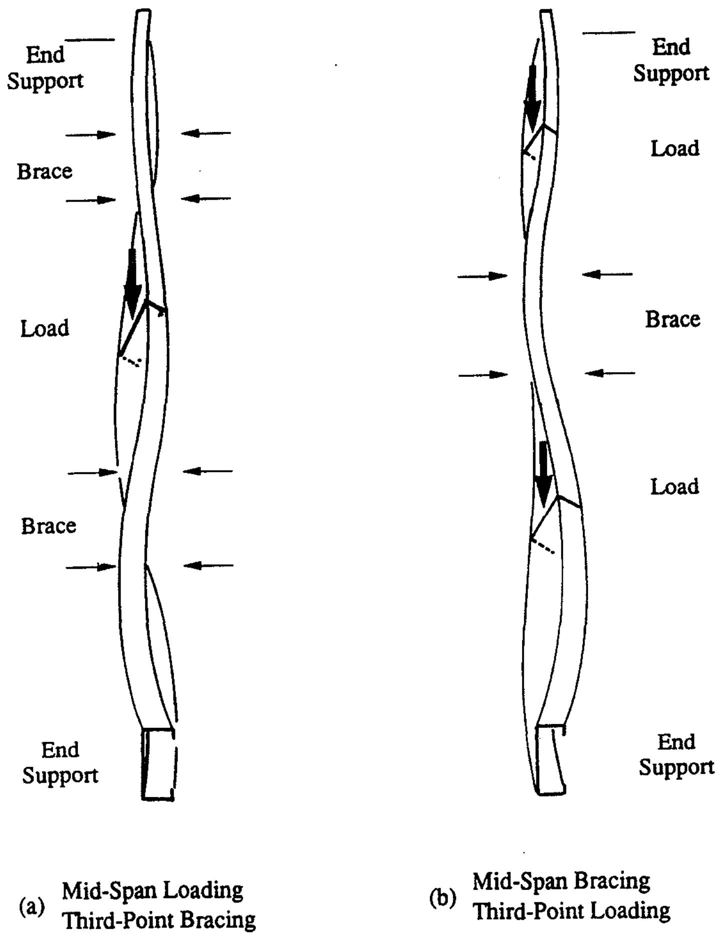
2. American Iron and Steel Institute, Addendum to the Specification for the Design of Cold Formed Steel Members, 1989.

3. Chajes, A., "Principles of Structural Stability Theory," Prentice Hall, 1974.

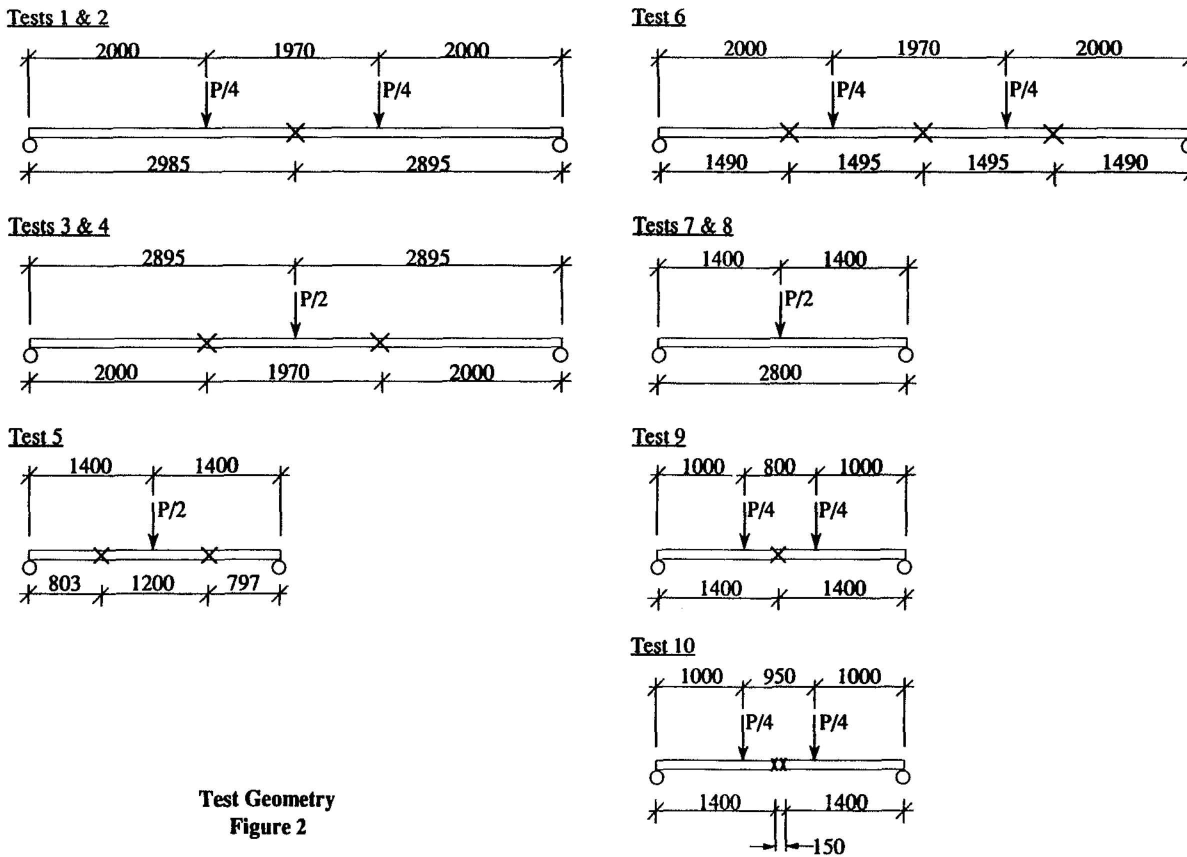
4. Krajcinovic, D., "A Consistent Discrete Elements Technique for Thin-Walled

5. Kavanagh, K. T., "Failure of Cold Formed Steel Purlins Near Points of Concentrated Load", Proceedings, 7th Australasian Conference on the Mechanics and Structure of Materials, 1980.

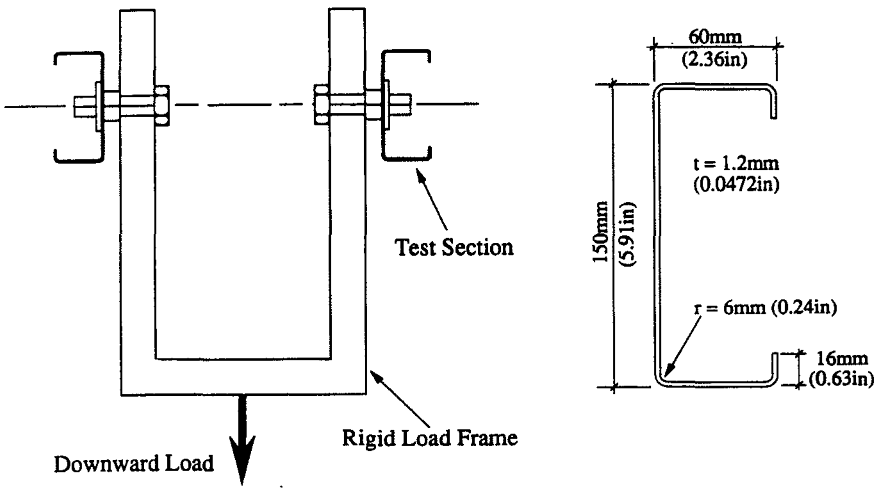
6. Timoshenko, S., and Gere, J., THEORY OF ELASTIC STABILITY, McGraw-Hill, 1961.



Deflected Shapes of Test Channels
Figure 1



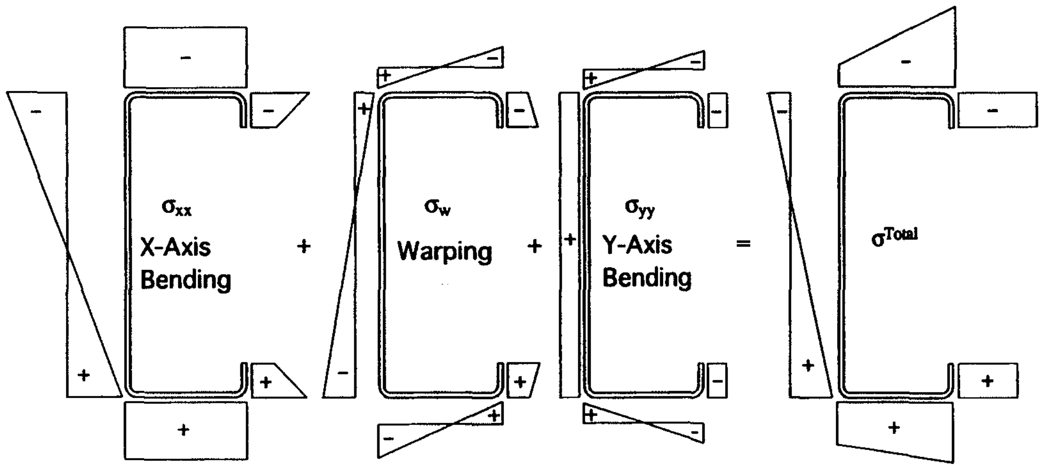
Test Geometry
Figure 2



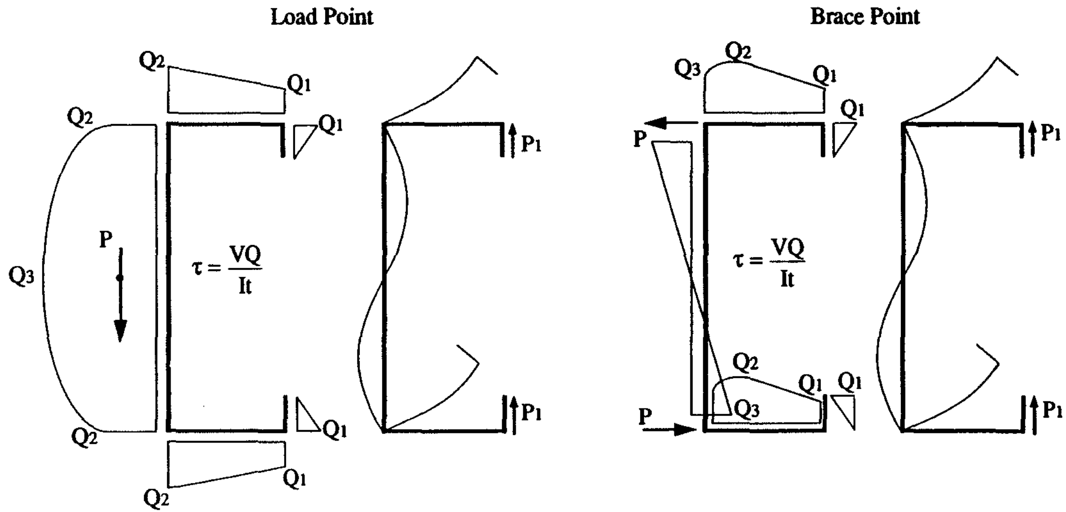
(a) Loading Frame

(b) Cross-Section

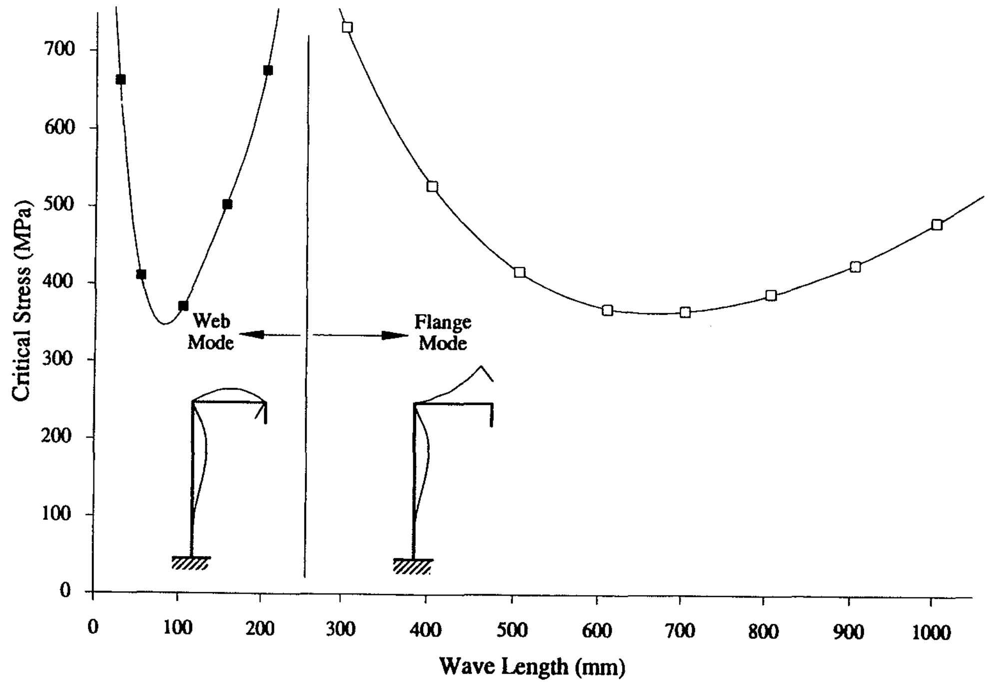
Section and Test Apparatus
Figure 3



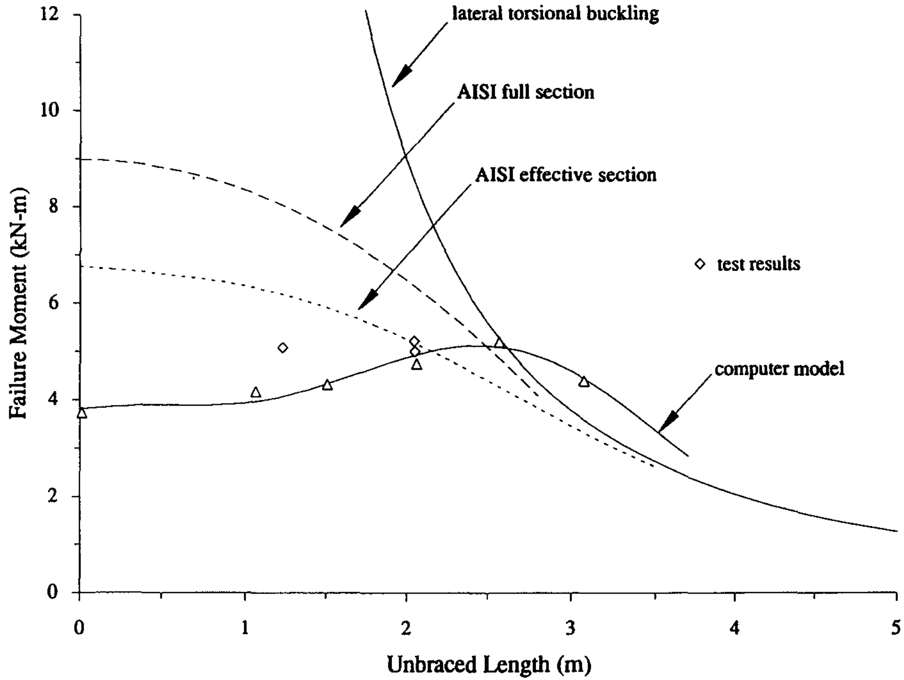
Stress Distributions
Figure 4



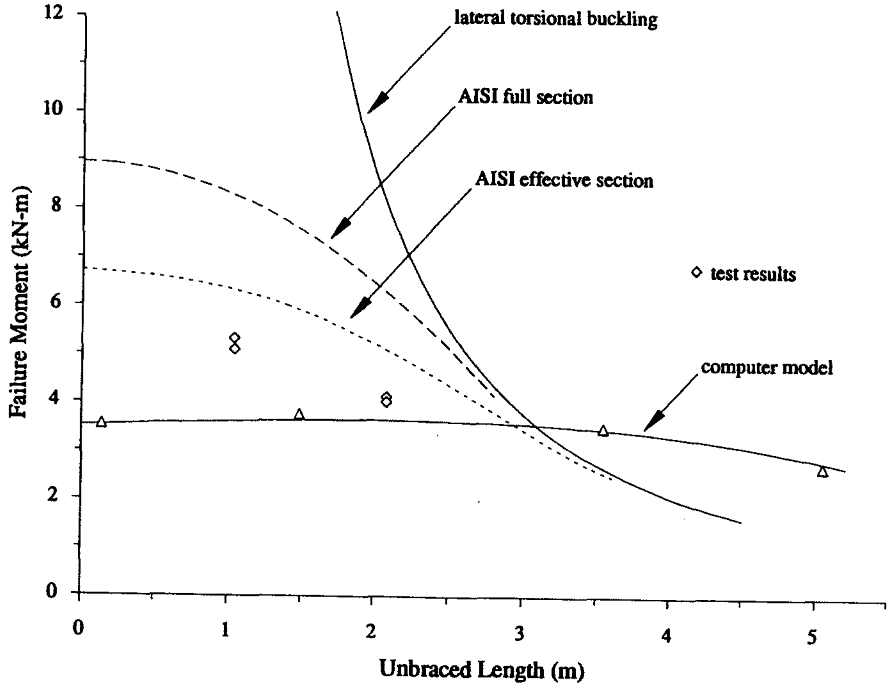
Distortion at Load and Brace Points
Figure 5



Distortional Buckling
Figure 6



Experimental and Analytical Results
Figure 7



Experimental and Analytical Results
Figure 8

A Study on the Stability and Deformation Capacity of Knee Members in System Frames

Tetsuro Ono[1], *H. Ohki*[2], *M. Yazaki*[2] *and K. Shimono*[3]

1) *Professor, Dr. Eng., Nagoya Institute of Technology*
2) *Manager, Nippon Steel Corporation*
3) *Graduate Student, Nagoya Institute of Technology*

1. Introduction

Low-rise structures are mainly one storied and long in span, as represented by factory buildings and school gymnasiums. How to erect a roof is the main design consideration for such structures. The roof may be erected above a one-storied building by arranging gabled or arched plane frames parallel in the ridge direction and connecting them into a framework, or by building a space framework. The roof may be built of H-shaped section members, arched rigid frames, or truss members. Vertical loads, such as dead load, live load and snow load, and horizontal loads, such as seismic load and wind load, produce large bending stresses in the corner knee members of frames. The ultimate strength and deformation capacity of the corner knee members determine the stability of the entire frame. In fact, there are reported many cases in which school gymnasiums, prefabricated steel frames, and truss frames failed apparently due to out-of-plane buckling in the frames containing the corner knee members.

The safety of individual structural members, such as beams and columns, is studied in the structural design of these frames. This procedure is based on the assumption that the structural members are fully supported at their joints to restrain their out-of-plane buckling and deformation in terms of design and construction. Some structures are not fully supported at corner points. In actual frameworks, purlins and furring strips, or tie beams in the ridge direction, and cross beams passing through the apex of the corner knee members are attached to beams and columns. Such members are usually connected to the outer flange or outer chord members of frames. No lateral bracing members are attached to the inner flanges and inner chord members of frames. Bending stresses due to vertical loads, such as snow load, and horizontal loads during an earthquake or a storm become compressive stresses in the inner chord members of frame corners. The apex of the inside of the corner is thus considered to laterally move out of plane, resulting in buckling. The bracing and stability of the corner knee members are an important issue in this respect. The results of two experiments conducted concerning the problem are reported here. The bracing design of corner knee members is also discussed.

2. Outline of Experiments

One experiment (experiment 1 [5]) was conducted concerning the corner knee members of a low-rise system frame, and the other (experiment 2 [1]) concerning comparison between solid web members and open web members.

Specimens in experiment 1 are beam-column corners knee members of system frame. The cross section of specimen is built-up tapered H-section and maximum height is 450mm and minimum height is 300mm. The inner flange of the specimens has a radius of curvature of 300 mm. The corner bracing problems are concerned with the bracing position, bracing rigidity, and presence or absence of stiffeners in the bracing point. The parameters of experiment 1 are the bracing position, the presence or absence of a stiffener at the corner, and the shape of the corner stiffener and loading condition. In this experiment, bracing rigidity is infinite as full bracing. Corner apex bracing types and loading conditions are illustrated in Figure 1. A total of ten specimens were used in experiment 1. Their dimensions are listed in Table 1.

Experiment 2 is concerned with the bracing and stability of solid web and open web members. As shown in Table 2, three truss members and two built-up H-shaped section members were used as specimens. Purlins, furring strips, or cross beams in the ridge direction for the corner apex are attached to the outer chord member or outer flange of a main frame. These members not only carry external forces, but also restrain the out-of-plane deformation of the main frame. In experiment 2, therefore, bracing members with strength and rigidity comparable to the purlins, furring strips, and cross beams were attached to the outer chord members of the main frames, as shown in Figure 2. The rigidity ratio of tie beams to main frame members in actual structures was obtained and used as the flexural rigidity transfer ratio to determine the cross section of bracing members used in experiment 2. Loading system is showed in Figure 2.

3. Experimental Results

3.1 Results of Experiment 1

Figure 3 shows the in-plane load-deformation curves of the specimens when the bracing method was changed. The maximum strength of each specimen reached the yield load. Compared with specimen A-0 having the out-of-plane deflection of the compression and tension flanges restrained, specimen A-3 having the tension flange alone restrained has a greater drop in deformation capacity after the maximum strength. Specimen A-8 with the rotational restrained at the apex has a deformation capacity similar to that of specimen A-0. If there is a stiffener at the bracing point, axial rigidity bracing with the tension flange alone is effective as bracing against buckling. To provide a desired deformation capacity after the maximum strength, axial rigidity bracing with the tension flange alone is not enough, but rotation must be restrained. This suggests the importance of studying the bending rigidity of tension bracing members. Web plates undergo local buckling relatively early after the maximum strength, partly under the influence of drift stress, because the inner flange of the specimens has a radius of curvature.

Figure 4 shows the in-plane load-deflection curves of the specimens when the stiffener shape was changed. The maximum strength of specimen A-2 without stiffener is about 30% of that of specimen A-3 with stiffeners on both sides. The effective buckling length of specimen A-2 is approximately the same as the distance between the supporting points at loading points. When there is no stiffener at the apex bracing point, bracing on the tension side alone is not enough. Specimen A-5 with stiffeners of 2/3 sectional depth attached on both sides has a higher strength than specimen A-2, but its maximum strength is 129 kN. Specimen A-7 with a full-depth stiffener attached on one side alone is as strong as specimen A-3 with stiffeners attached on both sides. Its loss of strength after the maximum strength is so large, however, that the desired deformation capacity cannot be expected.

Figure 5 shows the in-plane load-deflection curves of specimens A-3 and A-9 loaded by different methods. As shown in Figure 1, specimen A-9 has the outer flange subjected to compressive stress, while specimen A-3 has the outer flange subjected to tensile stress. Since specimen A-9 has the out-of-plane deformation of the compression flange restrained, it does not suffer lateral buckling deformation, and its strength rises after yielding strength under the influence of strain hardening.

3.2 Results of Experiment 2

The strength of the truss members greatly varies with whether or not the inner chord member subjected to compressive stress at the corner is restrained against out-of-plane deformation. Figure 6 shows the load-deflection curves of specimens T-1 and T-2. When the outer chord member in tension is braced and the inner chord member in compression is not braced, specimen T-2 with the highest bracing rigidity has a strength of only $P/Py = 0.7$. When there is no horizontal displacement at the column head in such arched frames, stress testing as separately performed on the beams and columns in the design of frame members. The design strength according to this consideration is $P/Py = 0.55 \sim 0.63$, and the experimentally determined maximum strength is 1.10 to 1.26 times higher. This means that the redundant strength with respect to the design strength is low and that only a small safety factor is provided when the columns and beams are separately designed. The maximum strength of specimen T-3 having the compression inner chord member and the tension outer chord member braced similarly as shown in Figure 7 is $P/Py = 0.98$ and close to the yield load Py. The maximum strength P is 1.86 times as high as the maximum strength $P/Py = 0.6$ of specimen T-1 with the outer chord member alone braced under the same conditions. This shows that bracing the inner chord member with compression stress is useful for buckling.

The solid web members were studied only in such types that the outer flange alone was braced. Figure 8 shows the load-deflection curves of specimens F-1 and F-2. As evident from the experimental results, the maximum strength exceeds the yield load Py=161 kN and reaches the full plastic moment load of the corner knee member. The maximum strength is nearly two times as high as the design load.

4. Relationships Bracing Conditions, Rigidity and Strength, Deformation Capacity of Frame

When the inner and outer chord members are restrained at the corner of open web members like truss members, the maximum strength is much higher than when the outer chord members are restrained. The buckling mode is of double-curvature type with respect to the corner apex. When the outer chord member alone is braced the corner apex of the inner chord member moves out of plane, because that the out-of-plane rigidity of the web is relatively low. The buckling mode including the beam and column members is single curvature, and the bracing effect is small. This shows that when the outer chord members alone at the corners of frame composed of truss members are braced, it is inappropriate to design separately test columns and beams against buckling at connection of beam and column, considering immoval point for lateral displacement at the apex of corner of frame . Solid web members like H-shaped sections exhibit the buckling bracing effect and provide sufficient strength when the outer chord members alone are braced. If the rigidity of the panel zone at the corner apex and the bracing strength and rigidity of the outer chord member are high enough, bracing of the outer chord member subjected to tensile stress is effective against buckling.

When the stress imposed on each solid web members is calculated backward from the reading of the strain gauge attached to the brace, it is clear that the brace is subjected to a large bending moment as well as axial force. This tendency is similar to that reported for beams[4]. Figure 9 shows the bracing rigidity to secure the deformation capacity of member based on numerical results. If the bending moment is taken as bending moment due to the lateral torsional deflection of the compression flange, it can be considered as bracing moment M=0.025Fh when 2.5% of the yield axial force is imposed as horizontal force F as shown in Figure 10.

5. Conclusions

The results obtained from the above-mentioned experiments may be summarized as follows:

(1) When the main frame is composed of truss members and when the outer chord member in tension is restrained by a cross beam or purlin, the inner chord member deforms out of plane at the corner to lose the bracing effect, because the out-of-plane rigidity of the web is small.

For the truss members, therefore, both chord members should be restrained at the corner by ridge tie beams and similar members (Figure 11). In addition, the inner chord member in compression should penetrate through the outer chord member.

(2) When the main frame is composed of solid web members like H-shaped sections, it is desirable that the out-of-plane deformation of both the inner outer chord members should be restrained at the corner. Unlike open web members, solid web members call for the restraint of only the outer flange in tension if enough strength and rigidity are provided around the panel zone composed of the column and beam. In this case, it is recommended that a stiffener be provided at each bracing point or that the braces attached to the compression flange and the tension flange be connected by tie members like knee braces.

(3) When attached to the inner and outer chord members or the compression and tension flanges, the corner knee members is 2.5% of the yield axial force of a T-shaped section containing the compression flange as shown Figure 10.

(4) When the tension flange alone of a solid web member is braced, it is necessary to pay full attention not only to the above-mentioned axial force, but also to the bending moment. If the bending moment is taken as bending momente due to the lateral torsional deflection of the compression flange, it can be considered as bracing moment M=Fh when 2.5% of the yield axial force is imposed as horizontal force F as shown in figure 10.

(5) Due care must be exercised when joining and attaching members restraining inner chord members in tension to the truss members.

References

1) T. Suzuki and T. Ono ; " Stability of Frame of School Gymnasiums ", Quarterly COLUMN 1977,Vol. 58, pp. 15-19

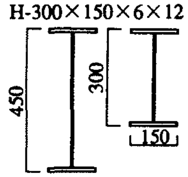
2) T. Ono, M. Araya and M. Iwata ; " Development and Ultimate Strength Test of New Steel Gable Roof Structure " IABSE Colloquium, " Thin-Walled Metal Structures in Buildings ", 1986, pp. 207-214

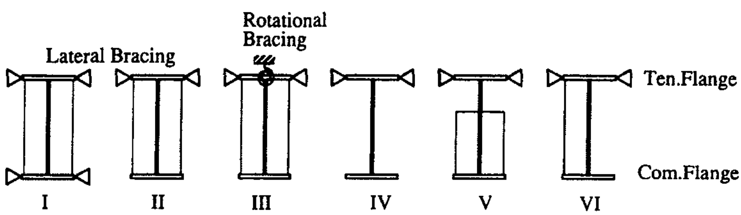
3) B. Mutton and N, Trahair ; " Stiffeness Requirements for Lateral Bracing " 1973, October, ASCE, ST10, pp. 2167-2182

4) T. Ono and T. Suzuki ; " Bracing Rigidity of Plastic Designed Steel Beams " Transaction A.I.J., Vol.202, 1972. 12, pp.31-39

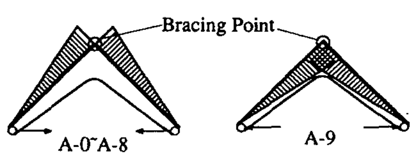
5) T. Ono, K. Shimono ; "Experimental Study for Corner Knee Members of System Frame", Trans. of Tokai branch of A.I.J., Vol.31, 1993, 2, pp.221-224

Table 1 Specimen of Experiment 1

Specimen	Section	Yield Stress		Bracing Condition
		σ_{yf}(N/mm^2)	σ_{yw}(N/mm^2)	
A-0	H-450×150×6×12	264	369	I
A-1	H-300×150×6×12	264	369	II
A-2		264	369	IV
A-3		264	369	II
A-4		264	369	II
A-5		264	369	V
A-6		264	369	II
A-7		264	369	VI
A-8		264	369	III
A-9	R=300	264	369	II



Bracing Condition



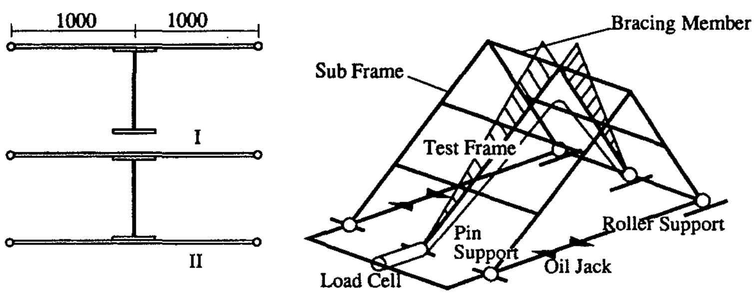
Loading Condition

Fig.1 Bracing and Loading Condition

Table 2 Specimen of Experiment 2

Specimen	Chord Member		Lacing Member		Bracing	Bracing
	Section	σ_y(N/mm²)	Section	σ_y(N/mm²)	Condition	Member
T 1	2L-65×65×6	327	2L-45×45×4	354	I	▭-11.5×100
T 2		327	2L-45×45×5	354	I	▭-21.5×100
T 3		327	2L-45×45×6	354	II	▭-11.5×100

Specimen	Member	Yield Stress		Bracing	Bracing
	Section	σ_{yf}(N/mm²)	σ_{yw}(N/mm²)	Condition	Member
F 1	H-350×150×6×12 H-175×150×6×12	281	304	I	▭-11.5×100
F 2		281	304	I	▭-21.5×100



Bracing Condition

Loading System of Frame

Fig.2 Bracing and Loading Condition

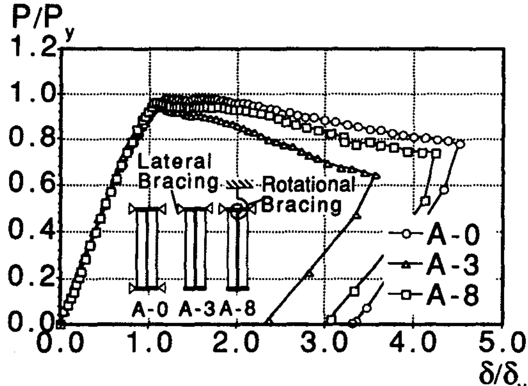


Fig.3 Load-Deflection Curves

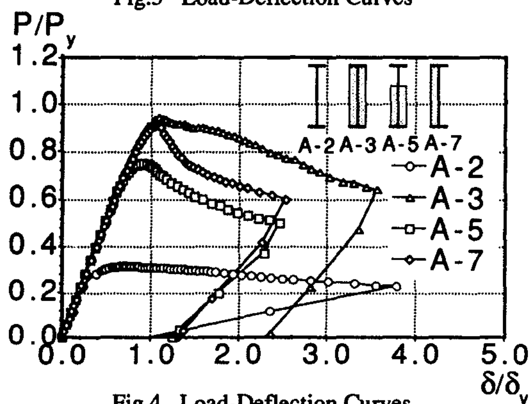


Fig.4 Load-Deflection Curves

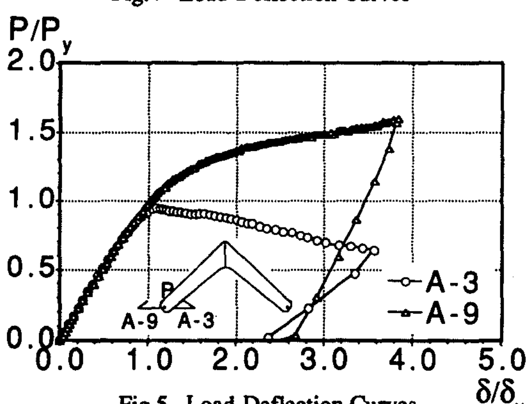


Fig.5 Load-Deflection Curves

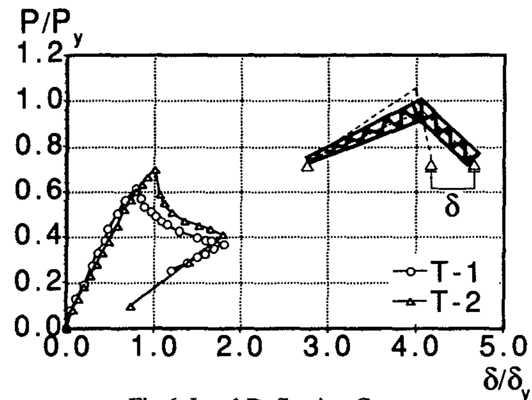


Fig.6 Load-Deflection Curves

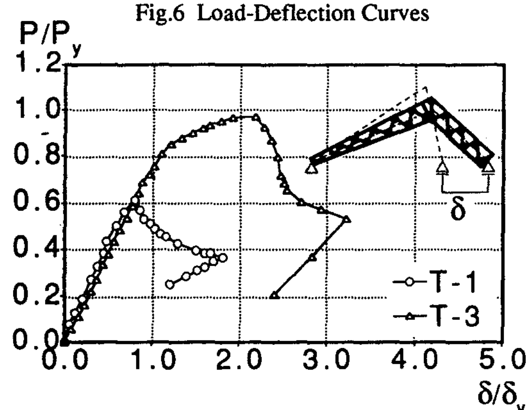


Fig.7 Load-Deflection Curves

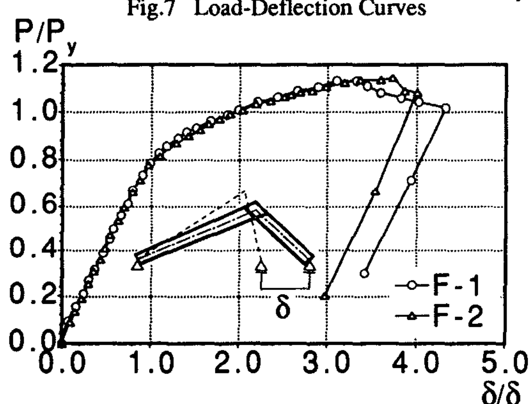


Fig.8 Load-Deflection Curves

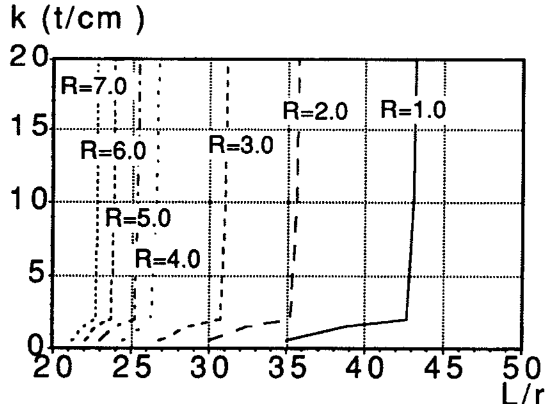


Fig.9 Bracing Rigidity for Deformation Capacity

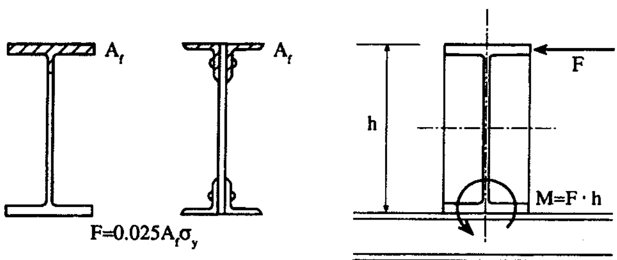


Fig.10 Bracing Force

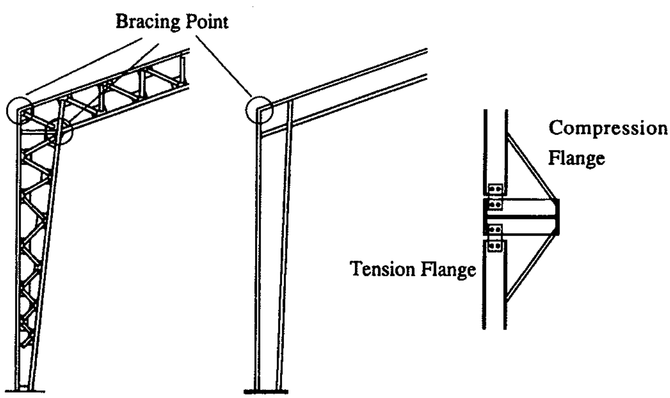


Fig.11 Bracing Point

BRACING DESIGN FOR INELASTIC STRUCTURES

Joseph M. Ales Jr. and Joseph A. Yura
The University of Texas at Austin

INTRODUCTION

Most theoretical research on bracing requirements for structures are based on elastic concepts. A summary of elastic bracing solutions for beams and columns is given by Trahair and Nethercot (1984). Pincus (1964) used a simple model to demonstrate that the bracing stiffness requirements for inelastic columns are greater than those for elastic columns. Based on the work of Pincus (1964), Trahair and Nethercot (1984) conclude that the summary of their stiffness requirements will progressively underestimate the required bracing stiffness as the column slenderness decreases, but no adjustments are given to account for the increased brace requirements as a result of the inelastic behavior. For full bracing, i.e. the bracing is sufficient to force buckling between the braces, Winter (1958) showed that the bracing requirements could be derived using a rigid link model. Winter's full bracing requirements are a function of the column load and the distance between braces, not column elasticity. Winter's approach is the basis for most bracing design recommendations. There is no experimental evidence available for bracing requirements of inelastic columns to evaluate the contention of Pincus.

For beams in the inelastic range, most research has been concerned with the spacing of the braces, not the properties of the braces. The Commentary on Plastic Design in Steel (ASCE, 1971) gives requirements for bracing at plastic-hinge locations. In the ASCE recommendations the lateral brace must have axial strength, axial stiffness and flexural stiffness. Experiments on simply-supported beams do not verify the need for flexural stiffness in the lateral braces. In addition, the brace requirements for axial stiffness and strength are less than those required by Winter's approach. A design example illustrating both Winter's approach and the ASCE approach (neglecting the flexural stiffness requirement) is given in Salmon and Johnson (1980). Both approaches give the same size brace in the example. Wong and Nethercot (1989) conducted a theoretical study of brace stiffness and strength requirements for beams with a concentrated load at midspan. Their study verified the Winter approach especially on the need to use at least twice the ideal full bracing stiffness in order to reduce the brace forces. The brace forces were less than 1% of the flange force when the recommended stiffness was provided. The results appear to verify Winter's approach for use with inelastic beams, but the loading condition considered only involved a small amount of inelasticity near midspan. Nakamura (1988) presents a few experiments which appear to follow the trends suggested by Winter's approach but no test details are given, including the loading condition.

While there are only a few documented studies on bracing requirements for inelastic beams and columns most design is in the inelastic range of behavior in order to achieve maximum utilization of material. Obviously, closer spacing of bracing must occur in order to reach the plastic region, but this does not necessarily mean the brace requirements are substantially changed by the inelastic behavior of the member to be braced. Undocumented failures of test setups in experiments when instability occurs in the inelastic range, has contributed to the notion that inelastic structures require larger bracing than elastic structures. Similar observations have been noted by colleagues when examining actual structural failures due to earthquake, wind and snow. However, in the authors' experience, when a lateral bracing failure occurs in a load test into the inelastic range, it usually happens <u>after</u> a local flange or web buckle occurs, which causes the w-shape beam to become unsymmetrical. The loss of symmetry of the section causes shifts and inclinations of the principal bending axes which can cause very substantial lateral and torsional forces, much like those in channel sections not loaded through the shear center. Lateral bracing forces caused after local buckling occurs are very substantial but are beyond the scope of this presentation. However, since most local buckling occurs in the plastic range, bracing failures are often associated with inelasticity rather than local buckling.

The objective of this paper is to provide bracing design recommendations for beams and columns loaded into the inelastic range before local buckling occurs. These recommendations apply to flexural buckling only; torsional buckling must be addressed separately. The recommendations are based on Winter's model for discrete bracing and the tangent modulus concept for continuous bracing. In the next sections, Winter's model is reviewed and the Pincus theory evaluated. Two beam experiments are presented which support the use of the Winter Model.

WINTER'S MODEL

The effect of brace stiffness on the elastic buckling strength of a simply-supported straight column with a mid-height brace is shown in Figure 1 (Timoshenko and Gere, 1961). There is an almost linear relationship between P_{cr}, the column buckling load, and β_L, the brace stiffness, until the maximum load corresponding to buckling between the braces is reached. At $\beta_L = 2P_e / L_b$ full or ideal bracing is achieved. Further increases in brace stiffness do not affect the column strength. The general buckling solution for any value of brace stiffness shown in Figure 1 is complicated, but Winter (1958) developed a simple solution for the particular case of full bracing.

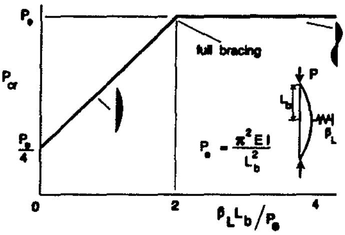


Figure 1

Winter's model for the determination of the ideal brace stiffness is shown in Figure 2. The column is represented by two straight rigid links, pinned at the center at the bracing location. Assuming a small movement, Δ, at the brace point when the column load P is applied, equilibrium or one-half of the column requires $P\Delta = \beta\Delta L_b / 2$ or $\beta = 2P / L_b$, the same solution shown in Figure 1. This model gives the correct ideal brace stiffness for any number of equally spaced braces (Salmon and Johnson, 1980). Winter's solution does not require the use of elastic principles. His model indicates that only the magnitude of the load, the distance between braces, and the number of braces affects the stiffness required for ideal bracing. Say, two columns have the same length and bracing spacing, but have different sizes of members and may be constructed from two different materials. If it so happens that the load corresponding to buckling between the braces is the same for both cases, then the ideal brace stiffness will be the same for both columns. Similarly, if the two columns are constructed from the same material, but because they have different sizes, one buckles in the elastic range and the other column buckles at the same load but is in the inelastic range of behavior, there will be no difference in the bracing requirements.

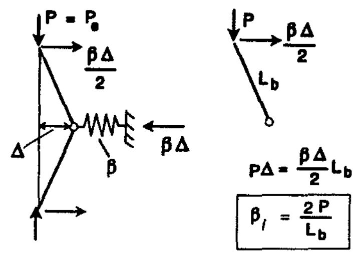


Figure 2

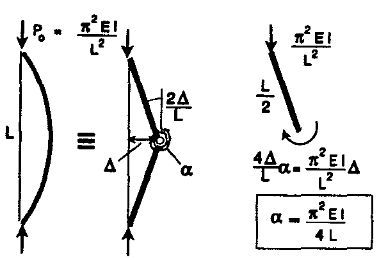


Figure 3

PINCUS MODEL

The rigid link model developed by Pincus (1964) is shown in Figure 3. His model is similar to Winter's model with the addition of a rotational spring at the midspan, which represents the bending stiffness of the column itself. When there is no brace at the mid-height, the column will buckle at $P_o = \pi^2 EI / L^2$. For the rigid link model to give the same answer, a rotational spring with a stiffness of $\pi^2 EI / 4L$ is required. When a lateral brace is attached to the linkage system, as shown in Figure 4, equilibrium for one half the column gives

$$P = \frac{\pi^2 EI}{L^2} + \frac{\beta L_b}{2} \tag{1}$$

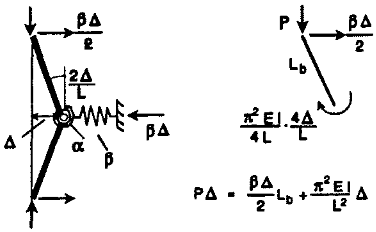


Figure 4

Pincus reasoned that if the column buckled in the inelastic range, then the π^2 EI / L^2 contribution would diminish based on the tangent modulus concept, thus increasing the brace stiffness required to reach the same load as an elastic case.

The conclusion reached by Pincus is questionable because his model does not give the correct value of ideal stiffness at full bracing, even when the column is elastic, as follows. For elastic behavior, at ideal bracing $P_{cr} = \pi^2$ EI / L_b^2. Since L = $2L_b$, Equation (1) gives a required β_{id} = 1.5 π^2 EI / L_b^3 = 1.5 P / L_b. The exact stiffness at full bracing is 2P / L_b, as shown in Figures 1 and 2. At full bracing there is an inflection point at the brace point (zero moment) and Winter's model which satisfies this condition gives the correct answer. No rotational spring at midspan is necessary at full bracing so the Pincus model is incorrect for full bracing. His model would be more appropriate at brace stiffness substantially below the ideal case.

BRACING RECOMMENDATIONS IN THE INELASTIC RANGE

Based on the previous discussion, Winter's method is recommended for both the elastic and inelastic ranges of behavior. Winter's method is applicable to bracing systems which are attached at a discrete number of points along the length, and the load level corresponds to buckling between the brace points. In this case the brace requirements are simply a function of the magnitude of the load and the unbraced length. Since Winter's recommendations are documented elsewhere (Salmon and Johnson, 1980), they will not be repeated here.

If the bracing system is continuous, Winter's simply approach will not give the exact results. For continuous column bracing, Timoshenko and Gere (1961) give the following elastic relationship between P_{cr} and the continuous brace stiffness $\overline{\beta}$,

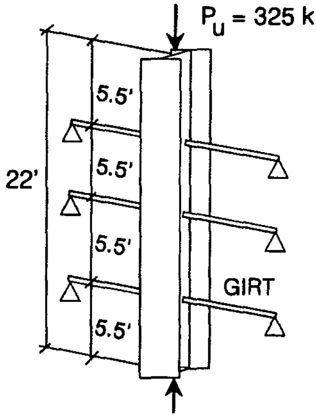
$$P_{cr} = P_o \left(n^2 + \frac{\overline{\beta} L^2}{n^2 \pi^2 P_o} \right) \tag{2}$$

where $P_o = \pi^2$ EI / L^2, L is the span length and n is the number of half sine waves in the buckled shape. To use this solution with a given $\overline{\beta}$, substitute integer values of n = 1, 2, 3, etc. in Eq. (2) and use the smallest result. Equation (2) has no limit except $P_{cr} \leq$ yield load. If P_{cr} is in the inelastic range of column behavior, it is recommended that the tangent modulus concept be used. That is, replace E in P_o with E_T. For the CRC column curve the tangent modulus is

$$\frac{E_T}{E} = 4 \frac{P}{P_y} \left(1 - \frac{P}{P_y} \right) \tag{3}$$

where P is the applied column load and P_y is the yield load. For other column curves such as that given in the AISC steel specification, the ratio E_T/E is the same as the "stiffness reduction factor" given in tabular form in both the ASD and LRFD versions of the specification. A design example follows illustrating the approach for continuous bracing of inelastic columns.

EXAMPLE PROBLEM



WHAT GIRT STIFFNESS IS NECESSARY TO ADEQUATELY BRACE THE COLUMN?
W12X40 : A36 STEEL

DETERMINE THE STIFFNESS USING THE CONTINUOUS BRACE FORMULA

$$P_u = \tau P_E \left[n^2 + \frac{\overline{\beta}_L L^2}{n^2 \pi^2 P_E \tau} \right]$$

- SOLVING FOR $\overline{\beta}_L$

$$\overline{\beta}_L = \frac{n^2 \pi^2}{L^2} (P_u - \tau P_E n^2)$$

- LET $\overline{\beta}_L$ = $3\beta_{id}$ / L where β_{id} = ideal girt stiffness
  L = total column length

$$\beta_{id} = \frac{n^2 \pi^2}{3 L} (P_u - \tau P_E n^2)$$

CALCULATE P_E

KLy = 22' : P_E = 135 kips : AISC - LRFD p. 2 - 25

CALCULATE τ

$$\frac{P_u}{\phi A} = \frac{325}{0.85\ (11.8)} = 32.4\ ksi\ :\ \tau = 0.258 \quad : \text{AISC - LRFD p. 2 - 8}$$

STIFFNESS REQUIRED

$$\beta_{id} = \frac{n^2 \pi^2}{3(22)(12)} (325 - 0.258(135)\, n^2) = \frac{n^2}{80.25} (325 - 34.83\, n^2)$$

- n = 1 : β_{id} = 3.62 k / in
- n = 2 : β_{id} = 9.26 k / in ✓ CONTROLS
- n = 3 : β_{id} = 1.29 k / in

DESIGN STIFFNESS

$$\beta_{DESIGN} = \frac{2\beta_{id}}{\phi} = \frac{2(9.26)}{0.75} = 24.7\ k\ /\ inch$$

WINTER'S FORMULA FOR A COLUMN WITH THREE DISCRETE BRACES

$$\beta_{id} = \frac{3.41\ P_u}{L_b} = \frac{3.41(325)}{5.5(12)} = 16.8\ k\ /\ inch$$

$$\beta_{DESIGN} = \frac{2(16.8)}{0.75} = 44.8\ k\ /\ inch$$

EXPERIMENTAL TESTS

Two tests were conducted on simply supported S6X12.5 beams. Two point loads were applied to produce a constant moment along the center section of the beam. The test setup is shown in Figure 5. The end supports and the loading locations were rigidly braced out - of - plane; at the center span location, a rigid brace was used in the first test and an elastic brace was used in the second test. The span lengths were calculated to cause inelastic buckling of the constant moment spans. A plastic moment test on a short section of an S6X12.5 was conducted to determine the moment - curvature relationship. From this relationship, a moment vs. modulus of elasticity curve was calculated; this curve is shown if Figure 6. Buckling was targeted to occur at an inelastic moment of 356 in-kips, which corresponds to a tangent modulus of 3650 ksi (.13E). The plastic moment was 373 in - kips. The center spans were treated as the buckling spans and the end spans were treated as the restraining spans.

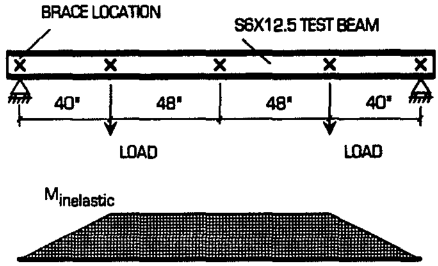


Figure 5: TEST SETUP

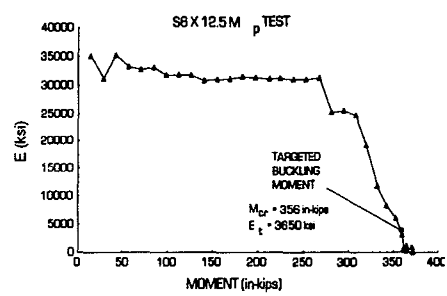


Figure 6: E vs. M : M_p TEST

DETERMINATION OF BRACE STIFFNESS FOR TEST #2

Winter's model shows that for a column with an elastic support in the center, the stiffness necessary to make the elastic support act as a rigid support is β_{id} = 2P / L_b, where β_{id} is the ideal spring stiffness required, P is the column buckling load, and L_b is the distance from the brace to the end support. See Fig. 2. This model can be used for determining the bracing requirements for beams by replacing P with the force in the compression flange of the beam when bucking occurs. For these tests, the buckling moment was 356 in-kips; dividing this by the distance between the flange centroids, 5.75", the force in the compression flange at buckling was P = 356 / 5.75 = 62 kips. The ideal brace stiffness required was β_{id} = 2*62 / 48 = 2.58 k/in. The ideal stiffness cannot be used in actual tests because any imperfections in the member will cause the brace force to approach infinity. Therefore, the ideal brace stiffness was increased by 20% to β_{req} = β_{id} (1.2) = 2.58 * 1.2 = 3.1 k/in.

SUPPORT FIXTURES

The test setup is shown in Figure 7. At the beam support locations, rollers provided the in - plane support and brace plates provided the out - of - plane support. See Figure 8. At the load points and at center span, out - of - plane bracing was used. The bracing allowed the in - plane deflection of the beam, caused by the applied loading, and the out - of - plane rotation, caused by lateral buckling. A schematic of the fixture for the load point bracing is shown in Figure 9. The bracing elements are held by inclined double angle supports that are bolted to the test floor and welded together at a point above the test beam. A steel plate <A>, centered between and bolted to the inclined double angles, supports another steel plate <B> that has a strip of teflon epoxied to its surface. Plate <B> is free to rotate about the edge of plate <A>. Aluminum clips, with teflon pieces epoxied to its surface, are attached

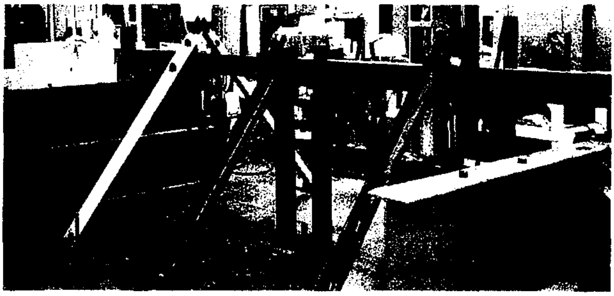


Figure 7: TEST SETUP

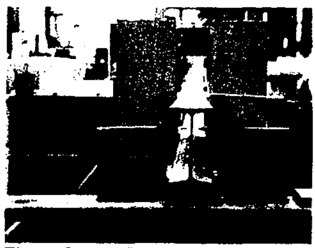


Figure 8: END SUPPORT

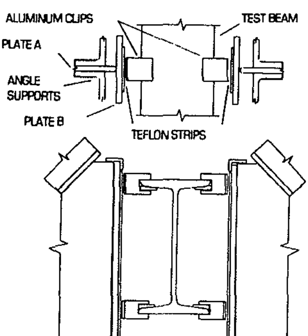


Figure 9: LOAD POINT BRACE

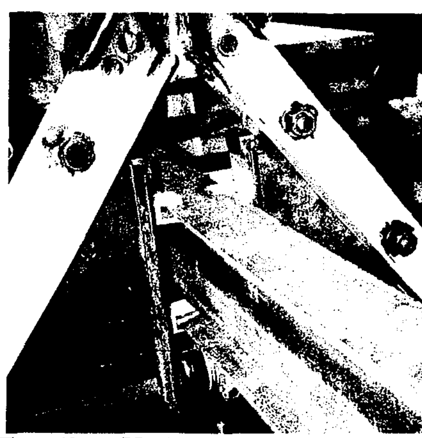


Figure 10: BRACING ELEMENTS

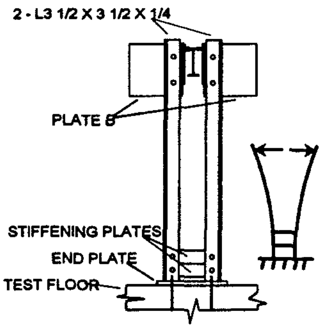


Figure 11: ELASTIC BRACE

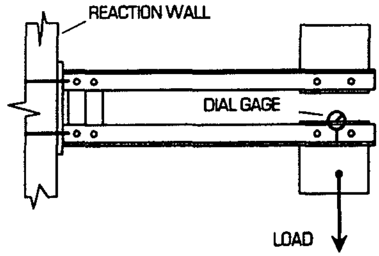


Figure 12: STIFFNESS CALIBRATION

to the top and bottom flanges of the test beam. The low friction teflon surfaces of the clips and plate <B> allows free vertical deflection of the test beam. A picture of the bracing elements is shown in Figure 10.

For the center span location, it was necessary to design a brace with rigid supports for the first test and a brace with elastic supports for the second test. The bracing elements used at the load points were also used at the center span location; only the top flange was braced. A schematic of the elastic brace support for Test # 2 is shown in Figure 11. It consists of L 3 1/2 X 3 1/2 X 1/4 angles, placed back - to - back, that are welded to an end plate, which is bolted to the test floor. The flexural stiffness of the cantilevered angles provides the lateral brace stiffness. The calibration of the brace support stiffness is shown in Figure 12. The brace support, bolted to a reaction wall, is loaded with steel plates at the bracing location. Two dial gages that can read to 0.001" measure the deflection at the bracing location. The stiffness of the brace support was the slope of the load - deflection curve, using the average of the slopes of two calibration tests. The calibrated stiffness of the brace support was 2.96 k/in.

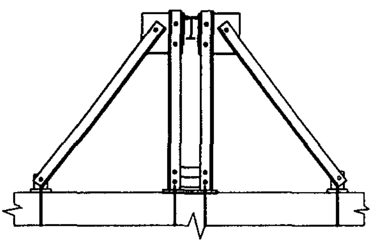


Figure 13: RIGID BRACE

The rigid brace was constructed by stiffening the existing elastic brace with inclined double angle buttresses; the schematic of the rigid brace support is shown in Figure 13.

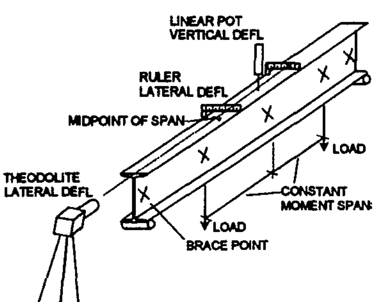


Figure 14: INSTRUMENTATION

TEST SETUP

The test beam was loaded by two 12 kip capacity rams. Load was measured with a pressure transducer. A linear potentiometer measured the vertical deflection at the center brace. Lateral deflections were measured at the center of the two interior spans by using a ruler, graduated in hundredths of an inch, and a theodolite. See Figure 14. For the test with the elastic brace support, dial gages at the braces measured lateral deflection. The beams were painted with whitewash to identify yielding.

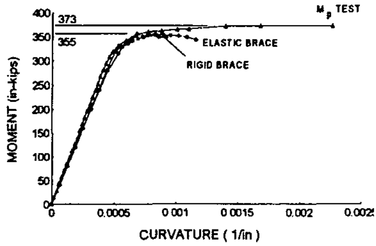


Figure 15: M - ϕ CURVES

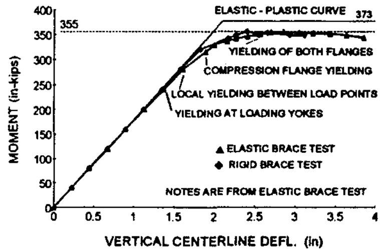


Figure 16: M - Δ_{CL} CURVES

Table 1: Sumary of Results				
TEST	BRACE STIFF k/in.	INITIAL IMPERFECTION at midspan	M_{cr} in - kips	S-Shape
1	∞	0.30"	355	YES
2	2.96	0.031"	355	YES

Load was applied statically in increments of 1 kip, up to a load of 8 kips; at this point, the first major yield lines appeared. Once yielding began, load was deflection controlled; readings were taken after each 0.15" increment in the centerline vertical deflection. Each load step was paused 5 minutes and the reading was taken after the pause. A real - time plot of the load vs. centerline vertical deflection was used to gage specimen behavior.

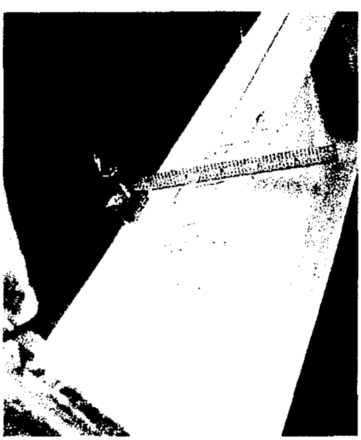


Figure 17: TOP FLANGE YIELDING

RESULTS

A summary of the test results is shown in Table 1. The results show that the brace stiffness was sufficient to cause S - shape buckling of the beam. In Figure 15, the moment - curvature curves of the two tests are compared with the curve from the plastic moment test. In both of the test beams, the moment peaked at 355 in - kips and then fell off, due to the lateral buckling. This moment is below the plastic moment of 373 in - kips and is essentially the same as the predicted buckling moment of 356 in - kips.

The moment vs. vertical centerline deflection plot is shown in Figure 16. Behavior was linear until the moment reached approximately 320 in - kips. Initial yielding began at the load yoke points; yield lines then appeared on both the top and bottom flanges; the yield lines on the compression flange indicated the horizontal curvature of the buckled shape. This is shown in Figure 17. Yielding occurred along the entire length of the constant moment spans.

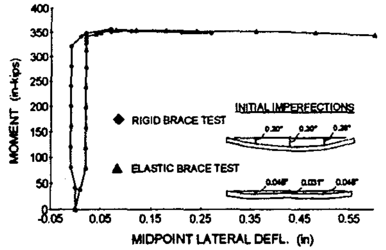


Figure 18: M - Δ_{LAT}

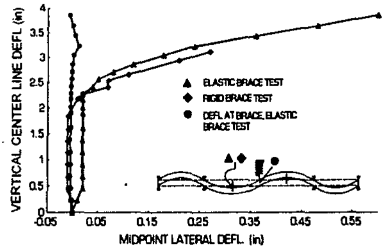


Figure 19: Δ_{CL} - Δ_{LAT}

The moment vs. lateral deflection curves for both tests are shown in Figure 18. After initial lateral settlement of the bracing fixtures, no additional lateral deflection occurred until the yield load was exceeded. The initial imperfections for the two beams are shown in the lower right corner of the graph. The initial out - of - straightness of the beam section between the center span brace and the load points was approximately L / 700 for Test # 1 and L / 3000 for Test # 2. Plots of the vertical midspan deflection vs. the south end lateral deflection for both tests are shown in Figure 19. Also shown is the lateral deflection of the elastic brace used in Test # 2. The generic buckled shape of the beam for both tests is shown in the lower right corner of the graph.

CONCLUSIONS

The experimental test program showed that Winter's model is valid for members loaded into the inelastic range. Nakamura has also published results that verify this hypothesis. Winter's model for discrete braces is a function of the column load and the unbraced length; the column elasticity does not affect the bracing requirements. For continuously braced columns, the elasticity of the column is addressed by using τ in the bracing equation. The bracing equation for continuously braced columns can be used to determine the bracing requirements for columns with multiple discrete braces. The Pincus model, which shows that inelastic columns require more bracing than elastic columns, has been shown to be incorrect due to a flawed extrapolation from the Winter rigid bar model. Future research, focusing on experimental testing, is necessary to determine the bracing requirements for columns and beams loaded into the inelastic range and to verify the bracing equations presented in this paper.

REFERENCES

Salmon, C.S. and Johnson, J.E. (1980) Steel Structures - Design and Behavior, 2nd Ed., Harper and Row, New York

Wang, Y.C. and Nethercot, D.A. (1989) "Ultimate Strength Analysis of Three-Dimensional Braced I-Beams," Proc. Institution of Civil Engineer, Part 2, 87, March, London, p. 87-112.

American Society of Civil Engineers (1971) "Commentary on Plastic Design in Steel," 2nd Ed., ASCE Manual No. 41, New York.

Pincus, G. (1964) "On the Lateral Support of Inelastic Columns," AISC Engineering Journal, 4th Quarter, pp. 113-115.

Trahair, N.S. and Nethercot, D.A. (Rhodes J. and Walker, A.C., eds.) (1984) "Bracing Requirements in Thin Walled Structures," Developments in Thin-Walled Structures - 2, Elsevier, London, pp. 93-130.

Nakamura, T. (1988), "Strength and Deformability of H-Shaped Steel Beams and Lateral Bracing Requirements," J. Construction of Steel Research, 9, 217-228.

Timosenko, S.P. and Gere, J.M. (1961), Theory of Elastic Stability, McGraw-Hill, New York.

Winter, G. (1958), "Lateral Bracing of Columns and Beams", J. Structural Division, Proc. ASCE, 84, ST2, March 1561-1-1561-22.

BRACING OF TRUSSED BEAMS

by
Theodore V. Galambos
Department of Civil and Mineral Engineering
University of Minnesota

INTRODUCTION

Trussed beams, more popularly known in North America as *steel joists*, are mass produced trusses used as direct load-carrying elements for supporting roof or floor loads, or as primary structural members which support the steel joists (*joist girders*). *These* structural members are selected from load tables in the Steel Joist Institute (SJI) Load Tables. The designs of the trusses are based on the SJI Specification (Steel Joist Institute, 1992).

Such structural components are strong and laterally stable once they are in their final erected state under the service loading, but they are extremely slender in the lateral direction during the various stages of construction, and, therefore, great care must be exercised in providing suitable lateral bracing. The ratio of stiffness in the plane of the truss to the out-of-plane stiffness is very large, and some of the more slender joists are difficult to erect without special bridging. The SJI Specification has many provisions that deal with lateral bracing. This paper will present some of the background for these rules.

The research for the development of the SJI bracing criteria is reported in the MS or Ph.D. theses of the following persons: Leigh (1971), Minkoff (1975), Westerheide (1979), Masoumy (1980) and Xykis (1988). The following publications deal with the bracing requirements of steel joists: Hribar and Loughlin (1968), Galambos (1988) and Galambos and

Xykis (1991).

The design of the individual steel joists is the purview of the joist manufacturer who provides trusses which will support the load specified in the Load Tables. The design engineer can thus easily select a joist from the tables. *However, it is the responsibility of the design engineer to define the bracing necessary in the various stages of erection and in the final service condition.*

Consideration must be given to the following stages in the process of erecting a steel joist roof or floor system:

Stability under self-weight.

Stability under the weight of an erector on the joist.

Stability under construction loading.

Stability under service loading.

STABILITY UNDER SELF WEIGHT AND UNDER WEIGHT OF ERECTOR

In general, steel joists are underslung trusses, and they are erected in the following sequence: First the joists are placed in their approximate location between the supporting walls or beams. One end is welded to the end-support in its exact location. An erector then proceeds to install the lateral bracing (also called *bridging*), at the same time drawing the member into its correct location. When all the bridging is in place, the ends of the bridging lines are anchored, and the other end-seat of the joist is welded to its support. The placing and fastening of the bridging is the most dangerous part of the erection sequence. Once the bridging is in place, both ends of each joist are welded down, and the bridging lines are properly anchored, construction may proceed by installing the deck, the flooring, the fireproofing, or the roofing and insulation. Under no circumstance is it wise to place construction load on the system until the bridging installation is complete.

Two types of bridging are used in steel joist construction: diagonal bridging and horizontal bridging (Fig. 1). Horizontal bridging is used exclusively if the joist length is less than approximately 40 ft, a central diagonal brace is used in the range of around 40 ft to 60 ft, while above that span all bridging is of the diagonal variety. If diagonal bridging is used, it (the center row) must be installed before the crane hoisting cables are released. Only then can the erector be allowed to climb on the joist.

The critical length when the central diagonal brace must be in place before the erector can venture on the joist is determined by assuming a uniform beam with simple supports in the plane of the truss, ends prevented from twist and lateral deflection, and loaded by a uniformly distributed force acting through the centroid of the cross section and a central concentrated load acting at the level of the centroid of the top flange. The uniformly distributed load is the self-weight of the joist, and the concentrated load is the weight of the erector, which is taken to be 300 lbs. The answer is found by solving for the length, L, in Eq. 1 (Minkoff, 1975). This equation is derived by the Rayleigh-Ritz method with an assumed sinusoidal lateral and torsional deformation. The critical lengths in the SJI Specification are determined by this formula.

$$P^2\left[\frac{\pi^2+4}{16}\right]^2+P\left[\frac{(\pi^2+3)(\pi^2+4)wL}{192}-\frac{\pi^4EI_y}{2(KL)^3}\left[\frac{\pi^2+4}{16}\beta_x-a\right]\right]+$$
$$\left[\frac{\pi^2+3}{24}\right]^2(wL)^2-wL\left[\frac{\pi^4EI_y}{2(KL)^3}\right]\left[\frac{\pi^2+3}{24}\beta_x-\frac{yo}{2}\right]- \tag{1}$$
$$\left[\frac{\pi^4EI_y}{2(KL)^3}\right]\left[\frac{\pi^4EC_w}{2(KL)^3}+\frac{\pi^2GJ}{2KL}\right]=0$$

In this equation the terms are defined as follows:

P=300 lbs, weight of erector

w=weight of joist per unit length

E=modulus of elasticity

G=shear modulus

L=length of joist

I_y=out-of-plane moment of inertia of top and bottom chord

K=effective length factor, 1.00 if ends are not welded down, 0.85 if one end is welded down

β_x=a cross-sectional property defined in Fig. 2

A_b=area of bottom chord

A_t=area of top chord

I_x=moment of inertia of cross section about its major axis

a=distance between shear center and point of load application (neg. if above shear center)

y=centroidal distance between centroid of top chord and centroid of cross section

y_o=distance between shear center and centroid, (neg. if shear center if above centroid)

d_e=effective depth of section

C_w=warping constant

J=St. Venant torsion constant

STABILITY UNDER CONSTRUCTION LOAD

The construction load consists of the weight of decking and the weight of the workmen as they install the decking by welding it at intervals of no more that 36 in to the top chords of the joists. The whole system is assumed to be braced only by the bridging. Two problems need to be considered: a) the spacing of the bridging and b) the magnitude of the anchorage forces. The first requirement of bracing was investigated by Hribar and Loughlin (1968) and Leigh (1971), while the bracing forces were studied by Masoumy (1980). Xykis (1988) and Masoumy (1980) also studied the bracing problem as a three-dimensional frame system. Lateral buckling tests on full-scale individual joists (Hribar and Loughlin, 1968 and Leigh, 1971) and on model-sized multiple joist systems, as well as the theoretical studies, demonstrated that the following approximations can provide a sufficiently accurate basis for developing the design criteria that underlie the bridging rules in the SJI Specifications:

Bridging spacing:

$$\frac{L_{cr}}{ry} = \sqrt{\frac{\pi^2 E}{0.9F_{cr}}} \tag{2}$$

where L_{cr} = spacing bridging lines

r_y = radius of gyration of top chord about its y-axis

F_{cr} = assumed stress in the top chord under construction loads

(17 ksi for open-web joists and 12 ksi for longspan joists)

Anchorage forces:

$$F_{required} = 0.0025\ n\ P_{allowable} \tag{3}$$

where n - number of joists in a row

$P_{allowable}$ = allowable stress in top chord

The required anchorage forces in the SJI Specification for the construction load condition are determined for eight joists in a row loaded to produce the critical stresses listed above. Equation 3 is based on an initial crookedness of 1/920 of the distance between bridging lines. Figure 3 shows the variation of the bridging anchorage forces with the number of loaded joists. A comparison is made between the predictions from Eq. 3 and an analytical calculation. The variation of the forces in the center row of horizontal bridging are shown in Fig. 4 for a roof system of ten joists in a row with seven lines of bridging. The joists are assumed to be fully loaded with their design service loads, as is the case when the decking cannot provide lateral bracing (see comments about standing-seam roofs below). These forces were determined by analysis of the system of braced chords. It should be noted that the bridging forces can be in compression despite the fact that both ends of the bridging lines are anchored.

STABILITY UNDER SERVICE LOADING

The joist system is amply stabilized under gravity loads by the rigid decking (steel deck and/or concrete slab) when the decking is welded to the compression chord at a distances of at most 36 in. Usually the welds are placed closer than this limit. However, there are two conditions when it is necessary to be concerned about bracing in the completed structure:

a) The strength of the bottom chords and the bottom chord bracing spacing must be checked to provide adequate resistance to uplift loads due to wind. It is especially important that the first bottom chord joint is braced (see Fig. 1), since this point must now brace the end of the first web diagonal which is in compression under uplift loading.

b) Standing-seam roof decking cannot be counted on to provide lateral bracing to the top chords of steel joists. Therefore it is necessary that lateral bracing be provided by the bridging lines under the full service loading. This means that the bridging lines will need to be spaced closer (i.e., F_{cr} is larger in Eq. 2, and thus L_{cr} is smaller), usually 4 ft on center, and that the bridging anchorage forces will be larger (i.e. F_a in Eq. 3 is larger). It will be advantageous to interrupt the horizontal bridging lines with a "box" made of two adjacent joists laced together by diagonal braces to form sufficiently strong intermittent anchorages when there are more than eight or ten joists in a row.

SUMMARY AND CONCLUSIONS

Steel joists are very efficient when they are in their final location with the decking in place and the loads are due to gravity. In the fully erected stage it is necessary, however, to check lateral bracing if the loads are uplift loads due to wind, and to provide adequate bracing in the case of standing-seam roofs. During the construction phases of building it is also important to consider the stability of the joist under its self-weight and under the weight of an erector. Furthermore, bridging spacing and bridging strength need to be accounted for during the placing of the decking on the joist system. The criteria to be considered during the various stages of construction are discussed in this paper, and the background of the rules in the Steel Joist Institute Specification is presented.

Galambos, T.V., "Bracing of Light Steel Trusses," in *Stability of Steel Structures*, Akademia Kiado, Budapest, 1988, p. 549.

Galambos, T.V. and Xykis, C., "The Effect of Lateral Bracing on the Stability of Steel Trusses," *J. Construct. Steel Research*, 20(1991) 251-258.

Hribar, J.A. and Loughlin, W.P., "Lateral Stability of Welded Light Trusses," *Journal of the Structural Division*, ASCE,, Vol. 94, No. ST3, March 1968.

Leigh, J.M., "Bridging Spacing for Longspan Steel Joists," Research Report No. 19, Washington University, Dec. 1971.

Masoumy, G., "Lateral-Torsional Buckling of Steel Joists and Joist Systems," D.Sc. Thesis, Civil Engineering Dept., Washington University, St. Louis, May 1980.

Minkoff, R.M., "Stability of Steel Joists During Erection," Research Report No. 39, Washington University, Aug. 1975.

Steel Joist Institute, "Standard Specification, Load Tables, and Weight Tables for Steel Joists and Joist Girders," 1992.

Westerheide, J.D. III, "Horizontal Versus Diagonal Bridging for Longspan Joists," M.S. Thesis, Civil Engineering Dept., Washington University, St. Louis, May 1979.

Xykis, C., "Lateral Torsional Stability of Non-Linear Rigid Space Frame Systems," Ph.D. Thesis, University of Minnesota, Minneapolis, 1988.

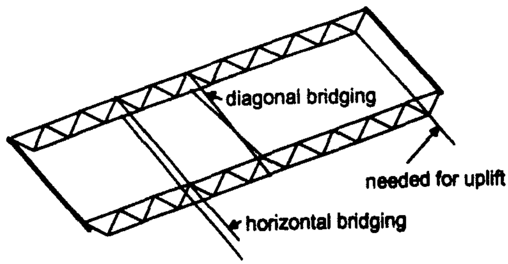


Fig.1 Types of lateral bracing

A_T =area of top chord, A_B=area of bottom chord
I_{yT}= I_y of top chord, I_{yB}= I_y of bottom chord
C= centroid of total cross section
S= shear center
t_T, t_B = thickness of angles, top and bottom chords,respectively

$$A=A_T+A_B$$

$$J_T=\frac{A_T t_T^3}{3}$$

$$J_B=\frac{A_B t_B^3}{3}$$

$$J=J_B+J_T$$

$$I_y=I_{yB}+I_{yB}$$

$$I_x=A_T\bar{y}^2+A_B\left(d_e-\bar{y}\right)^2$$

$$\bar{y}=\frac{A_B d_e}{A}$$

$$y_0=-y+\frac{I_{yB} d_e}{I_y}$$

$$C_w=\frac{d_e^2 I_{yB} I_{yT}}{I_y}$$

$$\beta_x=\frac{A_B\left(d_e-\bar{y}\right)^3-A_T y^3}{I_x}-2y_0$$

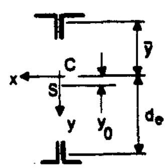


Fig.2 Formulas for determining section properties

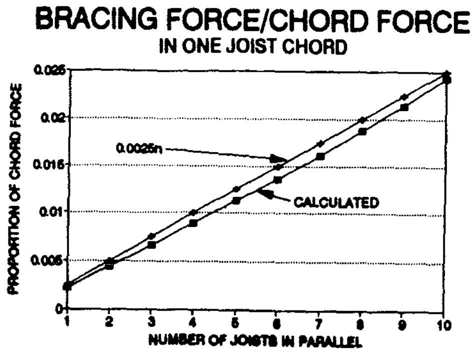


Fig.3 Variation of bracing force with number of joists

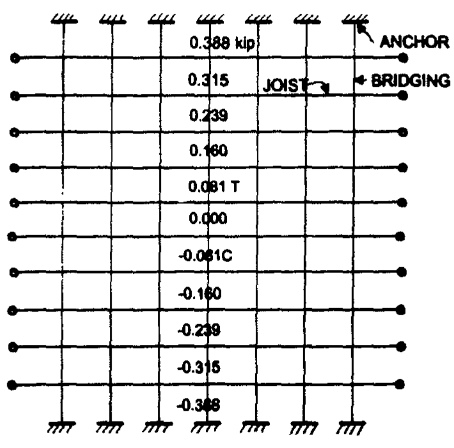


Fig.4 Bracing forces in center row of bridging

BRACING IN CANTILEVER-SUSPENDED SPAN CONSTRUCTION

Hesham S. Essa and D.J. Laurie Kennedy

Department of Civil Engineering, University of Alberta
Edmonton, Alberta, T6G 2G7

ABSTRACT

Secondary framing members such as open-web steel joists can enhance the stability of beams in cantilever-suspended span construction markedly. This enhancement results from both the lateral and torsional bracing provided to the top flange of the beam by the joists. With such restraints the buckling of the beam is characterized by web distortion. A distortional finite element model, verified by 31 full-scale tests, is used to investigate the stability of the beam. A simplified expression for the bending stiffness of open-web steel joists, which brace the beam torsionally, is presented. The strength of the minimum specified welded connections for the joists is checked. The stiffness of practical joists is sufficient and the strength of the minimum specified welded connection for joist shoes is likely to be sufficient to stabilize the beam through torsional bracing. Because torsional bracing enhances the stability of the beam much more than lateral bracing, its effect should be considered to achieve economical designs for beams in cantilever-suspended span construction. Numerical comparisons are given for the buckling capacity of beams with different bracing systems.

INTRODUCTION

After the collapse of a parking roof of a commercial building in Burnaby, British Columbia, Canada, in April 1988 (Closkey 1988), the question was raised about the adequacy of the bracing design for cantilever-suspended span construction. In this system, the beams of alternate bays cantilever over the top of columns and a simple span is suspended between the ends of the cantilevers in the other bays. Open-web steel joists are supported on the top flange of the beam. The welded joist shoe connection may provide both lateral and torsional restraint to the top flange and thereby enhance the beam stability significantly.

Solutions currently available for designing beams in cantilever-suspended span construction against lateral-torsional buckling (CISC 1989) account for neither the beneficial effect of torsional bracing nor the adverse effect of the height of load application above the shear centre. The results can be either overly conservative or unconservative, depending on the boundary conditions involved.

The problem is further complicated by web distortion and the twisting of the braced flange between bracing points. A finite element model (Albert et al. 1992) has been developed to predict the lateral-torsional buckling of a beam under any combination of loading and restraint conditions, taking into account cross-sectional distortion. This model uses a line element to simulate the flanges and a plate element to simulate the web. A series of 31 full-scale tests, conducted to verify this model, gave a test/predicted ratio of 0.99 with a coefficient of variation of 0.064.

TORSIONAL RESTRAINT STIFFNESS

The input data of the finite element program require an estimate of the effective torsional restraint stiffness, K_t, provided to the beam through the joist shoe connection by the flexural action of the joist. A spring model (Milner 1975) can be used to determine K_t as

$$\text{[1]} \qquad \frac{1}{K_t} = \frac{1}{K_j} + \frac{1}{K_c}$$

where K_j is the in-plane bending stiffness of the open-web steel joist and K_c, the stiffness of the joist shoe connection, depends on its moment-rotation characteristics. When a welded connection is used, which is the usual case in cantilever-suspended span construction, Milner (1977) recommended an infinite value for K_c based on some experimental results. Milner and Rao (1978) determined values of K_c for bolted connections experimentally.

Consider the open-web steel joist shown in Fig. 1(a), where the top chord is extended beyond the end of the web member to provide for the joist seat. This joist can be modelled as a beam with a moment of inertia I_1 for the central portion between the end panels and a moment of inertia I_2, which is taken equal to that of the top chord member, for the end panels, as indicated in Fig. 1(b). Applying an in-plane bending moment, M, at the end q, and assuming simply supported end conditions, the end rotation is given as

$$\text{[2]} \qquad \alpha_q = \frac{M}{3 E I_1 L I_2}[\, I_1 L^2 - (I_1 - I_2)(L^2 - 3LL_p + 3L_p^2 - 2L_p^3/L)]$$

Because $I_1 >> I_2$ and $L >> L_p$, the expression for end rotation can be approximated as

$$\text{[3]} \qquad \alpha_q \approx \frac{M L_p}{E I_2}$$

Therefore, the in-plane bending stiffness of the open-web steel joist is obtained as

$$\text{[4]} \qquad K_j = \frac{M}{\alpha_q} = \frac{E I_2}{L_p}$$

The expression for K_j given by [4] was confirmed using a plane frame program. When no information is available about the cross sections of the joist chords, it is suggested that, based on a study of practical joist sizes of two different manufacturers, a value of 3×10^7 Nmm/rad. is generally conservative for K_j. For interior beams, with joists on both sides, K_j is obtained by adding the contribution of the joists on each side. For joists acting compositely with a concrete slab, significant enhancement of the torsional stiffness is achieved. Conservatively, the moment of inertia of the composite section of the concrete and the top chord acting together is used for I_2 in [4] of the joists on one side of the beam where the concrete is in compression as the beam tends to rotate, and that of the steel top chord alone on the other side where the concrete is in tension.

STRENGTH OF MINIMUM WELDED CONNECTION

The reliability of the torsional restraint in enhancing the stability of the beam depends on the strength of the welded joist shoe connection to the top flange of the beam. CSA Standard CAN/CSA-S16.1-M89 (CSA 1989) requires a minimum specified length and size of the two parallel fillet welds of 40 mm and 5 mm respectively for this connection.

With a torsional restraint stiffness, K_t, and an angle of twist, θ_t, of the top flange at the buckling load at a bracing location, the minimum required strength for the connection is

[5] $$M_c = K_t \theta_t$$

However, because the finite element model treats the beam buckling as a bifurcation problem, only a normalized buckled shape is obtained and the values of the twist remain unknown. Therefore, the model was modified to include the effects of initial imperfections in order to determine the bracing forces (Trahair and Nethercot 1984). A W360x39 beam of 9 m span, under uniform moment, with five intermediate bracing points on the top flange, and a maximum sinusoidal sweep of 1/1000 of the span, the maximum specified tolerance for out of straightness, was analyzed. For a torsional restraint with a stiffness, K_t, of 10^8 Nmm/rad. and rigid lateral restraint at the bracing points, a rotation of $1.3\text{x}10^{-2}$ rad. was obtained for the maximum angle of twist for the top flange at a bracing point when the factored moment was applied. The resulting restraining moment of $1.3\text{x}10^6$ Nmm, under these extreme conditions, when shared equally by joists framing from both sides, is about 55% of the factored moment resistance of the minimum specified weld. For sweeps less than the maximum tolerance and for less extreme moment diagrams, the strength required would be less.

EFFECT OF TORSIONAL BRACING STIFFNESS

In cantilever-suspended span construction, practical open-web steel joists offer essentially rigid lateral bracing and finite or limited torsional bracing to the top flange of the beam. Bracing should be provided, where the beam is continuous over the column, to both the column and the bottom flange of the beam because the compression in the beam flange is greatest there. Extension of the bottom chord of the open-web steel joist on the column line and the joist shoe connection on the top flange provide the beam with lateral restraint for both flanges, i.e. a fork support. If the lateral bracing of the bottom flange, or alternatively web stiffeners are not provided, the only torsional bracing that can be provided to the bottom flange of the beam is through a rigid connection with the column. This does not provide much lateral restraint and is not preferred as discussed subsequently.

Figs 2(a) and 2(b) show two interior bay W460x74 beams with the same loading conditions but with different support conditions. The cantilever tip loads, as is approximately the case in practice, are applied at the shear centre. Open-web steel joists load the top flange of the main span and provide rigid lateral restraint and torsional restraint (which can be selected as desired) as indicated by the filled circles. Open circles and open squares represent complete (rigid) translational and complete rotational restraint respectively, each without the other. The bending moment diagram selected for this example is shown in Fig. 2(c). All the bottom flange is in compression. Cross-sectional dimensions are: depth, 457 mm; flange width and thickness, 190 mm and 14.5 mm; web thickness, 9 mm. The yield strength of the steel is 300 MPa, the modulus of elasticity is $200\text{x}10^3$ MPa, and Poisson's ratio is taken as 0.3. Not knowing the residual stress pattern (which the finite element model can accommodate), the finite element program was used to obtain the elastic lateral-torsional buckling resistances and then, if necessary, the corresponding inelastic buckling resistances were determined using the procedure given by CSA Standard CAN/CSA-S16.1-M89 (CSA 1989). The effect of torsional restraint stiffness on the inelastic buckling resistances of the beam with the two bracing systems is demonstrated in Fig. 3 where the ratio of the inelastic moment resistance to the fully plastic moment, m_i, is plotted against the torsional restraint stiffness. The lower plateaus, for both support or bracing systems, up to K_t values of about 10^6 Nmm/rad., represent the situation where the bottom flange is restrained only by twisting of the cross section without web distortion. The rotational restraint of the top flange is insufficient to cause distortion of the web as the compression flange buckles. Within the range of torsional bracing stiffness of 10^6 to 10^9 Nmm/rad., full torsional restraint of the top flange develops and, as the bottom flange attempts to buckle, web distortion occurs. Greater bracing stiffnesses do not further enhance the critical moment and the upper plateaus are reached.

Because the lower limit of the practical range of the in-plane bending stiffness of open-web steel joists is about 10^7 Nmm/rad., joists properly fastened to beams increase the beam resistances significantly.

It is also noted that, for all values of the torsional bracing stiffness, the beam with fork supports has significantly greater resistance than the beam without.

EFFECT OF DIFFERENT BRACING SYSTEMS

The effect of different bracing systems is illustrated by Table 1 where 27 different cases, all for the W460x74 beam considered previously, are presented. The bending moment diagram is that of Fig. 2(c) with the entire bottom flange in compression. The restraint symbols are as discussed previously. The torsional stiffness, used, is the intermediate value of 10^7 Nmm/rad. (see Fig. 3).

Consider cases 1 to 9, which all have fork supports and therefore allow comparisons to be made of the bracing provided at the load points. Comparisons are based on the inelastic moment resistance ratios, m_i, determined by prorating the elastic moment ratio $m_e = M_e / M_p$ to the inelastic value using the equation of CSA Standard S16.1 (CSA 1989) when m_e is greater than 0.67. (When m_e is less than 0.67, $m_i = m_e$).

Case 1, without restraints at the load points gives the least value of the ratio m_i of 0.530 and by providing fork supports at one end and then the other, cases 2 and 3, the ratio is increased to 0.648 and then to 0.750, an overall increase of 42%.

Cases 4, 5 and 6, with lateral restraints at the main span load points, parallel cases 1, 2 and 3 without, and show that fork supports at one and then two cantilever tips increases the moment ratio, m_i from 0.573 to 0.720 to 0.862 or 57% overall. This is also evident for cases 7, 8 and 9, with the addition of rotational restraint at the load points, where the moment ratio, m_i, is increased from 0.826 to 0.869 to 0.955 or 16% overall. The increase is less here as the significantly restrained beams are approaching their limiting moment capacity of M_p.

A comparison of the three parallel cases of 1 with 4, 2 with 5, and 3 with 6, the first three in each pair without lateral restraint and the last three with, show increases of 8%, 11% and 15% only. These improvements are relatively small because the beams buckle without significant lateral displacement of the tension flange, but are enhanced when interactive buckling occurs in cases 3 and 6.

In contrast, a comparison of the three parallel cases of 4 with 7, 5 with 8, and 6 with 9, the first three in each pair having lateral restraint only and the latter three lateral and torsional restraint give increases of 44%, 21% and 11%, respectively. Even though in the latter two cases the increase is compromised by the beam resistance approaching the limiting value of M_p, it is evident that torsional restraint has a marked effect on increasing the moment resistance of the beams.

The torsional restraint provided by the joists can cause beams with fork supports to behave very favourably failing by inelastic buckling at moments approaching M_p in the most restrained conditions.

Cases 10 to 18 parallel cases 1 to 9, the difference being that for cases 10 to 18 torsional restraint only is provided at the top of the columns. Fork supports are not provided and the top of the column, torsionally fixed to the beam flange, is free to translate (This approximates the bracing conditions in Burnaby (Closkey 1988), where one column had no lateral support and the other had fork supports. Using the finite element program to analyze that structure as loaded gave an inelastic moment ratio of 0.81 as compared to that calculated at failure of 0.86, i.e. the failure load was 1.06 times the predicted load.

Comparisons of pairs 1 with 10 through to pairs 9 with 18 shows that by allowing the bottom flange to translate at the top of the column, the inelastic moment ratio is reduced to between 52% to 74% of that for the comparable case with fork supports. Conversely the fork support bracing system is, for the cases cited, 136% to 192% as strong. Again the differences are reduced somewhat when moment resistances are limited due to inelastic behaviour.

Above all else, these comparisons illustrate the importance of providing lateral restraint at the top of the column.

Cases 19 through 27 differ only from the parallel cases 10 through 19 in the fact that the top flange of the beam over the top of the column is restrained laterally and torsionally in the former and only laterally in the latter. The increase due to this increased restraint varies between only 5 and 9% for the different pairs and is a measure of the decreased distortion of the beam web over the top of the column when the top flange is restrained locally as compared to the case when it is allowed to rotate at that location.

In Fig. 4, variations of the inelastic moment resistance ratio with the ratio R, of the static moment on the main span to the end moments on this span, are presented for a beam with fork supports and for a beam with translation of the supports allowed for the cases (4 and 13, respectively, Table 1) where only translational restraint is provided by the open-web steel joists and the cantilever tips are not laterally restrained.

A ratio R=0 means that no transverse loads are applied to the main span and that the entire length of the bottom flange is uniformly compressed. A ratio R=1 corresponds to the moment diagram of Fig. 2c for which Table 1 was computed. A ratio of R=2 means the static moment is twice the end moment or the positive moment at midspan equals the negative moment at the supports and 70% of the bottom flange is in tension with the same length of the top flange, which is laterally supported, in compression.

For the beam with fork supports the inelastic moment resistance ratio increases continuously from 0.331 for R = 0, to 0.573 for R = 1.0 (as given in Table 1) to 0.831 for R = 2.0 even though the load on the beam is increased 2.43 times. The reason is that the critical unbraced compression flange, tending to buckle laterally, is less and less heavily stressed as the value of R increases.

For the beam where translation (but not rotation) of the support is allowed, two phenomena have opposite effects on the variation of the inelastic moment resistance ratio, m_i, and its value increases from 0.313 for R = 0.0 to 0.396 for R = 1.0 and then decreases to 0.360 for R = 2.0. The ratio tends to increase as less of the laterally unsupported bottom flange is heavily stressed in compression but this phenomenon is overcome due to increased load on the top flange causing increased destabilization and tendency of the web to distort over the supports.

It is also noted that when R = 2.0 that the moment resistance of the beam with translation of the supports allowed is only 43% of that with forked supports.

CONCLUSIONS AND RECOMMENDATIONS

1. The moment resistance of the double-overhanging main span beams in cantilever-suspended span construction depends on the loading and bracing systems provided.

2. Open-web steel joists welded (or bolted) to the top flange of the main span beam provide both lateral and torsional bracing to the beams. The torsional bracing enhances the moment resistance substantially more than the lateral bracing.

3. Overall the inelastic moment resistance of the main span beams with fork supports is about halved from the maximum value when the joists provide rotational and translational restraint and the cantilever tips have fork supports to the case when no restraint is provided by the joists or at the cantilever tips.

4. In the most restrained case the moment resistance of the main span beams approaches the full cross-sectional strength.

5. The moment resistance of the main span beams when translation of the supports is allowed may be less than one-half that of beams with fork supports, with other restraining and loading conditions remaining the same.

6. The moment resistance of the main span beams with fork supports increases as its midspan moment becomes less negative and finally takes on positive values.

7. The moment resistance of the main span beams where translation of the supports is allowed remains low irrespective of the shape of the moment diagram.

ACKNOWLEDGEMENT

The financial assistance provided by the Natural Sciences and Engineering Research Council of Canada is greatly appreciated.

REFERENCES

Albert, C., Essa, H.S. and Kennedy, D.J.L. 1992. Distortional buckling of steel beams in cantilever-suspended span construction. Canadian Journal of Civil Engineering, Vol. 19, No. 5, pp. 767-780.

Canadian Institute of Steel Construction (CISC). 1989. Roof framing with cantilever (Gerber) girders and open web steel joists. Willowdale, Ontario, Canada.

Canadian Standard Association (CSA). 1989. Limit states design of steel structures. National Standard of Canada CAN/CSA-S16.1-M89, Canadian Standard Association, Rexdale, Ontario.

Closkey, D.J. 1988. Report of the commissioner inquiry Station Square development. Government of British Columbia, Victoria, British Columbia, Canada.

Milner, H.R. 1975. The elastic buckling of equal flanged beams torsionally restrained by discrete braces. Proceedings of the 5th Australian conference on the Mechanics of Structures and Materials, Melbourne, Australia. pp. 405-420.

Milner, H.R. 1977. The Design of simple supported beams braced against twisting on the tension flange. Civil Engineering Transactions, Institution of Engineers, Australia, Vol. CE19, No. 1, pp. 84-91.

Milner, H.R. and Rao, S.N. 1978. Strength and stiffness of moment resisting beam-purlin connections. Civil Engineering Transactions, Institution of Engineers, Australia, Vol. CE20, No. 1, pp. 37-42.

Trahair, N.S. and Nethercot, D.A. 1984. Bracing requirements in thin-walled structures. Developments in thin-walled structures-2, (editors J. Rhodes and A.C. Walker), Elsevier Applied Science Publishers, Barking, pp. 93-130.

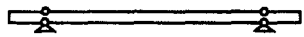
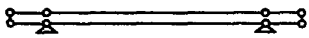
Case	Restraint diagram	Elastic moment resistance ratio $m_e = M_e / M_p$	Inelastic moment resistance ratio m_i
1		0.530	0.530
2		0.648	0.648
3		0.805	0.750
4		0.573	0.573
5		0.749	0.720
6		1.117	0.862
7		0.993	0.826
8		1.146	0.869
9		1.649	0.955

Table 1 Buckling resistances for different restraints

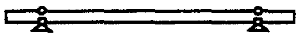
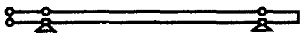
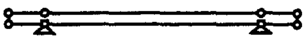
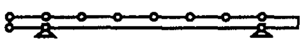
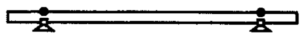
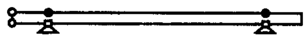
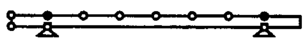
Case	Restraint diagram	Elastic moment resistance ratio $m_e = M_e / M_p$	Inelastic moment resistance ratio m_i
10		0.369	0.369
11		0.384	0.384
12		0.391	0.391
13		0.396	0.396
14		0.420	0.420
15		0.451	0.451
16		0.606	0.606
17		0.614	0.614
18		0.724	0.705

Table 1 Continued

Case	Restraint diagram	Elastic moment resistance ratio $m_e = M_e / M_p$	Inelastic moment resistance ratio m_i
19		0.395	0.395
20		0.413	0.413
21		0.422	0.422
22		0.425	0.425
23		0.458	0.458
24		0.498	0.498
25		0.663	0.663
26		0.674	0.672
27		0.783	0.739

∘ Lateral restraint ▫ Torsional restraint • Lateral and torsional restraint

Table 1 Continued

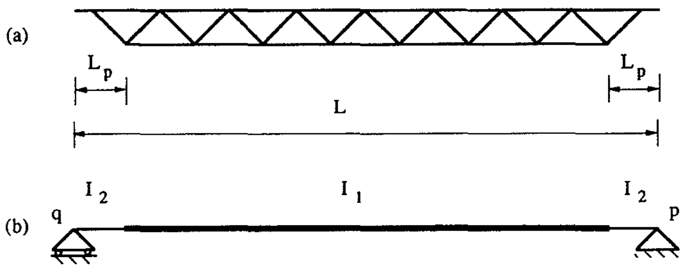


Fig. 1 (a) Open-web steel joist (b) Equivalent beam model

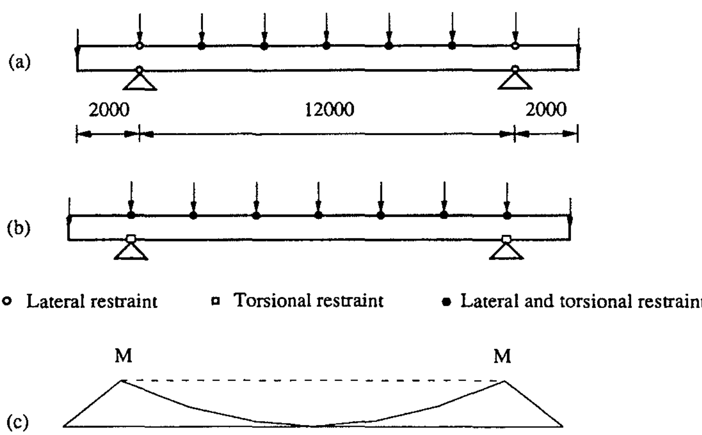


Fig. 2 Restraint and loading conditions. (a) Fork supports
(b) Translation allowed at supports (c) Bending moment diagram

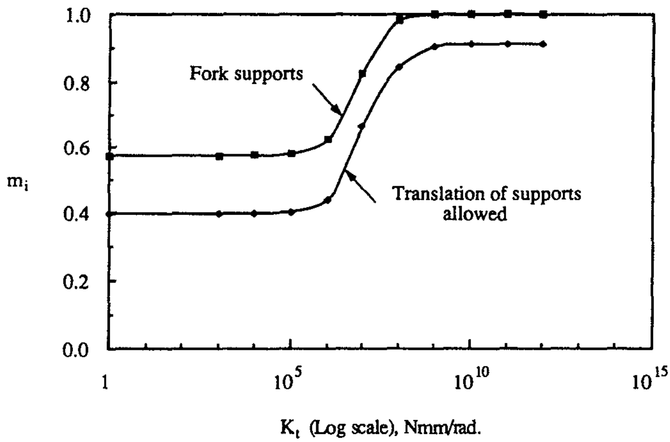


Fig. 3 Effect of torsional restraint stiffness

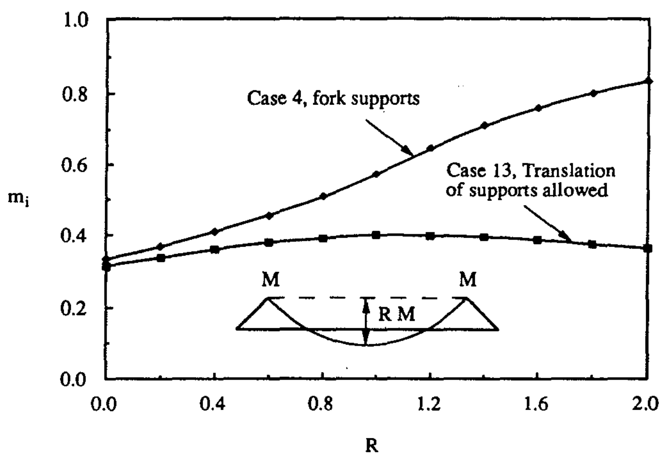


Fig. 4 Variation of inelastic moment resistance ratio with static moment ratio

BRACING, A SECONDARY LOAD PATH IN A FRACTURE CRITICAL BRIDGE.

R.L. IDRISS, K.R.WHITE, C.B. WOODWARD, JOHN MINOR and D.V. JAUREGUI

DEPARTMENT OF CIVIL ENGINEERING
NEW MEXICO STATE UNIVERSITY
Presenting Author: R.L. IDRISS

Abstract

The I-40 bridges over the Rio Grande are scheduled to be razed in the summer of 1993 due to geometry and traffic safety considerations. The bridges are classified as "fracture critical" two girder steel bridges. A bridge will be tested to determine the impact of defects on the redistribution of loads, on the load capacity, and on the potential for collapse. After data is accumulated for this fracture critical bridge in pristine condition, defects consisting of one or more cuts in critical locations will be made and the data retaken.

A three dimensional computer model was created to predict the before and after fracture response of the bridge. Response of the intact structure was determined. Then a defect in the form of a near full depth crack in one of the girders was introduced. The after fracture behavior of the structure was evaluated. The role of the different elements in load redistribution was evaluated and used for a) gage placement, b) safety considerations.

In the real structure the fractured girder will be initially shored and the shoring reduced. Strains will be measured at strategic locations under dead load and specific live loads. The field test data will be subsequently used to check, validate, or modify the analytical model.

This paper presents preliminary results of the finite element analysis that precedes the testing.

Introduction

The I-40 bridges over the Rio Grande in Albuquerque are due to be razed in mid 1993 due to geometry and traffic safety considerations. The bridges represent a common design in the U.S. and are classified as non-redundant"fracture critical" two girder steel bridges. Fracture critical members are tension members of a bridge whose failure will probably cause a portion of or the entire bridge to collapse. It is assumed by AASHTO that fracture of a non-redundant structure is severe, and will lead to catastrophic collapse. These concepts are based on the simplified AASHTO assumptions used in the design of two-girder steel bridges, which do not account for the actual three dimensional behavior of the as-built structure.

The NCHRP Report No. 319[2] "Recommended Guidelines for Redundancy Design and Rating of Two-Girder Steel Bridges" gives guidelines for design and analysis of alternate load paths. These guidelines are based on analytical studies. Actual in-situ testing of the as-built structure is necessary to verify the analytical approach.

Experience shows that two-girder highway bridges although classified as "fracture critical" typically do not collapse when a fracture occurs in a girder. In many instances, they remain serviceable, and damage sometimes is not even suspected until the fracture is discovered incidentally or during inspection. Much still needs to be learned about the after fracture behavior of these structures: how does the load get redistributed when fracture occurs and how does the fractured bridge in many instances carry not only it's dead load but also the vehicles on it?

Analytical Evaluation

A three dimensional finite element computer model of the bridge was developed using SAP 90 on a 486 desktop computer. A near full depth crack was modeled in one of the girders. This analysis was necessary from a safety standpoint, to determine the sensor locations, optimize the quality and quantity of the data acquisition devices, and to assist in planning the nondestructive testing procedure of the structure by predicting the after fracture response. Safety requires a temporary bent under the fractured girder. The predicted forces expected in this shoring are needed for proper design. Since one can only have a finite number of data acquisition channels, placement of the sensors is important. This model allows proper placement of sensors of proper sensitivity. The bridge is also going to be used for nondestructive testing techniques, to determine their sensitivity.

Bridge Description

The current structure, built in 1963, is a 1,275 foot long, non-composite bridge consisting of three 3-span continuous units with spans of 131 ft.-163 ft.-131 ft each. The structural unit is a two-girder welded with bolted splices design with a floor system (Fig. 1,2,and 3).

Loading

Two HS-20 Truck loadings were used in the computer studies in addition to the dead load of the structure. The trucks were positioned to create maximum positive moment at midspan of the central span of a 3-span group.

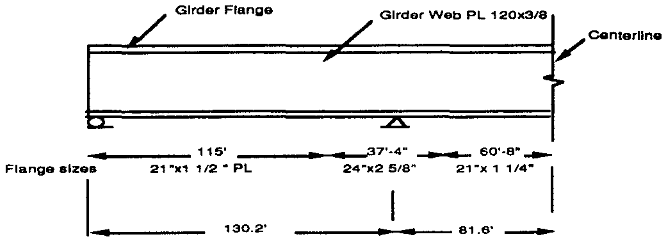


Figure 1. Girder side view.

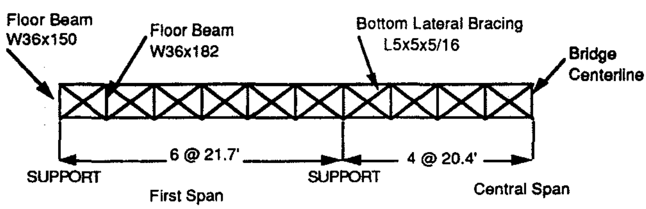


Figure 2. Floor Beams & Bottom Lateral Bracing

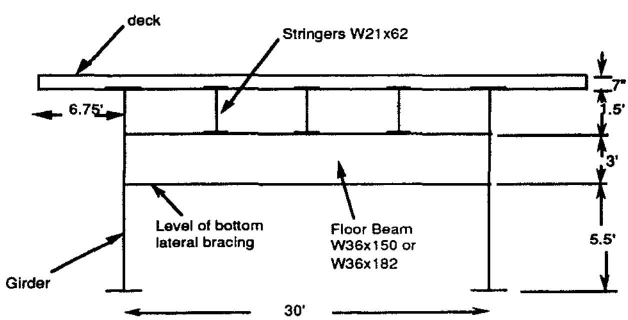


Figure 3. Bridge Cross-Section

Computer Model

A detailed three dimensional analysis of the bridge was necessary to account for the full interaction of the elements and to model the complex three dimensional behavior of the as-built structure (Fig.4).

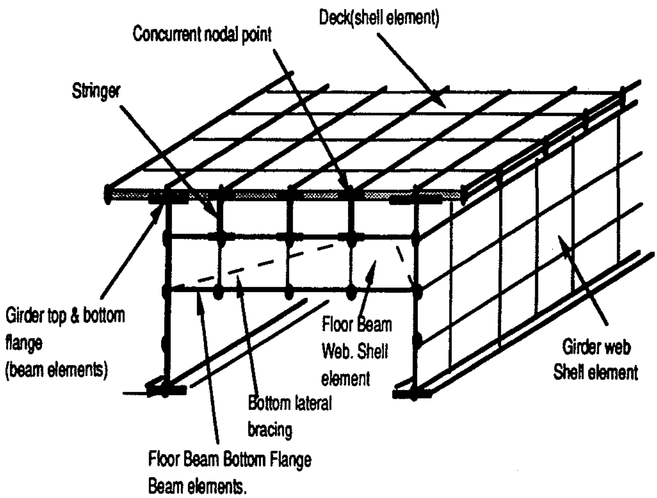


Figure 4. Three Dimensional Model

The elements used in this model are:

The Flanges. Beam elements are used to model the top and bottom flanges of the girders, stringers and floor beams.

The Webs are modeled using shell elements. In a previous model, plane stress elements were used to model the webs. When using plane stress elements, out of plane instability can occur, and it is necessary to restrict out of plane degrees of freedom. When using shell elements, this suppression of out of plane degrees of freedom is not necessary and a better representation of the full three dimensional movement of the actual fractured structure can be achieved.

The web stiffeners are modeled using beam elements.

The deck is modeled using shell elements.

Bottom lateral bracing diagonals are modeled as truss elements.

Supports: The expansion supports were modeled as rollers (translation restraints in the vertical, and transverse direction) , with the fixed supports modeled as hinges (translation restraints in the vertical, transverse, and longitudinal directions).

Crack. A near full depth crack is modeled at midspan of the central span of one of the girders (Fig. 5, 6).

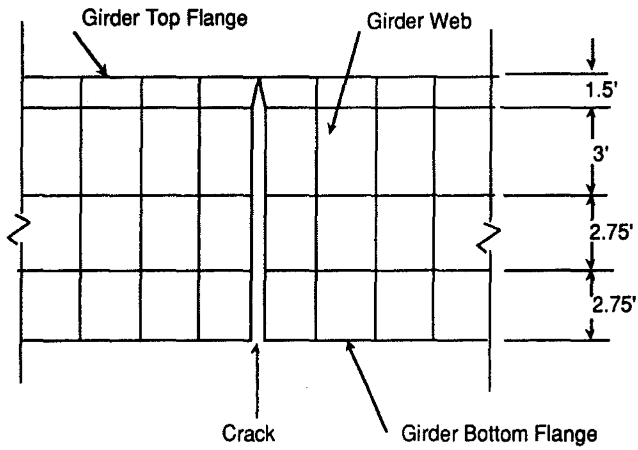


Figure 5. Modeling of the crack

Systems. Two basic models were examined: (1) a model of the intact structure; (2) a model of the bridge with a crack in one of the girders. A near full depth crack was modeled at midspan of the central span, extending through the bottom flange and the web. As a first step, an elastic analysis of both systems was conducted.

To determine the alternate load paths, the forces and moments generated in the different elements of the structure were monitored.

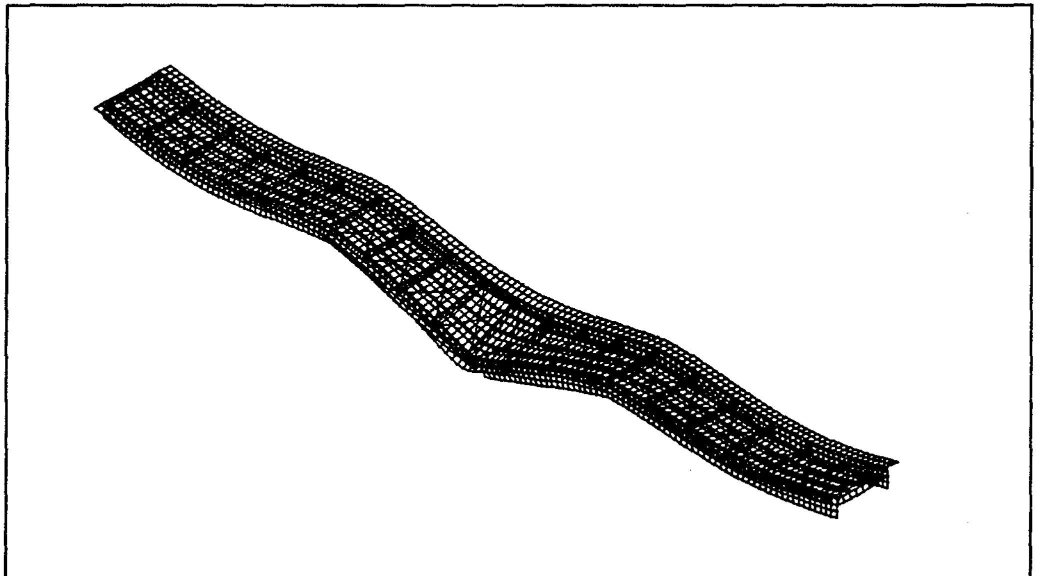


Figure 6. The three dimensional model of the fractured bridge.

Results

When a crack was introduced in the girder there was a shift in load paths throughout the structure. When the results were interpreted, it was apparent that the crack had simply created a new, but still stable structure.

Deflections: At the crack location, the deflection under dead+live load went from 1 in before the crack to 3 in after fracture.

Load redistribution: The load redistribution was observed under dead load when the crack occurred.

Longitudinal load redistribution: Most of the load was redistributed longitudinally via the cracked girder and the deck-stringer system. **Cracked Girder:** The two portions of the cracked girder acted as cantilever beams, and transferred the load to the interior supports, with a 30% increase in the negative moment at these supports. **Stringers:** The stringers showed a large increase in moment at the vicinity of the crack. The closer the stringer to the crack, the larger the increase in load. **The deck** : The deck showed a large increase in moment. The closer to the crack zone, the more pronounced the increase. At the crack, the moment computed exceeded the plastic moment capacity of the deck.

Transverse load redistribution: Part of the load was redistributed to the intact girder through the torsional rigidity of the deck, floor beams, and bracing system. This transverse redistribution of load is most pronounced at the crack zone. **Intact Girder:** There was a 20% increase in moment at mid span of the central span of the intact girder, showing the transverse redistribution at the crack zone. There was only a 4% increase in negative moment at the interior supports. **Floor Beams:** The beams B1, B2, and B3 (Fig. 7) at the vicinity of the crack had the most significant change in moment. Floor beam B2, adjacent to the crack carried less load due to the lack of support at one end created by the fracture in the girder. Floor beam B3 showed a 20% increase in positive moment. There was a negative moment at the floor beams connection with the intact girder, showing the load redistribution to the intact girder. **Lateral bracing:** There was a change in load pattern for the lateral bracing throughout the structure, with shifting for some of the bracing from tension to compression, due to the twisting of the structure. The largest increase in load was detected in the two panels at the vicinity of the crack (Fig. 8). These forces in the bracing were still well below yield and buckling.

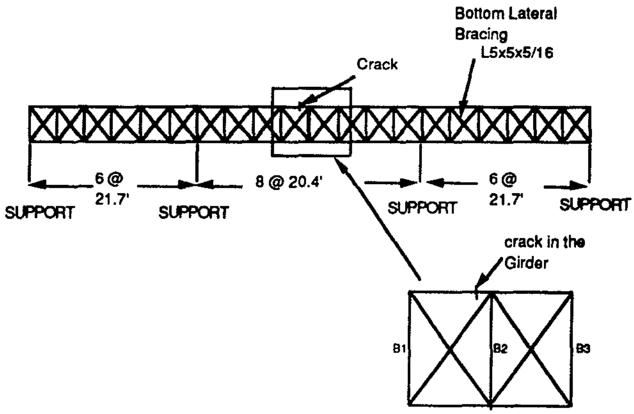


Figure 7. Bottom Lateral Bracing .

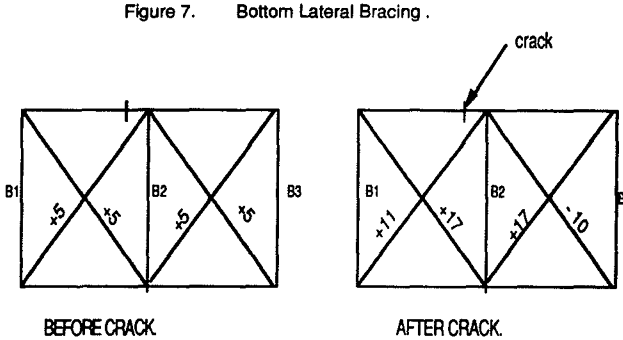


FIGURE 8. FORCES (KIPS) IN THE BRACING, AT THE VICINITY OF THE CRACK UNDER DEAD LOAD.

Bracing Effects. To further evaluate the role of the lateral bracing in the load redistribution, the bottom lateral bracing in the structure was removed and the simulated tests repeated.

Transverse load redistribution to the intact girder was reduced as was expected. The moment at mid span of the central span of the intact girder increased by only 15% compared with 20% when the bracing was in place. The deflection under dead + live load at the crack only slightly increased, staying within 3 in. This analysis leads to the conclusion that the bracing did play a role in redistributing the load; however, the major redistribution is in the deck-stringer system. The bracing would be more effective in creating torsional resistance, hence more resistance for this type of failure, if it was placed closer to the bottom of the girder.

Conclusions

1. When a crack was introduced in the bridge, it changed the existing structure into a new, but still stable structure. Vertical deflection predicted at the crack under dead+live loading was small, at about 3 in.

2. For this specific structure, most of the load was observed to be redistributed longitudinally, via the girders and the deck and stringers to the interior supports.

3. There was some load redistribution in the transverse direction to the intact girder due to torsional rigidity of the deck, floor beams and bracing system. This occurred mainly at the vicinity of the crack.

4. Although it contributed to the torsional stiffness of the bridge, due to it's location, at floor beam level, the bottom lateral bracing was not very effective as an alternate load path in redistributing the load to the intact girder.

5. With the exception of the deck, where the moment measured at the crack location exceeded the yield moment capacity, the forces computed in the different members of the structure were not high enough to cause yielding.

Further Study

These results are the analytical predictions of a computer model. The next step is the testing of the bridge. Although the computer model was instrumental in setting up the test, the test in turn will be used to check, validate, or modify the analytical model.

REFERENCES

1. American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges", AASHTO Code, 13th ed., 1983.

2. Daniels, J.H., Kim, W., and Wilson J.L., "Recommended Guidelines for Redundancy Design and Rating of Two Girder Steel Bridges", NCHRP Report 319, 1989.

3. R.L. Idriss and K.R. White, "Secondary Load Paths in Bridge Systems", Third Bridge Engineering Conference, Transportation Research Board, No. 1290, Vol. 2, March, 1991.

4. R.L. Idriss and K.R. White, "Alternate Load Paths in Damaged Bridges", Second International Conference on Computer Aided Assessment and Control (Southampton), Computational Methods in Fracture Mechanics, Vol. 2, Elsevier , 1992.

5. R.L. Idriss, K.R. White and John Minor, "Testing of a Fracture Critical Bridge", Third Workshop on Bridge Engineering Research in Progress, La Jolla, Ca, November 1992.

TORSIONAL BRACING REQUIREMENTS FOR BEAMS AND COLUMNS

By

Joseph A. Yura
Warren S. Bellows Centennial Professor in Civil Engineering
University of Texas at Austin

ABSTRACT

Very little information is available in literature on torsional bracing requirements for columns. Usually it is assumed that column braces must prevent movement in the two principal bending directions. Most column bracing are not designed to prevent twist at the brace point. The importance of torsional bracing is illustrated through a case study of an actual collapse and from theoretical solutions for columns with lateral braces attached at various locations on the cross sections. A common case of a column braced laterally along one flange is discussed and design examples presented. Torsional brace requirements for column braces are developed. In addition the torsional brace requirements at the ends of the beams are presented and compared with the torsional restraint provided by typical shear tabs and web connection angles.

This paper was unavailable at time of publication. Those wishing a copy of the paper may contact Prof. Yura as follows:

Dr. Joseph A. Yura
Department of Civil Engineering
University of Texas
Austin, TX 78712-1076
FAX: 412-471-1944

BRACING FORCE AND STIFFNESS REQUIREMENTS TO DEVELOP THE DESIGN STRENGTH OF COLUMNS

by

M.J. Clarke and R.Q. Bridge

Centre for Advanced Structural Engineering
The University of Sydney
Australia

INTRODUCTION

The in-plane strength of slender columns (axially loaded compression members) can be increased by the use of bracing at intermediate positions along the length of the column in the plane of buckling. In design, the strength of a column is related to the elastic buckling load for an equivalent pin-ended "effective" length member through the use of a column curve (AS 4100-1990, AISC-LRFD-1986). The bracing is usually considered as decreasing the effective length of the column and hence increasing the buckling load, which in turn enhances the ultimate strength. The buckling load will depend on the stiffness of the bracing. For perfectly straight elastic columns, the relationship between buckling load and brace stiffness can be derived analytically and standard solutions are available (Bleich, 1952; Timoshenko and Gere, 1961). These solutions, however, give no indication of the forces likely to be developed in the braces.

Real columns have geometric and material imperfections and will deflect immediately on the application of load and introduce forces into the braces. The problem of an elastic column with a sinusoidal out-of-straightness and a single central brace has been studied (eg. Green et al., 1947; Zuk, 1956). For a brace of infinite stiffness (a rigid brace), the relationship between the elastic buckling load and the brace force for any given maximum out-of-straightness at midheight has been derived analytically. For a brace of finite stiffness (an elastic brace) and where buckling occurs in a symmetric mode, an approximate expression relating the brace force to the applied load has been proposed (Green et al., 1947). The analytical expressions predict that the brace force asymptotes to infinity as the applied load approaches the buckling load for the symmetric mode. To be useful for design purposes, limitations on the brace stiffness and the column buckling load and mode are therefore desirable. Winter (1960) developed some simplified expressions by assuming a hinge formed at the brace point and proposed some lower limits on the required strength and stiffness of the bracing.

Mutton and Trahair (1975) also analysed an elastic imperfect column with an elastic central brace taking into account the lack-of-fit between the brace and the column and the out-of-straightness of both the column and the brace. The ultimate strength (as opposed to the elastic buckling load) was considered by calibrating the magnitude of the initial out-of-straightness such that a first yield criterion for an elastic imperfect column matched test strengths which included the effects of residual stresses as well as geometric imperfections. This is essentially the form of the column curve used in AS1250-1972 based on the well-known Perry-Robertson formulation. Brace force and brace stiffness requirements were developed by Mutton and Trahair (1975); their initial studies indicated that, for practical braces, the brace force requirement alone may well govern the design of the brace.

With the development of methods of advanced nonlinear analysis of structures (White et al., 1991; Clarke et al., 1992) that can account directly for nonlinear material properties, second-order effects, geometric imperfections and residual stresses, the ultimate strength of structural systems such as column-brace configurations can be determined in a rational manner. In this paper, the problem of an imperfect column with a single central brace at or near the column midheight is investigated. The parameters considered are the column initial out-of-straightness, the column slenderness, the bracing stiffness, the force in the brace, and the offset of the brace from the perfectly central position. The minimum bracing stiffness required to develop the near-ultimate strength of the column assuming an infinite bracing stiffness is established, and the force in the brace at the ultimate strength of the column is also determined. The results are compared with some of the bracing force and stiffness requirements in current design codes and specifications from Australia and Europe.

PARAMETRIC STUDY

The advanced analysis developed initially for stressed arch frames (Clarke and Hancock, 1991) and subsequently for any two-dimensional structural frame (Clarke et al., 1992) has been used in this study to determine the ultimate strength of the column-brace system. The analysis accounts for second-order effects and uses a plastic-zone model of material inelasticity to model the spread of yielding in the members of the structure. Other factors that affect strength and stability, such as residual stresses and geometric imperfections, are also modelled explicitly.

A simple brace configuration is shown in Fig.1 for a pin-ended column of overall length L with a central elastic brace dividing the column into two segments, each of length ℓ. The brace is modelled by an elastic spring of stiffness "a" which is pinned to the column so as to provide only lateral translational restraint and no rotational restraint. The brace can be offset from the perfectly central position by a small amount x to account for construction tolerances. A force P_b is generated in the brace from the application of the axial load P. The out-of-straightness of the column is assumed to be distributed sinusoidally with a maximum value of δ at midheight. The fabrication tolerance specified for δ in AS4100-1990 for columns is L/1000; this has been used in the study unless otherwise stated. The column cross-section is a compact Universal Column section 200UC46.2 (BHP Steel, 1991) bent about the minor (weak) axis and has a cross-sectional area A, a second moment of area I, and a radius of gyration r. The assumed residual stress pattern is that used by Galambos and Ketter (1959) with a maximum compressive stress in the flange tips of $0.3f_y$. The stress-strain relationship assumed for the steel is linear elastic-plastic with an elastic modulus E of 200,000 MPa and a yield stress f_y of 250 MPa. The following quantities referred to henceforth throughout the paper are also defined here:

- The normalised slenderness λ of the column between braces, $\lambda = \frac{1}{\pi}\frac{\ell}{r}\sqrt{\frac{f_y}{E}}$
- The Euler buckling load P_e of the column without braces, $P_e = \frac{\pi^2 EI}{L^2}$
- The strength P_y of the short column, $P_y = Af_y$
- The elastic critical load of the column-brace system is denoted P_{cr}.
- The ultimate strength of the column-brace system is denoted P_u.

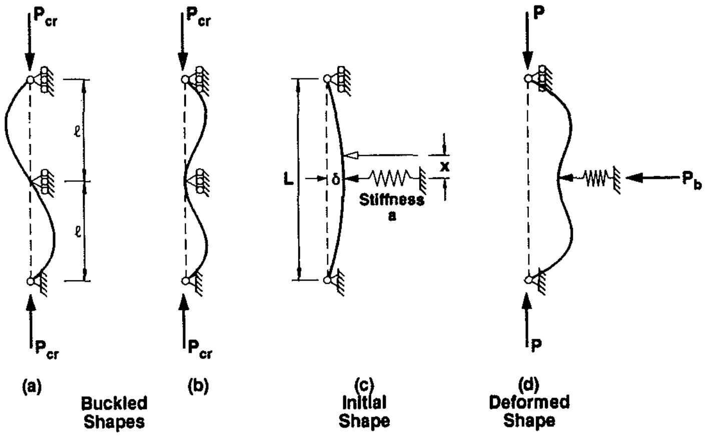


Fig. 1 Pin-ended column with central brace

Rigid braces

The column curves in Fig. 2 have been generated for a column with a single central brace assuming the bracing stiffness to be infinite. It should be noted that while the initial out-of-straightness $\delta = L/1000$ is representative of a fabrication tolerance for the overall length L of the column , the normalised slenderness λ is expressed in terms of the effective length ℓ between braces for the asymmetric buckling load. The pinned ends can be considered as rigid braces that prevent translation but not rotation. It can be seen in Fig. 2 that the column with the perfectly central brace is considerably stronger than the equivalent pin-ended column without any brace but with the same effective length ℓ. The difference can be attributed to at least two factors: the relative out-of-straightness of each column segment from the chord line joining adjacent brace support positions is different in both cases; and the shape of the initial out-of-straightness is such that the deformations of the column with a perfectly central brace are predisposed to a symmetric configuration with zero rotation at the brace position (see Fig.1(b)).

In this example, symmetry will obviously have a considerable influence on the strength. To investigate this phenomenon, the position of the brace was shifted slightly from the perfectly central position towards one end by a small offset distance x (see Fig. 1(c)). The variation in column strength P_u with offset distance x is shown in Fig. 3 for two different cases of column slenderness λ. It can be seen clearly from this figure that there is a critical position at which the failure mode suddenly switches from the symmetric mode (similar to Fig. 1(b)) to the asymmetric mode (similar to Fig. 1(a)). This highlights the importance of the positioning of the brace as only a small shift from the perfectly central position can result in a change in failure mode and a marked reduction in the strength. It is also pertinent to note that the determination of the offset position x/L at which the failure mode switches from symmetric to asymmetric is quite numerically sensitive and may depend on the degree of precision of the nonlinear analysis (the degree of refinement of the member and cross-section sub-divisions, the size of the load increments and the incremental-iterative solution strategies employed).

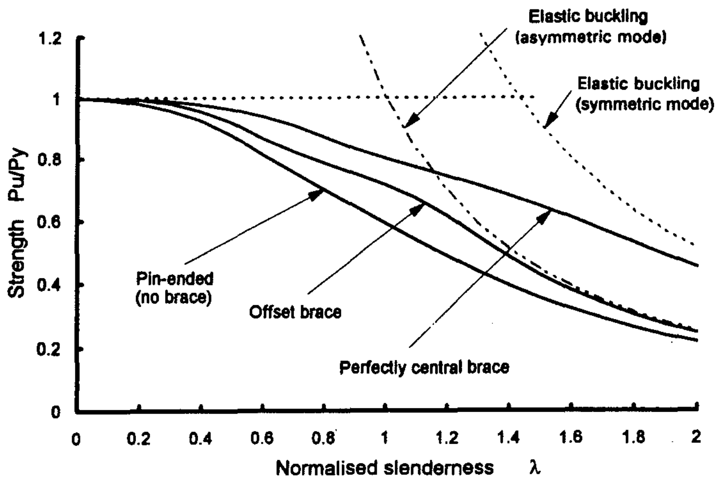


Fig. 2 Column curves for compression members with and without a central brace

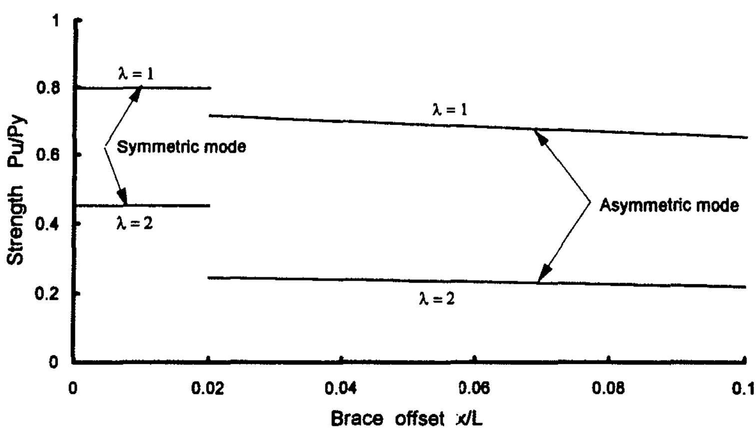


Fig. 3 Influence of brace position on column strength.

Therefore, a perfectly central brace is of limited use in analytical studies and should not be contemplated from a practical viewpoint. In the studies presented henceforth in this paper characterised by an asymmetric failure mode, a brace offset of 2.5% of the overall column length L, to account for "construction tolerance" has been used. Using this offset, the column curve has been recalculated for the asymmetric mode and is shown in Fig. 2. This curve is still higher than the equivalent pin-ended column curve; the reasons for this are the differences in relative out-of-straightness from the chord lines between points of support and the restraint provided by the slightly shorter segment to the slightly longer segment as the ultimate strength is approached.

Bracing Stiffness

(i) Elastic Behaviour

As the normalised column slenderness λ increases towards infinity, the behaviour of an axially loaded column approaches the fully elastic behaviour whereby the ultimate strength P_u can be taken as the elastic buckling strength P_{cr} (as indicated by the column curves in Fig. 2). The elastic buckling solution for a straight column with a central elastic brace has been derived by Timoshenko and Gere (1961), in which P_{cr} was derived in terms of the spring stiffness "a" as

$$\frac{aL}{P_e} = \frac{16\beta^3}{\pi^2(\beta - \tan\beta)} \tag{1}$$

where
$$\beta = \frac{\pi}{2}\sqrt{\frac{P_{cr}}{P_e}} \tag{2}$$

Equation (1) is shown as the upper curve in Fig. 4 for $\lambda = \infty$. For a brace stiffness aL/P_e of zero, the buckling load P_{cr} is equal to its minimum value of P_e for a pin-ended member of effective length = 2.0ℓ = L. With increasing spring stiffness aL/P_e, the buckling load P_{cr} asymptotes to $8.183P_e$ which is the buckling load (effective length = 0.6992ℓ) for the symmetric mode shown in Fig. 1(b) with no rotation at the brace position. This buckling load of $8.183P_e$ is the load that a second-order elastic analysis of the column with an initial out-of-straightness and a perfectly central brace will converge to. For the perfectly straight column, a bifurcation point occurs at $P_{cr} = 4P_e$ (effective length = 1.0ℓ), which corresponds to the asymmetric mode shown in Fig. 1(a) with rotation at the brace position. The bracing stiffness corresponding to this point is given by $aL/P_e = 16$ and the buckling load remains constant for any further increase in bracing stiffness as shown in Fig. 4.

(ii) Inelastic Behaviour

The variation of ultimate strength P_u with bracing stiffness aL/P_e is shown in Fig. 4 for a column with a perfectly central brace (symmetric failure mode, shown as dashed lines). By way of comparison, the lateral stiffness $48EI/L^3$ of a column with a central lateral load is equivalent to a non-dimensionless spring stiffness aL/P_e of $48/\pi^2$, or approximately 5. For inelastic columns with slenderness λ in the practical range, it can be seen in Fig. 4 that, for each particular value of λ, there is a limiting (minimum) brace stiffness aL/P_e beyond which the ultimate strength P_u

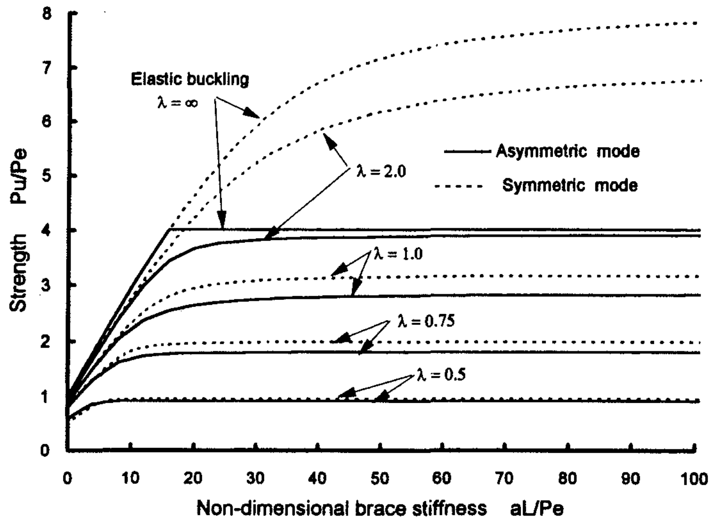


Fig.4 Ultimate strength of columns of $\lambda = 1$ with central brace of varying stiffness aL/P_e

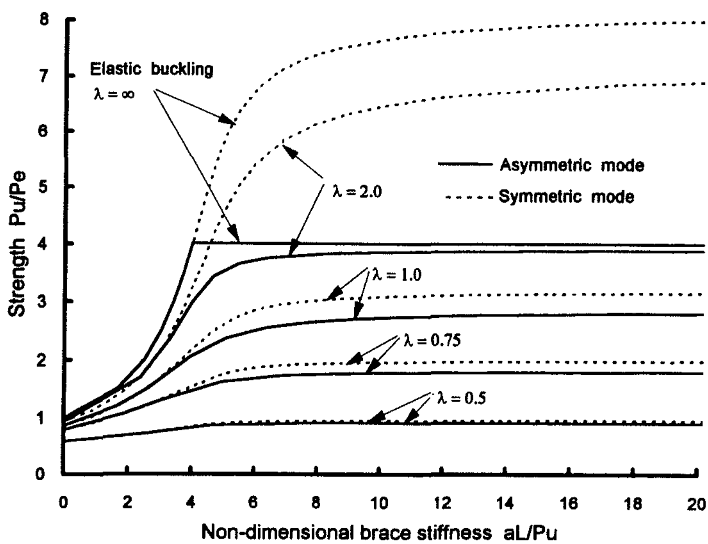


Fig.5 Ultimate strength of columns of $\lambda = 1$ with central brace of varying stiffness aL/P_u

effectively corresponds to the strength of the column with an infinite brace stiffness (to within, say, 5% tolerance). This limit, which increases as the slenderness λ increases, can be considered the limit for a "fully braced" column. However, as can be seen in Fig. 5, when the brace stiffness is non-dimensionalised using the ultimate strength P_u instead of P_e (i.e. giving a non-dimensional brace stiffness of aL/P_u) this limiting brace stiffness required to approach the "fully braced" condition is less variable, with a value of aL/P_u in the range of 8 to 12.

The variation of ultimate strength P_u with bracing stiffness aL/P_e is also shown in Fig. 4 for the case of a column with an offset central brace (asymmetric failure mode, shown as solid lines). The elastic buckling mode is anti-symmetric as shown in Fig. 1(a), and the buckling load P_{cr} has a maximum value of $4P_e$. The curve for a column slenderness $\lambda = \infty$ defines the elastic buckling strength P_{cr}. As for the symmetric mode discussed above, for inelastic columns with slenderness λ in the practical range and which fail in an asymmetric mode, it can be seen in Fig. 4 that there is a limiting brace stiffness aL/P_e beyond which the ultimate strength P_u corresponds effectively to the strength of the column with an infinite brace stiffness (to within, say, 5% tolerance). This limit increases with increasing column slenderness λ. However, as for the symmetric mode above, when P_u rather than P_e is used to non-dimensionalise the brace stiffness (i.e. giving aL/P_u), this limiting brace stiffness aL/P_u is less variable, being confined to the range 6 to 10 (Fig. 5).

Bracing Force

(i) Elastic Behaviour

For a column with a perfectly central brace of infinite stiffness (i.e. rigid) and a sinusoidal out-of-straightness of maximum amplitude δ as shown in Fig. 1(c), the bracing force P_b as a function of the applied axial force P was derived by Zuk (1956) and can be expressed as

$$P_b = \left(\frac{\delta}{L}\right)\frac{16P\beta^3\left[1+\cos 2\beta + 2\sin^2\beta\right]}{\left(\pi^2-4\beta^2\right)\left[2\tan\beta - \beta - \beta\cos 2\beta - 2\beta\sin^2\beta\right]} \tag{3}$$

where $\beta = \frac{\pi}{2}\sqrt{\frac{P}{P_e}}$

For $P = P_{cr} = 4P_e$ (effective length = 1.0ℓ), which corresponds to the antisymmetric buckling mode shown in Fig. 1(a) with rotation at the brace position, the bracing force P_b is

$$P_b = \left(\frac{\delta}{L}\right)\frac{16P_{cr}}{3} \tag{4}$$

For $P = P_{cr} = 8.183P_e$ (effective length = 0.6992ℓ), which corresponds to the symmetric buckling mode shown in Fig. 1(b) with no rotation at the brace position, the bracing force P_b is infinite.

For an elastic central brace of finite stiffness "a", the bracing force P_b can be approximated by (Green et al., 1947)

$$P_b = \frac{\delta a P}{P_{cr} - P} \tag{5}$$

where P_{cr} is obtained from the solution of Eq. (1).

The elastic solutions of Eqs. (3) and (5) are compared in Fig.6 with the results of the nonlinear analysis for a column of slenderness $\lambda = 1$ assuming linear elastic material behaviour. It can be seen in Fig. 6 that Eq. (3) and the second-order analysis results are indistinguishable. For brace stiffness with values of aL/P_e greater than 16 in the practical range where the column can be considered "fully braced" (see Fig. 4), the brace force P_b associated with the asymmetric mode reaches a limiting value exceeding 0.5% of the applied axial force P. The limits for $aL/P_e = 32$ and ∞ are shown in Fig. 6. The brace force increases with decrease in brace stiffness, and approaches infinity as the value of aL/P_e reduces to 16, which is the brace stiffness for which the symmetric and antisymmetric buckling loads coincide. The approximate Eq. (5) is reasonably accurate for values of aL/P_e less than 16, but is of little use as the braces are not "fully effective" in this range of stiffness. In the practical range of brace stiffness (aL/P_e greater than 16), the accuracy of Eq. (5) deteriorates as the deformed shape is no longer close to a sinusoidal half-wave (implicit in the deduction of the approximate Eq. (5)), but is characterised by increasing reversed curvature at the brace position (similar to Fig. 1(b)) as the brace stiffness increases.

(ii) Inelastic Behaviour

The variation of the brace force P_b with the applied axial load P, up to the ultimate load P_u and into the post-ultimate region, is shown in Fig. 7 for the case of slenderness $\lambda = 1$ and either a perfectly central brace (symmetrical failure mode) or an offset brace (asymmetrical failure mode). The following brace stiffnesses were chosen: $aL/P_e = 8$ corresponding to a partial restraint for which the buckling mode involves a displacement of the brace (sway mode); $aL/P_e = 32$, where the column is close to "fully braced" (P_u close to the value for a rigid brace); and $aL/P_e = \infty$ where the brace is rigid. The results for the offset brace (resulting in an asymmetric failure mode) are shown as the solid lines in Fig. 7. For each value of brace stiffness, it can be seen that, at the ultimate load P_u, the brace force P_b varies from approximately 0.5% to 2% of P_u. The brace force, as a percentage of the applied axial force, increases with decrease in brace stiffness.

The elastic solutions of Eqs. (3) and (5) indicate that the bracing force P_b generated in the brace by an applied axial force P is directly proportional to the magnitude δ of the out-of-straightness of the column. For the inelastic behaviour obtained from the advanced analysis, the variation of the brace force P_b with axial force P for values of δ of L/1000 and L/500 are compared in Fig. 7 for a column with a central brace of stiffness $aL/P_e = 32$ and a slenderness λ of 1 (all of the curves shown in Fig.7, except for the curve labelled "δ = L/500", were computed assuming δ = L/1000). It can be seen in Fig. 7 that the ultimate load P_u was not reduced to any significant extent by doubling the out-of-straightness δ. However, the brace force P_b can be seen to be essentially proportional to the magnitude of δ for all values of P. From the practical viewpoint of the design of bracing members, it is therefore important that the level of out-of-straightness of the column should be determined, based on acceptable fabrication tolerances.

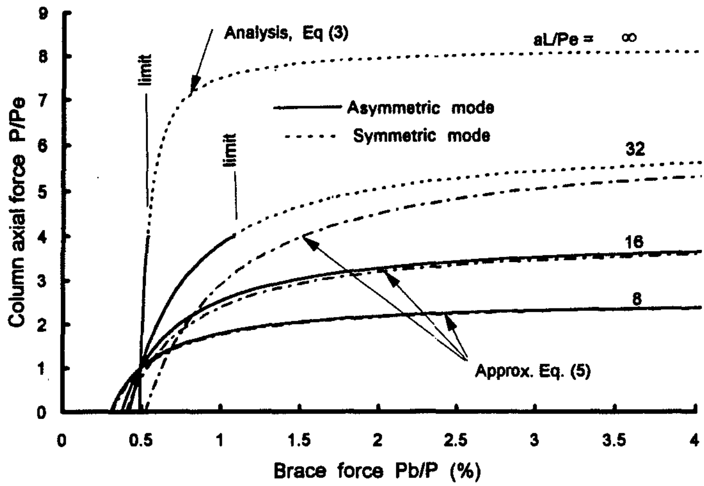


Fig. 6 Variation of bracing force with applied axial force, elastic behaviour, $\lambda = 1$

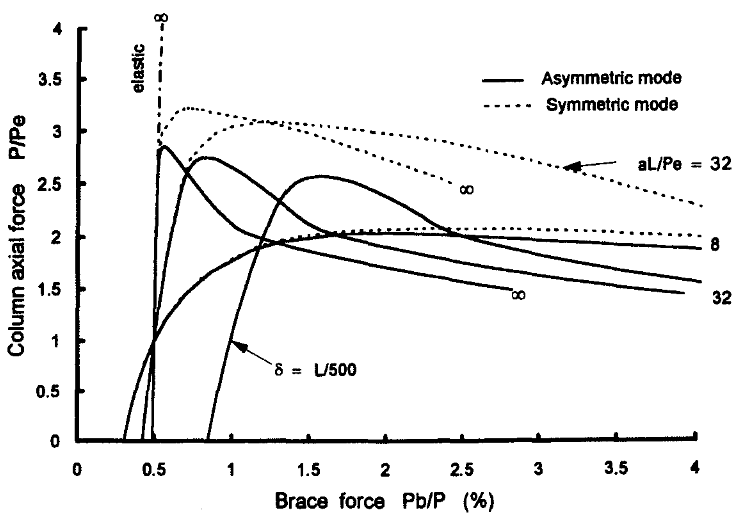


Fig. 7 Variation of bracing force with applied axial force, inelastic behaviour, $\lambda = 1$

Bracing Lack-of-Fit

The effect of the lack-of-fit between the brace and the column was not investigated in the parametric study. Assembling the brace-column system in which there is lack-of-fit may increase or decrease the out-of-straightness of the column and may subject the brace to an initial tension or compression. Subsequent loading may increase or decrease the bending in the column, and may increase or decrease the force in the brace. Using an elastic analysis, combined with a column curve to account for inelasticity and residual stresses, Mutton and Trahair (1975) studied the problem of lack-of-fit of the column-brace system for a column with a single central brace. This study concluded that the case of perfect fit was generally the most critical, exceptions being for some stocky columns where the force-fit of the brace may decrease the column strength slightly. This result needs to be verified for a range of parameters using a rigorous advanced analysis.

DESIGN SPECIFICATIONS

Some codes and specifications give guidance on the minimum force and stiffness requirements for the bracing in order for the column to be considered fully braced and achieve its full strength P_u based on an effective length ℓ between braces. The minimum bracing force P_b at a point of bracing is usually expressed as some percentage of the maximum compressive force P_{max} in the column. In the limit, P_{max} cannot exceed the design ultimate strength ϕP_u (P_u/γ where $\phi \cong 1/\gamma$).

BS 5950:Part 1:1990 has the following requirements for the minimum brace force P_b/P_{max}:

Single brace	2.5%	
Intermediate braces	1.0%	for any one brace
Multiple braces	2.5%	summed over all the intermediate braces.

In BS 5950:Part 1:1985, the corresponding figure for a single brace was 1%. The increase to the present figure of 2.5% was most likely in recognition of the "possibility of overloading the more flexible bracing systems leading potentially to the failure of the supporting members" (Nethercot and Lawson, 1992). BS 5950:Part 1:1990 has no requirements for bracing stiffness.

AS 4100-1990 has the following requirements for the minimum brace force P_b/P_{max}.

Single brace	2.5%	
Intermediate brace	2.5%	for any one brace except that a lesser value, proportional to the brace spacing, may be used where the braces are more closely spaced than is required to ensure that the applied design load P is equal to the design ultimate strength ϕP_u ($\phi = 0.9$).

AS 4100-1990 contains no requirements for bracing stiffness.

AS 1250-1972 was the first edition of the metric working stress code that preceded the current limit states code AS4100-1990. The force requirements in AS 1250-1972 for the minimum brace force P_b/P_{max} were:

Single brace	2.5%	
Multiple braces	2.5%	distributed evenly over all the intermediate braces provided the braces are uniformly spaced.

The requirements for minimum brace stiffness were:

Single brace $10P_w/L = 6P_{max}/L$

Multiple braces $4P_w/L = 2.4P_{max}/L$ for each individual brace

P_w is the maximum compressive force in the column at working loads which, for the implied load factor of 1/0.6 in AS 1250-1972, is equivalent to $0.6P_{max}$ in strength limit state format. The stiffness requirement of $6P_{max}/L$ for the single brace is shown plotted on Fig. 5 assuming $P_{max} = P_u$. The stiffness requirement can be seen to be not unreasonable, but perhaps a little on the low side. It is interesting to note that in an investigation of the bracing stiffness requirements of AS 1250-1972, Mutton and Trahair (1975) found that the stiffness requirements were inadequate but fortunately, "the brace force requirements always governed the design of the brace and the inadequacy of the stiffness requirements was unimportant." Based on this finding, stiffness requirements were not incorporated in AS 4100-1990.

Eurocode 3 (1990) takes a different approach to the other codes for the expression of bracing force and stiffness requirements. The requirements for a column of overall length L are:

Single brace		Not explicitly defined but could be interpreted as 2.0% for a deflection Δ of the bracing system (equal to the lateral deflection of the column at the brace position) of less than L/2500.
Multiple braces	2.0%	distributed evenly over all the intermediate braces provided the deflection Δ of the braces is less than L/2500.
	$(1+500\Delta/L)1.67\%$	distributed evenly over all the intermediate braces, if the deflection Δ of the bracing system is greater than L/2500.

Alternatively, the bracing forces can be determined using a second-order analysis of the column-brace system with an initial column out-of-straightness δ for the column of L/500.

CONCLUSIONS

A nonlinear (advanced) analysis has been applied to an investigation of the behaviour of an I-section steel column bent about its weak axis and restrained by a single central brace . It was found that the behaviour was sensitive to the positioning of the central brace and that a small offset from the perfectly central position should be included to model the construction tolerance and to ensure an asymmetric, rather than a symmetric, mode of failure.

The stiffness of the brace, beyond which the column ultimate strength P_u is within 5% of the strength of a column with an infinite brace stiffness, was found to be in the range $6P_u/L$ to $10P_u/L$. This range is reasonably consistent with the minimum stiffness of $6P_u/L$ implied by the superseded code AS1250-1972.

The determination of the force in the brace was based on an initial column out-of-straightness δ of L/1000, as specified as a fabrication tolerance in AS4100-1990, and a typical pattern of residual stresses, with a maximum compressive stress in the flange tips of 30% of the yield stress. For a brace stiffness sufficient to provide effectively full restraint, the maximum force in the brace at the column ultimate strength (for the asymmetric mode of failure) was found to be in the range 0.5% to 1%. This finding indicates that the provisions of BS 5950:Part 1:1990 and AS 4100-1990, both of which require a single central brace to withstand 2.5% of the column axial force, are adequate. The brace force requirements of Eurocode 3 (1990), which imply that the brace must withstand a minimum of 2.0% of the column force, are less conservative than AS 4100-1990 and BS 5950:Part 1:1990 but still adequate. Finally, it is important to note that the bracing force is directly proportional to the column out-of-straightness δ (assumed to be L/1000 in the paper). Acceptable and realistic fabrication tolerances therefore need to be established.

Multiple braces have not be considered here but are the subject of further studies, using the principles developed in this paper, to determine appropriate force and stiffness requirements to develop the design strength of columns.

REFERENCES

AISC-LRFD-1986 *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago.

AS1250-1972 *Australian Standard - SAA Steel Structures Code*, Standards Association of Australia, Sydney.

AS4100-1990 *Australian Standard - Steel Structures*, Standards Australia, Sydney.

BHP Steel (1991) *Hot Rolled Products*, 1991 edition.

Bleich, F. (1952) *Buckling Strength of Metal Structures*, McGraw-Hill, New York, 508p.

BS 5950:Part 1:1985 *The Structural Use of Steelwork in Building - Part 1: Code of Practice for Design in Simple and Continuous Construction; Hot rolled Sections*, British Standards Institution, London.

BS 5950:Part 1:1990 *The Structural Use of Steelwork in Building - Part 1: Code of Practice for Design in Simple and Continuous Construction; Hot rolled Sections*, British Standards Institution, London.

Clarke, M.J., Bridge, R.Q., Hancock, G.J. and Trahair, N.S. (1992) "Advanced Analysis of Steel Building Frames", *Journal of Constructional Steel Research*, Vol. 23, pp. 1-29.

Clarke, M.J. and Hancock, G.J. (1991) "Finite-element Nonlinear Analysis of Stressed-Arch Frames", *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 117, No. 10, pp. 2819-2837.

EUROCODE 3 (1990) *Design of Steel Structures - Part 1: General Rules and Rules for Buildings*, Volume 1 (draft), Commission of the European Communities.

Galambos, T.V. and Ketter, R.L. (1959) "Columns Under Combined Bending and Thrust", *Journal of the Engineering Mechanics Division*, American Society of Civil Engineers, Vol. 85, No. EM2, pp. 1-30.

Green, G.G., Winter, G. and Cuykendall, T.R. (1947) "Light Gage Steel Columns in Wall-braced Panels", *Bulletin 35/2, Engineering Experimental Station*, Cornell University, Ithaca, pp. 1-50.

Mutton, B.R. and Trahair, N.S. (1975) "Design Requirements for Column Braces", *Civil Engineering Transactions*, Institution of Engineers, Australia, Vol. CE17, No. 1, pp. 30-36.

Nethercot. D.A. and Lawson, R.M. (1992) "Lateral Stability of Steel Beams and Columns - Common Causes of Restraint", *SCI Publication 093*, Steel Construction Institute, Berkshire.

Timoshenko, S.P. and Gere, J.M. (1961) *Theory of Elastic Stability*, 2nd edn., McGraw-Hill, New York, 541p.

White, D.W., Liew, J.Y.R and Chen, W.F. (1991) "Second-order Inelastic Analysis for Frame Design: A Report to SSRC Task Group 29 on Recent Research and the Perceived State-of-the-Art", *Structural Engineering Report No. CE-STR-91-12*, School of Civil Engineering, Purdue University, Indiana.

Winter, G. (1960) "Lateral Bracing of Columns and Beams", *Transactions*, American Society of Civil Engineers, Vol. 125, Part I, pp. 807-845.

Zuk, W. (1956) "Lateral Bracing Forces on Beams and Columns", *Journal of the Engineering Mechanics Division*, American Society of Civil Engineers, Vol. 82, No. EM3, pp. 1032-1 - 1032-11.

BRACING REQUIREMENTS FOR COLUMNS WITH EQUAL OR UNEQUAL SPANS

Raymond H. Plaut

Charles E. Via, Jr. Department of Civil Engineering
Virginia Polytechnic Institute and State University
Blacksburg, VA 24061-0105

ABSTRACT

Lateral bracing requirements for elastic columns are examined. The braces are represented by elastic translational and rotational springs. Results are presented for a uniform column with pinned ends and an internal brace. A compressive load is applied at the top of the column. The effect of the bracing location on the critical load is investigated. For imperfect columns with initial deflections, the relationship between the bracing stiffness and the column deflection is studied. For given initial deflection and maximum allowable additional deflection under a working load, the required stiffness and strength of the brace are determined. Suggested modifications to the present design guidelines are proposed.

INTRODUCTION

Only a few studies of bracing requirements for columns have treated cases with unequal spans. Buckling of columns having two unequal spans that are separated by a lateral elastic restraint has been studied by Urdal [1], Brush and Almroth [2], O'Connor [3], Rutenberg and Scarlat [4], Stanway et al. [5], Thevendran and Wang [6], and Plaut and Yang [7]. Bracing requirements for such columns have been discussed by Urdal [1], Galambos [8], Stanway et al. [5], and Plaut [9]. If the spans are unequal, the behavior may be quite different from that of a column with equal spans.

Standard design rules for bracing are based on the classic work of Winter [10]. Winter analyzed columns with equally-spaced braces having equal lateral stiffness. He assumed that the bending moment in the column at each bracing location is zero when the column buckles, which is not always true and may not even be a good approximation in some cases. Also, the standard American guidelines are based on the critical load of the perfect column, rather than the working load. In this paper, modifications to the guidelines are proposed for two-span columns with equal or unequal spans.

Perfect Columns

Consider the two-span column shown in the insert in Fig. 1. It has length L and constant bending stiffness EI relative to the plane of buckling, and is compressed by an axial load P. An internal brace, at a distance a_1 from the base, is modeled by a lateral translational spring with stiffness k and a rotational spring with stiffness c.

The column is said to be perfect (or ideal) if it is initially straight and the load is concentric. At the critical load P_{cr}, sometimes the buckling mode has no deflection at the brace, which is then as effective as if it were a rigid support. This is called "full bracing", and the lowest bracing stiffness for which this occurs is called the "ideal stiffness" k_{id}. For the column in Fig. 1, full bracing is only possible if $c = 0$ and $a_1 = L/2$, in which case $k_{id} = 16\pi^2 EI/L^3 = 157.91 EI/L^3$ and the corresponding buckling mode when $k > k_{id}$ and $P = P_{cr} = 4\pi^2 EI/L^2 = 39.48 EI/L^2$ is a full sine wave. For uniform, pinned-pinned columns with unequal spans, no ideal stiffness exists and full bracing cannot be achieved [3,5,7]. If k_{id} does not exist, perhaps a design guideline for bracing should not involve such a quantity (or an "equivalent" k_{id}).

For the case $k = 300 EI/L^3$ and for several values of c, the effect of the bracing location on the critical load is shown in Fig. 1. Similar curves are presented for $c = 0$ in Refs. 2, 5, and 7. The critical load decreases as the brace moves away from the center of the column. If $c = 0$, the limiting value $P_{cr} = \pi^2 EI/L^2$ as $a_1 \to 0$ or $a_1 \to L$ is one-fourth of the critical load when $a_1 = L/2$. For a given bracing location, the addition of rotational restraint naturally increases the critical load.

Imperfect Columns with Equal Spans

Actual columns are imperfect and possess some initial deflection. Consider the column shown in Fig. 2, with no rotational spring. The axial coordinate is x, the initial deflection (when $P = 0$) is $w_o(x)$, and the deflection under load P is $w(x)$. At the brace, the initial deflection (from the x axis) is d_o and the additional deflection under load P is d.

As a special case, Fig. 3 depicts a column with a central brace ($a_1 = L/2$). If $w_o(x)$ is symmetric, then the pre-buckling deflection $w(x)$ also will be symmetric, as illustrated by the solid curve. If $k \geq k_{id}$, bifurcation occurs when $P = 4\pi^2 EI/L^2$ (based on a linear analysis, which assumes that $(dw/dx)^2$ is negligible compared to unity). The column then buckles into an asymmetric shape, as shown by the dashed curve. Due to the symmetric part of w(x), the bending moment at the brace is not zero. Winter [10] approximated it as zero, and then obtained the relation

$$k = k_{id}\left(1 + \frac{d_o}{d}\right) \tag{1}$$

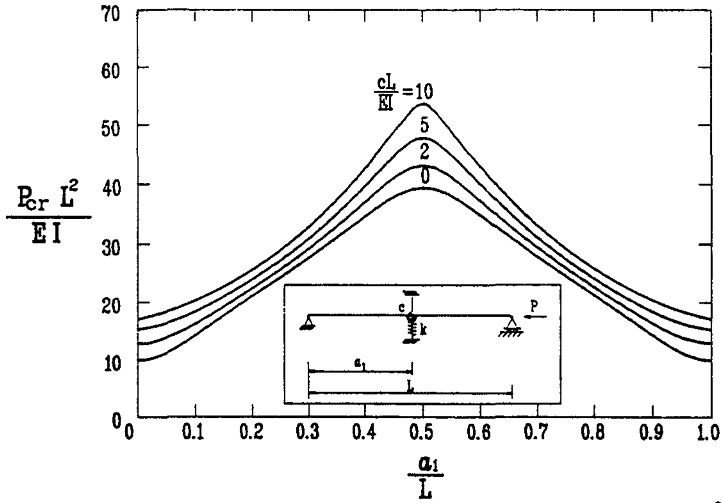


Fig. 1. Effect of bracing location on critical load; $k=300EI/L^3$

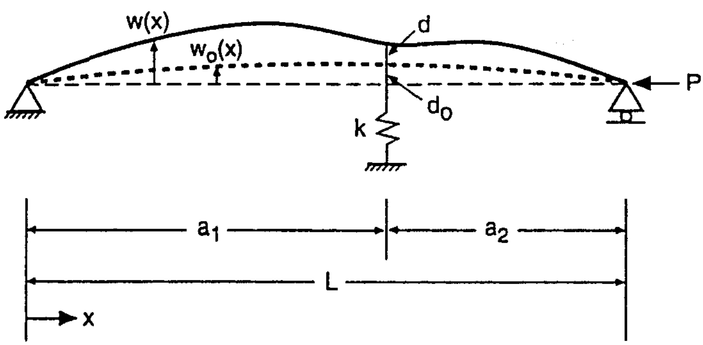


Fig. 2. Imperfect column before loading (dashed) and after loading (solid)

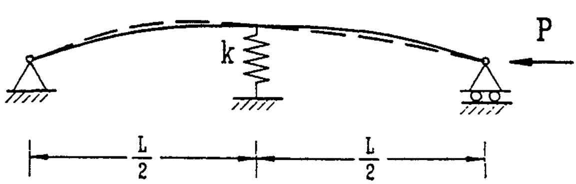


Fig. 3. Deflection before buckling (solid) and after (dashed); $a_1=L/2$, $k>k_{id}$

when P is equal to the critical load of the corresponding perfect column. The force F in the brace is

$$F = kd. \tag{2}$$

The bracing stiffness k given by Eq. 1 and the bracing strength F associated with it have been suggested as design guidelines in Refs. 8, 11, 12, and others.

In order to test the accuracy of Eq. 1, the column in Fig. 3 was analyzed for several initial deflections $w_o(x)$. In all cases, a relation of the form

$$k = k_{id}\left(1 + \eta\frac{d_o}{d}\right) \tag{3}$$

is obtained (when $a_1 = L/2$ and $P = P_{cr}$). If $w_o(x)$ is a half sine wave, which is the buckling mode of the column when there is no brace, the factor η in Eq. 3 has the value 4/3. If $w_o(x)$ is parabolic, one finds that $\eta = 1.41$. Other initial deflections might provide higher values of η. Therefore Winter's approximate formula, Eq. 1, may underestimate the required bracing stiffness. For example, if one wants the additional deflection d to be not greater than the initial deflection d_o, Eq. 1 says that a bracing stiffness $k = 2k_{id}$ would be sufficient; however, if that stiffness were used and if $w_o(x)$ were parabolic, the actual value of d when $P = P_{cr}$ would be $1.41d_o$ (according to a linear analysis).

On the other hand, columns may only be subjected to loads that are substantially lower than the critical load, and it may be more appropriate to base the bracing requirements on the working load instead of the critical load. The Australian Steel Structures Code requires that the bracing stiffness be at least $k = 10P/L$, where P is the working load in the column [13]. Stanway et al. [5] suggest that the stiffness of the central brace should satisfy

$$k \geq \max\{8P/L;\ k_{id}\}. \tag{4}$$

In Ref. 9, the proposed required bracing stiffness for this case is

$$k = \frac{4P}{L}\left(1 + 1.5\frac{d_o}{d}\right), \tag{5}$$

which involves the deflections d_o and d.

With regard to bracing strength, the Australian code requires $F \geq 0.025P$ [13], the British code BS 5950 requires $F \geq 0.01P$ [5], and Eurocode 3 requires $F \geq 0.01P$ if the restraint is rigid ($k = \infty$) and a higher strength if the brace is flexible [5]. A common guideline is that the bracing force F will usually be less than two percent of the axial load in the column [8, 11].

IMPERFECT COLUMNS WITH UNEQUAL SPANS

Consider the column in Fig. 2 with no rotational spring ($c = 0$) and $a_1 \neq L/2$. As mentioned earlier, no ideal stiffness exists and full bracing cannot be achieved with a flexible support in this case. If k is increased, the critical load of the perfect column increases and approaches the value P_∞ corresponding to a rigid brace. For $a_1 = 0.6L$, $0.7L$, $0.8L$, and $0.9L$, respectively, one obtains $P_\infty L^2/EI = 36.78$, 31.76, 27.05, and 23.23.

The relationship between the axial load P and the ratio d_o/d of the initial and additional deflections at the brace depends on the bracing stiffness k, the bracing location a_1, and the initial deflection $w_o(x)$. Assume that $w_o(x)$ is parabolic and $a_1 = 0.6L$ or $0.4L$. The ratio d_o/d is plotted as a function of the bracing stiffness in Fig. 4. On each straight line, the axial load is fixed (e.g., $P = 18.39EI/L^2$ on the line with $P/P_\infty = 0.5$). One can determine the bracing stiffness required to satisfy a maximum ratio of d/d_o at a given load. For example, if the working load P is $0.7P_\infty$ and if the additional bracing deflection d should not be greater than the initial deflection d_o, then k must be greater than $265EI/L^3$.

The corresponding bracing force F can be determined with the use of Fig. 5, which again applies to the case of parabolic $w_o(x)$ and $a_1 = 0.6L$ or $0.4L$. For the specified load P and the stiffness k determined from Fig. 4 (at a given ratio d_o/d), the ordinate in Fig. 5 gives the bracing force as a percentage of the axial load if $d_o = L/1000$. For other initial deflections d_o, the bracing force percentage can be found by proportionality. For example, if $d_o/d = 1$ and $P/P_\infty = 0.7$, then $kL^3/EI = 265$ from Fig. 4 and $100F/P = 1.0$ from Fig. 5. Therefore, if $d_o = L/500$ and if $k = 265\ EI/L^3$, then when $P = 0.7P_\infty$ the additional deflection is $d = L/500$ and the bracing force is 2.0 percent of P.

Several bracing guidelines have been proposed for the column in Fig. 1 with $c = 0$ and unequal spans. According to Ref. 5, the bracing stiffness should be chosen so that the following three conditions are satisfied (if the column is to be "effectively subdivided"):

$$k \geq 16\pi^2\frac{EI}{L^3}; \qquad k \geq \frac{2PL}{(L-a_1)a_1}; \qquad P_{cr} \geq \frac{\pi^2}{10}\frac{EI}{L^2} + 0.9P_\infty. \tag{6}$$

The last condition in Eq. 6 leads to the requirement that kL^3/EI be greater than 246, 504, 1100, and 4027, respectively, for $a_1 = 0.6L$, $0.7L$, $0.8L$, and $0.9L$. (These are computed from Eq. 7 of Ref. 7.) In Refs. 1 and 8, the following formula is suggested:

$$k = \frac{\pi^2 EI}{a_1^3} + \frac{\pi^2 EI}{(L-a_1)^3}. \tag{7}$$

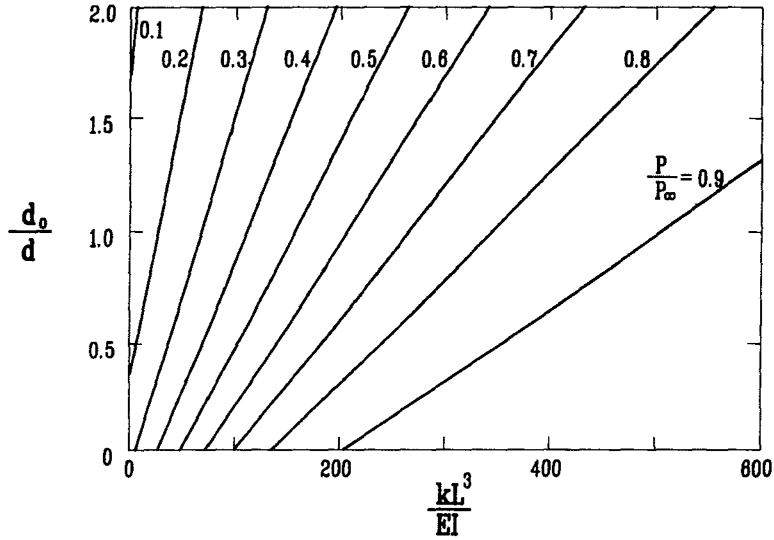


Fig. 4. Effect of bracing stiffness on deflection ratio; a_1=0.6L, parabolic w_o

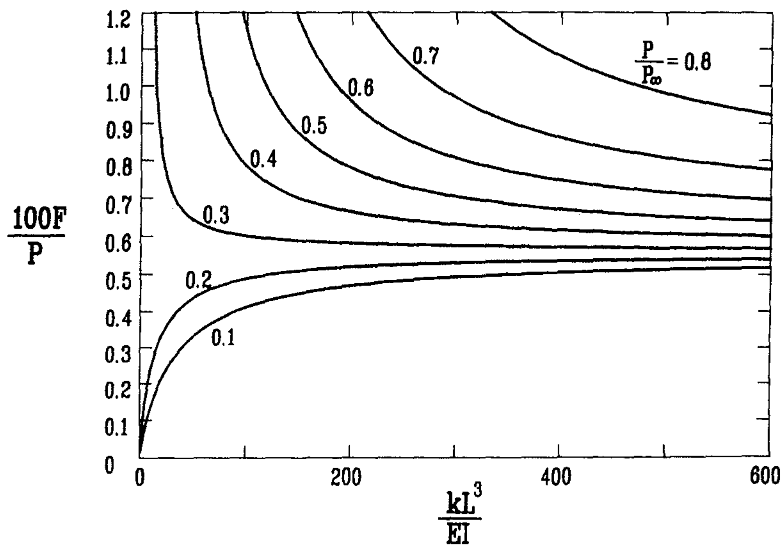


Fig. 5. Effect of bracing stiffness on bracing force;
a_1=0.6L, parabolic w_o, d_o=L/1000

Equations 6 and 7 do not involve d_o or d. Finally, Plaut [9] proposed that the required stiffness be

$$k = \frac{PL}{(L-a_1)a_1}\left(1+1.5\frac{d_o}{d}\right) \tag{8}$$

where $P < P_\infty$. The corresponding bracing force is given by $F = kd$.

These conditions can be compared for the example associated with Figs. 4 and 5, with parabolic $w_o(x)$ and $a_1 = 0.6L$ or $0.4L$. Assume $d_o/d = 1$ and $P/P_\infty = 0.7$, so that the actual required stiffness is $k = 265EI/L^3$. Equation 6 leads to $k = 246EI/L^3$, Eq. 7 gives $k = 200EI/L^3$, and Eq. 8 yields $k = 268EI/L^3$. The first two results do not provide adequate bracing stiffness in this case. As another example, assume that $w_o(x)$ is again parabolic, $d_o/d = 1$, $P/P_\infty = 0.5$, and $a_1 = 0.7L$ or $0.3L$. From calculated data, the actual required stiffness in nondimensional form is $kL^3/EI = 181$, whereas Eqs. 6, 7, and 8, respectively, furnish the values 504, 394, and 189. In this case, the bracing stiffnesses given by Eqs. 6 and 7 are much greater than necessary, whereas Eq. 8 again yields a stiffness that is slightly larger than the actual required value.

It is interesting to examine the effect of the bracing location on the required stiffness. Since P_∞ decreases as the brace moves away from the center, fixing the ratio P/P_∞ would not correspond to the same axial load at different values of a_1. Assume that the working load is $P = 15EI/L^2$, the additional bracing deflection d at this load should be equal to the initial bracing deflection d_o, and the initial shape of the column is parabolic. Results are presented in Table 1 for bracing locations $a_1 = 0.5L$, $0.6L$, $0.7L$, and $0.8L$, and are also applicable for the corresponding locations on the bottom half of the column. The actual required stiffnesses (in nondimensional form) are determined from a linear elastic analysis [7]. In Table 1, the required bracing stiffness increases as the bracing location moves away from the center. As in the previous examples, the stiffness based on Eq. 8 is closer to the actual required stiffness than the stiffnesses based on Eqs. 6 or 7, although Eq. 8 does not always provide a good approximation and is not conservative if $a_1 = 0.8L$ or $0.2L$.

TABLE 1
Required Values of kL^3/EI

a_1/L	Actual	Eq. 6	Eq. 7	Eq. 8
0.5	103	158	158	150
0.6	115	246	200	156
0.7	164	504	394	179
0.8	329	1100	1253	234

Concluding Remarks

Pinned-pinned elastic columns with an internal elastic brace have been considered. The restraint is modeled by a translational spring and (sometimes) a rotational spring, as shown in Fig. 1. The location a_1 of the brace is variable, and the displacements of the column are assumed to be small in the analysis.

For perfect columns (with no initial deflection), the critical load P_{cr} decreases as the bracing location moves away from the center (Fig. 1). If rotational restraint at the brace is significant, less translational stiffness k is required to obtain a given critical load. If the brace is not at the center (or if it has rotational resistance), no ideal stiffness k_{id} exists and full bracing (for which there is no deflection at the brace when buckling occurs) cannot be achieved.

Imperfect columns with initial deflection and no rotational spring at the brace ($c = 0$) have been analyzed (Fig. 2). For a central brace ($a_1 = L/2$), the standard rule for required bracing stiffness is Eq. 1, in which d_o and d are the initial and additional deflections at the brace, respectively. This rule is based on an axial load equal to the critical load and on the approximation of zero bending moment at the brace. It may underestimate the required bracing stiffness at that load.

It seems logical that the required bracing stiffness k should be based on the working load P, the expected or allowable initial deflection d_o, and the maximum allowable additional deflection d. It can be obtained from curves such as those in Fig. 4, where the brace is located at $a_1 = 0.6L$ (or $0.4L$) and the initial deflection is parabolic. (The quantity P_∞ is the critical load for the perfect column with an immovable support at a_1, i.e., with $k = \infty$.) The corresponding required bracing strength (as a percentage of the axial load) can be determined from Fig. 5. Similar curves for a central brace and the same initial shape are presented in Ref. 9.

It is convenient to have simple formulas which could be applicable for any initial shape and any bracing location. Equations 6, 7, and 8 have been proposed in Refs. 5, 8, and 9, respectively. Comparisons between the actual required bracing stiffness and the values from these equations for a few cases indicate that Eq. 8 gives the most accurate approximation and is usually conservative.

Equation 8 is proposed for pinned-pinned columns with an internal brace. The bracing strength is given by $F = kd$ and is less than two percent of the axial load P for most practical designs. If the pinned support at the top of the column is replaced by a flexible support with the same translational stiffness k as the internal brace, the following formula is proposed for required stiffness [9]:

$$k = \frac{1.3\,PL}{(L-a_1)\,a_1}\left(1 + 1.1\frac{d_o}{d}\right). \tag{9}$$

Again, $F = kd$ gives the force in the brace.

For pinned-pinned columns with N equal spans of length S separated by N−1 braces with the same translational stiffness k, Winter [10] also recommended Eq. 1, and k_{id} is given by [14]

$$k_{id} = 2\left[1 - \cos\left(\pi - \frac{\pi}{N}\right)\right]\frac{\pi^2 EI}{S^3}. \tag{10}$$

Stanway et al. [5] suggested a required bracing stiffness of

$$k = \max\left\{3\pi^2\frac{EI}{S^3};\, \frac{6P}{S}\right\}. \tag{11}$$

For unequal spans, Ref. 8 defined an equivalent ideal stiffness

$$k_{id}^{(i)} = \frac{\pi^2 EI}{S_i^3} + \frac{\pi^2 EI}{S_{i+1}^3} \tag{12}$$

for pairs of adjacent spans of length S_i and S_{i+1}, and recommended that the bracing stiffness be at least twice this value. Equation 1, with k_{id} given by Eq. 10, is based on an axial load equal to the critical load and on the approximation of zero bending moment at the braces, whereas Eqs. 11 and 12 do not involve constraints on the deflection of the column. Further development of bracing requirements for columns with more than one internal brace and with equal or unequal spans is needed.

ACKNOWLEDGMENT

The author is grateful to Yu-Wen Yang for carrying out the numerical computations.

REFERENCES

[1] Urdal, T. B. (1969). "Bracing of continuous columns." *Engrg. J.*, AISC, 6(3), 80-83.

[2] Brush, D. O., and Almroth, B. O. (1975). *Buckling of bars, plates, and shells*. McGraw-Hill, New York, N.Y.

[3] O'Connor, C. (1979). "Imperfectly braced columns and beams." *Civ. Engrg. Trans.*, Instn. Engrs., Aust., CE21(2), 69-74.

[4] Rutenberg, A., and Scarlat, A. (1990). "Roof bracing and effective length of columns in one-story industrial buildings." *J. Struc. Engrg.*, 116(10), 2551-2566.

[5] Stanway, G. S., Chapman, J. C., and Dowling, P. J. (1992). "A simply-supported column with a transverse elastic restraint at any position. Part 1: behaviour; Part 2: Design models." *Proc. Instn. Civ. Engrs., Strucs. & Bldgs.*, 94(2), 205-228.

[6] Thevendran, V., and Wang, C. M. (1993). "Stability of nonsymmetric cross-bracing systems." *J. Struc. Engrg.*, 119(1), 169-180.

[7] Plaut, R. H., and Yang, J.-G. (1993). "Lateral bracing forces in columns with two unequal spans." *J. Struc. Engrg.*, to appear.

[8] Galambos, T. V., ed. (1988). *Guide to stability design criteria for metal structures*, 4th ed. Wiley, New York, N.Y.

[9] Plaut, R. H. (1993). "Requirements for lateral bracing of columns with two spans." *J. Struc. Engrg.*, submitted.

[10] Winter, G. (1958). "Lateral bracing of columns and beams." *J. Struc. Div., ASCE*, 84(ST2), 1561-1 - 1561-22.

[11] McGuire, W. (1968). *Steel structures*. Prentice-Hall, Englewood Cliffs, N.J.

[12] Salmon, C. G., and Johnson, J. E. (1990). *Steel structures: Design and behavior*, 3rd ed. Harper & Row, New York, N.Y.

[13] Trahair, N. S., and Nethercot, D. A. (1984). "Bracing requirements in thin-walled structures." *Developments in thin-walled structures - 2*, J. Rhodes and A. C. Walker, eds., Elsevier, London, 93-130.

[14] Zhang, H.-Y., Beliveau, J-G., and Huston, D. (1993). "Minimum lateral stiffness for equally spaced braces in columns," *J. Engrg. Mech.*, 119, to appear.

BRACING CONCEPTS FOR SKYLIGHT AND CURTAINWALL FRAMING

by

LeRoy A. Lutz, Ph.D., PE[1]

Introduction

Bracing of skylight and curtainwall framing members differs in many respects from the bracing of other structural systems. The main reason is that the structural system itself is typically different in a number of ways:

1) The skylight and curtainwall framing members are made of aluminum. They are smaller in size than the members with which most are familiar. Most members are less than 3 inches in width with depths typically ranging between 3 and 6 inches. However, skylight members of 8, 10 or even 12 inch depth are used for large skylight structures.

2) The shapes of the cross-sections vary considerably. The ease of extruding or bending different shapes in aluminum and the need to facilitate assembly, accumulate condensate and provide exterior aesthetics all contribute to this variation in shapes. Although many sections can be identified with the customary I-shape or rectangular tube shape used in steel design, others can not (Figure 1).

3) The connections between the secondary and primary framing members can vary considerably. The secondary framing can be tightly attached to the primary so that there is no question that flexural continuity is achieved (Figure 2 (a)). In other systems only one end of a secondary member is attached tightly while the other is detailed to be loose to permit thermal expansion and contraction to occur without restraint (Figure 2 (b)). In some systems both ends of the secondary member are attached in such a way that rotational restraint of the primary member is indeed questionable and translation restraint occurs only after a gap for thermal movement is closed (Figure 2 (c)).

The series of primary members, which are vertical mullions in curtainwalls and typically rafters in a vertical plane for skylights, are usually not braced horizontally to a "brace point" such as a building column or wall. These members are not attached to in-plane bracing that could be created by adding diagonal framing members to the system. There is some lateral stability from the frame action of the vertical and horizontal members but the strength and stiffness of this frame is typically very poor. In most skylights and curtainwalls the de facto bracing element is the glazing.

The glass is intended to float between the aluminum framing, so that it can move under load and temperature change without making any glass-aluminum contact or avoid any other local stress condition that could damage the entire glass panel.

However, the connection between the glass panel and the aluminum support framing must also be weather tight. This can be accomplished in one of two ways. The glass may be clamped between aluminum components using gaskets (called glazing strips) under a prescribed pressure (Figure 3) or a structural sealant can be applied to hold the glass and provide the weather tight seal (Figure 4). There are systems in use where the

[1] Vice President, Computerized Structural Design, Inc., Milwaukee, WI

glazing panel is clamped on all four sides, where the panel is clamped on two sides with structural sealant on the other two sides and where structural sealant is used on all four sides. Currently four-sided sealant is recommended by the sealant manufacturers to be shop applied only.

The clamping pressure recommended is 4 to 10 pounds per linear inch of glazing strip. Unpublished tests conducted by Computerized Structural Design, Inc. found that an ultimate long term friction force of 4 to 5 pli of contact length existed for one manufacturer's product. The shear stiffness of a 50 durometer glazing strip is in excess of 200 psi; i.e., 200 lbs. per inch of strip per inch of displacement.

The structural sealant produced by several sealant manufacturers has an allowable shearing stress set at 20 psi when subjected to short term loads. A common contact width of sealant to glass is one-quarter inch which leads to a shear capacity of 5 pli. The stiffness of the sealant under short term loads varies considerably with the aspect ratio of the height to the length in the direction of load. The stiffness can exceed 500 psi for aspect ratios of 0.5 or less.

Due to creep of the sealant under long term loads, the long term allowable stress permitted by sealant manufacturers is significantly lower than the 20 psi short term value. Thus, little structural use can be made of the sealant except for wind and seismic loads until higher limits are permitted.

Two aspects of the bracing of skylight and curtainwall framing will be addressed. The first is the lateral bracing provided by the glazing panel to the primary section between adjacent secondary members. The second is the overall bracing of an entire system of framing.

Lateral Bracing of Member between Brace Points

The primary framing member can be subjected to bending inward, bending outward and in the case of skylights also to substantial axial compression. The glazing panel via the glazing strip (or sealant) has some ability to brace the section between points where cross members brace the section. This is most useful and most critical when long glass lights are supported by the section such that the distance between cross members is relatively large.

When examining this situation one finds that the glazing panel is extremely strong and stiff in its plane. There is no need to question the integrity of the glazing as will be shown. In fact, the stiffness of the glazing strip is usually much more than necessary for bracing. The weak link is typically the strength of the glazing strip; i.e., the shearing limit before the glazing strip slips along the glass.

This is examined by considering the section in question being out-of-straight by $L_b/500$ as shown in Figure 5. Thus, the first mode of buckling is considered as critical. The straightness of aluminum extrusions is typically better than that found for steel shapes. This along with the attachment details between components and the esthetics desired might justify use of $L_b/1000$ out-of-straightness. The shape of the member, the movement under load and the resulting shear stress developed from an axial load P_m are considered to vary sinusoidally considering an $L_b/500$ initial eccentricity.

The resistance to moment produced by the axial load on the out-of-straight member is considered to be due entirely to shear intensity V_s in pounds per inch developed between the glass and the glazing strip. Since the lateral displacement is very small, the elastic resistance of the member is ignored.

Thus,

$$M = V_s L_b^2 / \pi^2 = P_m (L_b / 500) \qquad \text{(Eq. 1)}$$

which leads to

$$P_m \cong 50 V_s L_b \qquad \text{(Eq. 2)}$$

where V_s is the allowable shear intensity between the glazing strip and glass.

Consider a member in bending with a flange in compression being braced by the glazing and consider $L_b = 96"$, $L_b/r_T = 120$ with $I_T = 0.4$ in.^{4} and $A_T = 0.625$ in.^{2} (A_T is the area of the compression flange plus one-sixth of the web, I_T is the moment of inertia of this area about the web).

If V_s is 2 pli for the two glazing strips, then $P_m = (50)(2)(96) = 9600$ lbs. and $P_m/A_T =$ 15,360 psi.

Conservatively according to the Aluminum Association Specifications this section with an $r_y = 0.8$, if unbraced, would have

$$F_b = 87{,}000 / \left(L_b/r_y\right)^2 = 6.04 \text{ ksi} = 6040 \text{ psi}$$

At an out-of-straightness of 96"/500=0.192" = e, the unbraced flange can deflect laterally

$$\delta = \delta_i / (1 - P / P_E)$$

from an axial load P with $\delta_i = PeL_b^2 / \left(\pi^2 EI\right)$

With P = 6040 psi (.625) = 3775 lbs,

$$\delta_i \cong 3775(.192)(96)^2 / \left(\pi^2 10E6(0.4)\right) = 0.169"$$

and δ is about twice δ_i.

However, the two glazing strips each with 1 pli shear force and a 200 psi stiffness would permit the member to displace laterally 1/200 = 0.005".

Thus, it appears that the glazing strip with properties as illustrated in the above example is capable of providing complete lateral bracing for the flange. A compression stress limit F_b of 15,360 psi could be used for this member if it is made of 6061-T6 aluminum with a 21,000 psi yield-based allowable. For a 6063-T5 material the stress would, of course, have to be limited by the material allowable of 9,500 psi or perhaps 4/3 that value with bending from a wind load. Obviously a skylight or curtainwall manufacturer would have to test their system or systems to verify that this bracing would indeed be achieved for their system(s).

If a member in bending has the <u>tension</u> flange braced by the glazing, the bracing force required is smaller than that given above for a given flange compression force. Considering the relative stiffness from warping and St. Venant torsion measured by the parameter $K = \sqrt{\pi^2 E I_w / (GJL_b^2)}$, it can be shown that sinusoidal lateral load to the compression flange produces a lower resisting force at the braced tension flange because part of the load transfers across the span L_b. This force ratio is $1/(2K^2+1)$ which will approach 1 for tubular sections having small I_w/J ratios.

This concept, if extended to axially loaded members, produces a $(K^2+1)/(2K^2+1)$ ratio. However, for axially compressed members, the entire cross sectional area would be used in computing a limit stress from the P_m value in Eq. 2. That is,

$$F_a = \frac{P_m(2K^2+1)}{A(K^2+1)} \qquad \text{(Eq. 3)}$$

For beams it will mean that the limiting stress for I shaped members will be based on the critical moment (1)

$$M_{cr} = \frac{\pi^2 E I_c h}{L_b^2} + \frac{JG}{h} \qquad \text{(Eq. 4)}$$

which applies when the tension flange is laterally restrained. The effective r_y, instead of that computed from the equations of Section 4.9 of the Aluminum Association Specifications(2), would be

$$\text{effective } r_y = L_b\sqrt{M_{cr}/(ES_c)}\,/\,(1.2\pi) \qquad \text{(Eq. 5)}$$

using M_{cr} in Eq. 4. For axially loaded members which can be shown to be laterally braced on one flange, the P_{cr} developed by Timoshenko & Gere (3) (their Eq. 5-56) can be used as a basis for obtaining a limiting axial stress by using an appropriate safety factor.

No benefit is attributed to the rotational bracing that is provided by the attachment of the glazing to the aluminum framing. When the glass panel is loaded, the glass can deflect significantly. The rotation induced at its attachment to the framing will likely enhance the normal force and the slip resistance between the cap and the rafter. The rotation of the two glass lites will actually tend to confine the framing member against lateral translation as well. However, differences in rotation of the two lites or the presence of a glass lite on one side only may actually induce rotation in the aluminum member. This rotation may force the glazing to supply rotational restraint to keep the member from buckling laterally. Thus, the rotational restraint of the glazing to framing may well provide further assurance of lateral restraint, but may actually be detrimental to the rotational behavior of the member. Clift and Austin (4) did examine analytically benefits of both rotational and lateral bracing provided to a curtainwall mullion by the glass, but did not supply experimental data to demonstrate the level of bracing achieved.

Overall Bracing of Skylight or Curtainwall System

First consider a series of vertical curtainwall mullions that must be laterally braced. The horizontal framing is not tight between mullions due to need for thermal movement. Either the horizontal member is loose on one end or there is a split in the vertical mullion to permit movement.

One may be tempted to apply a lateral restraint criteria to each vertical which will accumulate to a significant lateral force at one side of the wall or the other. However, often due to presence of corners or for other reasons there may be no place to take the bracing load. Furthermore, one shouldn't be attempting to transfer load across a series of expansion points in the framing unless it is absolutely necessary.

Rotational Restraint

A much more logical approach is to realize that a flexural member can be laterally braced by rotational restraint as well as by lateral restraint [4]. There are several benefits to considering rotational bracing from the horizontal members.

1. The vertical member only needs the appropriate attachment to one horizontal member to achieve bracing at that connection. A bracing moment is transferred into the horizontal bracing member.

2. The rotational restraint provides bracing which is essentially independent to the direction of load or location of load on the member being braced.

The rotational stiffness can be considered to be the lateral (translational) stiffness multiplied by $h^2/4$ (Ref. 5). The attachment of the bracing member must be such as to restrain both flanges of the member being braced. Figure 6 illustrates primary members that appear to be well braced rotationally via the attached horizontal members. If the attachment of the bracing member allows for too much flexibility in the web as may be the case in Figure 7, only partial rotational restraint may exist unless some means of stiffening or strengthening is added to the web.

If full rotational restraint is provided, no lateral restraint is theoretically required. However, a minimal amount of lateral restraint is required to provide some resistance to any component of load that might occur parallel to the glazing. Lateral restraint may be necessary to supplement the rotational restraint if the rotational restraint is not sufficient or when the primary members are subjected to axial loads.

Lateral Integrity

Lateral bracing loads or other in-glass-plane loads can exist in curtainwalls and skylights. An example of an in-plane load is seismic load. A typical glazed curtainwall weights approximately 8 psf and has vertical framing at a five foot spacing. The seismic load could be as much as 24 plf per member. This load would have to span between supports located at the floor levels. If there are split mullions, the 24 plf should be taken by the 5 foot deep system shown in Figure 8. The mullion halves represent the flanges of a plate girder, the horizontal members are the stiffeners and the glass lites represent the web or diaphragms. The shear in the glass lites reaches 24(14'/2) = 168 lbs. The maximum shear intensity developed in the glass lites is 168/60 = 2.8 pli assuming a uniform distribution or 11.2 psi for a quarter inch thick glass. This stress can be transferred to and from the glass by structural sealant and also by shear on glazing strips if an adequate clamping pressure can be insured.

An alternative means of transferring the shear load into the glass lite is by placing elastomeric spacer blocks between the glazing and aluminum framing. The weight of the glass lite is always supported at two locations by elastomeric pads. These pads are typically located at the quarter points of the lower side of the glass lite, but are sometimes moved toward the corners to reduce the stress and deformation of the horizontal supporting member. It is acceptable to use a bearing stress of up to 60 psi on the edge of the glass lite for this permanent load. Elastomeric stops are often used on the sides to insure that the glass lite will never contact the aluminum directly (Figure 3). These stops can also be placed so as to be able to transfer shear load across the panel. They should be detailed to permit thermal movement, but able to pick up load by bearing without significant distortion (racking) of the framing. A swinging door with a glass lite (or lites) and an aluminum frame achieves its structural integrity by blocking the glass tight to the frame.

In curtainwall framing, the lateral support is often not located at a horizontal mullion as illustrated in Figure 8. This requires insuring that the 168 lbs. shear force discussed above can properly be transferred from the adjacent horizontal mullion to the support by weak axis bending of the vertical mullion. If this lateral force were smaller, it may be possible to show that the load can be transferred directly from the glass to the support by structural sealant.

Other examples of framing in need of lateral bracing integrity are the skylights shown in Figures 9 and 10. In each case there are a series of parallel frames interconnected by horizontal members. The primary rafters carry both axial loads and bending moments, i.e., they are beam-columns. The aluminum frame may have the ability to brace itself with proper sizing and connection of the aluminum members. However, lateral movement is necessary to develop the bracing moments from the frame action. Once the glazing is installed, the diaphragm created doesn't permit the lateral movement required to develop the frame action. If anyone has ever had the chance to push laterally on aluminum skylight frame before and after glazing, one will realize the tremendous improvement in stiffness and apparent strength improvement that is obtained from the glazing. One will also realize that the lateral frame action will not occur with the glass in place. The rafters of the ridges shown in Figure 10 have a significant axial design load. These rafters are typically considered as laterally braced at each of the horizontal members about the weak axis and at the ridge for strong axial buckling.

Mock-up tests of portions of skylight and curtainwall systems are often required on major projects. The structural tests, which follow ASTM E330, require testing the mock-up to 150% of the design load. Testing is also done to evaluate its ability to resist water infiltration. Obviously the structural framing stability benefits from presence of the glazing in these tests. In fact, the aluminum framing usually deflects noticeably less than predicted by calculation since the glass and glazing caps actually act compositely with the aluminum cross section. With the test the benefit of the glass in improving the lateral stability of individual members and even the overall structural stability is considered. Designers are often required to demonstrate analytically that individual members are satisfactory without any benefit from the glass.

Seldom though is there dispute that the framing system is adequately braced. One wonders why there is less concern with the overall bracing. Fortunately, the glass does work to produce an adequate diaphragm to brace most skylight and curtainwall systems. Low profile vaults with relatively long spans, which develop large axial forces, should have their lateral bracing more seriously examined. Some probably should be laterally braced by steel or aluminum structure rather than the glass.

The skylights shown in Figures 9 and 10 are all subjected to lateral load from wind (and seismic) forces. These lateral forces plus the bracing forces are usually resisted using the diaphragm created by the glazing. Enough of the framing is constructed tight with the sealant and/or clamping friction to the rafters to develop a diaphragm used to transfer the end wall load by shear to the supports. Alternately blocking of the glazing panel in the framing can also create the diaphragm necessary to transfer the end wall wind or the lateral seismic load to the skylight supports.

Summary and Conclusions

The primary aluminum framing members in skylights and curtainwalls can be braced by the secondary framing members. However, to brace for flexural loads it is better to use the rotational bracing provided by the secondary members in order to stabilize the primary members so as to not have to rely on developing lateral forces.

The attachment of the glass panels to the aluminum framing has the ability to transmit structural loads. Use of structural sealant or insurance of sufficient clamping of the glass to the rafter will permit a glass panel to laterally brace the rafter between secondary members.

The use of glass panels as shear diaphragms due to presence of the structural sealant, clamping of the glass or by blocking the glass panel in the aluminum framework permits the framing system to take lateral bracing and other in-plane (seismic) loads. For skylights, the glass panels are employed as shear diaphragms to laterally brace members with significant axial loads as well as transfer lateral wind loads to the skylight supports.

References

1. Winter, G., "Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending", Transactions, ASCE, Vol. 108, 1943, p. 247.

2. Specification for Aluminum Structures, Fifth Edition, the Aluminum Association, Inc., Washington, D.C., 1986.

3. Timoshenko, S.P., and Gere, J.M., "Theory of Elastic Stability", Second Edition, McGraw-Hill, New York, NY, 1961.

4. Clift, C.D., and Austin, W.J., "Lateral Buckling in Curtain Wall Systems", Journal of Structural Engineering, Vol. 115, No. 10, Oct. 1989, p. 2481.

5. Mutton, Bruce R., and Trahair, Nicholas S., "Stiffness Requirements for Lateral Bracing", Journal of the Structural Division, ASCE, Vol. 99, No. ST10, October 1973, p. 2167.

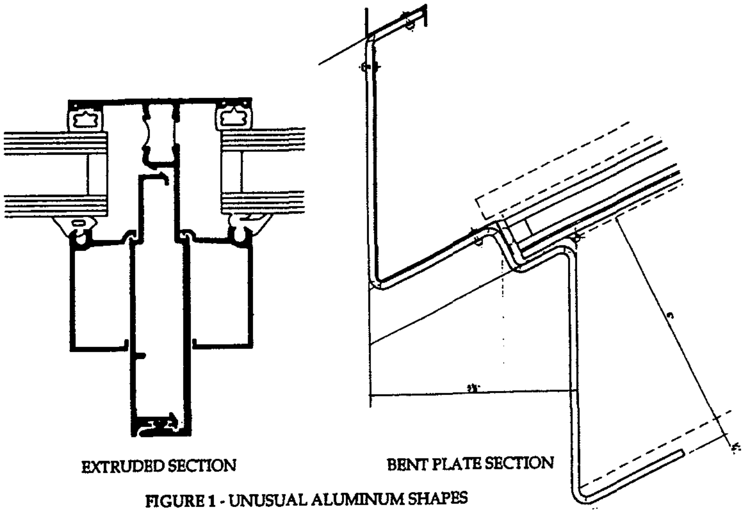


FIGURE 1 - UNUSUAL ALUMINUM SHAPES

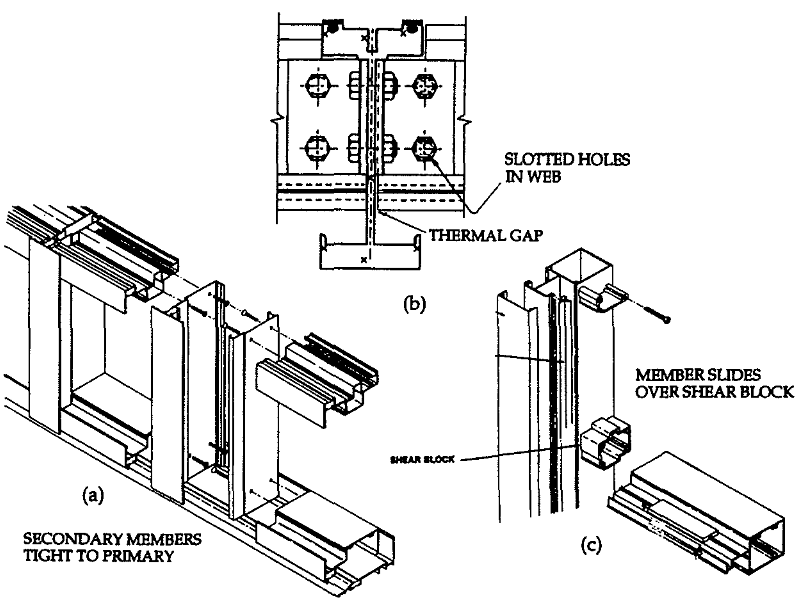


FIGURE 2 - DIFFERENT CONNECTIONS FOR SECONDARY MEMBERS

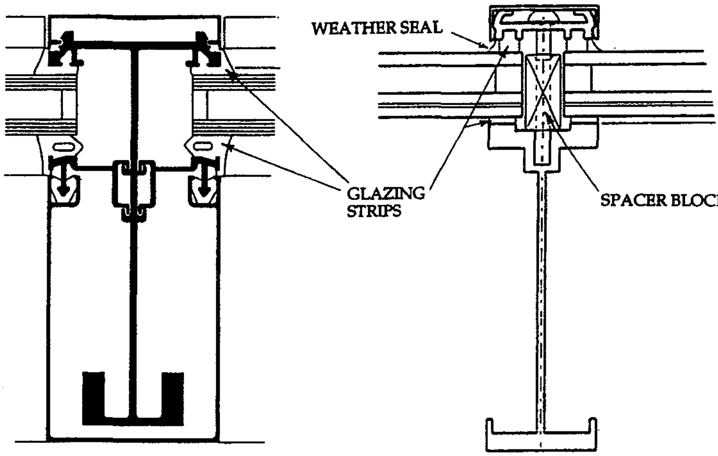


FIGURE 3 - GLASS CLAMPED IN PLACE

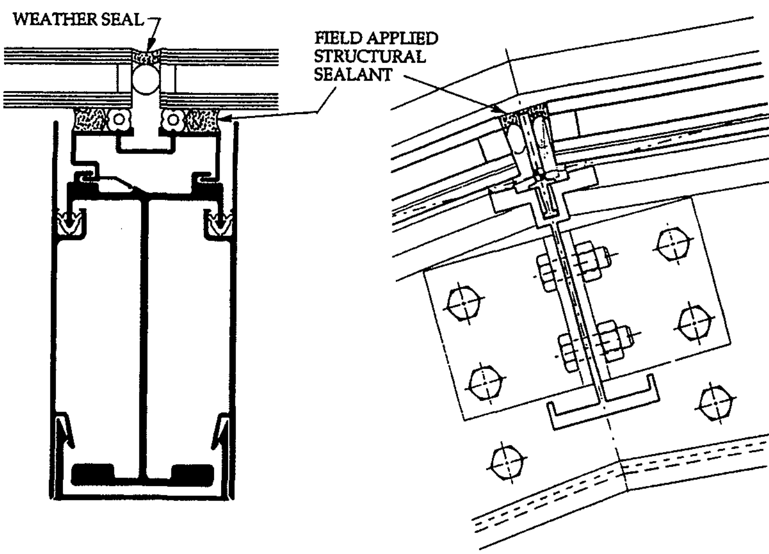


FIGURE 4 - GLASS HELD BY STRUCTURAL SEALANT

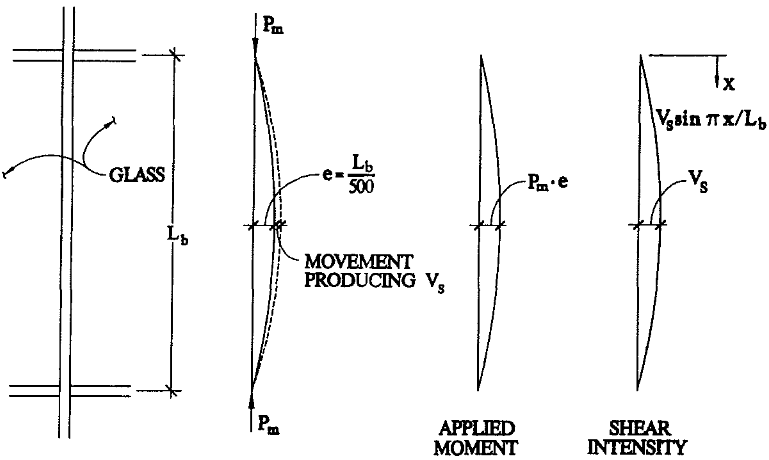


FIGURE 5 - BRACING OF OUT-OF-STRAIGHT MEMBER BY GLASS

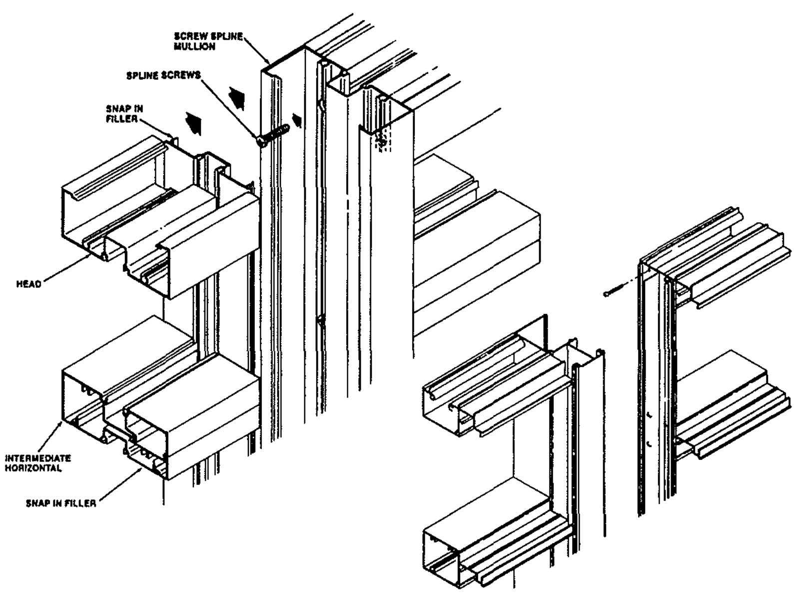


FIGURE 6 - HORIZONTAL MEMBERS ROTATIONALLY WELL ATTACHED

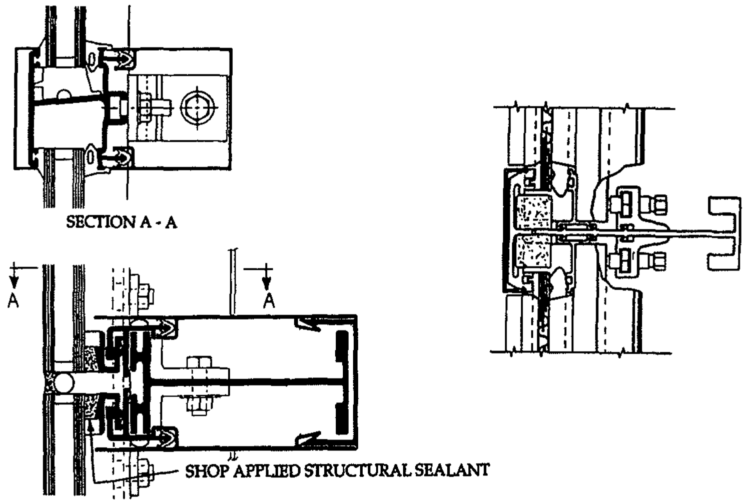


FIGURE 7 - HORIZONTAL MEMBERS WITH FLANGES RELYING ON WEB STIFFNESS

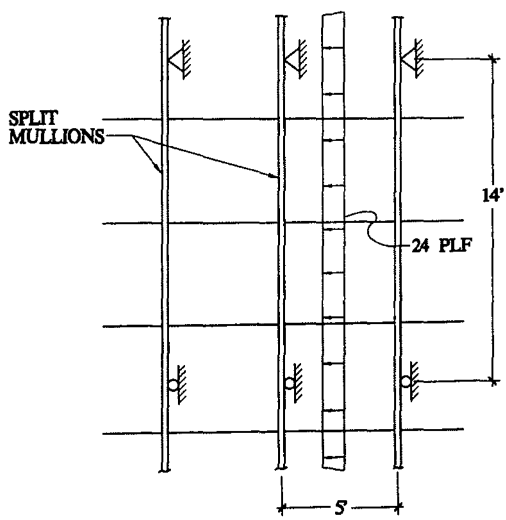


FIGURE 8 - CURTAINWALL FRAMING WITH SPLIT MULLIONS

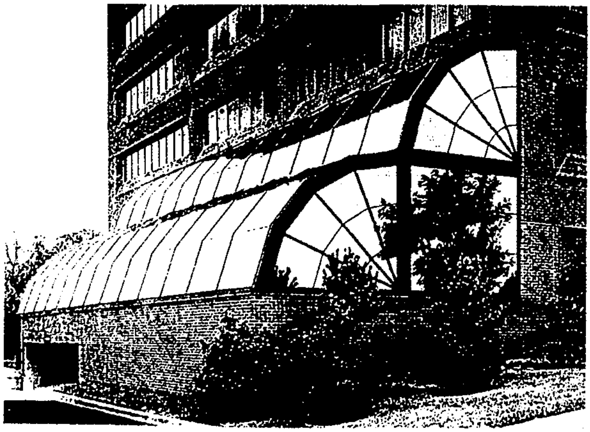
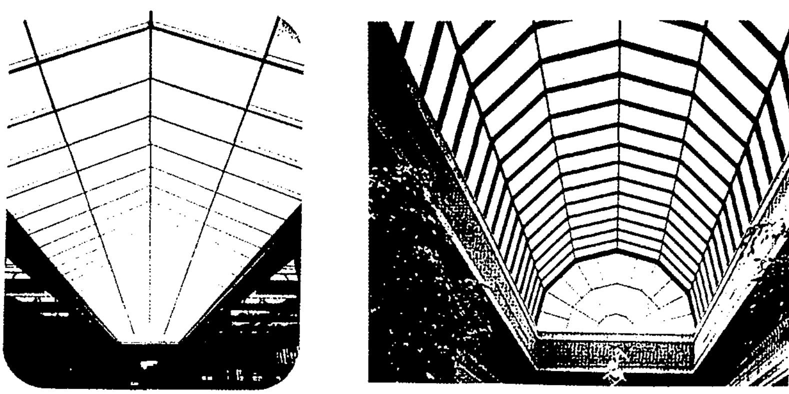


FIGURE 9 - HALF VAULT SKYLIGHT WITH END WALLS



RIDGE

BARREL VAULT

FIGURE 10 - LONG SKYLIGHT STRUCTURES WITH END WALLS

LEAN-ON BRACING SYSTEMS

By

Joseph A. Yura
Warren S. Bellows Centennial Professor in Civil Engineering
University of Texas at Austin

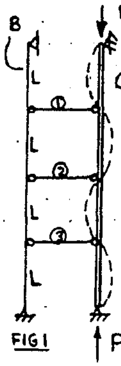
ABSTRACT

Columns and beams may rely on the bracing effect of adjacent members to increase their buckling strength. Such a system is illustrated in Fig. 1 attached. It is shown that these "lean-on" systems are not the same as discrete braces. Exact solutions are presented for such systems and compared with discrete bracing. A practical design approach is developed and several design examples are presented. A case study of the application of the design method for a major power plant structure is discussed.

This paper was unavailable at time of publication. Those wishing a copy of the paper may contact Prof. Yura as follows:

Dr. Joseph A. Yura
Department of Civil Engineering
University of Texas
Austin, TX 78712-1076
FAX: 412-471-1944

LEAN-ON SYSTEMS

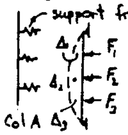


$P_e = \frac{\pi^2 E I_A}{L^2}$

FIG 1

Col A is supported by member B. What I_b is req'd for full bracing?

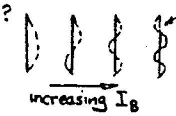
This system is usually modeled as a single point brace system but the spring stiffness is difficult to evaluate as follows:



Is $F_1 = F_2 = F_3$?

There are many possibilities
What is correct?

Will col A bend through the various modes associated with single point bracing as the I_B is increased?



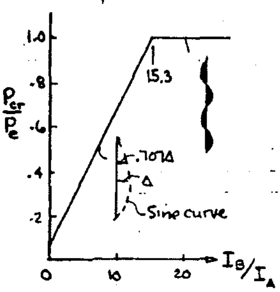


FIG 2

The exact solution to the system shown in Fig 1 indicates that the response is not similar to the single point system. As the I_B is increased, the buckling load increases linearly until the ideal brace situation is reached when buckling occurs between the supports. The response shown in Fig 2 indicates that the buckled shape is always a half sine curve until the full bracing is achieved when $I_B = 15.3 I_A$. There is no switching from one shape to the next higher mode as shown for single point bracing.

$\boxed{\text{LEAN-ON BRACING IS NOT THE SAME AS SINGLE POINT BRACING}}$

A lean-on system is one in which the "bracing" member must have the same shape as the "buckling" member. Such systems can be solved using the **ΣP concept**.

EX 1. - Problem above
Elastic behavior

What I_B is necessary to permit $P = \pi^2 E I_A / L^2$

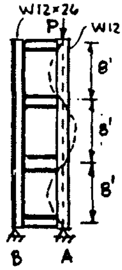
B: $\frac{\pi^2 E I_B}{(4L)^2}$ A: $\frac{\pi^2 E I_A}{(4L)^2}$

$\Sigma P_{cr} = P$

$\frac{\pi^2 E I_B}{16L^2} + \frac{\pi^2 E I_A}{16L^2} = \frac{\pi^2 E I_A}{L^2}$

$\therefore \underline{I_B = 15 I_A}$ - close to 15.3

EX 2. - Crane column - Problem from Lutz-Fisher, AISC Jour, 4th Q, 1985
$\boxed{\text{ASD}}$ - A36 steel



Is the W12×26 sufficient to fully brace the W12×40?

From AISC Manual $P_{allow} = 217^k$ for $L = 8'$

$\Sigma P_{concept}$: W12×40, $A = 11.8 in^2$, $I_y = 44.1 in^4$, $r_y = 1.93$
W12×26, $A = 7.65$ $I_x = 204 in^4$, $r_x = 5.17$

A: $\frac{P}{A} = \frac{217}{11.8} = 18.4$ ksi, $\tau = .301$; $P_A = \frac{12}{23}\frac{\pi^2(29000)(44.1)}{(288)^2}(.301) = 24^k$

B: $\tau = 1.0$, $P_B = \frac{12}{23}\frac{\pi^2(29000)(204)}{(288)^2} = 367^k$

$\Sigma P_{cr} = 367 + 24 = 391k > 217k$ OK

W12×19 would work also

Bracing Practices in Metal Building Systems

Richard C. Kaehler, P.E.
James M. Fisher, Ph.D., P.E.
Computerized Structural Design, Inc.
Milwaukee WI

ABSTRACT

Topics discussed include bracing load paths, out of plane bracing provided by secondary members and diaphragms, and bracing of secondary members by roof and wall diaphragms. Stability load paths, capacities of columns with different bracing spacing on each flange, and the axial capacity of strut-purlins are covered in detail.

1. INTRODUCTION

Although *metal building systems*, sometimes mistakenly called pre-engineered metal buildings, are engineered almost exclusively by metal building manufactures, virtually all engineers involved in the design, specification or review of steel buildings encounter metal building systems at one time or another. Plant and consulting engineers specify, approve, and modify such structures. Building code officials review them for code conformance. A number of stability related questions regularly arise. Some of these questions are equally applicable to low-rise light steel buildings designed by those outside the metal building industry.

Key characteristics of Metal Building Systems include:

A. The primary structural system consists of *unbraced* single span or multi-span steel frames. The rafters and exterior columns are fabricated plate girders. Rafters and columns are usually tapered to optimize the material cost required to resist the forces and moments at every point along their length. Interior columns are generally prismatic shapes, either built-up W sections, or rolled wide flange shapes, pipes or tubes. The exterior columns are usually rigidly connected to the adjacent rafters, while interior columns are usually pinned at the top.

B. The secondary framing is generally cold-formed, light gage structural shapes. These members are usually Z shaped, but C shaped sections or steel joists are sometimes used.

C. The secondary members are often sheathed with cold-formed light gage panel. Wall panels are screwed to the secondaries, while roofs may be screwed down (*through fastened*) or attached with sliding clips (*standing seam roof*). Where architectural or other concerns dictate, a variety of conventional wall and roof systems are used.

To date, metal buildings in the United States have been designed using the allowable stress design approach. Consequently, all references to design equations for primary structure refer to the current AISC ASD Specification[1], while references to design equations for cold-formed light gage members refer to the current edition of the AISI ASD Cold-Formed Specification[2].

2. OUT OF PLANE STABILITY OF FRAME MEMBERS

In most cases, bracing to resist out of plane stability forces from the primary framing is provided by the secondary members and the elements to which the secondary members are attached. Only when frame members become very deep with large flanges is an independent bracing system added.

2.1 Stability Bracing Load Paths

The outside flanges of frame members are braced by girts and purlins framing directly into the primary members. A typical cross section is shown in Figure 1. These secondary members may be simply supported or continuous. Continuous members are attached to the frame with varying degrees of moment continuity. Lateral bracing forces from the primary frame members are transferred through the connections to the secondary members and induce axial forces into the secondary members, as shown in Figure 2.

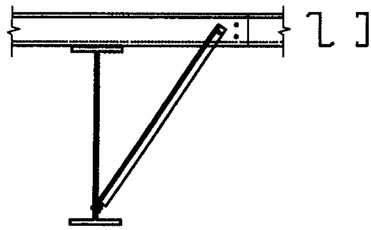


Figure 1 - Typical Cross section with Diagonal Brace

Diagonal Flange Braces

The inside flanges of frame members are braced by diagonal members attached to girts and purlins. Steel angles are usually used for diagonal bracing members; however, the cross section varies from manufacturer to manufacturer. Connection to the primary and secondary members is usually accomplished with a single bolt at each end. Other details of the connection vary. The lateral bracing forces from the inside flanges are transferred to the purlins and girts, where they are resolved into axial and transverse forces.

Diagonal braces are required at or near interior column locations if stiffeners are not used in the rafters to laterally brace the column tops.

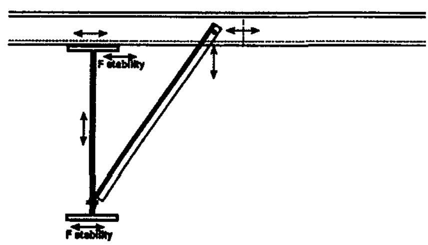


Figure 2 - Bracing Forces Induced into Secondary Members

Rod and Cable Bracing

Tension-only X bracing is generally provided in the plane of wall and roof bays near the building end walls. It is sized to resist longitudinal wind forces. Additional bays of bracing are often provided in long buildings to provide additional seismic resistance and/or frame stability. Braced bays that are provided for stability in long buildings will generally be of the same materials as exterior bays and spaced based on the manufacturer's rule of thumb, rather than specific calculations for the building in question. A typical roof plan is shown in Figure 3.

These braced bays will join the frame at the eave and at every 20 to 30 feet along the roof, providing the eave strut and purlins near the X brace attachment points with a means of resisting the axial bracing forces accumulated from the primary framing.

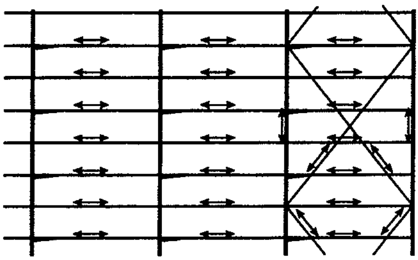


Figure 3 - Bracing Forces in Roof Bracing

At the points of attachment to the frames, the X bracing provides a system of *relative* brace points. If these were the only assumed brace points for the primary framing, the X bracing could, in most cases, be conservatively design for 0.8 percent of the tributary flange forces applied to each brace point; however, purlins and girts between those near X brace attachments must also be assumed to act as brace points to provide economical primary framing.

Wall and Roof Diaphragms

Through diaphragm behavior, the wall and roof panels are generally assumed to resist any axial bracing forces in girts or purlins that are not near X brace lines. If the X brace lines are assumed to be fixed against significant lateral motion, then the diaphragm can provide relative bracing points to the intermediate girts and purlins.

Through fastened panels on cold formed members, and steel deck either screwed to or welded to steel joists, create relatively stiff and strong diaphragms. For ordinary light steel structures, these systems perform their stability bracing function adequately and are seldom subject to analysis unless the diaphragm is being used to carry calculated seismic or wind forces.

The structural details of standing seam roof systems differ significantly among manufactures, resulting in diaphragm stiffnesses that vary over a wide range. In all practical cases, the diaphragm stiffness of a standing seam roof is significantly lower than the stiffness of a through fastened panel system. Nevertheless, virtually all manufacturers assume that their standing seam roof has sufficient stiffness and strength to provide support for axial stability bracing loads in the cold-formed light gage purlins and girts that are positioned between X brace points of attachment. The authors are not aware of any structural failure to date that can be attributed to the inability of a standing seam roof to adequately perform this function.

2.2 Unbraced Lengths of Primary Frame Members

The unbraced length of a primary frame compression flange is taken as the distance between girts or purlins on the outside flange and the distance between diagonal flange braces on the inside flange. Usually, not every girt and purlin has a diagonal brace attached to it; consequently, the distance between braces on the inside flange will be larger than the distance between braces on the outside flange at some locations. In this case, the question of what effective length should be assumed for L_y when computing axial capacity arises.

To simplify computations, individual manufactures will generally choose either the longer or shorter length as a standard approach. The true effective length will lie between these extremes.

If moment continuity in the strong axis plane of the secondary members between the secondary and primary members is achieved, some bracing is provided to the inside flange by the through thickness bending of the web near the connection. As shown in Figure 4, the three elements contributing flexibility to this bracing path are the flexibility of the secondary member in flexure, the flexibility of the connection to the frame and the flexibility of the frame web.

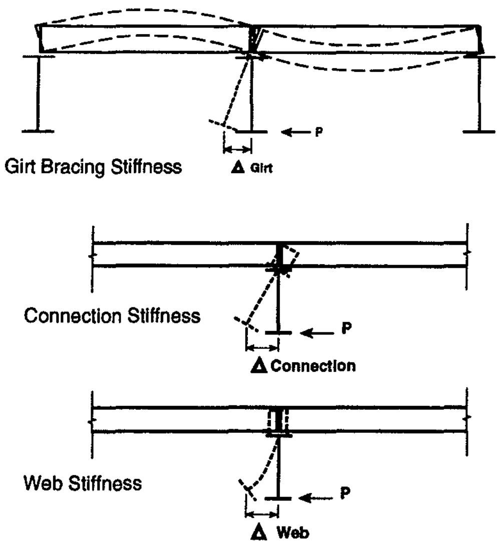


Figure 4 - Sources of Flexibility in Bracing Load Path

If sufficient strength and stiffness are provided, the inside (*indirectly*) braced flange will buckle in the same mode as the outside (*directly*) braced flange, so the column can be considered to be fully braced. As shown in Figure 5, if insufficient strength or stiffness is provided, the indirectly brace flange will buckle in a lower mode at a lower load.

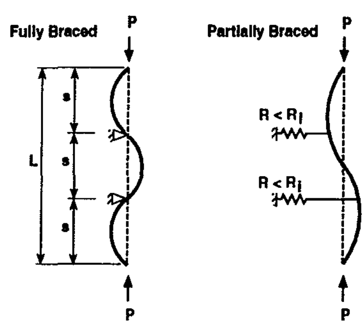


Figure 5 - Buckling modes of fully braced versus partially braced column halves

Summation Approach

In an attempt to develop a practical design approach to this issue, a research program was sponsored by the Metal Building Manufacturer's Association having the following elements:

1. a test program to determine rotational stiffnesses of typical girt to column connections,
2. a theoretical study to develop an equation to predict the net bracing stiffness and strength contributed by the bracing path described above,
3. a test program to measure the actual total bracing stiffness contributed by various girt to column details,
4. a test program to measure the axial compression capacity of columns braced by girts,
5. a test evaluation/theoretical study to derive a practical approach to predicting the capacities of columns with secondary members of known stiffness attached.

The principal findings of this research were:

1. Actual connection rotational stiffness is quite dependent on the connection configuration and must be determined experimentally. In tests, reliable rotational stiffnesses were obtained for secondary members whose flanges were bolted with two bolts to the column flanges. Reliable rotational connection stiffnesses were also obtained for secondary members bolted to the column through bracket plates only when the bolts in shear were fully tensioned.
2. Total bracing stiffness contributed to the unbraced flange, neglecting the increase in torsional stiffness, can be computed as:

   $R = 1 / (F_{girt} + F_{Connection} + F_{web})$

   Where:

   $$\frac{d_{rafter}{}^{2}L}{8EI_{girt}} \leq F_{girt} \leq \frac{d_{rafter}{}^{2}L}{6EI_{girt}} \text{ (inch per kip)}$$

   $F_{connection}$ is determined by test

   $$F_{web} = \frac{d_{rafter}{}^{2.54}}{0.69 b_f{}^{0.38} t_w{}^{1.95} t_f{}^{1.01} E} \text{ (inch per kip)}$$

   A factor of safety on stiffness of 2 is often applied, thus:

   $R_O = R / 2$

3. The total column capacity could be reasonably well predicted by evaluating the braced and indirectly braced halves of the cross section separately and summing their capacities, with the following restrictions applying:

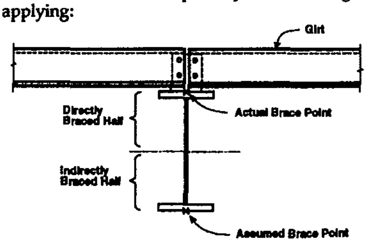


Figure 6 - Directly Braced and Indirectly Braced Column Halves

In the case of a simple or bracketed connection, buckling of the unbraced flange would probably represent the limit of the column capacity. If column top rotation is restrained by an adequate moment resisting connection, as often happens in the exterior columns of a metal building frame, the column can be capable of carrying additional load after the capacity of the indirectly braced half is achieved, up to the point where the braced flange buckles laterally with nodes at the brace points.

4. If bracing stiffness and strength from the braced side is used to increase the capacity of the indirectly braced side, the capacity of the indirectly braced side can be taken as:

   1 Brace:

   $$\frac{P_{cr}}{P_e} = \frac{3R_oL}{16P_e}$$

   2 Braces:

   $$\frac{P_{cr}}{P_e} = \sqrt{\frac{R_oL}{P_e}}$$

   3 or more Braces:

   $$\frac{P_{cr}}{P_e} = 0.9\left(\frac{L}{S}\right)^2\sqrt{\sqrt{\frac{R_oL}{P_e}\left(\frac{S}{L}\right)^3+1}-1}$$

   $$\frac{R_oL}{P_e} \le 4\left(\frac{L}{S}\right)^3$$

   where:

   P_e = Elastic capacity of the column half unbraced
   L = Total column length
   S = Spacing between braces

   The capacity of the indirectly braced half should not be taken as more than one half of P_y (one quarter of P_y for the full column). Even when the bracing stiffness provided appeared sufficient to produce column capacities of the indirectly braced half in the inelastic range, these capacities were not achieved in any test.

Practically speaking, the full capacity of a column based on the smaller outside brace spacing will generally only be achieved in a situation with an excellent secondary to primary connection, a relatively thick web and a required allowable stress in the elastic range of the column buckling curve. When bending moments also exist simultaneously, the allowable axial stresses for the flange under consideration should be used, not the average allowable axial stress.

3. STABILITY OF SECONDARY MEMBERS

Lateral bracing of secondary members is provided by a combination of:

1. rotational and translational support provided by connection details
2. translational support provided by discrete bracing lines
3. diaphragm and rotational stiffness provided by the wall or roof panel.

3.1 Gravity Loads

Screw down deck is generally assumed to provide full lateral bracing to the top flange. The full moment capacity is assumed in the positive moment region. The negative moment region is often designed as unbraced, assuming that the inflection point is a brace point.

The capability of standing seam roof to provide bracing to the top chord under gravity loads varies from manufacturer to manufacturer. No guidance is provided in the AISI specification. Testing by each manufacturer is required to establish the adequacy of this assumption.

Roof slopes and unsymmetrical shapes generate substantial stability forces that must be resisted. AISI section D3.2.1 provides a method for calculating the required anchoring forces when diaphragms are used to laterally brace purlins. In the case of standing seam roof or screw down roofs with very steep pitches, additional bracing lines may be provided at third or half points that must be properly anchored.

3.2 Uplift

Purlins and girts under wind uplift have the majority of their compression flange unbraced and do fail in a lateral stability mode at load levels well below those predicted for fully braced sections. Until recently, very little guidance was available for the design of such members that took into account the favorable effect of the roof or wall sheeting. Very simple provisions for determining the uplift capacities of these members were added to the 1989 Addendum to the 1986 AISI Specification. The nominal capacities permitted are the full strength of the section (Se * Fy) multiplied by the reduction factor R, where R = :

0.4 for simple span C sections
0.5 for simple span Z sections
0.6 for continuous span C sections
0.7 for continuous span Z sections

No analytical method for the rational design of secondary members under uplift sheathed with standing seam roof are available at this time. Some manufactures provide discrete bracing lines and assume that the secondary member is unbraced between brace lines. Others base their designs upon test programs.

3.3 Axial Forces

Roof purlins are usually required to resist and transfer compression loads created by direct wind application to the endwalls and to act as wind truss struts. MBMA and AISI have funded a multi-phase research program on the axial compression capacity of so called strut-purlins.

The first phases of the research, conducted by Hatch et al[3], established that the theoretical method of determining axial capacity developed by Simaan[4] could be applied successfully to a simple span configuration with through fastened roof panel and that the traditional beam-column interaction equations provide a conservative design when both axial and strong axis bending moments are present. The objectives of the last phase of the research, conducted by CSDI, were to simplify the Simaan method sufficiently that it could be used as a practical design method and to determine whether the method is applicable to continuous span systems and standing seam roofs.

Primary Conclusions

1. A typical plot of length versus critical stress is shown in Figure 7. Except for very long and very short members, the axial capacity is not a function of length. The capacity of long members is governed by strong axis buckling.

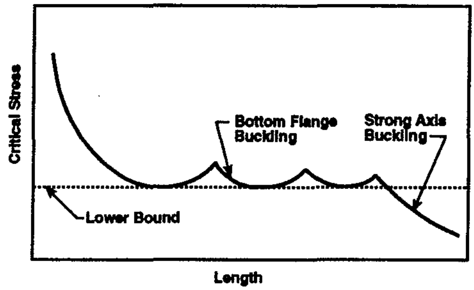


Figure 7 - Strut Purlin Ultimate Stress

The capacity of commonly used, medium length members are governed by a lateral buckling of the unsheathed flange occurring approximately simultaneously with failure of the deck at the fastener locations, as shown in Figure 8.

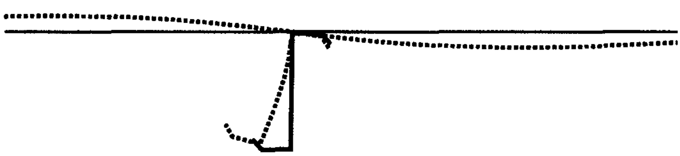


Figure 8 - Lateral Flange Buckling

2. The lower bound ultimate axial stress achieved for medium length members can be represented by the equation:

$\sigma_{cr} = (0.79x + 0.54)(1.17t + 0.93)(2.5b - 1.63 + 22.8)$

where

σ_{cr} = critical axial stress (ksi)
x = a / b as shown in Figure 9
t = thickness of purlin (inches)
b = flange width (inches)
h = web depth (inches)

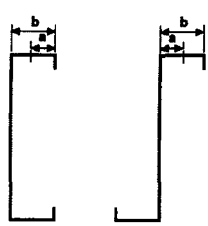


Figure 9 - Fastener Location

For members complying with the limitations below, the range of σ_{cr} is from about 11 ksi to 23 ksi.

3. The capacity of continuous span systems is not substantially different from that of simple span systems.

4. The capacity of systems with standing seam roof must be established by test, but should not exceed the value given by the equation above.

Parameters other than those listed in the equation were considered in the development of the equation, but were found to have insufficient impact on the results to justify their inclusion. The following limitations apply to the use of this equation:

1. C and Z sections with depths from 6 to 12 inches
2. Span lengths from 15 to 30 feet
3. Panels fastened to purlin at 12 inches on center or less and having a minimum rotational stiffness of 0.0015 as determined by the AISI test procedure
4. Yield strengths from 33 to 60 ksi
5. C and Z thickness not more than 0.125 inches

The standing seam systems tested were able to provide substantial axial bracing to the purlins; however; the test loads achieved were significantly lower than those achieved with through fastened panel and the deformations of the purlin before reaching ultimate load were much larger. Given the great variation between manufactures in the detailing of the clips, panels and seams, a large range in bracing capacity should also be expected.

3.4 Open Web Steel Joists

Section 5.8.e of the SJI Standard Specification[5] requires a maximum top chord fastener spacing of 36 inches and requires that each point of attachment be capable of resisting a lateral force of at least 300 pounds. Since even relatively stiff sliding clip systems can be moved by hand, joist manufactures have been reluctant to make the assumption that standing seam roofs provide adequate top chord bracing for joists. Many joist suppliers add extra top chord bracing and design the top chord based on the lateral support provided only by the bridging. Bridging is also supplied to provide stability for the top chord under erection conditions and to provide stability for the bottom chord under uplift conditions.

Recent tests conducted by CSDI have shown that standing seam roofs with sliding clips can provide significant lateral bracing to the top chord of joists. This testing, conducted with several relatively stiff standing seam roof systems, showed that the lateral bracing stiffness and/or strength of these standing seam systems was the governing factor in the capacity of joists with large chords and spans, but that the full capacity of smaller joists could be developed. Because of the variations in joist and standing seam construction, it is essential that the effectiveness of the standing seam roof be determined by test for the specific combination of joist and standing seam roof.

4. CONCLUSIONS

As shown above, each of the components of a metal building system plays an important role in providing stability bracing to members around it. Even though each element is usually sized as required to perform its primary function, it must be recognized that each element is carrying additional bracing forces, sometimes of considerable magnitude.

Capacities of secondary members braced by standing seam roofs must be determined by test. Standard uplift and axial tests should be developed and codified.

The results of the strut-purlin research is ready for review and consideration for inclusion in the AISI light gage specification.

6. REFERENCES

1 American Institute of Steel Construction, "Specification for Structural Steel Buildings - Allowable Stress Design and Plastic Design", American Institute of Steel Construction (AISC), One East Wacker Drive, Chicago, Illinois 60601, June 1, 1989

2 American Iron and Steel Institute, "Specification for the Design of Cold-Formed Steel Structural Members - August 19, 1986 Edition with December 11, 1989 Addendum", American Iron and Steel Institute(AISI), 1133 15th Street, NW, Washinghton, DC 20005-2701, December 11, 1989

3 Hatch, J., Easterling, W.S., Murray, T.M., "Strength Evaluation of Strut-Purlins", Report No. CE/VPI-ST90/08, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA 24061, 1991

4 Simaan, A., "Buckling of Diaphragm-Braced Columns of Unsymmetrical Sections and Application to Wall Studs Design", Report No. 353, Department of Structural Engineering, Cornell University, Ithaca, New York, August 1993

5 Steel Joist Institute, "Standard Specifications for Open Web Steel Joists, K Series", Steel Joist Institute, Richmond, Virginia 23229, November 15, 1989

RATIONAL CRITERIA FOR CLASSIFICATION OF FRAMES AS BRACED FOR GRAVITY LOADS

Mrinmay Biswas

Duke University
Department of Civil and Environmental Engineering
Durham, North Carolina 27708-0287

ABSTRACT

Stability requirement of design specifications defines structural frames as braced when "adequate" lateral bracings are provided. This paper presents rational criteria for adequate values of bracings for frames to be classified as braced to sustain gravity loads.

To address bracing requirements, the gravity load problem is separated from the lateral load problem. For the purpose of gravity load problem, the bracing criterion is identified as a "stiffness" requirement. Bracings act as a secondary system to allow the primary compression members to sustain full designated loads that may be "factored" loads, or "service" loads with consideration of factor of safety.

Then, to address the bracing requirements for gravity loads, general bracing system of a frame is first segregated into two groups, namely, internal bracings and external bracings, since they would require two different mathematical treatments.

To solve the internal bracing problem, the lateral stiffness matrix of the frame structure is expressed symbolically in terms of story stiffness of internal bracing. The limit state of stability of the frame is expressed as its lateral stiffness matrix being positive semi-definite. This results in a set of inequalities. Closed-form solutions are obtained which directly provide the criteria for internal bracings.

The external bracing problem, in which bracings are attached at the floor levels, appears to be simpler than the internal bracing problem, but mathematically it is far more complex. The requirement of the lateral stiffness matrix being positive semi-definite, symbolically expressed in terms of bracing stiffness at floor levels, results in a set of non-linear inequalities. The order of non-linearity is equal to the number of floors in a building frame.

The problem is resolved by using an arbitrary but simple objective function as the optimality criterion for the bracings and using the non-linear inequalities as constraints. The inequalities are derived by using symbolic programming. Numerical experiments are then performed using frames from two to four stories high with different configurations and loadings. Resulting numerical non-linear optimum design problems are then solved by using an available computer program. Examination of results of numerical experiments provides a clue to the design rule for external bracings.

Having arrived at two separate rules for internal and external bracings, respectively, a third rule is derived for integrated internal and external bracings. The rules are then verified by using the stability analysis component of a general purpose structural analysis program.

In spite of invocation of some rigorous mathematical and computational tools in the development stage, the rules that emerge are very simple and require only basic arithmetic operations. Bracings can be easily designed by tabular calculations using a hand-held calculator or by using a simple spread-sheet tool.

INTRODUCTION

Design specifications classify structural frames as "braced" when they are deemed to have "adequate" bracing. (AISC, 1986 and 1989) In effect such a classification allows one to design columns with a so called effective length factor, K, value of 1 or less. Traditionally designers use certain rules of thumb to provide adequate bracing.

A set of rational criteria is provided here for classification of frames as braced. When bracing amount meets or exceeds these values then the frame can be considered as fully braced. Conversely, if any component of a bracing system is less than the corresponding limiting value, then the frame could not be considered as braced. Such a set of limiting stiffness values of bracing is termed *threshold stiffness*. (Biswas, 1983; Galambos, 1988)

Previous research on the subject indicated that when building frames are subject to gravity load only, for the purpose of providing adequate lateral constraint to maintain stability, bracing criteria would be *stiffness* based, as opposed to being *force* based. (Biswas, 1983 and 1988)

Fig. 1 shows a schematic model of a typical plane frame of a multistory, multi-bay building, subject to gravity loads and with general forms of bracing, e.g., diagonal bracing, shear walls, and attachment of floor slabs and roof deck to an adjacent structure. Each of these types of bracing, used separately or together, can provide lateral stiffness. Bracing system in the forms of diagonal bracing and shear wall, etc., constructed within a building structures is termed as *internal bracing*. Internal bracing provides lateral stiffness within a story against inter-floor motion. A system providing lateral stiffness at the floor and/or roof level is termed as *external bracing*.

The frame shown in Fig. 1 is articulated at joints. As members, especially columns, of a real building frame are expected to have some degree of continuity at joints, limit values of stiffness obtained on the basis of such a model will be considered as *upper-bound* values of threshold stiffness.

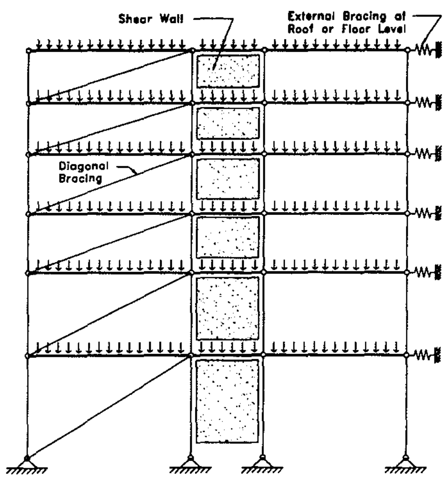


FIG. 1. General Plane-Frame Model

As consideration of internal bracing stiffness and external bracing stiffness would require different mathematical treatment, these two systems are de-coupled. Once their respective solutions are obtained, then the combined situation of internal and external bracing is addressed.

Both internal and external bracing stiffness values are obtained in terms of two sets of basic frame parameters, i.e. 1) *story loads* and 2) *story heights*, as shown in Fig. 2. Stories and floors are numbered top down, the roof deck being the level number 1. Story load is defined in two different ways, that would yield the same result: 1) Story load is the sum of loads on all columns in the plane of the frame, in that story; and 2) Story load is the sum of loads on the plane frame from all floor above. For the *ith* story, the story load C_i is given by:

$$C_i = \sum_{j=1}^{m} c_{i,j} \quad \ldots\ldots (1.a)$$

in which,

$c_{i,j}$ = load on a typical *jth* column in the *ith* story
m = number of columns in the *ith* story

Alternatively,

$$C_i = \sum_{j=1}^{i} P_j \quad \ldots\ldots (1.b)$$

in which, P_j = load on a typical *jth* floor of the plane frame.

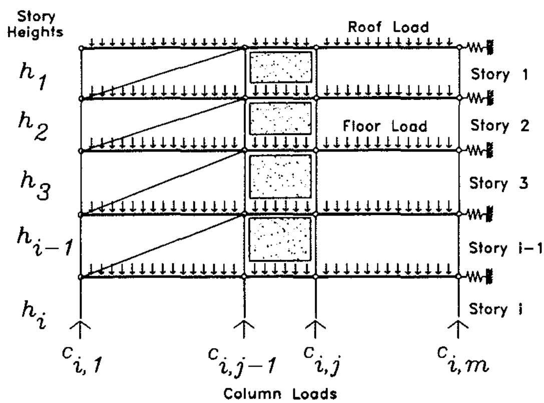


FIG. 2. Frame Bracing Design Parameters

INTERNAL BRACING

The model for the solution of the internal bracing problem is shown in Fig. 3 as a one bay multistory frame including story levels 1 through n. The typical design unknown is the internal lateral *story* stiffness k_i^s applied in the *ith* story. Using only the lateral translational motion of the frame at its floor levels as the degrees of freedom, the stiffness matrix **K** is given by:

$$\mathbf{K} = \mathbf{K}_E + \mathbf{K}_G$$

in which,

$\mathbf{K}_E$ is the so called *Elastic Stiffness Matrix* comprised entirely of k_i^s, and

$\mathbf{K}_G$ is the so called *Geometric Stiffness Matrix*, indicating the effect of load on the structure as expressed in terms of story load C_i and story height h_i.

The stiffness matrix $\mathbf{K}$ is tri-diagonal.

Defining:

$$q_i = k_i^s - \frac{C_i}{h_i} \quad \ldots\ldots. (2)$$

The elements of the first, *ith* and *nth* rows of the matrix $\mathbf{K}$ is given as:

$$k_{1,1} = q_1$$
$$k_{1,2} = -q_1$$

$$k_{i,i-1} = -q_{i-1}$$
$$k_{i,i} = q_{i-1} + q_i \quad \ldots\ldots. (3)$$
$$k_{i,i+1} = -q_i$$

$$k_{n,n-1} = -q_{n-1}$$
$$k_{n,n} = q_{n-1} + q_n$$

Stability of the frame and its marginal state, neutral equilibrium, require that the stiffness matrix $\mathbf{K}$ be positive semi-definite, or

each Leading Principal Minor (*LPM*) ≥ 0. .... (4)

To find story stiffness k_i^s that would satisfy the inequality (4), the following solution scheme is used: (Biswas, 1988)

By using row and column matrix operations, the tri-diagonal stiffness matrix is converted to an upper-triangle matrix.

The Leading Principal Minors of the upper-triangle matrix are formulated as:

$$LPM_i = q_1 \times q_2 \times \cdots \times q_i \quad \ldots\ldots (5)$$

Requiring that $LPM_i \geq 0$, we get:

$$q_1 \times q_2 \times \cdots \times q_i \geq 0, \; i = 1 \cdots n \quad \ldots\ldots. (6)$$

By successively solving the inequality (6), solution for story stiffness is given by:

$$k_i^s \geq (C_i / h_i) \quad \ldots\ldots (7)$$

Verbally stated, internal *Story-Stiffness* must be greater than the ratio of *Story Load* and *Story Height*. The story-stiffness thus obtained for the single bay model can be allocated to one or more bays of the plane frame as desired.

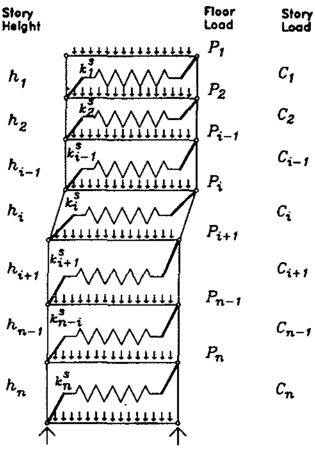


FIG. 3. Frame Model with Internal Bracing

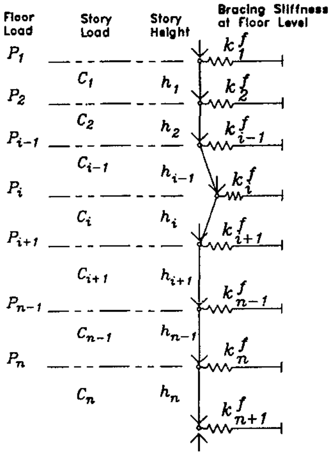


FIG. 4. Frame Model with External Bracing

EXTERNAL BRACING

The model for the solution of the external bracing problem is shown in Fig. 4 as an articulated single stick including story levels 1 through n and corresponding floor levels 1 through $n+1$. The typical design unknown is the external lateral stiffness k_i^f applied at the *ith* floor. Using only the lateral translational motion of the frame at its floor level as the degrees of freedom, the stiffness matrix $\mathbf{K}$, as before, is given by:

$$\mathbf{K} = \mathbf{K}_E + \mathbf{K}_G.$$

Also, in this case, the stiffness matrix $\mathbf{K}$ is tri-diagonal.

Defining a new term:

$$r_i = \frac{C_{i-1}}{h_{i-1}} - \frac{C_i}{h_i} \quad \ldots\ldots\ (8)$$

The elements of the *ith* row of the matrix $\mathbf{K}$ is given as:

$$\begin{aligned} k_{i,i-1} &= r_{i-1} \\ k_{i,i} &= k_i^f - r_i \quad \ldots\ldots\ (9) \\ k_{i,i+1} &= r_i \end{aligned}$$

Again, stability of the frame and its marginal state, neutral equilibrium, require that the stiffness matrix $\mathbf{K}$ be positive semi-definite, or

each Leading Principal Minor (LPM) ≥ 0.

In this case the LPM_i becomes a polynomial of *ith* order and the $LPM_i \geq 0$ condition yields a set of *n* non-linear inequalities of order *n*, indicating non-unique numerical solutions. Such a formulation did not yield a closed form symbolic solution. A non-linear numerical optimization solution scheme was used, as described in the following:

To force a single solution, a heuristic objective function was used:

$$\min.\ (\sum_{i=1}^{n+1} k_i^f) \quad \ldots\ldots (10)$$

Using a symbolic computer program, the leading principal minors were formulated and inequalities were used as non-linear constraints. Then, a non-linear optimization program was used to solve this class of problems. A population of numerical solutions were generated using typical but varying values of number of stories, story loads and story heights. Also, closed form solutions were obtained for two story frames. By examining the patterns of the population of numerical results, a general symbolic solution was postulated. Using the postulated symbolic solution, for realistic building frames, a number of bracing designs were generated. These design were tested using the stability analysis component of a general purpose finite element structural analysis computer program.

Fortuitously, the following very simple symbolic formula emerged:

$$k_i^f \geq 2\ \frac{C_{i-1}}{h_{i-1}} + 2\ \frac{C_i}{h_i} \quad \ldots\ldots (11)$$

Verbally stated, external stiffness at a floor level must be equal to or grater than the sum of twice the ratio of Story Load and Story Height immediately above and twice the ratio of Story Load and Story Height immediately below the floor level. Alternatively stated, external stiffness at a floor level must be equal to or greater than the sum of twice the required story stiffness of the story immediately above and that of the story immediately below the floor level. The stiffness thus obtained may be allocated to one or both sides of the plane frame as desired. Also, because of inherent stiffness of floor system as a diaphragm, the stiffness may also be allocated to other sister frames. A sister frame may be sufficiently stiff by virtue of its own internal story stiffness provided by diagonal bracing or shear walls.

Building Frame

Fig. 5 shows the model of the special case of building frame where the base is fully constrained in the lateral direction. In this case the following two specific boundary condition exists:

$C_0 = 0$, and

$k_{n+1}^f = \infty$.

Besides the inequality (11), the following two formulas can be used:

$$\text{Stiffness at the roof level},\ k_1^f \geq 2\ \frac{C_1}{h_1}$$

$$\text{Stiffness at the level above the base},\ k_n^f \geq 2\ \frac{C_{n-1}}{h_{n-1}} + 1\ \frac{C_n}{h_n} \quad \ldots\ldots (12)$$

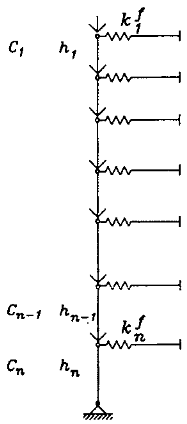


FIG. 5. Building External Bracing

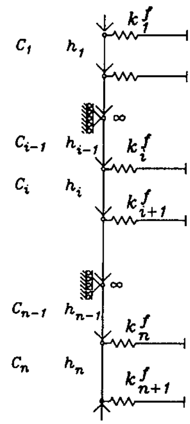


FIG. 6. General Case Of External Bracing

General Frame

Fig. 6 shows the stick model of an externally braced general frame which had full lateral constraints at certain floor levels. In such a case a general expression for the required external bracing can be given as:

$$k_i^f \geq \alpha \frac{C_{i-1}}{h_{i-1}} + \beta \frac{C_i}{h_i} \quad \ldots\ldots. (13)$$

in which,

$$\left.\begin{array}{l} \alpha = 1 \text{ when } k_{i-1}^f = \infty, \text{ else } \alpha = 2 \\ \beta = 1 \text{ when } k_{i+1}^f = \infty, \text{ else } \beta = 2 \end{array}\right. \quad \ldots\ldots. (14)$$

COMBINED INTERNAL AND EXTERNAL BRACING

Based on rules developed for de-coupled internal and external bracing systems, the following rule was deduced for combined bracing condition. Combined bracing system would be required when one system, say the internal bracing system provided, is less than the corresponding threshold value. Then, external lateral stiffness needs to be provided to compensate for the deficit.

Let the k_i^s be internal story stiffness provided and it is less than the threshold value of C_i / h_i. Let the deficit of story stiffness, $k_i^{s'}$ be given by:

$$k_i^{s'} = \frac{C_i}{h_i} - k_i^s \quad \ldots\ldots. (15)$$

Then, the required external lateral stiffness is given by:

$k_i^f \geq \alpha k_{i-1}^{s'} + \beta k_i^{s'}$ .... (16)

Using these postulated symbolic solutions, for realistic building frames, a number of combined internal and external bracing systems were designed. These designs were tested using a general purpose finite element structural analysis program.

SUMMARY

- For design of internal, external or combined bracing systems, that would classify a multistory, multi-bayframe as braced, three theorems have emerged. These theorems generalize the single design rule for one-story, one-bay frame model (Galambos, 1964) and those for multistory columns (Urdal, 1969).
- In spite of the use of rigorous mathematical formulations and computational tools for their development, the resulting design rules are very neat and only simple arithmetic operations will be needed for their use. Tabulated calculations using only hand-held calculators or simple spread-sheet tools will suffice.
- Rules are valid for both LRFD and ASD.

REFERENCES

AISC (1989) *Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design*, American Institute of Steel Construction, Chicago, Illinois., Section C2.1. and Commentary Section C2.

AISC (1986) *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois., Section C2.1. and Commentary Section C2.

Biswas, M. (1988) "Criteria for Design of Internal Bracing," *Stability of Tall Steel Buildings, Workshop Proceedings of the Third International Conference on Tall Buildings*, Council Report M383, Council on Tall Buildings and Urban Habitat, Bethlehem, Pennsylvania, pp. 79-90.

Biswas, M. (1983) "Threshold Bracing Stiffness for Two Story Frames," *Proceedings of the George Winter Memorial Session of the Third International Colloquium on the Stability of Metal Structures*, Structural Stability Research Council, Bethlehem, Pennsylvania, pp. 389-409.

Galambos, T. V. (Ed.) (1988) *Guide to Stability Design Criteria for Metal Structures*, 4th Ed., Structural Stability Research Council, John Wiley & Sons, , pp. 567-568.

Galambos, T. V. (1964) "Lateral Support for Tier Building Frames," AISC *Engineering Journal*, Vol. 1, No. 1, pp. 16-19.

Urdal, T. B. (1969) "Bracing of Continuous Columns", AISC *Engineering Journal*, Vol. 6, No. 5, pp. 80-83.

BRACING FORCES IN DIAPHRAGMS AND CROSS FRAMES

Todd A. Helwig, Joseph A. Yura, and Karl H. Frank

Department of Civil Engineering, University of Texas
Austin, Texas

ABSTRACT

Cross frames and diaphragms are frequently used in bridges to brace plate girders and stringers. Such brace systems are called torsional braces. Torsional brace stiffness requirements have been previously developed, however, torsional brace strength requirements have not been addressed except to assume that classical rules of thumb such as the 2% rule apply.

This paper concentrates on the strength requirements for torsional braces of beams. The results of a finite element study on single and multiple torsional bracing systems are presented. The study considered brace size as well as shape and magnitude of initial imperfections. The results show that in general brace force requirements control the member size of the diaphragm or cross frame, not brace stiffness. It is also shown that the torsional brace system increases the load on the beam or plate girders so that the applied buckling load is reduced. A practical design formula for torsional brace forces is developed and a design example is included.

INTRODUCTION

Current design practice for steel bridges often makes use of composite construction in which the concrete slab will provide continuous lateral restraint to the top flange of the girder in the finished bridge. The critical stage for lateral torsional buckling for these girders is typically during construction before the concrete deck has cured. Bracing of the girders is usually accomplished by providing torsional bracing which prevents the twist of the girder cross section. Diaphragms and cross frames are the two primary types of torsional bracing which are used in bridge construction. Other systems such as joists and joist girders also use cross frames to stabilize the members before the slab is constructed. A torsional brace permits the same lateral displacement of both flanges, however relative lateral displacement (twist) is prevented.

In order for a brace to be effective it must satisfy both stiffness and strength criteria.[1] Stiffness requirements for torsional braces have been developed by Ojalvo[2], Trahair[3], and Tong[4]. The evaluation of the stiffness of a torsional bracing system must include several elements such as diaphragm (or cross frame) stiffness, girder stiffness, and cross sectional distortion. Many published solutions do not include girder stiffness or cross sectional distortion. Current AASHTO provisions do not adequately address either stiffness or strength criteria of bracing systems.

This paper will present the results of a study that has made use of the finite element program ANSYS to perform several large displacement analyses on a twin girder system.

Several variables were considered in the study such as shape and magnitude of imperfection, brace size, and number of braces. Equations will be presented for calculating the required torsional brace stiffness and strength.

BRACE STIFFNESS

When bracing is provided by a discrete bracing system, the following equation can be used to determine the ideal brace stiffness, β_{Ti}[5]:

$$\beta_{Ti} = \frac{1.2LM_{cr}^2}{nEI_yC_{bb}^2} \tag{1}$$

where L is the girder length (use 0.75L for one brace, n=1), M_{cr} is the buckling moment, n is the number of braces along the girder length, E is the modulus of elasticity, C_{bb} is the factor for the moment gradient of the braced girder, and I_y is the moment of inertia of the cross section about the y axis. For singly symmetric sections, substitute I_{yeff} for I_y in Eq. 1. I_{yeff} can be calculated using the following equation:

$$I_{yeff} = I_{yc} + \frac{d_t}{d_c} I_{yt} \tag{2}$$

where I_{yc} is the y-axis moment of inertia of the compression flange, I_{yt} is the y-axis moment of inertia tension flange, and d_t and d_c are the respective depths of the web in compression and tension.

Winter has shown for lateral bracing that providing the ideal brace stiffness leads to systems which experience very large lateral displacements. Yura followed the same approach for torsional bracing, and has recommended supplying at least twice the ideal stiffness.[5] Providing at least twice the ideal stiffness will lead to a bracing system that provides much better control over torsional displacements and brace forces.

The stiffness of a typical diaphragm and cross frame is shown in Fig. 1. The stiffness of the cross frame system depends on whether it is defined as a compression system or a tension system. The top and bottom chords are required in a tension system, whereas a compression system only requires the diagonals.

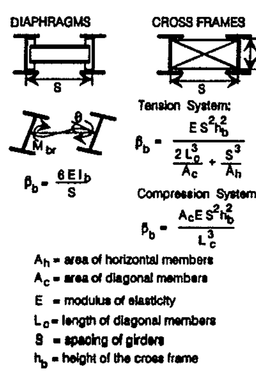


Figure 1 Diaphragm and cross frame stiffness

In addition to including the stiffness

of the diaphragm or cross frame, it is also important to consider the girder stiffness when calculating the stiffness of the entire bracing system. As forces begin to develop in torsional braces, overturning forces also develop at the ends of the brace. The overturning forces cause one of the girders to deflect downwards and the other girder to deflect upwards, which imposes a rigid body rotation on the bracing member. The expression for the girder stiffness, β_g, can be calculated by dividing the brace moment by the rigid body rotation. For a single brace at the centerline, the girder stiffness is shown in the following equation:

$$\beta_g = \frac{M}{\theta_{bg}} = \frac{12 s^2 E I_x}{L^3} \tag{3}$$

Where s is the girder spacing, E is the modulus of elasticity, I_x is the strong axis moment of inertia of the girder, and L is the girder length.

When torsional bracing systems are used, it is very important to include the effect of cross sectional distortion on the system stiffness.[5] Web stiffeners are usually required to control the distortion. The following equation can be used to determine the stiffness due to cross sectional distortion, β_{sec}:

$$\beta_{sec} = \frac{3.3\,E}{h}\left[\frac{1.5\,h\,t_w^3}{12} + \frac{t_s\,b_s^3}{12}\right] \tag{4}$$

where E is the modulus of elasticity, h is the depth of the girder, t_w is the web thickness, and the stiffener dimensions are t_s and b_s.

For partial depth stiffeners shown in Fig. 2, the stiffness of the various sections of the web can be evaluated separately, and then combined to determine the system stiffness, β_T as follows:

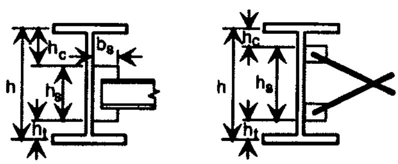


Figure 2 Partially stiffened webs

$$\beta_j = \frac{3.3\,E}{h_j}\left[\frac{h}{h_j}\right]^2\left[\frac{1.5 h_j t_w^3}{12} + \frac{t_s\,b_s^3}{12}\right] \tag{5}$$

$$\frac{1}{\beta_T} = \left[\frac{1}{\beta_c} + \frac{1}{\beta_s} + \frac{1}{\beta_t}\right] + \left[\frac{1}{\beta_b}\right] + \left[\frac{1}{\beta_g}\right] \tag{6}$$

In Eq. 5, β_j= the stiffness of the web section under consideration and h_j is the corresponding web depth. The remaining variables have the same definitions as in Eq. 4. In Eq. 6, β_c, β_s and β_t are the corresponding stiffnesses of the different web sections; β_b is the stiffness of the diaphragm or cross frame; and β_g is the girder stiffness. It should be noted that the system stiffness, β_T, will be smaller than or equal to the smallest of the various components.

BRACE STRENGTH

The brace size in torsional bracing systems is often controlled by strength requirements. The following equations can be used to predict the rotation of the girder at a brace point θ_T and the brace moment M_{br}:

$$\theta_T = \frac{\theta_o}{1 - \frac{\beta_{Ti}}{\beta_T}\frac{M^*}{M_{cr}}} \tag{7}$$

$$M_{br} = \beta_{Ti}\,\theta_o \frac{\frac{M^*}{M_{cr}}}{1 - \frac{\beta_{Ti}}{\beta_T}\frac{M^*}{M_{cr}}} \tag{8}$$

where θ_o = initial twist (radians), β_{Ti} = ideal brace stiffness, β_T = actual brace stiffness, M^* = actual girder moment, and M_{cr} = buckling moment.

Eq. 7 is synonymous with the classic equation for amplification of initial imperfections in columns when the initial shape has the same geometric configuration as the buckled shape. The equation for the brace moment was derived by assuming the brace moment is equal to the brace stiffness multiplied by the rotation at the brace.

Both of the equations have the factor β_{Ti}/β_T in the denominator. This factor takes into consideration the reduction in twist and brace moment when the stiffness provided is larger than the ideal stiffness. Referring to Eq. 7, if the actual stiffness provided is equal to the ideal stiffness, β_{Ti}/β_T is equal to 1.0 and the rotation goes to infinity as the buckling moment is approached. Conversely if the actual stiffness is much larger than the ideal brace stiffness, the denominator tends to 1.0 which gives $\theta_T = \theta_O$.

For design it is recommended to use an initial imperfection (θ_o) of 1 degree and a brace stiffness (β_T) equal to twice the ideal stiffness (β_{Ti}). Eq. 8 then reduces to the following equation:

$$M_{br} = \frac{0.04\ L\ M^2}{n\ E\ I_y\ C_{bb}^2} \tag{9}$$

where M is the design moment for the girder, and the remaining variables are as defined in Eq. 1. For singly symmetric sections, $I_{y\,eff}$ from Eq. 2 can be used in place of I_y.

These equations were developed by Yura[5] following the concepts presented by Winter[1] for columns. The purpose of the paper is to check the accuracy of these simple design equations for actual three dimensional structures.

ANALYTICAL STUDY

The finite element program ANSYS was used to perform several large displacement type analyses on a twin girder system. The first part of the study was performed on a twin girder system similar to that shown in Fig. 3.

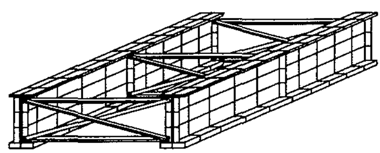


Figure 3 Finite element model

The cross sections of the girders were built up using 8 node shell elements. The model had a cross frame at the centerline of the girders. The cross frames were modeled using truss elements connected at the top and bottom of the web so that there was no effect of cross sectional distortion.

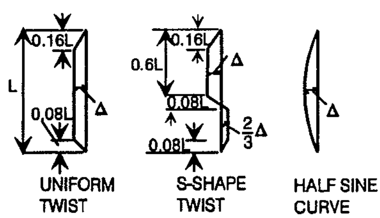


Figure 4 Imperfections considered

The girders were simply supported and free to warp at the supports. The type of loading consisted of a uniform moment. The study considered several variables. In addition to varying the size of the brace, three different shapes of imperfections were considered. The bottom flanges were straight, however Fig. 4 shows the lateral displacements which were applied to the top flange. The lateral displacements were varied to produce three different magnitudes of the initial twist: 0.5, 1, and 2 degrees.

RESULTS FOR SINGLE CROSS-FRAME AT MIDSPAN

Fig. 5 shows a plot for the uniform twist imperfection and $\beta_T = 2.3\beta_{Ti}$. The girder twist normalized by the initial twist is plotted on the vertical axis. The bracing moment is plotted on the horizontal axis. For a θ / θ_0 of 1.75, the 2 degree imperfection gave a brace moment of 92.7 k-in. The corresponding values for the 1 degree and 0.5 degree initial twists are 45.4 k-in and 22.5 k-in, respectively. This shows that the bracing moment is a linear function of the initial twist. If the imperfection is doubled, then so is the brace moment.

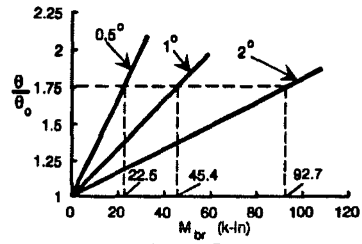


Figure 5

Fig. 6 shows a plot for the 1 degree half sine curve imperfection and $\beta_T = 2.3\beta_{Ti}$. The bracing moment from ANSYS (M_{br}) divided by the bracing moment from Eq. 8 (Eq.. M_{br})is plotted on the vertical axis. The girder moment (M^*) divided by the buckling moment (M_{cr}) has been plotted on the horizontal axis. M^* is the actual applied moment including the overturning force from the cross frame. The overturning force from the cross frame exerts a point load at the midspan of the girders which causes a moment gradient which can be accounted for with a C_b factor of 1.75. The moment from the overturning force was transformed into a "uniform moment" by dividing by C_b =1.75. M^* was calculated by adding the "uniform moment" from the cross frame directly to the uniform moment applied to the girder.

The plot in Fig. 6 shows the importance of including the girder stiffness in the calculation of the system stiffness. If both the cross frame and the girder stiffness are considered, ANSYS and Eq. 8 have very good correlation at the buckling load. If the girder stiffness is neglected, however, Eq. 8 estimates the brace force by about 20 percent less than ANSYS. Therefore it is unconservative to neglect the girder stiffness when evaluating the system stiffness.

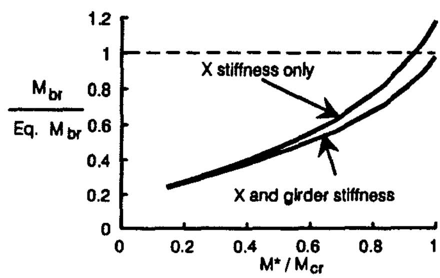


Figure 6

Fig. 7 is a plot for a 1 degree twist of the three different shapes of imperfections with $\beta_T = 2.3\beta_{Ti}$. On the vertical axis M_{br} from ANSYS / Eq.. M_{br} is plotted while M^* / M_{cr} is plotted on the horizontal axis. When the girders have the s-shaped imperfection, Eq. 8 provides conservative estimates for the brace force compared to ANSYS. As in Fig. 6, the half sine curve imperfection has good correlation between ANSYS and Eq. 8. The equation is unconservative, however, for the uniform twist imperfection; it underestimates the brace force by about 20 percent when compared to ANSYS.

Fig. 8 has the same format as Fig. 7 except $\beta_T = 6.8\beta_{Ti}$. Eq. 8 again gives conservative estimates for the s-shape imperfection and unconservative estimates for the uniform twist imperfection. The equation is also slightly unconservative for the half sine curve imperfection.

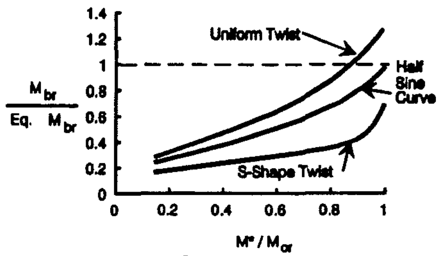


Figure 7

Table 1 summarizes the results of the last two figures for the half sine curve imperfection. The brace area for $\beta_T = 2.3\beta_{Ti}$ was equal to 0.1 in^2. The brace moment from Eq. 8 and ANSYS were almost identical. When the brace area was increased by a factor of 10, the system stiffness only increased by a factor of about 3. This was due to the presence of the girder stiffness in the calculation of the system stiffness. Eq. 8 predicted about a 33 percent reduction in the brace force when the system stiffness was increased to $6.8\beta_{Ti}$ whereas ANSYS only had about a 26 percent reduction. If a designer wants to consider the reduction in brace force when stiffnesses in excess of $2\beta_{Ti}$ are provided, it is necessary to bear in mind that the equation will slightly underestimate the brace moment.

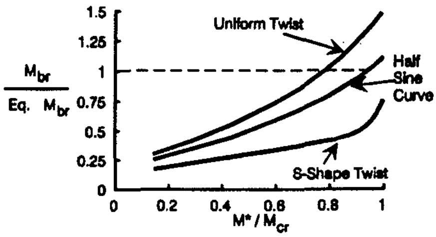


Figure 8

Table 1: Degree Half Sine Curve Imperfection

System Stiffness	Brace Area	Equation 8	ANSYS
$2.3\beta_{Ti}$	0.1 in^2	46.0 k-in	46.2 k-in
$6.8\beta_{Ti}$	1.0 in^2	30.2 k-in (33 % less)	34.4 k-in (26 % less)

TWIN GIRDER WITH MULTIPLE BRACES

The second part of the study considered a twin girder system with multiple braces as shown in Fig. 9. A twin girder system with 4 diaphragms within the span was considered. The girders were simply supported and free to warp at the supports. The type of loading consisted of a uniform load applied at the top flange.

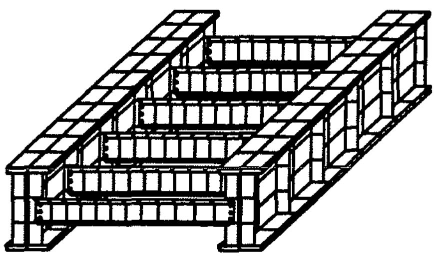


Figure 9 Finite element model

As shown in Fig. 10, three shapes of imperfections were considered. The magnitude of the initial imperfection was 1 degree. When multiple braces are used, the problem is considerably more complicated than the case with a single brace. When full bracing is supplied so that the girders buckle between the braces, the reactions from the brace may vary in direction along the length of the girder.

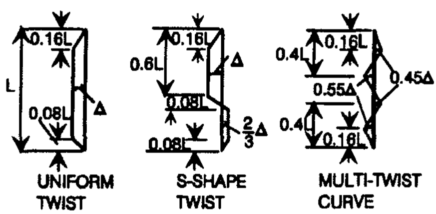


Figure 10 Imperfections considered

Fig. 11 shows the resulting brace forces for the three different shapes of imperfections. The applied load to the girders was $0.9W_{cr}$. The figure shows that the direction of the brace forces is dependent on the initial imperfection. The uniform twist imperfection had the brace forces oriented in the same direction, whereas the other two imperfections had brace forces which varied in direction along the length. The s-shape imperfection had the largest magnitudes of brace forces.

It is also important to notice in Fig. 11 that although the applied load was only $0.9W_{cr}$, in the case of the uniform twist and s-shape imperfection, the actual girder moment was 6-7 percent higher.

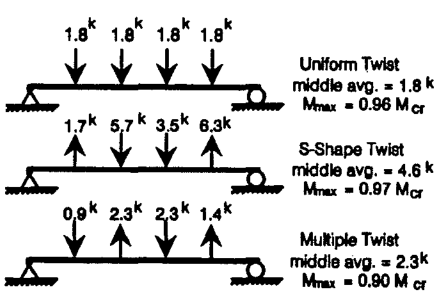


Figure 11 Brace Forces - 0.9 W_{cr}

Evaluating the girder stiffness when multiple braces are used is very complicated due to the alternating directions and magnitudes of the brace forces along the length of the girders. A simple approach to assessing the girder stiffness would be to place a single brace reaction at the centerline of the girder regardless of the number of torsional braces. The girder stiffness would then be based on the centerline deflection which yields the same equation derived previously for a single cross frame at the centerline:

$$\beta_g = \frac{M}{\theta_{b_g}} = \frac{12s^2EI_x}{L^3} \tag{6}$$

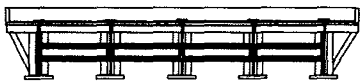
A two dimensional analysis was conducted on the girders considering the brace forces from Fig. 11. The centerline deflection is presented in Table 2 along with the proposed design approach. The table shows that the proposed method of assessing the girder stiffness is somewhat conservative, however, it is a simple solution to a complex problem. The brace force which was calculated using Eq. 8 gives an overturning force of 6.4 kips. This is slightly larger than the forces from ANSYS shown in Fig. 11.

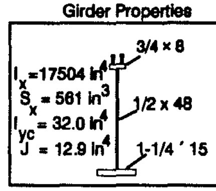
Table 2: Midspan Deflections

CASE	DEFLECTION
DESIGN APPROACH	-0.250 in
UNIFORM TWIST	-0.195 in
S-SHAPE TWIST	-0.193 in
MULTIPLE TWIST	+0.006 in

TORSIONAL BRACING - DESIGN EXAMPLE

Girder Properties





Span = 80 ft. ; 9 in. concrete slab ; 5 girders @ 8 ft spacing, A36 steel

Design a torsional bracing system to stabilize the girders during the deck pour. Use a diaphragm system as shown. The form supports transmit some load to the bottom flange so assume centroid loading.

Use Load Factor Design for the construction condition - L.F. = 1.3

Steel girder: $A = 48.75 \text{ in}^2$; wt = 165 lb/ft

Conc. slab: $8' \times \frac{9}{12} \times 150 \text{ lb/ft}^3 = 900 \text{ lb/ft}$

$w = 1065 \text{ lb/ft} = 1.065 \text{ k/ft}$

$$M = \frac{1}{8} w L^2 \times L.F. = \frac{1}{8}(1.065)(80)^2 1.3 = 1108 \text{ k-ft}$$

$$M_y = 36\,(561)/12 = 1682 \text{ k-ft} > 1108 \text{ k-ft}$$

Try 4 torsional braces @ 16-ft spacing

Check lateral buckling - center 16-ft is most critical (AASHTO 10-102c)

M_r = 1251 k-ft > 1108 k-ft   4 braces required

Brace Strength: Design for $\theta_o = 1^o$ and $\beta_T = 2\beta_{Ti}$: ⇒ Use Equation 9

$$M_{br} = \frac{0.04\, L\, M^2}{n\, E\, I_{y\,eff}\, C_{bb}^2} \qquad I_{y\,eff} = I_{yc} + \frac{d_t}{d_c} I_{yt}$$

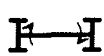
$$I_{y\,eff} = 32 + \frac{18.8}{32.2} 352 = 244 \text{ in}^4$$

$$M_{br} = \frac{0.04\,(80 \times 12)\,(1108 \times 12)^2}{4(29000)\,244\,(1.0)^2} = 240 \text{ k-in}$$

A36 Steel: $S_{x\,req'd} = 240 / 36 = 6.7 \text{ in}^3$

For example: C9 x 13.4 - S_x = 10.6 in^3

Check lateral buckling of the diaphragm

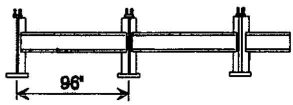


(AASHTO) M_r = 565 k-in > M_y = 10.6 x 36 = 382 > 240 k-in = M_{br}

Brace Stiffness:

$$\beta_{T\,req'd} = 2\beta_{Ti} = 2\,\frac{0.6\,L\,M^2}{n\,E\,(I_{y\,eff}/2)\,C_{bb}^2} = 2\,\frac{0.6\,(80\times12)\,(1108\times12)^2}{4\times29000\times(244/2)\times(1.0)^2} = 14400 \text{ k-in/rad}$$

Diaphragm Stiffness:



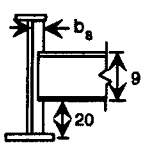
The stiffness of the diaphragms on the exterior girders is $6EI_{br}/S$. Since there are diaphragms on both sides of each interior girder, the stiffness is $2 \times 6EI_{br}/S$. The average stiffness available to each girder is $(2 \times 6 + 3 \times 12)/5 = 9.6\,EI_{br}/S$.

$$\beta_b = 9.6\,(29000)\,47.9/96 = 139000 \text{ in-k/rad}$$

Girder Stiffness:

$$\beta_g = \frac{12\,s^2 E\,I_x}{L^3} = \frac{12\times(96)^2\times29000\times17504}{(960)^3} = 63450 \text{ k"/rad}$$

Cross Section Distortion: use 1/2" thick stiffener



Note: The portion of the web along the depth of the diaphragm can be treated as rigid. Then use Equation 5 to get stiffness of web above and below the diaphragm ($h_c = h_t = 20$ in)

$$\beta_j = \frac{3.3\,E}{h_j}\left(\frac{h}{h_j}\right)^2\left(\frac{1.5\,h_j\,t_w^3}{12} + \frac{t_s\,b_s^3}{12}\right)$$

$$\beta_c = \beta_t = \frac{3.3\,(29000)}{20}\left(\frac{49}{20}\right)^2\left(\frac{1.5\times20\times0.5^3}{12} + \frac{0.5\,b_s^3}{12}\right) \qquad (1)$$

$$\underbrace{\frac{1}{14400}}_{\text{required stiffness}} = \underbrace{\frac{1}{139000}}_{\text{diaphragm stiffness}} + \underbrace{\frac{1}{63450}}_{\text{girder stiffness}} + \frac{2}{\beta_c}\;; \qquad \beta_c = 43000 \qquad (2)$$

Equation (1) and (2) gives b = 3.05 in. - say 3.5 in. Use 3.5x0.5 stiffener

CONCLUSIONS

Based on the ANSYS results the following conclusions may be made:

1. The brace moment is equal to the torsional brace stiffness multiplied by the twist at the brace location.

2. The brace moment is a linear function of the initial imperfection θ_0.

3. The stiffness of the girder should be included in calculating the system stiffness.

4. For any number of braces, the girder stiffness used for calculating the system stiffness can be conservatively estimated by the equation:

$$\beta_s = \frac{M}{\theta_{bs}} = \frac{12s^2EI_x}{L^3}$$

5. The design recommendations gave good correlation with the ANSYS results when the initial shape of the girder matched the buckled shape.

6. For the imperfections considered with multiple braces, the design recommendations gave results which were slightly conservative, but reasonable when compared to the ANSYS results.

REFERENCES

1. Winter, G., 1958, "Lateral Bracing of Columns and Beams," *Journal of Structural Division*, ASCE, ST2, March, pp. 1561-1565.

2. Taylor, A.C., and Ojalvo, M., 1966, "Torsional Restraint of Lateral Buckling," *Journal of the Structural Division*, ASCE, ST2, April, pp. 115-129.

3. Trahair, N.S. and Nethercot, D.A., 1982, "Bracing Requirements in Thin-Walled Structures," Chapter 3, *Developments in Thin-Walled Structures - Vol. 2*, Rhodes and Walkér - Ed., Elsevier Applied Science Publishers, pp. 93-129.

4. Tong, G.S., and Chen, S.H., 1988 "Buckling of Laterally and Torsionally Braced Beams," *Construction Steel Research.*

5. Yura, J.A., Phillips, B., Vegnesa, S.,Webb,S.,"Bracing of Steel Beams in Bridges," Report No. 1239-4F,Center for Transportation Research, The University of Texas at Austin, October 1992.

ERECTION BRACING OF STRUCTURAL STEEL FRAMES

James M. Fisher, Ph.D., P.E.
Michael A. West, AIA, P.E.

Computerized Structural Design, Inc.

While it is uncommon for structures to collapse during their erection, it occurs far more often than the collapse of completed structures. The standards and criteria which apply to the design of buildings are focused on the completed structure. The standards for materials and workmanship also apply to completed work and finished products. Very little regulatory or codified standards or contract requirements apply directly to partially completed assemblies or subassemblies. In the design of a structure this is largely due to that fact that what is designed is a complete structure which is ready for occupancy. Likewise, Building Codes are written to protect the general public in its use of completed buildings as owners, tenants, employees and customers.

Regulations and standards as they relate to a structure under construction approach the construction as a work place not as a building per se. Thus, in the construction contract between the owner and the builder, the builder is given responsibility to build the proposed structure and the builder is given the freedom to choose the "means and methods" necessary to achieve the specified end product as described in the plans and specifications which are part of the owner/builder contract.

This control of the "means and methods" of construction is well expressed in these citations taken from Document A201, "General Conditions of the Contract for Construction" (14) published by the American Institute of Architects:

> "3.3.1 The Contractor shall supervise and direct the Work, using the Contractor's best skill and attention. The Contractor shall be solely responsible for and have control over construction means, methods, techniques, sequences and procedures and for coordinating all portions of the Work under the Contract, unless Contract Documents give other specific instructions concerning these matters.
>
> "3.3.2 The Contractor shall be responsible to the Owner for acts and omissions of the Contractor's employees, Subcontractors and their agents and employees, and other persons performing portions of the Work under a contract with the Contractor.
>
> "3.3.3 The Contractor shall not be relieved of obligations to perform the Work in accordance with the Contract Documents either by activities or duties of the Architect in the Architect's administration of the Contract, or by tests, inspections or approvals required or performed by persons other than the Contractor.
>
> "3.3.4 The Contractor shall be responsible for inspection of portions of Work already performed under this Contract to determine that such portions are in proper condition to receive subsequent Work."

Similar provisions are found in AIA A201/CM, General Conditions of the Contract for Construction,Construction Management Edition with the exception that "overall co-ordination" is a duty of the Construction Manager and that the Construction Manager is included with the Architect in the paragraph comparable to paragraph 3.3.3 cited above.

The delegation of responsibility for "means, methods, techniques, sequences, and procedures of construction" is also found in "Standard General Conditions of the Construction Contract" 1910-11 prepared by the Engineers Joint Contract Documents Committee (25), published by the American Society of Civil Engineers and others. The paragraphs which address the points cited from AIA A201 are 1910-11 paras. 6.1, 6.9.1 and 6.30.2.

The foregoing clearly establishes the Contractor's sole responsibility for the "means and methods" of construction, i.e. the responsibility for the safety and stability of partially completed assemblies and sub-assemblies until the final completion of the proposed structure.

The erection of structural steel is one specialized activity within the overall construction process. It is preceded and followed by the work of other separate trades and is supported by and supports other materials and systems. The erection of structural steel can be carried out by:

1. The General Contractor.
2. A division of the Fabrication/Erection subcontractor.
3. A separate erection subcontractor.

AISC Specifications

The requirements for the complete work are spelled out in the Plans and Specifications. Both the Specifications and the Building Code will in all likelihood make reference to the :"Specifications for Structural Steel Buildings" (24) with commentary published by the American Institute of Steel Construction. This specification gives its requirements for erection in section M4 and its requirements for bracing in paragraph M4.2 as follows:

2. **Bracing**
   "The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in the *Code of Standard Practice* of the American Institute of Steel Construction. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice,* wherever necessary to take care of all loads to which the structure may be subjected, including equipment and operation of same. Such bracing shall be left in place as long as may be required for safety.

   "Wherever piles of material erection equipment or other loads are supported during erection, proper provision shall be made to take care of stresses resulting from such loads."

AISC Code of Standard Practice

The "Code of Standard Practice" (7) cited above is the "Code of Standard Practice for Steel Buildings and Bridges" (adopted effective June 10, 1992) published by the American Institute of Steel Construction. It addresses erection bracing in sub-section 7.9, entitled "Temporary Support of Structural Steel Frames". The general requirements for temporary bracing are set out in paragraph 7.9.1 which reads as follows:

> "Temporary supports, such as temporary guys, braces, falsework, cribbing or other elements required for the erection operation will be determined and furnished and installed by the erector. These temporary supports will secure the steel framing or any partly assembled steel framing against loads comparable in intensity to those for which the structure was designed resulting from wind, seismic forces and erection operations, but not the loads resulting from the performance of work by or the acts of others, nor such unpredictable loads as those due to tornado, explosion or collision."

In paragraphs 7.9.2 and 7.9.3, the Code of Standard Practice draws a distinction between "Self-supporting Steel Frames" and Non-Self-supporting Steel Frames". A Self-supporting steel frame is "one that provides the required stability and resistance to gravity loads and design wind and seismic forces without interaction with other elements of the structure", whereas a Non-Self-Supporting steel frame is "one that, when fully assembled and connected, requires interaction with other elements not classified as Structural Steel to provide stability and strength to resist loads for which the frame is designed". The classification of Structural Steel is given in Section 2.0 of the Code of Standard Practice. Sub-section 2.1 provides a list of elements and material defined as "Structural Steel "and Subsection 2.2 lists other material not classified as Structural Steel. Many of these items are likely part of the permanent lateral bracing and are thus important to the distinction between the two types of steel frames. Some of the items noted in subsection 2.2 as not being Structural Steel are:

Cables for permanent bracing.
Cold-formed steel products.
Embedded steel parts in precast or poured concrete.
Grating and metal deck.
Open-web, long span joists and joist girders.

Other elements of construction such as cast-in-place or hollow core slab floors, cast-in-place or precast shear walls and masonry shear walls can also serve as the lateral bracing of a non-self-supporting steel frame.

When a steel frame is not self-supporting the elements which provide lateral stability to the frame must be identified in the contract documents.

The current edition of the Code of Standard Practice also requires detailed information regarding loadings on temporarily braced non-self-supporting frames. In addition it requires information on the sequence and timing of the installation of "major elements not characterized as structural steel" to be included in the contract documents. These requirements as stated in full in the second paragraph of paragraph 7.9.3 which cited in its entirety below.

> "When elements not classified as structural steel interact with the structural steel elements to provide stability and/or strength to resist loads, the owner is responsible for the installation, structural adequacy during installation, and timely completion of all such elements. The contract documents must specify the sequence and schedule of placement of such elements and the effects of the loads imposed on the structural steel frame by partially or completely installed interacting elements. The erector furnishes and installs temporary support as necessary in accordance with this information but does not thereby assume responsibility for the appropriateness of the sequence specified."

In the Code of Standard Practice, contract documents are defined as "The documents which define the responsibilities of the parties involved in bidding, purchasing, supplying and erecting structural steel. Such documents normally consist of a contract, plans and specifications". As stated previously the plans and specifications are prepared to depict a completed building. Also the preparer of the plans and specifications does not control the sequencing of operations or their timing. Thus while the Code of Standard Practice is making a valid request for loads, sequence and times, no one should expect that such information would necessarily be included in the plans and specifications. In most situations the information relating to sequencing would most appropriately be supplied by the General Contractor or Construction Manager.

The consequences of the distinction between self-supporting and non-self supporting structures are two-fold. First, in a non-self-supporting frame the non-structural elements may impose additional stability loads on the temporary bracing of the structural frame and these forces must be accounted for in the design of the bracing. Secondly, the self-supporting vs. non-self-supporting distinction affects the timing of the removal of the bracing. In fact, the current edition of the Code of Standard Practice specifies that in the case of a non-self-supporting structure, the temporary bracing is to be removed by the "owner" (or a designate, of course) and returned in tact to the erector, such removal taking place only after the erector gives consent.

It should be noted that the distinction between frame types does not in any way relieve the erector from the general requirement that temporary bracing will be "determined and furnished and installed by the erector".

One special situation is covered in paragraph 7.9.4. This situation occurs "when the design concept of a structure is dependent upon the use of shores, jacks or loads which must be adjusted as erection progresses to set or maintain camber or prestress". In such cases, requirements are to be "specifically stated in the contract documents". For example, this might apply in such cases of transfer trusses or girders supporting several stories of framing above or cantilevers whose back spans are counterbalanced by loads from subsequent construction.

The Code of Standard Practice gives no specific requirements for the design or placement of temporary erection bracing. This is completely left to the erector's judgment.

The Code of Standard Practice specifically states in its Commentary that the Code does not apply to the erection of "metal building systems" or "standard steel joists", nor by inference would it cover the erection of anything else which is not structural steel.

OSHA Requirements

Because structural steel is erected in the construction site work place, it is subject to requirements of the Occupational Safety and Health Administration, Department of Labor. These requirements are set forth in Subpart R - "Steel Erection" of the Code of Federal Regulations, Title 29, Chapter XVII, Part 1926 (5). Subpart R is divided into three sections.

Section 1926.750 is concerned with temporary and permanent flooring. It has no specific requirements with regard to temporary erection bracing.

Section 1926.751 contains important requirements with regard to the assembly of the frame.

Section 1926.752 indicates in its title that it contains requirements on "plumbing-up". These are also cited in toto:

"(d) Plumbing-up (1) Connections of the equipment used in plumbing-up shall be properly secured.
"(2) The turnbuckles shall be secured to prevent unwinding while under stress.
"(3) Plumbing-up guys [and] related equipment shall be placed so that employees can get at the connection points.
"(4) Plumbing-up guys shall be removed only under the supervision of a competent person."

This is the totality of the requirements of OSHA as they relate to erection bracing. It is not even stated that "plumbing-up" guys are intended to be temporary bracing.

ANSI Requirements

The American National Standards Institute Inc. has prepared its American National Standard for "Construction and Demolition Operations - Steel Erection - Safety Requirements" (pub. no. A10.13-1989) (2). This document was prepared by a group representing diverse backgrounds in the construction industry. It is divided into sixteen sections covering a variety of issues related to Steel Erection Safety. The erection of steel for buildings is covered in subsections 8.6 "Plumbing" and 11.1 "Buildings" and the paragraphs which specifically address erection and erection bracing are as follows: 8.6.1, 8.6.2, 8.6.3, 8.6.4, 8.6.5, 11.1.1, 11.1.9, 11.1.11, and 11.1.12. It should be noted as in OSHA the emphasis is on "plumbing-up" rather than bracing in reference to cables and guys.

While the requirements are useful such as they are, ANSI A10.13 is not adopted by the Code of Federal Regulations or the AISC Code of Standard Practice. Also, both ANSI A10.13 and the OSHA (CFR) do not define "structural steel' and each uses it in a context which appears to be broader than the narrow definition put forth by the AISC. Lastly the AISC, ANSI and OSHA (CFR) exist separately without cross reference. It would be useful for both AISC and OSHA to adopt ANSI A10.13 by reference. There is ample past precedent for doing so.

FM Requirements

Another source offering a recommendation for minimum erection bracing is the Factory Mutual System. It gives the following in its Loss Prevention Data 1-7 "Wind Forces on Buildings and Other Structures" (31):

"The erecting contractor is responsible for installation of some type of bracing to prevent collapse during construction. Installation of bracing is facilitated when end-connecting plates (to which the bracing will be attached) have been shop welded to the steel work. Cable bracing can be effectively tightened by jacks or turnbuckles and is needed only in a few of the total bays.

"Unless a steel framework under construction is properly braced to a heavy existing structure (such as a building or retaining wall) or permanent bracing has been installed, temporary cable "X" bracing should be provided generally in every third bay of all column lines. Framework should be braced in all four directions. If connections for cable "X" bracing have not been provided, they should be installed in the field. Bracing should be in the plane of column center lines. All beams and girders should be connected to columns prior to bracing."

The authors caution the reader that the final sentence of the above paragraph obviously does not mean what it states literally. Bracing must be installed as the frame is erected. The authors concur in FM's minimum bracing approach, but recommend it be located in every fourth bay rather than FM's every third bay.

Adequacy of Current Standards

As can be seen from the foregoing the requirements for erection bracing are very broad and not at all specific with the exception of those of Factory Mutual. Because of this, it is exceedingly difficult for an outside party to tell whether the requirements are being met or not. For example, could an OSHA inspector easily determine if the correct number of "plumbing guys" were being used? Could the inspector readily determine if there was a problem if no plumbing cables were being used? Obviously the answers to these questions must be favorable to confirm that structures are being properly temporarily braced.

The foregoing review of the standards and regulations as they relate to erection bracing demonstrates that there is little control or standardization and that the matter is very dependent on the knowledge and skill of the individual erector and its employees. The degree to which engineering and pre-planning is employed in execution and follow-up in the field is subject to vast variation and poor performance can often be rewarded if no problems occur. This allows a degree of luck to enter into the process which is unacceptable in other aspects of the construction process. This paper is written to propose procedures and practices which, if used, would make the provision of temporary erection bracing more dependable and easier to monitor.

Temporary Supports of Allied Systems

Prior to the presentation of specific recommendations, a review of practices and standards in the erection of allied systems and components is in order.

Regulations for erection bracing of allied systems are established by OSHA, additionally ANSI has published standards. Lastly, recommendations are published by MBMA (16), SJI (15,26,27,28), ACI (4,13,22,23,30), PCI (17,21) and TPI (9,20). The reader is referred to these documents for requirements as they relate to:

1. Erection of pre-engineered metal buildings.
2. Erection of steel joists and joist girders.
3. Erection of formwork for cast-in-place concrete.
4. Erection of precast concrete.
5. Erection of tilt-up concrete wall panels.
6. Erection of pre-fabricated wood trusses.

In many cases the requirements for erection bracing for these allied systems are more exacting than the comparable requirements for steel erection. For example, ANSI A10.9-1983 (1) contains the following requirements with respect to cast-in-place concrete in two sections: 6. "Vertical Shoring" and 7. "Formwork".

> "6.1.1 The specifications and shoring and reshoring drawings shall be prepared or approved by a qualified designer." (A "qualified designer" is defined as "a person who, by possession of a recognized degree, certificate, or professional standing has demonstrated ability in design in the subject under consideration covered by this standard").

The standard gives minimum live and dead loads for design as well as allowable unit stresses in section 7.0 entitled "Formwork".

> "7.1.1 Formwork shall be designed, fabricated, erected, supported, braced and maintained so that it will support all vertical and lateral loads (sec. 7.3.3) [*sic*] that may be applied until such loads can be supported by the structure."

> "7.2.3 Braces, shores and vertical forms shall be designed to resist all foreseeable lateral loads such as wind, cable tensions, inclined supports, impact of placement, and starting and stopping of equipment. The assumed value of load caused by wind, impact of concrete and equipment, acting in any direction at each floor line, shall not be less than 100 pounds per

linear foot of floor edge nor less than 2 percent of the total dead load on the floor. The height of wall forms shall be taken into consideration when determining the wind load for the formwork design. Formwork should be designed to meet minimum wind load requirements of the local building code. The minimum wind design load should be 15 pounds per square foot, unless the local code specifically permits less. Bracing for wall forms shall be designed for a lateral load of at least 100 pounds per linear foot of wall, applied at the top of the wall."

"7.2.4. Formwork shall be designed for all special conditions of construction, such as unsymmetrical placement of concrete, impact of machine delivered concrete, uplift, and concentrated loads."

"7.3.2 All formwork, designed in accordance with this standard, shall be designed by or under the supervision of a qualified designer."

Both sections 6 and 7 cross reference ACI 347-1978 (22). It expands upon the issues presented in ANSI A10.9 and provides a detailed treatment on the subject of formwork. ACI No. 4 (13) is an even more lengthy treatment of the topic.

The "Recommended Practice for Erection of Precast Concrete" (21) in its Chapter entitled "Preconstruction Planning" contains the following:

"The design engineer for the precast concrete shall provide an erection and bracing sequence, developed in conjunction with the erector and engineer of record, to maintain stability of the structure during erection. If the design engineer or the engineer of record fails to provide an erection and bracing sequence, the erector should request in writing any limitations that should be considered. Limitations may state, for example, that loading of the structure shall be balanced, requiring that no elevation be erected more than a stated number of floors ahead of the remaining elevations; or limitations may involve the rigidity of the structure, requiring that walls should not be erected prior to completion of floor designed to carry lateral loads.

"Generally, procedures to ensure stability during erection such as bracing or temporary connection details are developed by a professional engineer engaged by the precast concrete manufacturer or their erector; these may be reviewed by the engineer of record."

In the chapter entitled "Field Considerations for Connections" the following statement is made. "Some of the most common problems in precast structures are caused by the failure of the designer or erector to consider stability, and equilibrium and its components, not only in its completed state, but during all phases of construction".

Erection of precast is covered extensively in the section "Product Installation" which is part of the chapter entitled "Rigging, Handling and Installation", which contains this paragraph:

"Consideration should be given to the erection drawings, bracing drawings, written procedures and calculations related to shoring, bracing and buying to meet local codes, seismic zone requirements, wind and eccentric loadings, and dead, live and super imposed loads applied during erection."

The design of precast concrete is generally performed by engineers employed by the precast manufacturer. These engineers design the precast elements for loading conditions in the completed structure but also for loads included in handling, transporting, and erection. This often includes design of temporary erection bracing, which is provided by the precast erector. As can be seen from the preceding citations it is the intent of the precast industry to have conditions requiring erection bracing worked out ahead of time and incorporated in the erection drawings.

The design of such erection bracing is given a very good treatment in Section 5.9 of the PCI Design Handbook (17). This Section covers such topics as:

- Loads
- Factors of safety
- Bracing equipment and materials
- Erection analysis, and concludes with a design example in which a detailed erection sequence and bracing scheme is presented.

ANSI A10.9-1983 gives the following requirements for temporary supports and bracing and loading for bracing in paragraph 9.4 "Temporary Supports and Bracing":

> "Precast concrete wall units, structural framing, or tilt-up wall panels shall be braced until permanent connections are completed. Temporary supports or bracing shall be designed by or under supervision of a qualified person in accordance with American National Standard Minimum Design Loads for Buildings and Other Structures, ANSI A58.1-1982, but not less than 15 pounds per square foot on projected surfaces. Permanent connections may be used in lieu of bracing provided they are designed to withstand all loads imposed during construction and attachments are made under the supervision of a qualified person."

Current Practice

Having presented the codes, standards and regulations as they apply to both structural steel erection stability and erection stability of allied structural types, the current state of the art and practice with regard to erection stability and the authors' recommended changes in practice can now be discussed.

First, it is the authors' contention that the majority of erection instabilities and failures occur in one story and low rise buildings. Secondly, these instabilities and failures are generally caused by insufficient temporary bracing. This insufficiency can be the result of many causes:

1. The total lack of any temporary bracing.
2. The lack of sufficient temporary bracing.
3. The untimely removal of temporary bracing.

Likewise, localized failures can occur for a multitude of reasons and these localized failures can initiate a progressive collapse of whole structures or significant portions thereof. These localized failures are caused by a variety of reasons, many of which are specific violations of the codes and standards cited previously.

Some of these causes are:

1. Failure to secure individual members such as tie beams and tie joists before moving onto other members.
2. Failure to tighten nuts on anchor bolts.
3. Failure to fasten joists to their supports.
4. Failure to install and anchor bridging prior to placing loads on joists.

The first key to temporary erection stability is having an experienced competent erector who has experienced competent employees. The current edition of ANSI A10.13 emphasizes the need for preplanning and preparation. The authors recommend that this preplanning be formalized to a significant degree. First. a significant improvement must be made to what are currently designated as "erection plans". For the most part erection plans show only the final location of and the piece marks for the elements of the complete structure, as well as, to a lesser extent, the field work for connections and the assembly of loose material.

In addition to the erection plans as currently prepared, the authors propose that a second set of plans be prepared. This second set of plans would show the temporary support scheme to be used and would include:

1. Temporary erection bracing, sizes and locations.
2. Other temporary supports.
3. The starting point of the erection process.
4. The sequence of erection from the starting point.
5. The order in which individual elements are to be set with respect to one another.
6. The order for the progressive installation of temporary bracing.
7. Details for the attachment of temporary bracing.
8. The timing and sequence for the removal of temporary bracing.
9. Reiteration of whether the structural frame is self-supporting or non-self-supporting.
10. Statement of the forces for which the temporary supports have been designed, including those forces from other collateral materials which must be supported by the temporary supports.

11. Each temporary support and bracing drawing shall bear the seal of a registered professional or structural engineer registered in the state in which the project is erected. The temporary bracing design must consider the erection sequencing, frame type and loads imposed.

The responsibility for the preparation and review of the temporary support and bracing documents is set forth in Section 16.11 "Shop Drawings for Temporary Construction" in "Quality in the Constructed Project" (18) published by the American Society of Civil Engineers. It states:

"The constructor has full authority and responsibility for these shop drawings, including design, preparation, review, and approval, since he or she develops the construction plan and has control of the construction process of which the temporary construction is a part. Construction or erection procedures, shoring, bracing for excavations, or other temporary construction requiring engineering analysis or design requires the seal of a qualified professional engineer affixed to the drawings and specifications."

"The design professional usually does not review temporary construction shop drawings except when necessary to determine compatibility with the design of the completed structure."

The authors agree that there is no need to review of the temporary bracing scheme by the engineer of record, chiefly because such a review would obscure the clear distinction of temporary supports as being the sole responsibility of the erector, in that it has to do with the means and methods of construction.

Fabricator - Erector Interaction

In addition to the changes proposed for the content of the erection drawings, the fabricator must play an active role in the development of the detailing for fabrication. Prior to fabrication the detailing must be reviewed to ensure that the erection of these fabrications can be done in the most efficient and direct manner. With forethought, connections can be detailed which eliminate the need to manipulate or reconnect a piece once it is initially secured in place. Such connections which should be minimized or eliminated are:

1. Those which require columns to be tilted or rocked on their bases.
2. Those which require spreading of parallel girders.
3. Those which require 100% of bolts to support opposing beams or girders on opposite sides of a common girder or column web.

Over the years thousands of steel frames have been erected without incident even though they contained detailing such as those in the list above. These details require extra care and attention on the part of the erector and as such they should be specifically recognized by the erector in the review of the fabrication drawings.

In "Detailing for Steel Construction" (11) published by the AISC, the connections of the sort described above are discussed. For example, mathematical and graphical methods are described to check beam lengths so that when erected they can be rotated either in a horizontal or vertical plane into their final locations without interference from projecting obstructions. Performing these clearance checks and detailing to eliminate obstructions eliminates most of the need to tipping columns to spread girders or columns during erection. Other detailing can also minimize tipping or spreading members. These are:

1. Eliminating column cap plates unless specifically required by the designer.
2. Detailing girders in roofs to set atop the column. It should be noted this can result in the instability of the girder webs unless stiffeners or flange braces or both are used.
3. Using single angles or shear tabs on girder webs or column webs.
4. Detailing wide flange column web connections so that the connections are made beyond the flange tips.
5. Use of tube columns so that the connections are automatically at the face of the columns.

"Detailing for Steel Construction" also addresses the situation where opposing framing angles must share fasteners through a common column or girder web. It states that for beams where the distance from beam bottom to top of girder flange is "up to about 4" difference" the erector may use blocking on the girder flange to support one beam "until drift pins and bolts can be entered through the common holes". Where deeper girders or column webs are the support, "Detailing of Steel Construction" suggests but does not demand that erection seats should be used. Their use should be mandated more specifically than is currently the practice. The location of seats is a function of the sequence of erection. Whether they are removed and reused represents a trade off between the material cost to install them in all locations versus the labor cost to install, remove and reinstall them as the work progresses. Normally erection seats would be left in place "unless they create an interference or detract from the architectural appearance." Because material and labor costs are involved, the provision of temporary erection seats is clearly a matter which must be resolved between the fabricator and the erector.

Other means have been proposed over the years to eliminate the problem of common bolts at webs. These are:

1. The use of permanent seated connections. This would primarily apply to column webs. It should be noted that permanent seats require a top stabilizer angle to complete the detail. The timing of the installation of the stabilizer angle is important for the erector to consider in the preparation of the sequence of erection and detailing.

2. Opposing angles can be detailed so that at least two of the bolts for one or both of the connected beams are not common to both connections. In using such a detail the sequence of erection could determine the actual joint configuration because when the first beam is installed only those bolts which are not common can be installed and these must be adequate to temporarily support the beam. The beam which was intended to be installed second can create a problem if it is installed first and it has not been provided with any bolts which are not common. Lastly, many beams may not be deep enough to accommodate the one or two extra rows of bolts to make the two connections independent.

3. The use of common holes can be eliminated with the use of shear tab connectors or single angle connectors.

4. If double angles are needed, it may be possible to connect one angle to the beam and the other to the column or girder web. While this detailing would be effective, it can be seen as significantly increasing both shop and field work. This may make the other methods worth considering before this method.

The last principal area of interaction between the fabricator and the erector is in the detailing of the column bases. In high rise construction and in other cases of very high loading, column bases are shipped and set separately from the column shafts. In the case of low rise and one story buildings, the base plates are almost universally shipped attached the column shafts. Again, almost universally, there is a layer of grout provided between the bottom of the base plate and the supporting pier or footing. This grout is used to correct for variance between the specified top of pier or footing and the elevation as constructed. It is also used to ensure a uniform contact between the base plate and the supporting pier or footing.

Commonly three methods have been used to account for the timing difference between column setting and base plate grouting. These methods are:

1. The use of leveling nuts and, in some cases, washers on the anchor bolts beneath the base plates.
2. The use of shim stacks between the base plate bottoms and top of concrete supports.
3. The use of 1/4" steel leveling plates which are set to elevation and grouted prior to the setting of columns.

All three of these methods have been successfully employed to erect thousands of columns. All are illustrated and discussed in AISC publication "Detailing for Steel Construction". All have been extensively discussed in the literature because they each have positive and negative attributes. No one method has been proven to universally preclude the use of the other two. There has also been extensive debate as to whether four bolts versus two is the minimum number to be used in a column base and whether or not if four bolts are used for a wide flange column they must be set outside the column profile or if they can be set inside the column profile.

It has been common for erectors to rely on anchor bolts to provide temporary lateral support for columns as can be seen in these citations:

1. "When a column is first 'stood-up' and the hook let go, there is a short period of time when it must stand alone before it is tied with beams or guy cables" - Ricker, AISC Design Guide No. 1 (8).

2. "Anchor bolts also serve to locate and prevent displacement or overturning of columns due to accidental collision during erection" - AISC Detailing for Steel Construction.

3. "9.7 When columns are being set on base plate or shims, and before lifting falls are unhitched, either the nuts on the anchor bolts shall be drawn down tight or temporary guys shall be affixed" - ANSI A10.13.

All of the foregoing clearly establishes that it is common practice for erectors to place reliance on the anchor bolts, piers and footings. This reliance is misplaced. Erectors must bear the responsibility to confirm by analysis the adequacy of the base plate, anchor bolts, shims, leveling nuts, leveling plates, concrete piers and/or footings to resist forces imposed on them during erection and until the permanent bracing is in place. The need for this independent verification is clear when one remembers that the design of the base plates, anchor bolts, and piers and/or footing has been made solely for the loads imposed by the complete structure. For example, in a single story structure with common bay sizes from 30-60 feet and good soil conditions, the spread footing may only be on the order of five feet square and twelve to fifteen inches thick. Such a footing obviously has not been designed for the overturning resistance to support a cantilevered column shaft. In fact it may not even restrain roof uplift and forces if part of the uplift resistance is supported by roofing dead load. As can be seen in this example it is possible for erection induced forces on the foundations to be greater than those of the completed structure. It thus is essential for the erector as part of the temporary support scheme to verify the adequacy of the foundation system for erection induced loads. When the column base, anchor bolts, foundation and temporary support of the base are analyzed, designed and properly detailed, any of the three temporary base support schemes can be used successfully. When this quantitative design is done, it will replace the current qualitative discussion of the merit of the various base support schemes.

The necessity of action on the part of the erector in the preparation of fabrication details can be seen in the foregoing. While such interaction has been and is common in many instances especially where the fabricator and erector are one firm, this interaction must be applied in all cases. One method which would formalize this relation would be for the erector to formally review and approve the fabrication drawings for issues which impact the erector of the structural framework as outlined above.

Design Loads for Temporary Bracing

As is stated in the AISC Code of Standard Practice, temporary supports for structural steel must be designed to "secure the steel framing, or any partly assembled steel framing, against loads comparable in intensity to those for which the structure was designed, resulting from wind, seismic forces and erection operations, but not the loads resulting from the performance of work by or the acts of others, nor such unpredictable loads as those due to tornado, explosion or collision."

A related requirement is given in ANSI A10.13 in paragraph 11.1.1 which states: "Consideration shall be given to the dead weight of the structure, the weight and working reactions of all construction equipment placed thereon, and all external forces that may be applied."

The loads which must be resisted by temporary erection bracing and supports are thus:

1. Dead loads of the structure and loads imposed upon it during erection.
2. Loads from erection apparatus.
3. Impact loads caused by erection equipment and pieces being raised within the structure.
4. Stability loads
5. Wind loads
6. Seismic loads

Gravity loads, loads from erection apparatus and impact loads are as relatively well understood in the industry. This is especially true in high rise construction and bridge construction, where provisions for sequence, temporary supports and apparatus loadings have been well understood for many years. In low rise construction and one story buildings the need for temporary gravity supports are seldom required and erection equipment is rarely anchored to the structure.

Lateral loads represent a much different situation. First, the industry regards temporary X-brace cables as "plumbing-up cables". Secondly, wind and seismic forces are intermittent and vary greatly in intensity during the course of erection. More often than not seismic forces would not be present and wind would likely never reach the intensity used in the final building design. Stability forces may be relatively minor in a light structure which is erected true and square. Lastly, high winds (even within design velocities) are often regarded as natural accidents and not associated to inadequate bracing. Thus, an individual erector's experience may likely not prompt an accurate appreciation for the lateral loads which may be imposed on steel frames. Because the need of lateral load bracing and the magnitude of forces may not be well understood in the field, minimum X-bracing requirements are needed and the design of X-bracing must be done in advance of the commencement of erection.

The nature and magnitude of lateral bracing forces are well understood in the design community. However, there are a few special issues which relate to the design of temporary X-bracing in steel frames. First, stability bracing at a technical level requires a detailed interaction of strength and stiffness. Yet, at a practical level, stability bracing can be relatively easily addressed using an equivalent lateral design force equal to two percent of the gravity load tributary to the bracing. This approach will produce generally conservative results. This approach has the benefit of simplicity and is the approach used in ANSI A10.9 for the stability bracing of concrete formwork. It should be noted however that in some cases, for example when cables are used, stiffness may govern the design. This case should be checked.

The design of temporary bracing for wind loads is a more complex topic. The AISC Code of Standard Practice states that "loads of comparable in intensity" are to be used. If the lateral wind forces of the completed building are used in the erection bracing, the bracing forces may be underestimated. This is due to the fact that the multiple bare frames may present a greater surface to the wind. A more appropriate statement regarding the design for wind is that the bracing shall be designed for the same wind velocity and exposure classification as the completed structure.

The determination of wind forces requires an evaluation to determine the correct drag co-efficient and the correct degree of shielding on multiple parallel members. It also requires the correct evaluation of the effects of wind on open web members.

This topic has been treated in the following documents:

1. Part A4.3.3 of the "Low Rise Building Systems Manual" published by the Metal Building Manufacturers Association.(16)
2. "Wind Forces on Structures", Paper No. 3269, ASCE Transactions, published by the American Society of Civil Engineers.(32)
3. "Standards for Load Assumptions, Acceptance and Inspection of Structures", No. 160, published by the Swiss Association of Engineers and Architects (29).
4. "Design Loads for Buildings", German Industrial Standard (DIN) 1055, published by the German Institute for Standards(10).

The use of the principles contained in the documents requires a degree of engineering judgment in their application to temporary bracing, however a few general observations can be made. First, the drag co-efficient for wide flange members and plate girders is on the order of two. Second, the drag co-efficient for open web members is a function of their solidity ratios, and can range from 1.6 to 2.0 depending on the solidity ratio. Lastly, the effect of shielding in ranks of parallel members must be accounted for. In multiple parallel girders shielding can be minimal. For multiple parallel joists the effect can be in the range of one half the projected area of all the joists. Additionally, there can be significant effects when quartering winds are considered.

Earlier in this paper, the authors recommended the minimum provision of X-bracing as being required in each frame line and in one bay for every four bays. Given common bay sizes of 30 feet to 60 feet and common framing, it is possible to generalize about the relative magnitude of wind forces on frames in terms of the wind pressure on the projected area of the exterior frames, accounting for drag and shielding. Based on the foregoing the design force for wind pressure in each frame line would be the basic wind pressure based on wind speed and exposure times the projected area of the frame times a factor between six and eight. This range of values represents the authors' synthesis of factors accounting for openness, shielding, and common bay dimensions. This would apply to both solid and open members, using depth times span as the projected area of the open web members. It should be obvious that a more detailed analysis is in order but this computation would establish the order of magnitude of the required design force.

The determination of the equivalent lateral load for resistance to seismic forces is well documented in building codes. In general it involves the determination of a percentage of the gravity weight of the frame which is to be applied as a static force.

One very useful source of wind and seismic loading data is ASCE/ANSI A58.1 "Minimum Design Loads for Buildings and Other Structures" (3). This document is the standard for loads referenced in ANSI A10.9, which also establishes a minimum design pressure of 15 psf for wind loading. This seems like a reasonable provision for the design of temporary erection bracing as well.

It is necessary to consider the effects of load combinations in designing bracing. The combinations for stability, wind, and seismic loads are:

a. Stability loading
b. 0.75 (stability loading plus wind loading)
c. 0.75 (stability loading plus seismic loading).

With regard to allowable stresses and load resistance, the values determined using the AISC Specification would be appropriate without further modification. The codes and standards published by the American Concrete Institute would be used to evaluate the affected concrete sections.

Recommendations

Based on the authors' review of the state of the art with regard to erection bracing as presented in this paper, the following changes in practice are recommended:

1. Cross reference should be made among the AISC Code of Standard Practice, ANSI A10.13 and OSHA.
2. Adoption of the self-supporting/non-self-support distinction should be made by ANSI and OSHA.
3. Change should be made in nomenclature from plumbing-up cables to temporary bracing cables.
4. Minimum bracing requirements should be established.
5. Requirements for positive anchorage points for the attachment of bracing should be established.
6. Review and approval of fabrication drawings by the erector should be established prior to fabrication for erection issues such as special sequences and temporary supports required, lugs and anchor devices, spreading of girders and tilting of columns.
7. Preparation of an erection bracing scheme containing the information outlined in the text above, including the forces from non-structural components which load the bracing in non-self-supporting frames should be required for all structures.
8. Evaluation by the erector of the foundations and anchor bolts for erection forces should be required.

As can be seen from the foregoing the state of the art in erection bracing demands attention and improvement. Many erectors are erecting structures with proper bracing. These recommendations will not affect these erectors. What these recommendations represent is a clarification of minimum requirements to help erectors who are unaware of the importance of these issues and provide a uniform minimum standard to the industry. The authors hope this paper will help the industry in advancing the state of the art of erection stability bracing.

References

1. "American National Standard for Construction and Demolition Operations - Concrete and Masonry Work - Safety Requirements", ANSI A10.9-1983, American National Standards Institute, New York, New York
2. "American National Standard for Construction and Demolition Operations - Steel Erection - Safety Requirements", ANSI A10.13-1989, American National Standards Institute, New York, New York
3. "American Society of Civil Engineers - Minimum Design Loads for Buildings and Other Structures", ASCE 7/ANSI A58.1-1988, American Society of Civil Engineers, New York, New York
4. "Building Code Requirements for Reinforced Concrete", ACI 318, 1989, American Concrete Institute, Detroit, Michigan.

5. "Code of Federal Regulations", Title 29, Subpart R, "Steel Erection", 1992, U.S. Government Printing Office, Washington, D.C.
6. "Code of Federal Regulations", Title 29, Subpart Q, "Concrete and Masonry Construction", 1992, U.S. Government Printing Office, Washington, D.C.
7. "Code of Standard Practice for Steel Buildings and Bridges", 1992, American Institute of Steel Construction, Chicago, Illinois
8. "Column Base Plates", AISC, Design Guide No. 1, 1990, American Institute of Steel Construction, Chicago, Illinois
9. "Commentary and Recommendations for Handling, Installing and Bracing Metal Plate Connected Wood Trusses", HIB-91, 1991, Truss Plate Institute, Madison, Wisconsin
10. "Design Loads for Buildings" German Industrial Standard 1055, 1986, German Institute for Standards, Berlin, Germany
11. "Detailing for Steel Construction", 1983, American Institute of Steel Construction, Chicago, Illinois
12. "Engineering for Steel Construction", 1984, American Institute of Steel Construction, Chicago, Illinois
13. "Formwork for Concrete", ACI No. 4, 5th Edition, 1989, American Concrete Institute, Detroit, Michigan
14. "General Conditions of the Contract for Construction", A201, 1987, American Institute of Architects, Washington, D.C.
15. "Handling and Erection of Steel Joists and Joist Girders", Technical Digest #9, Steel Joist Institute, Myrtle Beach, South Carolina
16. "Low Rise Building Systems Manual", 1986, Metal Building Manufacturers Association, Cleveland, Ohio
17. "PCI Design Handbook", 4th Edition, 1992, Prestressed Concrete Institute, Chicago, Illinois
18. "Quality in the Constructed Project: A Guide for Owners, Designers and Constructors", 1990, American Society of Civil Engineers, New York, New York
19. "Recommended Code of Standard Practice for Steel Joists and Joist Girders", 1987, Steel Joist Institute, Myrtle Beach, South Carolina
20. "Recommended Design Specification for Temporary Bracing of Metal Plate Connected Wood Trusses", DSB-89, 1989, Truss Plate Institute, Madison, Wisconsin
21. "Recommended Practice for Erection of Precast Concrete", 1985, Prestressed Concrete Institute, Chicago, Illinois
22. "Recommended Practice for Formwork", ACI 347, 1978, American Concrete Institute, Detroit, Michigan
23. "Specifications for Structural Concrete for Buildings", ACI 301, 1984, American Concrete Institute, Detroit, Michigan
24. "Specification for Structural Steel Buildings: Allowable Stress Design and Plastic Design", 1989, American Institute of Steel Construction, Chicago, Illinois
25. "Standard General Conditions of the Construction Contract", 1990, Engineers Joint Contract Documents Committee, American Society of Civil Engineers, et al
26. "Standard Specification for Joist Girders", 1990, Steel Joist Institute, Myrtle Beach, South Carolina
27. "Standard Specifications for Longspan Steel Joists, LH-Series and Deep Longspan Joists, DLH-Series", 1990, Steel Joist Institute, Myrtle Beach, South Carolina
28. "Standard Specifications for Open Web Steel Joists, K-Series", 1990, Steel Joist Institute, Myrtle Beach, South Carolina
29. "Standards for Load Assumptions, Acceptance and Inspections of Structures", 1956, No. 160, Swiss Association of Engineers and Architects, Zurich, Switzerland
30. "Tilt-up Concrete Structures", ACI 551, 1992, American Concrete Institute, Detroit, Michigan
31. "Wind Forces on Buildings and Other Structures", Loss Prevention Data: 1-7, 1983, Factory Mutual Engineering Corp., Norwood, Massachusetts
32. "Wind Forces on Structures", ASCE Transactions, Paper No. 3269, American Society of Civil Engineers, New York, New York

COLUMN BUCKLING CONSIDERATIONS IN HIGHRISE BUILDINGS WITH MEGA-BRACING

Ahmad K. Abdelrazaq,[1] William F. Baker, Jr.,[2] Jerome F. Hajjar[3] and Robert C. Sinn[4]

Abstract:

This paper addresses the practical concerns of designing a multi-story building whose primary lateral load resisting system is derived from mega-braced frames over multiple stories. Two types of mega-braced systems are recognized. "Discrete" systems involve independent global and local stability subsystems while "integrated" forms utilize intermediate connections between horizontal and diagonal members to ensure column stability. For either system, the local stability of the entire structure between the intersections of the diagonals and columns must be considered. One solution involves introducing a secondary stability frame (vertical truss or moment resisting frame), with sufficient strength and stiffness to stabilize the mega-brace panel as well as to transfer lateral shears to the panel points of the primary system. The results of an analytical study includes the effect of material non-linearity on the stiffness and critical mode shape of the localized stability frame/brace.

1 Senior Structural Engineer, Skidmore, Owings & Merrill, 224 South Michigan Avenue, Chicago, Illinois 60604

2 Associate Partner and Senior Structural Engineer, Skidmore, Owings & Merrill, 224 South Michigan Avenue, Chicago, Illinois 60604

3 Assistant Professor of Civil Engineering, University of Minnesota, 500 Pillsbury Drive S.E., Department of Civil Engineering, Minneapolis, Minnesota 55455

4 Associate and Senior Structural Project Engineer, Skidmore, Owings & Merrill, 224 South Michigan Avenue, Chicago, Illinois 60604

Introduction:

Highrise buildings with mega-bracing have unique stability concerns. A mega-braced lateral load resisting system is composed of a vertical truss with panel points located several floors apart. The system resists lateral loads on a global level and provides overall column stability at the panel points of the truss. Unlike buildings where the lateral load stability system is the same in each story, such as buildings with rigid frames or diagonal bracing in each individual story, mega-bracing separates the lateral stability of the full tower from the lateral stability of individual columns. The problem is one of overall story stability with all columns buckling simultaneously in a multi-story mode between the mega-braced points. This paper will address the stability of the columns between the panel points.

Highrise Structural Systems with Mega-bracing:

Primary Systems

Two classes of highrise mega-braced structural systems will be discussed. The first system is characterized as a "discrete" bracing system in which the mega-bracing is connected to the rest of the structure only at the panel points. This type of system can take the form of an exoskeleton, where the mega-bracing is disengaged from the building mass. An example of such a structure is the Hotel de las Artes Tower in Barcelona, Spain (Figure 1).

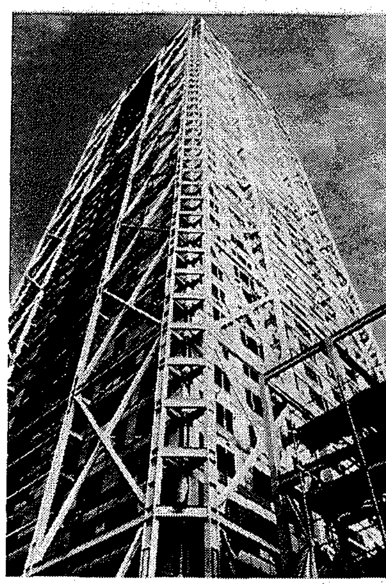


Fig. 1 Hotel de las Artes, Barcelona, Spain

The mega-bracing of the exoskeleton completely resists the lateral loads and provides stability to interior gravity columns at the floors that are connected to the panel points. A second class of structure is termed an "integrated" bracing system in which the diagonal bracing intersects intermediate columns between the panel points of the primary mega-braced system with the intersection of the column and diagonal always occurring at a floor level.

An example of an integrated bracing system is that of the John Hancock Building in Chicago (Figure 2). It is the benefit derived from these intersections with the intermediate columns that distinguishes an integrated bracing system from a displaced system.

Secondary Systems

It is often necessary to have a secondary stability system between the bracing points for both discrete and integrated mega-braced systems. The secondary system has the dual function of transferring local lateral loads applied at levels other than those of the primary system mega-module and providing column stability at the intermediate levels.

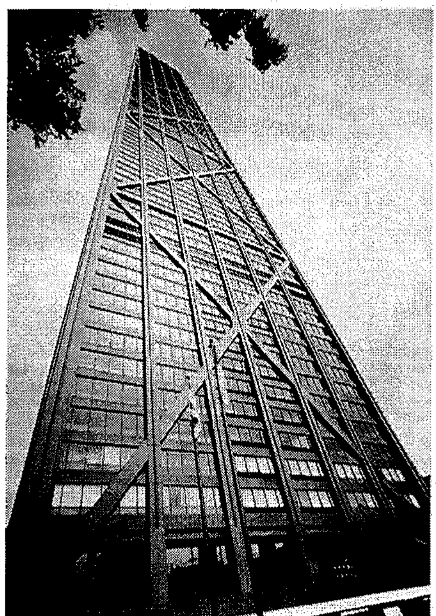


Fig. 2 John Hancock Center, Chicago, Illinois

It is the need for a secondary stability system that distinguishes the mega-braced systems from other highrise lateral load resisting systems. In a building with a rigid frame composed of girders at every floor or a braced frame with diagonal bracing in each story, the wind shears increase from the top of the building downward in a manner that is roughly proportional to the lateral stiffness that is required for stabilizing the columns in each story. Therefore, a bracing system that is designed to resist wind loads with reasonable limits placed on building movements will also provide adequate stability for the columns in each story (the accuracy of this statement depends on the particulars of each structure such as the magnitude of the lateral load, the design drift limits, the density of the building, etc.). In a mega-braced system, the wind loads are transferred from the secondary system to the primary mega-bracing system at regular intervals but the gravity loads continue to accumulate in the local system and are generally not transferred to the mega-bracing system. The simultaneous requirements of resisting lateral loads and stabilizing individual columns becomes somewhat separated in mega-braced systems (Figure 3).

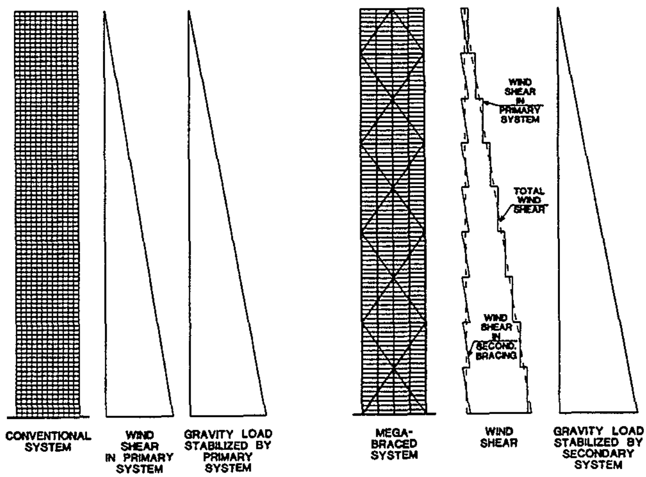


Fig. 3 Separation of Lateral Load and Stability Systems

The secondary system can take several forms. In an integrated mega-bracing system, all columns can be simply designed for longer unbraced lengths, and any reserve capacity in the mega-bracing columns and diagonals can be used to partially brace other columns (the sizing of the mega-bracing system columns and diagonals are often controlled by deflection limitations and may have reserve capacity at design loads). The columns and diagonals of the primary mega-bracing system then participate as flexural members in the secondary transfer of shears between panel points.

In a discrete mega-bracing system, intermediate lateral wind and stability shears must be transferred to the primary system by an auxiliary system, which could consist of a vertical truss that is designed to span between mega-bracing panel points. This system is discussed by Thornton, et al. (1988) and could take the forms shown in Figure 4. The secondary vertical trusses may be accommodated along the walls surrounding building core elements such as elevators, stairs, etc. Vertical trusses are also valid alternatives for the secondary system of integrated mega-bracing systems.

Another possibility for the secondary system is to create moment resisting frames that span between bracing points. Although this is an unbraced frame, it differs

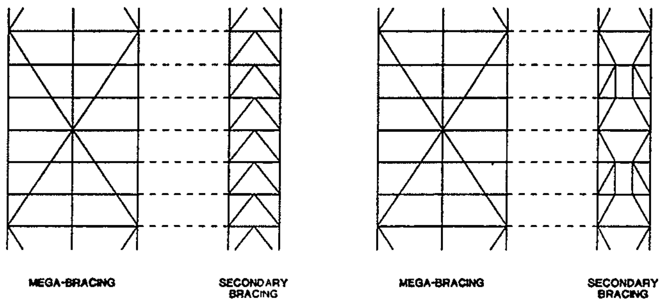


Fig. 4 Secondary Bracing System Forms (Source: Thornton, et. al. 1988)

from common unbraced frames in that both the bottom and the top level are restrained at the mega-module points and only the intermediate levels can sidesway (Figure 5).

The procedure for determining the required level of bracing for a single continuous column over multiple spans has been established (Yura, 1992) and may be extended to the case of a complete building system (Figure 6).The preliminary sizing of a complete mega-braced system might begin by the design of all columns to preclude individual single story weak axis buckling. Next, the secondary frame or truss systems in orthogonal directions should be sized for a multi-story buckling mode such that the column sizes remain unaffected. For this, the presence of strong axis columns in the orthogonal directions is utilized.

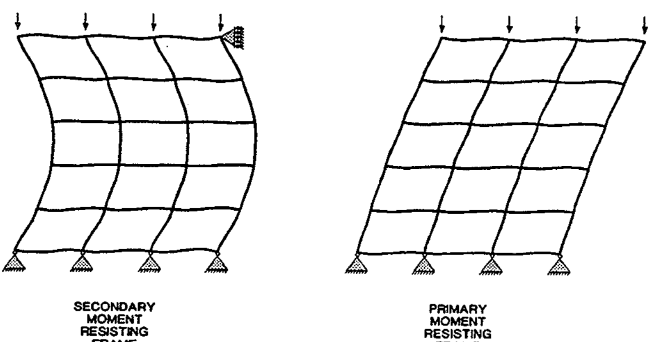


Fig. 5 Moment Resisting Frame Buckling Modes

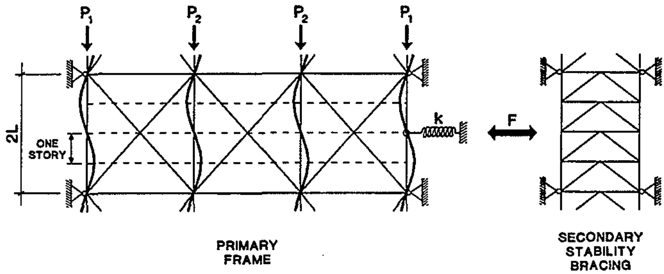


Fig. 6 Secondary Bracing System Design Concept

System Studies:

The behavior of discrete and integrated mega-bracing systems is demonstrated with the following system studies. The studies are based on eigenvalue buckling analyses where the member properties are corrected for inelastic effects.

Two types of corrections for inelastic effects were included: flexural stiffness and axial stiffness. The flexural stiffness of each column of the structure was reduced depending on the magnitude of axial stress present according to the following formulae (Baker, 1991):

$$\sigma = \frac{P_u}{\phi A_g} \tag{1}$$

$$EI = 0.877 EI_o \; ; \quad \frac{\sigma}{\sigma_y} \leq 0.39 \tag{2}$$

$$EI = [2.389 \frac{\sigma}{\sigma_y} \ln \frac{\sigma_y}{\sigma}] \, EI_o ; \quad 0.39 < \frac{\sigma}{\sigma_y} \leq 1.0 \tag{3}$$

$$EI = 0 ; \quad \frac{\sigma}{\sigma_y} > 1.0 \tag{4}$$

Using these corrections in the Euler buckling equation will give strengths that match column strength levels in the AISC LRFD specifications, see Figure 7. The above formulae can be viewed as correcting for both residual stress as well as initial out-of-straightness effects.

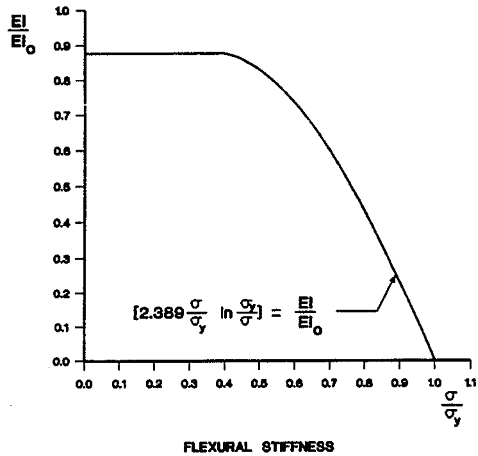


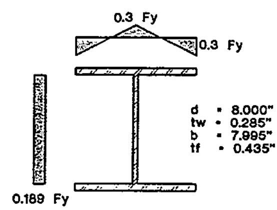
Fig. 7 Flexural Stiffness Correction

The decrease in axial stiffness of an element was based on the idealized residual stress pattern shown in Figure 8. This is the same residual stress pattern that was used in the work by Kanchanalai (1977) that lead to the AISC LRFD column equations. The following formulae were used for reducing column member axial stiffnesses (Figure 9).

$$EA = EA_o; \quad \frac{\sigma}{\sigma_y} \leq 0.70 \tag{5}$$

$$EA = [2.108 - 1.583\,\frac{\sigma}{\sigma_y}]\,EA_o; \quad 0.70 \leq \frac{\sigma}{\sigma_y} \leq 1.189 \tag{6}$$

$$EA = 0 \quad ; \quad \frac{\sigma}{\sigma_y} > 1.189 \tag{7}$$



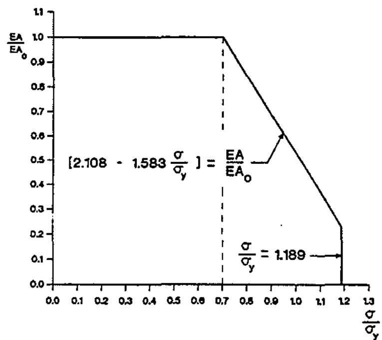


Fig. 8 Idealized Residual Stress Pattern

Fig. 9 Axial Stiffness Correction

It should be noted that although Equation (6) allows stresses in excess of yield, an individual beam element would buckle at less than yield due to Equation (3).

The system study solutions were conducted using the following iterative method:

1. An eigenvalue analysis was made of the structure using factored loads divided by ϕ and full elastic properties (elastic buckling analysis).

2. The member axial stresses were calculated and the member properties adjusted for inelastic effects. The buckling analysis was then repeated and a new eigenvalue calculated.

3. The loads were then increased, the new axial stresses calculated, and the member properties adjusted for inelastic effects. The buckling analysis was then repeated and a new eigenvalue calculated.

4. Step 3 is repeated until an eigenvalue near 1.0 is achieved indicating that the ultimate inelastic stability of the structure had been reached.

Example A: Hotel de las Artes Tower, Barcelona, Spain.

An example of a mega-braced building recently completed in Barcelona, Spain is shown in Figure 1. The exoskeleton form of the tower was created by placing the exterior braced frame 1.5 meters away from the plane of the window wall. The primary exterior lateral load-resisting system is organized on a 4-story (12m) bracing module. The isolated L-shaped braced frames in the four corners of the tower are linked together at three vertical locations, thus creating an equivalent cantilever system in the form of mega-portal frames.

All exterior columns are connected to the floor diaphragm at each level by floor beams extending through the curtain wall from the interior space and by horizontal diagonal bars located in the interstitial space between the cladding and the exterior frame. Preliminary designs indicated that a four-story column unbraced length design would result in a steel weight premium of some 900 tons (nearly 4 psf applied over the framed floor area) as compared to a one or two-story design. Next, the possibility of connecting the vertex of the exterior diagonals to the floor diaphragm, thus creating no more than a two-story buckling mode was explored. The strength required for these connecting members would have resulted in exposed elements contrary to the desired aesthetic of slender bar bracing. Consequently, interior secondary frames designed to span between the panel points of the primary system were discussed. While interior moment resisting frames in one direction were possible, the resulting interior beams depths would have produced clear height conflicts in the hotel rooms. Light, secondary, K and knee-braced frames were placed in the core area in orthogonal directions with the amount of steel expended being only 0.45 psf applied over the area of the building (Figure 10).

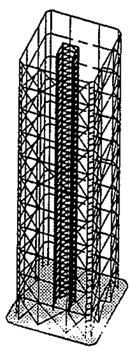


Fig. 10 Hotel de las Artes Vertical System

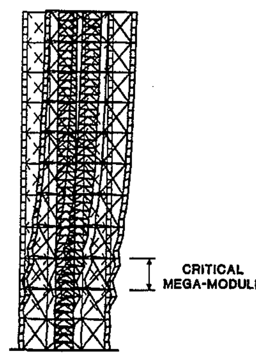


Fig. 11 Elastic Buckling Analysis

An elastic buckling analysis of the entire building frame (Figure 11) was used to determine the critical 4-story exterior bracing module. This single module was then analyzed both with and without the interior trusses at factored loads. The module was pinned at the bottom and free Vertical Systems at the top. This is a somewhat conservative simplification as seen by comparing the elastic eigenvalue for the full building analysis ($\lambda = 6.68$) versus that of the single module ($\lambda = 5.79$). It can be seen that the eigenvalue with the interior bracing present is nearly twice that without, testifying to the increased stability of the overall system upon introduction of the secondary trusses (Figure 12).

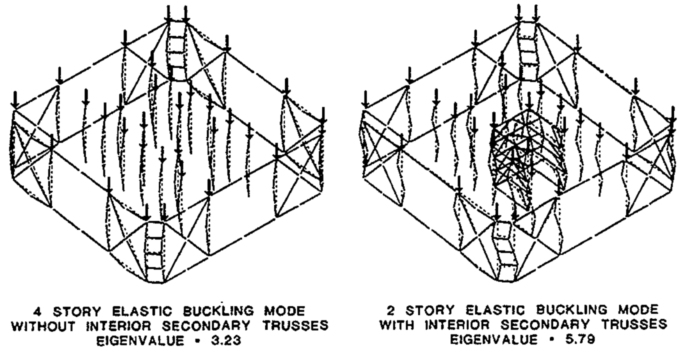


Fig. 12 Elastic Buckling Study With/Without Secondary Trusses

The column loads were then gradually increased beyond the design factored load with the member properties adjusted at each iteration based on inelastic effects. Figure 13 charts the decrease in eigenvalue versus increased column load. An equivalent moment resisting frame secondary system was analyzed as well. The results are indicated in Figure 14.

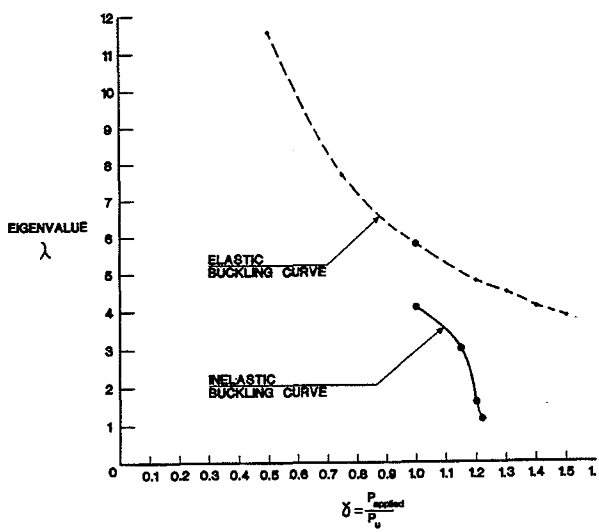


Fig. 13 Buckling Analysis Results

It should be noted that the buckling mode for this system is quite different than that for an unbraced frame free to sway at the top. The two-story mode shown demonstrates the equivalency and efficacy of the secondary moment resisting frame as compared to a secondary vertical truss.

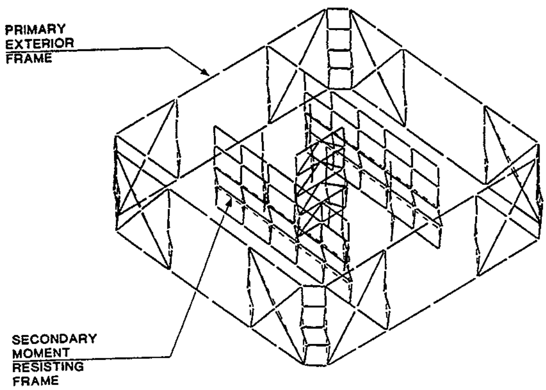


Fig. 14 Secondary Moment Resisting Frame Buckling Mode

Example B: Integrated Mega-Braced Tower

A hypothetical 90-story X-braced tower is shown in Figure 15 with diagonal/ column intersections occurring every 5 floors. All columns are organized in a symmetrical pattern with approximately one-half of the columns oriented along their strong axis in each direction. The column and diagonal members were sized for typical gravity loads assuming an unbraced length for individual column buckling of one story.

The behavior of the building under wind loading is indicated in Figure 16. Without a discrete secondary system, the integrated system includes local deformations between the panel points of the primary mega-bracing. Depending on the sizes of the primary members and the story heights, these local displacements may require the addition of a secondary load transfer system to prevent serviceability problems in architectural elements such as claddings, floor finishes and partitions. The overall global behavior of the tower under wind loading is nearly the same with and without the secondary interior vertical trusses.

An elastic eigenvalue buckling analysis of the entire building frame was performed with the fundamental mode shown in Figure 17. The elastic eigenvalue under factored loads is 1.12.

The five-story mode demonstrates that the diagonal/column intersection is in fact a point of bracing for a system proportioned with all perimeter columns of approximately equal size. A survey of the capacity of all columns buckling simultaneously over a five-story mode was found to be somewhat less than the applied factored load, confirming the low eigenvalue. In general, for buildings of this type, if column/diagonal intersections are spaced less than 3 or 4 stories apart, secondary trusses may not be necessary based on stability considerations alone. Secondary interior vertical trusses were introduced to the model, as shown in Figure 18. The elastic eigenvalue under factored loads increased to 2.10, while the mode shape is still over five-stories. The mode shape is one of overall symmetric torsional deformation. Inelastic effects were incorporated into the models with and without the interior core trusses. Figure 19 charts the lowered eigenvalues when correcting for these inelastic effects.

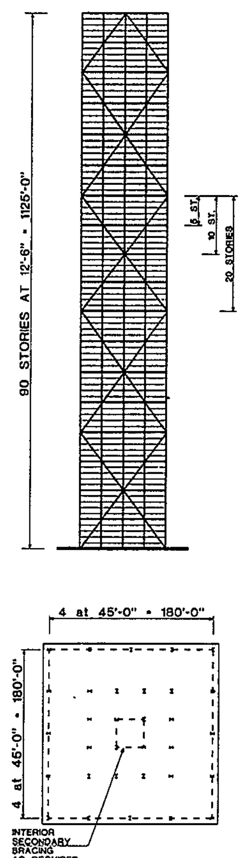


Fig. 15 Integrated Example

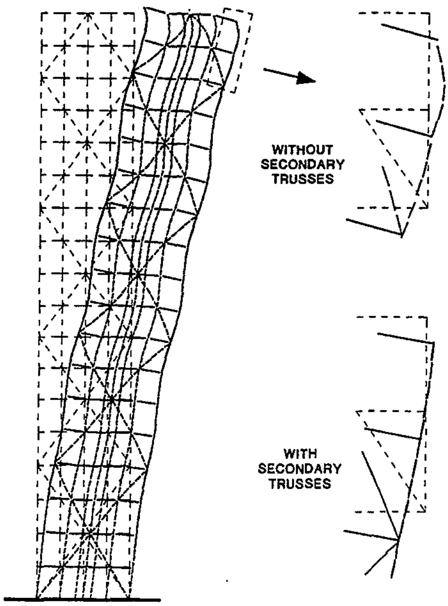


Fig. 16 Behavior Under Wind Load

Conclusion:

The special stability characteristics of tall buildings with mega-bracing have been discussed. Some guidance for the analysis and sizing of such systems is presented along with the potential system forms. A series of eigenvalue buckling analyses performed on both an actual building example and a hypothetical tower indicate that significant inherent stability exists in the system over 2 and 3 stories without requiring independent secondary stability framing. A method for modifying an elastic buckling model to include the effect of material non-linearity is included allowing for the accurate analysis of complex building forms.

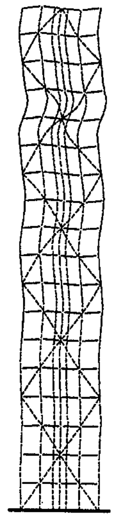


Fig. 17 Without Secondary Trusses

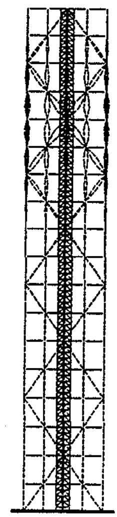


Fig. 18 With Secondary Trusses

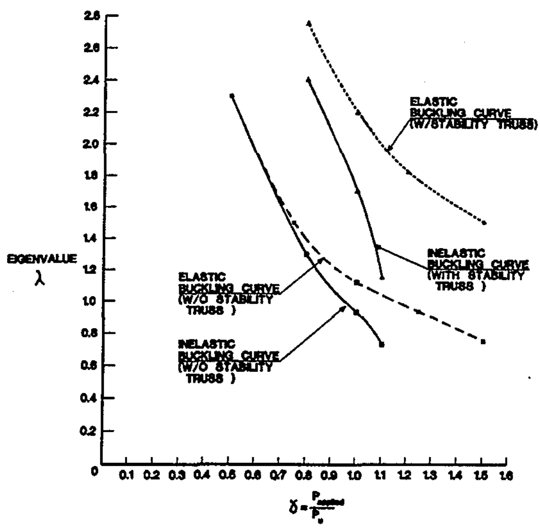


Fig. 19 Integrated Example Buckling Results

Appendix - References:

American Institute of Steel Construction, (1986). "Load and Resistance Factor Design Specification for Structural Steel Buildings," AISC, Chicago, Illinois.

Baker, W. F., (1991). "Practical Problems in Inelastic Steel Design." Proceedings, ASCE Structures Congress 1991.

Kanchanalai, T., (1977) "The Design and Behavior of Beam-Columns in Unbraced Steel Frames." Department of Civil Engineering, University of Texas at Austin, Austin, Texas.

Thornton, C. H., Scarangello, T. Z., and DeScenza, R. P. (1988) "Localized Stability Problems in Large Structural Systems." Proceedings, Structural Stability Research Council Annual Technical Session, Minneapolis, Minnesota.

Yura, J.A., (1992). "Summary of Bracing Recommendations." 1992 AISC National Steel Construction Conference.

BROADWAY CENTRE

Steven G. Judd, P.E.
MARTIN/MARTIN - UTAH, INC.

GENERAL BUILDING OVERVIEW

The Broadway Centre is situated on the corner of State Street and Broadway in the heart of the Central Business District of downtown Salt Lake City, Utah. The project includes a 192,000 sq. ft., 13-story office tower; a 26,000 sq. ft., 6-auditorium theater; and a 7-story, 450-car garage.

OWNER:	BROADWAY CENTRE LIMITED
ARCHITECT:	EDA ARCHITECTS
STRUCTURAL ENGINEER:	MARTIN/MARTIN - UTAH, INC.
FINANCING:	IBEW
LEASING:	COMMERCE PROPERTIES, INC.

This paper will focus on the tower/theater structure.

The office tower has a floor plate of about 16,000 sq. ft. with nine outside corners and a curved front. (See Fig. 1.) The mechanical penthouse and elevator equipment room sit on the roof behind a curved curtain wall facade. The foundation system consists of a mat foundation and spread footings.

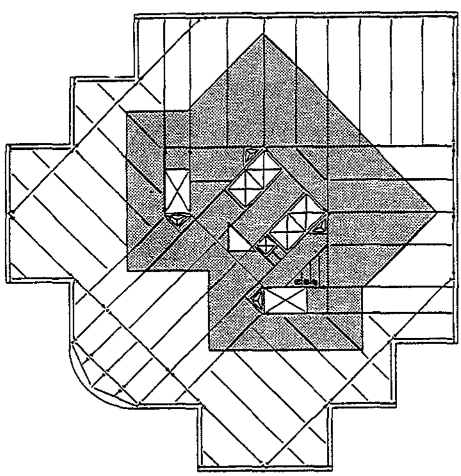


Fig. 1 TYPICAL OFFICE FLOOR PLATE
(Shaded area represents additional reinforcing at Levels 2 and 3)

Two of the six theater auditoriums sit within the footprint of the office tower. The other four auditoriums are contained in a 2-story high space that wraps two sides of the office tower. Three-and-one-half of the auditoriums, plus the lobby, are suspended over the service level/basement, while the balance of the auditoriums are slabs on grade. (See Fig. 2.)

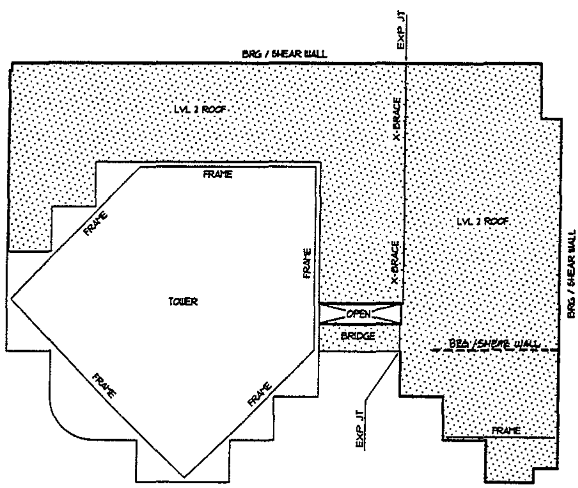


Fig. 2 FRAMING AT LEVEL 2 - 2ND FLOOR OFFICE TOWER AND CINEMA ROOF

All floor framing and the tower roof are light weight concrete slabs on a 2" composite metal deck. The deck spans roughly 10'-0 to composite steel beams and girders with shear studs. The cinema roof is 1½" type B deck on open web steel joists.

The 2-story high cinema roof is tied to the 2nd floor of the tower where they interface for lateral stability. Masonry shear walls and X-braced frames at the opposing perimeters of the building complete the lateral load-resisting system for the cinema roof.

The office tower is laid out in a 20'-0 X 20'-0 module with a perimeter frame which cuts diagonally across the curved front of the building. At the top of the tower, the corner offices drop off starting at the 11th floor, thereby truncating the top of the perimeter frame at corresponding levels. At the base of the tower, the perimeter frame terminates on top of the perimeter foundation walls along the west, southwest, and south sides. The balance of the perimeter frame continues down through the plaza framing and terminates on the mat foundation.

The mat foundation which parallels the outline of the tower floor plate is 4'-0 thick. It was crowned 3/4" at its center to offset the differential settlement expected between the middle of the mat and the edges. Settlement was monitored; and, after 75% of the structure dead load was on the tower, the perimeter foundation walls which bridged between the mat and other isolated spot footings were poured. The mat foundation was poured in 14 hours (one day) and contained 3,800 cubic feet of concrete.

Due to specific anomalies (to be discussed later), five Eccentric Braced Frames (EBF's) had to be introduced into the building from the 3rd floor down to the mat. The balance of this paper will focus on the lower portion of the building and issues related to the EBF's.

The project was designed under the jurisdiction of the 1988 UBC. Salt Lake City is in seismic Zone 3. Design Basic Wind Speed is 70 mph. Design Base Shear = $ZICW/R_w$ or $(ZIC/R_w)W$ where R_w is the lateral system coefficient. The higher the coefficient, the more ductile the system; the more ductile the system, the lower the force demand on the structure. R_w is similar in concept to 1/K in the BOCA and SBC codes, where K is the "stiffness" coefficient.

GEOPHYSICAL SETTING:

In order to understand the reasons for the use of the EBF, one must first understand the problem. The following criteria ultimately lead to the decision to incorporate EBF's into this project:

Geophysical setting
Tenant leasing requirements
Architectural response to these two phenomena

The project occupies a lot on the northeast corner of an intersection. The existing sidewalk and the adjacent street slope down from the farthest northwest portion of the site (reference elevation 104'-4) almost 6'-0 to the southeast end of the project next to the garage (98'-5½). The complex has tenant spaces which front onto, and access the sidewalk, so the interior plaza floor level had to vary to roughly follow the sloping sidewalk around the perimeter of the building. This created four of the distinct plaza level framing elevations.

The theater auditoriums slope from the theater lobby (103'-0) to the exit corridors (98'-6) creating inclined subdiaphragms and corridors which interface with other plaza level framing elevations. The theater requirements created five additional distinct plaza level framing elevations. (See Fig. 3)

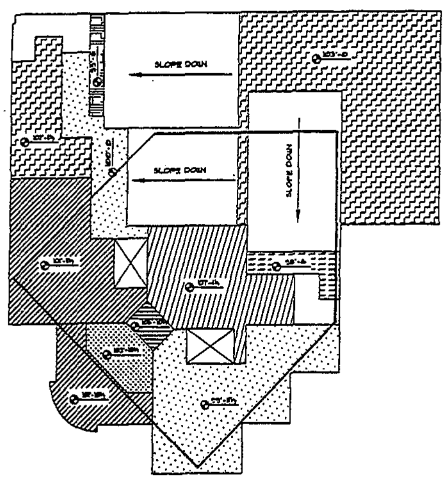


Fig. 3 PLAZA LEVEL FRAMING ELEVATIONS

In order to make aesthetic sense of the high first floor and relatively narrow elevator lobby, the building entrance lobby area stepped up via a "grand stairway." This stairway transitioned from street level (101'-10½) to the elevator lobby (107'-4½) with two intermediate landings. This added three more distinct framing levels.

As it turned out, the plaza level has at least twelve distinct framing elevations, plus three sections of sloping floors which slope down anywhere from 3'-0 to 4'-6. The plaza is anything but a flat, continuous diaphragm. The high ceilings also created a very tall first floor.

The lack of a continuous diaphragm at the plaza level made it extremely difficult to transition lateral forces from the perimeter frames to the foundation system. The 2-story high first floor created the potential of a "soft" or "weak" story at the base of the structure. The solution to these problems had to stiffen the base of the structure without unduly increasing the design forces (explained later), and had to provide a more direct path for the lateral forces to be resolved through the plaza level down to the foundation level. The solution to both problems was a relatively new structural system - the EBF.

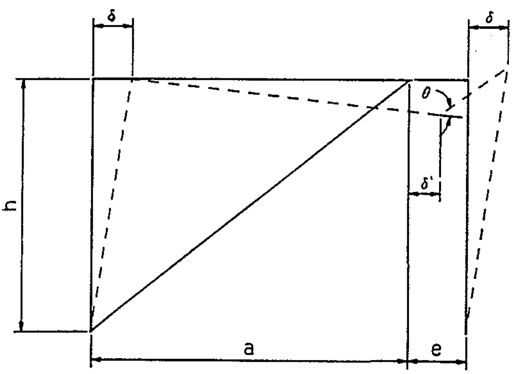
PRIMER ON EBF

An EBF is a diagonally braced frame which has one or both ends of the diagonal element intentionally offset from the traditional beam/column joint, along the axis of the beam. Each braced bay may have one or two diagonal elements.

The concept of the eccentric brace is an attempt to combine the best features of a concentric braced frame (CBF), which has good deflection control and strength but poor inelastic behavior, and a special moment resisting frame (SMRF), which has very good energy absorption/dissipation characteristics but is generally drift sensitive at it's lower floors.

EBF's were first proposed and rigorously tested in the 1970's by Egor Popov and Charles Roeder working at U.C. Berkeley. In development of the behavior and design procedures for the EBF, data originally investigated in 1949 by others on the phenomena of shear yielding was used.

EBF's effectively combine the best qualities of CBF's and SMRF's by introducing an energy absorbing "link" in the beam element between the end of the diagonal brace element and the beam column joint. (See Fig. 4)



$$\theta = \frac{\delta}{h}\left(1 + \frac{a}{e}\right)$$

Fig. 4 TYPICAL ECCENTRIC BRACED FRAME

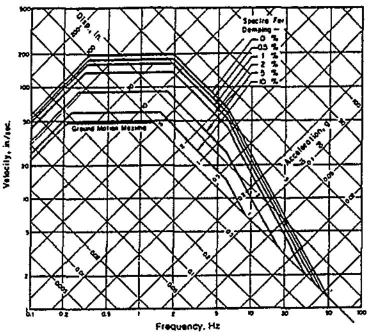
Introduction of the intentional eccentricity into the beam element is done to precipitate shear or flexure yielding in the short length of the beam element adjacent to the eccentric brace connection. The short length of beam is referred to as a "link" or "fuse" because it is used to limit the force in the bracing element and therefore reduces the potential of buckling of the brace which can lead to catastrophic failure. When properly designed, the link undergoes large deformations without fracture. Thus the system maintains stability even under large inelastic distortions.

The link, if it performs properly (i.e. designed properly), does two things physically:

1) Absorbs energy when it displaces which increases the damping effects of the bracing system.

2) Increases the fundamental period of the building by increasing the displacement.

Each of these two phenomena help reduce the demand of the building as shown on the normalized Basic Newmark-Hall design spectrum and associated amplification factors. From the design spectrum, it can be shown that as the frequency of the building decreases (period increase), the observed building acceleration also decreases. Likewise, from the table of amplification factors, as damping increases, acceleration decreases. Any decrease in observed building acceleration equates to lower design forces, hence lower demand on the building. (See Fig. 5 and 6)



Percent of critical damping	Amplification factor for Displacement	Velocity	Acceleration
0	2.5	4.0	6.4
0.5	2.2	3.6	5.8
1	2.0	3.2	5.2
2	1.8	2.8	4.3
5	1.4	1.9	2.6
7	1.2	1.5	1.9
10	1.1	1.3	1.5
20	1.0	1.1	1.2

Fig. 5 BASIC NEWMARK-HALL SPECTRUM

Fig. 6 TABLE OF AMPLIFICATION FACTORS

During moderate or low lateral forces, the EBF performs much like a CBF having very small displacements. As the load increases, and exceeds the design forces, the link yields, developing the energy dissipation fuse helping to reduce the demand on the structure.

Preliminary sizing of EBF's can be done the same way as for CBF's for axial loads in columns due to overturning moment and brace forces due to story shears, with members being sized for strength and serviceability. The beam element is initially sized for the axial load due to story shears combined with the shear and bending due to the eccentric brace connection point.

At this point the design departs from the typical CBF approach. Essentially, EBF's are designed for strength with a check for code drift limitations and inelastic beam rotation. First the basic frame geometry needs to be determined.

There are few established rules to determine the length of links. If the links are small, yielding of the beam (strong/column weak beam concept for stability) is in the form of a plastic shear hinge which tends to create a very rigid frame with deflections approaching those of CBF's. Long links produce more typical plastic moment hinges at the ends of the link, which add substantial deflection to the frame, making a relatively flexible structure in the inelastic range, more similar to a SMRF. Either way, it is important to make sure the link meets the ductility requirements of the model building code. Generally if the shear strength of the link is less than 2.2 times the Moment Strength ($V_S \leq 2.2xM_S$), then the link beam strength is assumed to be governed by shear yielding. When the link beam is governed by shear yielding, the axial and flexural properties for use in the interaction formulas used to design the link are based on the link beam flanges only. The contribution of the web in the plastic moment capacity and axial load capacity is neglected because it is assumed that the web has already yielded. Shear yielding is preferred as the limiting design mode because the majority of test data which supports the theory behind EBF's and predicts their behavior has been based on shear yielding of the link beam.

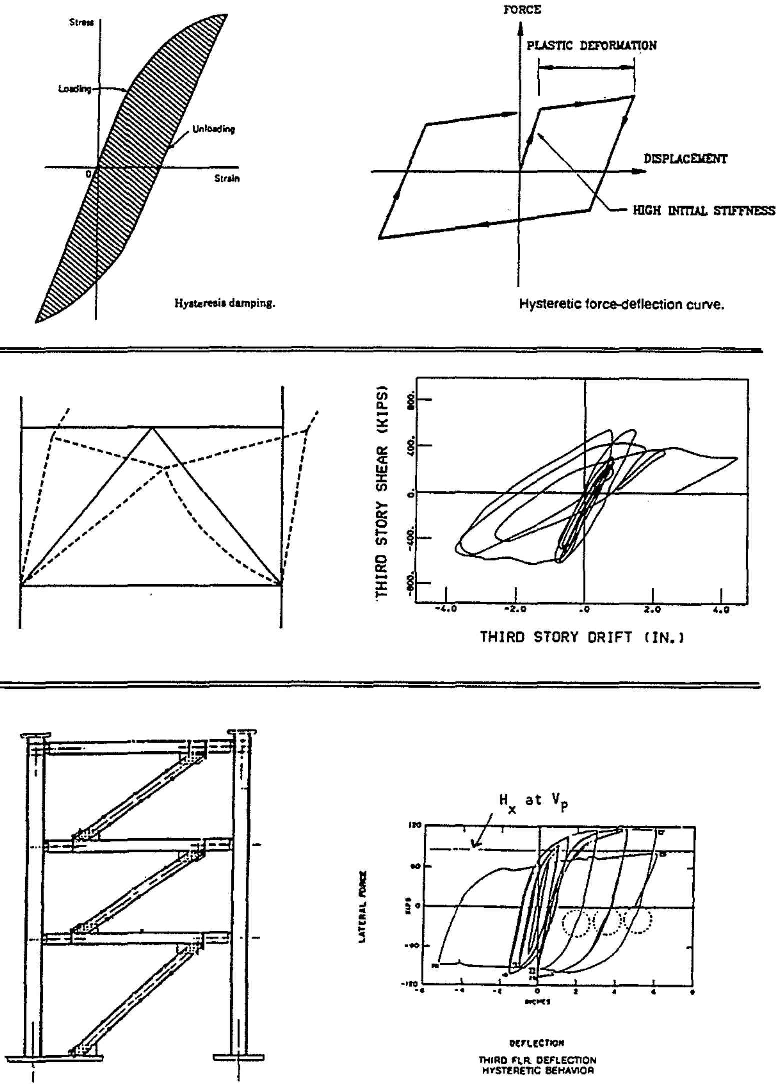


Fig. 7 THEORETICAL HYSTERETIC LOOP, CBF HYSTERETIC LOOP, EBF HYSTERETIC LOOP (Top to Bottom)

To ensure preservation of the system and to avoid buckling of the diagonal brace(s) and columns, these elements are designed for increased force levels. The compressive strength of the brace must be designed to exceed 1.5 times the axial force which corresponds to the controlling link beam strength. Likewise, the columns in the EBF must be designed to carry all gravity loads plus at least 1.25 times the axial force which corresponds to the link beam strength. This design procedure/check is done to ensure that the brace elements and columns remain well below their capacity when the link yields, preserving the vertical load capacity of the structure (strong column/weak beam theory).

The UBC puts limits on the amount of rotation the link portion of the beam can undergo relative to the rest of the beam at the inelastic force level (forces at $3R_w/8$ times the prescribed design lateral forces). These limits are based on the research and testing which was used to develop the EBF system. The rotational limits are as follows:

1) 0.060 Radians for link segments having clear lengths of 1.6Ms/Vs - ('shear yielding link')

2) 0.015 Radians for link segments having clear lengths of 2.6Ms/Vs or greater - ('moment yielding link')

Interpolation can be used for values between the upper and lower bounds.

The cyclic yielding of the web in shear can take place without failure and while preserving the hysteretic behavior. This can occur because the yielding occurs over a large segment of the beam web while tension field action takes over the load carrying mechanism to prevent failure. This provides a hysteretic loop having a large area representing good energy dissipation. (See Fig. 7 on previous page)

From a practical sense, EBF's have advantages in: simplified joint fabrication (two member verses three member joints); the geometry of the diagonal brace/link beam can allow egress paths through braced bays, not otherwise available with CBF's; structures can be designed for lower force levels.

On the negative side: design of EBF's are more complicated than CBF's; the link beam and beam/diagonal brace joint generally require stiffeners and braces not found in CBF's; framing elements of EBF's are heavier than CBF's for the same design load.

DUAL SYSTEMS

A dual system is a combination of a moment frame and shear walls or braced frames (CBF or EBF) designed in accordance with the applicable model building code requirements. We shall refer to the shear wall/braced frame option as 'rigid-bay systems.' Building codes allow systems to be combined in order to best use each system's strengths, and to accommodate the multitude of possibilities when confronted with site and design constraints.

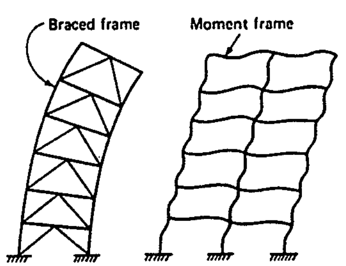


Fig. 8a BRACED FRAME AND MOMENT FRAME DEFLECTION CHARACTERISTICS

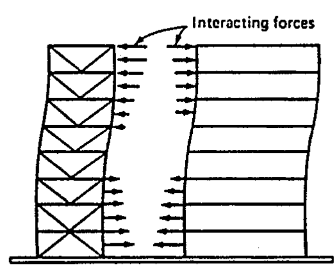


Fig. 8b DUAL SYSTEM LOAD INTERACTION

Moment frames tend to displace most (largest inter-story drift) at the base where the cumulative story shears are the largest. Rigid-bay systems have the largest inter-story drift at the top floors. When combined, the moment frame restrains the rigid-bay system at the top while the rigid-bay system restrains the moment frame at the bottom. (See Fig. 8) This load transference between systems can cause odd load paths. At the top of the dual system, shears in the moment frame can exceed the story shears at any given level, while at the base the opposite is the case.

A dual system, under the jurisdiction of the current UBC, must:

1) have an essentially complete space frame (non-bearing wall configuration) which provides support for gravity loads;

2) resist lateral loads via rigid bay and moment resisting frames (There are limitations as to types of frames depending on building height and seismic zone.);

3) combine to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels; and

4) be designed so that the moment frame will independently resist at least 25% of the design base shear.

NOTE: The '85 UBC required the shear walls or braced frames to be designed to carry 100% of the design base shear acting independently of the moment resisting frames.

Both systems - moment frame and rigid-bay usually extend the full height of the building, although they aren't required to by code.

Dual systems tend to be less ductile in the inelastic range than a pure moment frame, but more ductile than a rigid-bay system. To account for the behavior of Dual Systems, codes allow use of different coefficients than for SMRF or rigid-bay systems when considered acting alone. For instance: alone, steel framing lateral systems have the following coefficients:

System	R_W (UBC)	K (BOCA, SBC)
SMRF	12	0.67
EBF	10	N/A
CBF NON-BEARING	8	1.00
CBF BEARING WALLS	6	1.33

When combined, the systems have the following coefficients:

Dual System	R_W	(K)
SMRF+EBF	12	N/A
SMRF+CBF	10	0.8

If one were to choose a SMRF+CBF Dual System rather than a SMRF+EBF System, the static design base shear would be 20% higher. Generally, the reduction in design force obtained by utilizing an EBF more than offset increases due to the design constraints and detailing which tend to make EBF more expensive than CBF.

BROADWAY CENTRE'S EBF

As noted above, the most efficient way to stiffen Broadway Centre's soft building story was to add an EBF. The introduction of the stiff frame in the lower portion of the building also reduced the amount of force that was distributed to the subdiaphragms at the plaza level. Only about 30% of the total base shear of the structure remained in the SMRF at the plaza level. The other 70% was carried by the EBF to the mat. This met the 25% minimum force level required by code. Actual frame elevations are shown in Fig. 9.

Four EBF's having the single diagonals are used at the stair cores while one dual-diagonal brace was placed at the end of the elevator lobby.

Because the stiffness of the segmented subdiaphragms at the plaza level was extremely difficult to model, the design of the EBF's was bounded by the following conditions. First, the system was analyzed assuming there was no restraint at the plaza level for the EBF's. Second, the system was then re-run assuming the plaza level had an infinitely rigid diaphragm. The design for each element of the EBF's and SMRF's was based on the largest loads generated by either of the two analysis scenarios. This "bounding" approach also addressed a concern that the plaza level could act like a fulcrum and actually produce design loads beneath the plaza level which exceeded the base shear of the building. This ended up not being the case.

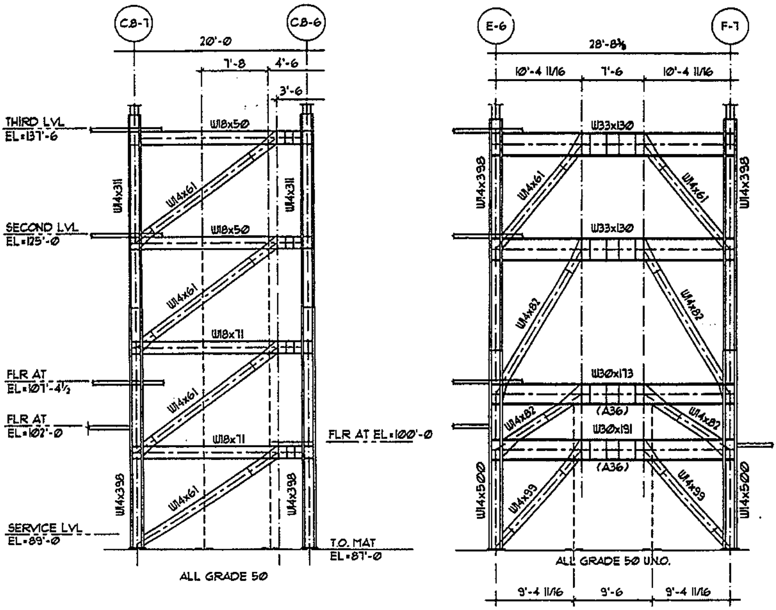


Fig. 9 BROADWAY CENTRE EBF ELEVATIONS

The top of the EBF's terminate at the 3rd floor of the tower. At this level, the load transfer from the SMRF to the EBF was at magnitudes which could be dealt with in the slab. The inter-story drift (lateral floor displacement relative to the next floor above or below) was controlled enough by the Dual System up to level 3 that the SMRF alone was sufficiently stiff from there up. About 2/3 of the total building shear at the 3rd floor is transferred from the SMRF to the EBF. Only about 8% of the total building shear is transferred to the EBF at the 2nd floor.

The 3rd floor diaphragm needed to be reinforced for the shear transfer to the EBF. Fairly heavy cord steel (reinforcing) was also added to the diaphragm for in-plane bending. (See Fig. 1, shaded area)

ELEMENT BRACING

One critical issue with EBF's is that the brace/beam joint must be restricted from lateral displacement so the beam will deform only in the plane of the brace. If lateral buckling of the EBF were to occur in the beam, the frame would become unstable and potentially precipitate failure. Normally the EBF beam coincides with the floor framing so buckling of the top flange, at least, is easily provided. Additional braces from the bottom flange to the floor soffit, or other framing, are common. At Broadway Centre, between the mat foundation and the 2nd floor, the EBF's pass through the various plaza levels, which don't necessarily correspond with the location of the EBF beam element. Out of plane buckling had to be restrained without the ability to brace to a floor.

Vertical elements were introduced which could be considered "lateral bracing girts" (vertical dashed lines in Fig. 9). These girts spanned from the mat or plaza framing to the underside of the 2nd floor. The girts are attached to the beam element of the EBF for lateral (not vertical) restraint only. Two girts per brace are used: for dual diagonal braces, one girt was placed at each brace/beam joint; where the EBF used a single diagonal, one girt was placed at the brace/beam joint and one girt at mid-span of the beam back-span (the long end opposite the link end).

CONCLUSION

Broadway Centre is unique in many aspects, not the least of which was the use of a fairly new structural bracing system, the EBF. The use of EBF's in this project solved two main problems: complicated force distributions through a maze of truncated subdiaphragms at the building plaza level; and stiffening a structural system with a soft first story. EBF's accomplished this without increasing the entire lateral load-resisting system's demand, unlike CBF's. The use of the EBF's at only the base of the structure minimized the negative aspects (heavier members, increased element bracing and fabricating complexity due to stiffeners) thereby reducing the adverse impact to the construction budget. SMRF+EBF's combined in a "Dual System" at Broadway Centre was the correct, appropriate use of this new structural system.

CODE COMPARISON: Treatment of EBF

1991 UBC (ICBO)	Identifies building type. (First appeared in '88 UBC.)
1991 SBC (SBCCI)	No specific reference to EBF
1990 BOCA (BOCAI)	No specific reference to EBF

ABBREVIATIONS

V_s	Shear Strength (of link beam) = 0.55(Fy)d(tw)
M_s	Moment Strength (of link beam) = (Fy)Z
UBC	Uniform Building Code
SBC	Standard Building Code ("Southern Building Code")
BOCA	Building Officials and Code Administrators

REFERENCES

Fig. 4 "Recommended Lateral Force Requirements and Commentary," Seismology Committee, Structural Engineers Association of California, Sacramento, California, 1990

Fig. 5 Farzad Naeim, "The Seismic Design Handbook," Van Norstrand Reinhold, New York, 1989

Fig. 6 Farzad Naeim, "The Seismic Design Handbook," Van Norstrand Reinhold, New York, 1989

Fig. 7 Bungale S. Taranath, "Structural Analysis and Design of Tall Buildings," Mc Graw hill, New York, 1988; Farzad Naeim, "The Seismic Design Handbook," Van Norstrand Reinhold, New York, 1989; "Recommended Lateral Force Requirements and Commentary," Seismology Committee, Structyral Engineers Association of California, Sacramento, California, 1990; Edward J. Teal, "Design of Eccentric Braced Frames," Steel Tips, Steel committee of California, Technical Information and Product Service, August 1987

Fig. 8 Bungale S. Taranath, "Structural Analysis and Design of Tall Buildings," Mc Graw Hill, New York, 1988

Thanks to my associates Don Weber and Farzad Naeim for help in understanding and designing Eccentric Braced Frames and to my wife, Jennifer, for understanding me and helping prepare this report.

EFFECTIVE BRACING OF TRUSSED TOWERS AGAINST SECONDARY MOMENTS

F. Al-Mashary[1], A. Arafah[1] and G. H. Siddiqi[2]

[1]Assistant Professors and [2]Associate Professor
Civil Engineering Department, King Saud University,
P.O. Box 800, Riyadh 11421
Kingdom of Saudi Arabia

ABSTRACT

A forensic study into the failure of six transmission towers revealed high values of secondary moments in some regions which were not accounted for in the design. These bending moments caused overstressing which led to the cascading failure of the towers. The study showed that improper bracing configuration of the main and/or secondary braces induced high secondary moments at these locations. High moments are also induced at the locations where the changes, however small, occur in the main leg alignment. It is possible to rearrange bracing scheme to modify the load path so that the secondary moments are effectively reduced. The study recommends consideration of secondary moments in both the analysis and design procedures.

INTRODUCTION

It is a common practice to model transmission towers as two-dimensional truss systems in the transverse and longitudinal directions. In three-dimensional modeling kinematic instability is encountered at the joints where the members lie in one plane. This problem is circumvented by introducing spring supports at such joints placed normal to the plane [1] or by modeling the main legs as continuous frame members [2]. When the analysis results in significant forces in the spring supports or moments in the main leg frame members, whichever is employed in the modeling, the suitability of bracing configuration should be investigated in order to effectively reduce these forces. It is advisable to proportion the main leg frame members as beam-column elements, and fabricate and splice them accordingly.

THE CASE STUDY

The authors were commissioned to investigate the failure of six 132 kV tangent transmission towers in Al-Qassim region of Saudi Arabia. The first and the last towers failed by bending of the top cross-arms, the three of the intermediate ones at their bases and the sixth above its first body. The towers had height of 45.45 m (Fig. 1) and were spaced over a span of 400 m. They had square planar configuration and were assembled from single angle continuous main legs and cross bracing. The bracing in adjacent faces was staggered.

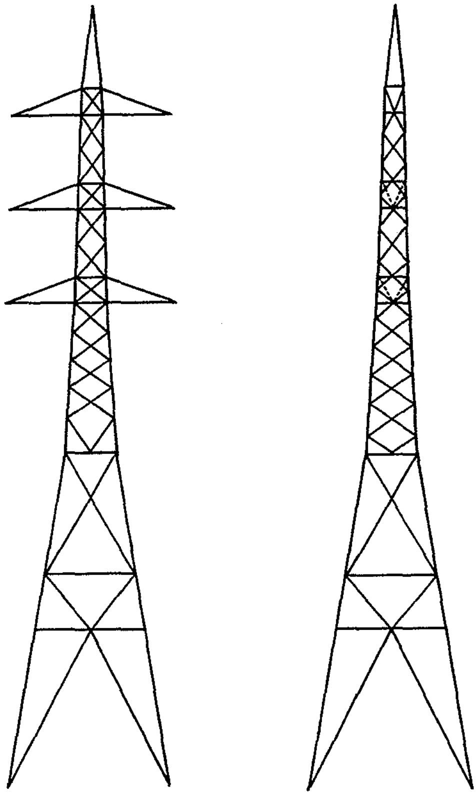
A meteorological report on the day of failure recorded at a nearby station did not show any sign of high wind nor the people living in the surrounding regions reported unusual wind. However, the designer in his post failure assessment attributed the failure to unforeseeable wind gust above the design speed of 170 km/hr (106 mph) [3]. The authors visited the site after the removal and restoration of the said towers. They examined the debris for possible clues. Laboratory test of the specimens from the material employed showed tensile strength in excess of the prescribed value of 345 MPa (50 ksi). The specimens also showed adequate ductility.

The governing design specifications [4] prescribed a design wind velocity of 170 km/hr (106 mph) and a gust speed of 220 km/hr (138 mph) in the region. The specified value is a good estimate of 50-year basic design wind speed recommended by an independent study [5]. The specifications require that a tower be analyzed for fourteen load cases covering full-loading, single circuit usage, broken conductors and under construction situations. They refer to ASCE Manual No. 52 [2] for design regulations.

ANALYSIS OF TOWERS

The towers were analyzed as three-dimensional structures with the main legs as frame members. This assumption is the closest to the actual detailing of the main legs. The applied loads when evaluated in accordance with the prescribed conditions were found to be adequate. The analysis was implemented on SAP 90 finite element package [6]. Presumably the designer employed two-dimensional truss model for his analysis.

The axial forces in most of the members, from the two analyses, were in agreement. However, the deviations between the two



(a) Transverse (b) Longitudinal

Fig. (1) Tower Elevation

analyses, in some of the members, is attributable to the difference in the methods of modeling and analysis [2].

The most noteworthy results of the analysis are the bending moments in the main legs which are significantly high compared to the meager bending resistance of the legs. These bending moments are neglected in the original design calculations. It is also noteworthy to mention that the maximum drift of 570 mm (22.5 in.) was encountered under the severe most loading condition.

EFFECTIVE BRACING OF TOWERS

Some of the generalized observations on the bending moments are that they are induced only when the main legs are modeled as frame members, that they do not buildup over the tower height, are local in nature and occur where reversal of moments takes place, and that they are small in value but significantly large compared to the bending capacity of the leg members. It was noticed that bracing configuration significantly affected these moments. Three major configuration faults were identified and are discussed below.

Single-End Bracing Joint

Secondary bracing members are usually assumed to have zero force in the pre-buckled state. This assumption is not true when the main members are modelled as frame members. At a single-end bracing joint with the main leg, the axial force in the brace is equilibrated only by the shear resistance of the leg which in turn induces significant value of bending moment in the main leg at the joint. Fig 2a shows a two-dimensional part model of the tower to illustrate this point. This situation can be avoided by either eliminating single-end bracing joints wherever possible or by adding a member to redirect the force away from the main leg. Fig 2b shows such a solution which reduces the maximum moments of 115, 151, 148 and 168 kN-mm at four joints to 21, 27, 7.8 and 8.3 kN-mm respectively.

Improper Main Bracing

Main bracing elements are employed to transmit the horizontal loads as direct forces and avoid moment build-up on the main legs. Improper arrangement of these braces is liable to generate the moments in the main legs. One such example is the use of K-bracing arrangement in the tower body at the attachment of the cross-arms. The use of cross-bracing reduces the moment in the main legs about

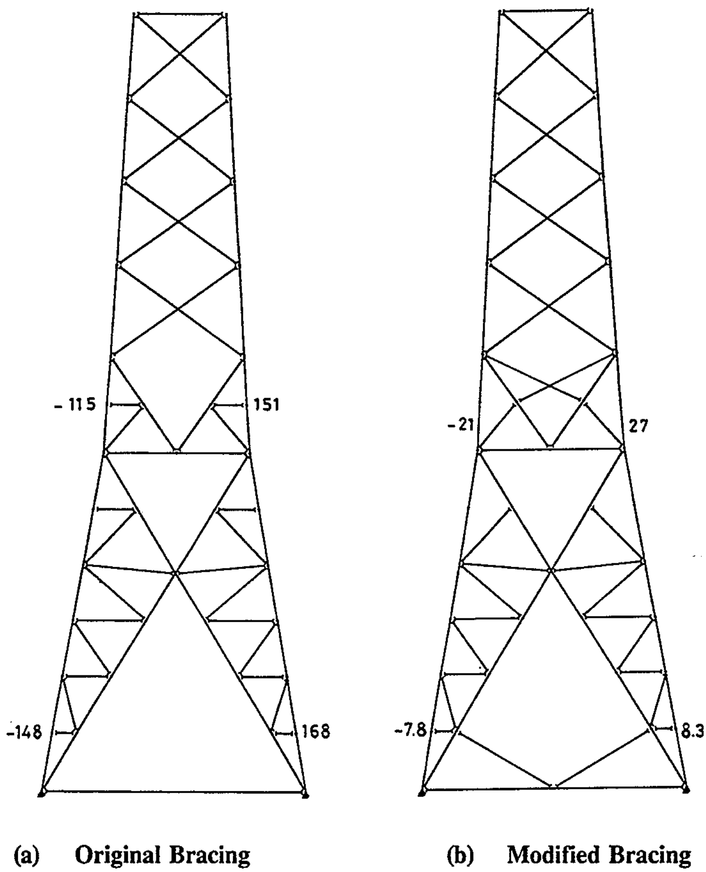


Fig. (2) A Two-Dimensional Partial Model of the Tower showing Maximum Moments (KN-mm)

65 to 75%. Therefore, careful attention must be paid during the selection of bracing system.

Lack of Leg Alignment

The analysis reveals that the secondary moments are sensitive to lack of leg alignment which may be caused by fabrication errors specially at the lap-joints and at the locations where intentional change of slope is introduced in the leg alignment. A 10 mm offset from the leg alignment at a lap joint in the middle part of a tower resulted in 40 kN-mm moment on the leg which is 240% of the moment at this joint in perfect alignment. This effect is produced by the normal to leg component of the axial force in the out of alignment part which is resisted by shear in the main leg. This problem can be resolved by providing an alternate path to the normal forces with the help of adequate bracing at splice joints, as a measure of precaution, and at joints where the main leg is realigned.

CONCLUSIONS

The analysis and design of transmission towers should consider both the axial force and bending moment in the main legs. These moments although dubbed as secondary and neglected in common design practice, are significantly high at locations to lead to unexpected failures. The bracing system and its arrangement play an important role in control of these moments. These moments can not be eliminated completely but can be managed to values affordable by the main leg section. It is shown that single-end bracing joint, improper bracing and lack of main leg alignment contribute to the bending moments. Provision of an alternate path, to channel the normal forces resulting from alignment offset at these locations, can lead to effective control of the secondary moments.

REFERENCES

1. Lo, David L. C. , Andrew Morcos and Surrendra K. Goel, "Use of Computers in Transmission Tower Design, Journal of the Structural Division, ASCE, Vol. 101, No. ST7, July 1975.

2. ASCE Manuals and Reports on Engineering Practice No. 52, Guide for Design of Steel Transmission Towers, Second Edition, 1988.

3. SAE SADELMI S.P.A., " Technical Report on Collapse of Towers," Milan, Dec. 3, 1991.

4. Surveyer, Nenniger & Chenevert Inc., "Tender Document for the 132 kV Transmission Line in the Qassim Region," Vol. II, Montreal Canada, Oct. 1978.

5. Arafah, A.; Siddiqi, G. H. and Dakheelallah, A.; "Extreme Wind Speeds in the Kingdom of Saudi Arabia," the Arabian Journal for Sci and Engr., Vol. 17, No. 3, King Fahd Univ of Petroleum and Minerals, Dhahran, Saudi Arabia, July 1992.

6. Wilson E. and Habibullah A., "SAP 90, Structural Analysis Programs," Computers and Structures Inc., Version 5.1, 1989.

A CONTRIBUTION TO THE DESIGN OF FRICTION BOLTED CONNECTIONS FOR CONCENTRICALLY BRACED STEEL FRAMES

R. Tremblay and S.F. Stiemer
Department of Civil Engineering, University of British Columbia,
Vancouver, B.C., Canada V6T 1Z4

ABSTRACT

An experimental study has been carried out to examine the performance of a braced framing system including friction-type bolted brace connections which can slip at a predetermined load level for dissipating input energy during severe ground shaking. A first series of tests was conducted on 42 one-bolt connection samples subjected to dynamically applied cyclic loading. Various faying surface materials, bolt sizes and frequencies of excitation were considered. The effects of varying the initial bolt load level and using conical disc spring washers with the bolts were also examined. These tests revealed that the desired performance could be achieved by using proper sliding material together with an appropriate bolt clamping force level. A second series of tests was carried out to investigate the behavior of a typical bracing bent undergoing severe interstorey drifts (up to 2%). Quasi-static cyclic loading tests on a full scale braced frame assembly including a HSS 203x203 diagonal bracing member with a 16-bolt symmetrical shear splice sliding connection demonstrated that the proposed system behaved in a very predictable and satisfactory manner.

INTRODUCTION

Of all the lateral load resisting systems for steel building structures, the Concentrically Braced Frames (CBF) system stands out by its simplicity of design, fabrication and construction, as well as by its high efficiency for low- and medium-rise structures. The performance of CBFs under severe earthquake ground motions has however recently been questioned, the main concern being the reliability and the effectiveness of the energy absorbing and dissipating mechanism formed by the bracing members undergoing cyclic tension yielding and compression buckling. Deterioration of the compressive strength, cumulative elongation, premature fractures are among the problems that have been observed upon inelastic response of bracing members of CBFs (Popov 1981; Foutch 1987).

Many avenues have been explored to enhance the seismic response of braced frames. The first one consisted in establishing proper detailing provisions for improving the performance of the braces and in specifying appropriate seismic design load levels which depend upon the detailing and the bracing configuration. These have been included in recent code documents (e.g. SEAOC 1990). Extensive efforts have also been devoted in developing systems in which the inelastic action would be restricted to structural elements other than the bracing members. Systems like Eccentrically Braced Frame (Roeder and Popov 1978), Disposable Knee Bracing (Aristizabal-Ochoa 1986) or Y-Shape braces (Seki *et al.* 1988) emerged from these research efforts. In both of these cases, however, the improved CBF system gains in complexity, loses its efficiency and is exposed to structural damage upon earthquake ground shaking.

A third and promising approach has also been proposed which consists in introducing a connection at one end of the bracing members which would slip at a predetermined load level (Elsesser 1986; Roik *et al.* 1988; FitzGerald *et al.* 1989). During strong seismic events, energy would be absorbed and dissipated by friction upon sliding of these connections, with no damage to the surrounding structural elements. Such a system, referred to herein as a Friction Concentrically Braced Frame (FCBF), can be used either for new or existing braced structures. Furthermore, braces with sliding connections can be installed in Moment Resisting Frames to improve their seismic response (Pall and Marsh 1982; Whittaker *et al.* 1987).

OBJECTIVES AND SCOPE OF THE STUDY

The critical element in a FCBF system is the brace sliding connection. It would desirably exhibit the following features: simple to design, fabricate and install, built with low cost and readily available material only, high slip resistance per unit of material and stable response upon cyclic sliding. The latter is of prime importance in earthquake resistance since a degradation of the strength would result in an inacceptable structural response whereas an increasing strength would have to be taken into account in the design of the surrounding structural elements which, in turn, would result in more costly structures.

The systems proposed by Roik *et al.* and FitzGerald *et al.* included many of these characteristics. The connections were made from slotted bolted unfinished steel parts sliding against each other and were found to respond in a stable manner upon cyclic slip tests. However, they involved rather complicated multistage slip mechanisms and could only develop relatively low friction forces because fairly uncommon small bolts were used. Further, the applied loading in the tests was deemed to be less critical than what could prevail during actual seismic events: Roik *et al.* imposed very small slip displacement in their dynamic tests (less than 1 mm) whereas the rate of loading used by FitzGerald *et al.* (0.25 Hz) is somewhat low when compared to the fundamental period of low-rise structures.

Also of interest was the overall behavior of the system upon sliding of the brace connections. Though members of CBFs are generally assumed as being pin connected in design, some beam-column joints can actually exhibit significant rotational restraint, the columns are commonly made continuous over two or three stories and, in most cases, the brace connections would in fact be capable of developing substantial in-plane bending moments. While the elastic response of braced frames is not strongly influenced by such additional restraint, significant bending moments can develop in the members when large interstorey drifts occur as sliding takes place in the connections of FCBFs. Since the beams, columns and braces must remain essentially elastic during an earthquake, these secondary moments must be accounted for in the design. Their magnitude obviously depends upon the degree of rotational restraint offered by the brace connections upon sliding, for which no information was available. Further, it was thought that significant bending moments developing in the brace sliding connections could diminish their resistance to longitudinal slip. Therefore, some knowledge on such interaction had to be obtained as well.

The objectives of this study were then to come up with a FCBF system that could be readily implemented in the steel industry, to investigate the performance of the sliding connections when subjected to extreme loading conditions and to examine the overall response of a typical braced frame assembly undergoing severe interstorey drifts.

STUDIED FCBF SYSTEM

The system addressed in our study included a symmetric shear splice connection (two shear planes) with slotted holes in the gusset plate and standard circular holes in the connecting plates extending from the brace (Fig. 1). In such a system, the length of the slot would be set to allow the maximum slip expected for the design basis earthquake level to take place. The undesirable overstressing of the connections and of the frame when the bolts suddenly come into bearing would be then avoided. A suitable strength hierarchy would have, however, to be provided in the connections to prevent detrimental failure modes (e.g. shear block failure of the gusset plate) and maintain the integrity of the connections in the case of extreme events larger than the design earthquake. Tentatively, common high-strength bolts were to be used and the connecting plates were to be made of ordinary structural steel in the clean mill scale condition.

The thickness of the connected plates was expected to change upon cyclic sliding: reduction due to the wear likely to occur at the contact surfaces and, oppositely, increase due to the temperature rise resulting from the heat generated by the friction. If the former was to dominate, the bolt clamping force would diminish, together with the slip resistance of the connections. If, on the other hand, the plates were to increase in thickness during sliding, the response would then depend whether the initial bolt tension was well below the bolt ultimate capacity or near this limit.

In the first case, the bolt load and, consequently, the slip resistance of the connections would likely increase, which would result in unwanted connection overstrength. Upon cooling, however, if they remained elastic during sliding, the bolts would return to their original tension level, minus the loss due to wear. In the second case, the bolts pretensioned to their ultimate capacity would deform inelastically (yield) and a rather constant clamping force would be maintained despite the increase in the plate thickness. This second scheme also appeared more efficient since the capacity of the bolts would be fully employed and bolt preload near ultimate can easily and reliably be achieved in the field by using the turn-of-nut installation method. However, the post-earthquake strength of the connections in such case would most likely be reduced because of the permanent elongation undergone by the bolts.

In order to evaluate the relative importance of the shortcomings exhibited by each approach, it was decided to investigate both torque levels in the experimental study. As proposed by Roik *et al.* and FitzGerald *et al.*, conical disc spring washers were to be used in the bolting assembly to mitigate the effects of the variation in the plate thickness.

EXPERIMENTAL PROGRAM

The experimental program included three phases: preliminary testing of connection components, testing of connection samples subjected to dynamic loading and quasi-static loading of a full-scale braced frame assembly. The work was carried out in the Structures Laboratories in the Department of Civil Engineering of the University of British Columbia between December 1991 and November 1992.

The behavior of a typical brace-sliding connection assembly was examined in the program. The brace was a 203x203x13 hollow tubular shape and the connection included A325 3/4" (19.1 mm) bolts with a parallel stack of 15 pre-stressed disc spring washers (No. AM602130). Such disc spring arrangement could carry the full bolt capacity and was preferred to other possible configurations including fewer and stronger washers because of its higher flexibility. Brace connecting plates and the gusset plate were 16 mm and 19 mm in thickness, respectively. The slotted holes could accommodate a maximum slip equal to 54 mm in each direction, plus some fabrication and installation tolerances. Such amount of slip can be experienced in typical braced frame configurations undergoing interstorey drift equal to 2% of the storey height, the latter value corresponding to current code limitations for drift under seismic loading.

In order to reflect current fabricating practices, all steel specimens were manufactured by local steel fabricators and no special attempt was made to control the flatness or the dimensions of the samples. Except otherwise specified, all of the steel plates used were in the clean mill scale condition.

PRELIMINARY TESTING

Preliminary testing included a calibration of the bolts, cyclic loading of stacked disc spring washers and static slip tests on two-bolt connection samples. Calibration of the bolts permitted to relate the bolt load, the bolt elongation and the amount of rotation of the nut upon torquing for bolts with and without disc springs. Such relationships were then used to predict the actual clamping force in the tested connections. Tests on disc springs were done in order to investigate their response upon cyclic loading. The load level, the number of cycles and the number of washers in the stacks were varied. The tests showed that the washers responded in a fairly stable manner. Some hysteretical behavior was, however, noted as the number of washers in the stack was increased. This was attributed to the friction between the individual units.

The main objective of the static slip tests was to obtain a first insight into the gross slip behavior of the connections. Three series of three tests were performed on two-bolt symmetric butt joints (Fig. 2). Specimens of the first two series where made from common structural steel (CAN3-G40.21M-300W, Fy = 300 MPa) whereas ASTM A514 grade B plates (Fy = 689 MPa) were used in the third series. In the second series, the faying surfaces were polished (medium grain size rotative disc) and then sand blasted (60-100 sieve). The bolts were torqued to their ultimate capacity. The load was gradually increased until slip occurred. The test was then pursued at a slow and constant rate until the bolt came into contact with the holes.

Fig. 2 shows typical response for each series. The most important finding from these tests was that all specimens but one sample of series 1 exhibited a significant gain in resistance as the displacement progressed past the initial slip. An average ratio of the maximum to the initial slip load of 2.52 (2 tests), 1.95 and 1.77 were measured in test series 1, 2 and 3, respectively. Inspection of the faying surfaces after the tests showed a few parallel scores up to 6 mm in width, 50 mm in length and 2 mm in depth, with localized enlargements showing evidence of plowing of the material of one surface into the opposite one.

The gain in strength and the severe frictional behavior were probably due to the breaking of the many films (oxide, grease, etc.) at the contact surfaces which led to the development of stronger adhesive bonds, together with the fact that the surfaces were made of the same material and therefore could interact more strongly (Rabinowicz 1965). Strain hardening of the steel in the damaged areas also likely contributed in the growing resistance to slip. Such performance after only 1/4 cycle of sliding suggested that the connection would develop substantial overstrength and suffer extensive surface damage under cyclic loading, which would be inappropriate for the studied FCBF system. Improvement was likely to be achieved by providing a more suitable material combination or, alternatively, by reducing the bolt tension load. Both of these approaches were considered in the dynamic testing phase of the program.

DYNAMIC TESTING OF CONNECTIONS

A series of 42 dynamic cyclic tests were performed on single bolt specimens as shown on Fig. 3a. Specimens D-01 to D-14 were made of the aforementioned ordinary structural steel (Fy = 300 MPa), specimens D-15 to D-29 included higher strength steels and cobalt-base alloy insert plates (Fy = 550 MPa) with a circular hole were used between the sliding surfaces in specimens D-30 to D-42. Tests D-01 to D-37 included a 3/4" bolt, torqued close to half its ultimate capacity (88 kN) except for 5 specimens with the insert plates for which the bolt was tightened to its full capacity. A325 1/2" (12.7 mm) bolts torqued to their full capacity were used in specimens D-38 to D-42. It is worth noting that the clamping force developed by a 3/4" bolt torqued to half its capacity is approximately equal to the full tensile strength of a 1/2" bolt.

Disc spring washers were used only in half the specimens D-01 to D-38. For the 3/4" bolts with the reduced initial tension, a parallel stack of 7 disc springs of the aforementioned type was provided on both sides of the connection. The imposed slip time history was a 32 cycle sinusoidal signal as shown on Fig. 3b. Three frequencies were applied: 0.02, 1.0 and 1.5 Hz. Some specimens were subjected to two or three successive runs.

Slip load time history and load-slip relationships for specimens D-01, D-27, D-32 and D-39 are given in Fig. 4. The first three specimens had a 3/4" bolt, with the reduced preload, and disc spring washers. In specimen D-27, the interior plate was made from an abrasion resistant plate material having a yield stress of 1370 MPa whereas its exterior plates were of the ordinary structural steel. All four specimens were subjected to a 1.0 Hz excitation. Fig. 5 shows the interior (gusset) plate of specimens D-01 and D-32 after the test.

Response and surface damage exhibited by specimen D-01 are representative of the specimens made entirely of ordinary structural steel and subjected to the 1.0 and 1.5 Hz frequency loadings. The load increased within the first few cycles, reached a plateau at approximately twice the initial slip load and increased again up to 3 to 4 times the initial slip load towards the end of the run. Such high variations in the slip load have also recently been observed by Grigorian *et al.* (1992). In most of the tests, evidence of very high temperature of the faying surfaces could be observed during the large amplitude phase of the loading: the steel of the middle plate turned blue and even red along the slotted hole, which indicates temperatures up to about 700 degrees C. Examination of the faying surfaces after the tests revealed significant damage (Fig. 5a): most of the contact area was severely scored with evidence of metal being pulled out and transferred from one plate to another (adhesive wear). The darker zone on the picture corresponds to the region where the highest temperatures were reached and where the most significant damage level was observed.

The initial increase in load probably had the same origin as the one observed in the static slip tests described earlier. The subsequent phase likely involved many interacting phenomena: wear, increase of the plate thickness and softening of the metal due to the rise in temperature, variation of the bolt clamping force and growing contribution of the roughness and plowing terms to the slip load as more surface damage was taking place (friction between undamaged flat surfaces mainly originates from adhesion). Moreover, as the tests progressed, the particles removed by wear likely contributed in damaging further the surface (abrasive wear). The increased slip resistance observed in the last part of the tests was probably the result of that surface damage, specially during the last cycles when the steel asperities recovered some strength as the plates cooled down (smaller slip amplitudes). This may also explain the higher loads that can be observed at the end of each cycle where the relative velocity of the plates was minimum.

Surprisingly, samples without disc springs behaved in a manner very similar to those with disc springs. Their response was even slightly more uniform, the temperature rise during the tests was less apparent and their surfaces less damaged. This has been attributed to the fact that the bolt clamping force likely diminished in the early stage of the tests due to wear, and the subsequent demand on the plates was therefore reduced. The tests run at 0.02 Hz were, on the other hand, interrupted after a few cycles only when seizure of the connecting plates occurred, regardless whether disc springs were used or not. Such seizure was probably due to the slow rate of slippage and the affinity of the contact surfaces which both permitted the creation of high adhesive bonds, as observed in the static slip tests.

Although the plates underwent severe damage, the response of this first group of connection samples would have been acceptable had it not been for the high increase in strength, and even seizure, which would make the system inefficient in practice. Thus, reducing the bolt preload was not sufficient by itself to obtain an acceptable response of the connection. It was found, however, that the usefulness of the disc springs was somewhat diminished due to the many other phenomena involved.

Using higher strength steels did not result in substantial improvements until, as expected, the difference in strength between the materials in contact became important, as this was the case for specimen D-27. For this configuration, the slip load remained fairly stable over the whole duration of the loading, the damage to the surfaces was significantly reduced and, while higher slip resistances were measured, a uniform response with no seizure was observed upon loading at 0.02 Hz. The role of the disc springs was found more important for these specimens. Samples without disc springs developed high overstrength during the tests, probably because the increase in thickness of the plates due to the heat generated by the friction dominated over the wear, which resulted in an increasing bolt tension.

For the first group of samples with insert plates, 3.2 mm thick cobalt-base alloy material and 3/4" bolts (D-30 to D-37) were used. Among these specimens, those with the reduced initial bolt tension and with the disc springs performed very well under high frequency loading. As shown on Fig. 4 (D-32), their resistance to slip remained fairly constant throughout the test despite the plates suffered some damage (Fig. 5b). Specimens with the bolt torqued to its full capacity developed a higher and increasing slip load when subjected to 1.0 Hz loading and the tests had to be interrupted because the capacity of the loading device was reached. In all tests run at 0.02 Hz, the samples exhibited a very smooth response.

The second group (D-38 to D-42) involved samples with 1.6 mm thick insert plates made from cobalt and bolts 1/2" in diameter without disc springs. All of these specimens behaved well regardless of the frequency of excitation. Except for the increase observed within the first 4 cycles, the slip load for the specimens tested at 1.0 Hz (D-39, Fig. 4) and 1.5 Hz remained constant over the entire duration of the test. Upon reloading (second run), these specimens exhibited approximately the same strength than the one measured at the end of the first run, even if the bolts were torqued to their full capacity and experienced significant inelastic elongation during the tests. The slip load eventually dropped within a few cycles to about half the initial load of the first run. In the tests performed at 0.02 Hz, the specimens showed the same initial gain in resistance, then the slip load slowly decreased until the end of the tests where it reached approximately the initial slip load level.

When insert plates are used, any increase in the friction force between the middle plate and the insert plates has to be resisted through bearing of the insert plates against the bolts. Samples with 1.6 mm thick plates subjected to high frequency loading showed evidence of such bearing as their hole was severely elongated. The discontinuities in the lower right and upper left corners of the load deformation diagram recorded during the last 5 cycles of the test on specimen D-39 are due to that phenomenon (Fig. 4). This suggests that these plates were too thin for such application. Plates 3.2 mm thick with 3/4" bolts behaved in a more acceptable manner (Fig. 5b).

FULL-SCALE BRACED FRAME TESTING

A single storey diagonally bracing bent mounted horizontally on a rigid floor was used for investigating the overall behavior of the proposed FCBF system (Figures 6 and 7). Beam-column joints were pinned and bending moments could then be induced in the system only by the bending stiffness of the brace and of its connections. At the lower end of the brace (lower left corner of the frame), a typical connection with the brace welded to a gusset plate pre-welded to the column was reproduced. In this case, the gusset was however bolted to the column by means of a slip-resistant connection for making easier the installation of the specimens. The sliding connection located at the upper end of the brace included 16 A325 1/2" bolts torqued to their ultimate capacity together with cobalt insert plates 1.6 mm in thickness. No disc springs were used.

Two series of tests were performed with different grades of steel for the gusset plate. The response was very similar in both cases and only the series for which ordinary structural steel (Fy = 300 MPa) was used for both the gusset and the brace connecting plates is described herein. The slotted holes were made wide enough to accommodate the relative rotation of the connected parts that could be expected assuming the connection would behave as a pin upon sliding. Strain gauges were mounted at two locations on the brace to obtain the bending moments in that member. The loading sequence used in the dynamic tests (Fig. 3) was also applied to the frame. The maximum amplitude of the storey drift was 70 mm, which resulted in comparable maximum slip in the connection (54 mm). The rate of loading was 0.5 cycle per minute.

The overall response of the frame is given on Fig. 8a. The variation of the slip load in time is almost identical to the one observed for the single-bolt connection specimen having similar characteristics and tested at 0.02 Hz. The sliding connection behaved very smoothly in all tests and no sign of severe bearing of the bolts against the sides of the slotted holes as slip was taken place could be noticed. As shown, the storey shear-storey drift diagram exhibits a slight post-slip stiffness which corresponds to the contribution to the storey shear from the members being deformed in bending. In Fig. 8b, the bending moments measured at the lower quarter-span and at mid-span of the brace are plotted against the storey drift. The value of the moment acting at the connection, as extrapolated from the two preceding measurements, suggests that, over the whole range of the storey drift experienced by the frame, the connection behaved mostly like a pin upon sliding. Indeed, calculations showed a good agreement between the measured moments and those computed assuming a pinned sliding connection. The very stable response of the frame within each cycle (Fig. 8a) indicates that the relative rotation of the connected parts did not impair the longitudinal slip resistance of the connection.

CONCLUSION

The experimental study reported herein showed that the proposed FCBF system represents a very promising alternative for buildings located in seismically active regions. The sliding connection samples exhibited a very high and stable energy dissipation capability under extreme loading conditions (amplitude of slip, number of cycles, high and low frequencies) provided that appropriate sliding material and bolt clamping force were used. The tests showed that employing dislike materials at the contact surface, such as different steels or plate inserts made from a different alloy, could result in a desirable stable response. Samples including cobalt-base insert plates and a 1/2" bolt torqued to its full capacity, without disc spring washers, responded in a very satisfactory manner and exhibited high post-earthquake resistance. The latter diminished significantly, however, upon reapplication of the cyclic loading.

Tests on the bracing bent revealed a very satisfactory and predictable overall response of the system. The proposed sliding connection behaved mostly like a pin and its resistance to slip was unaffected by the relative rotation of the connected parts. Further research is however needed to investigate the performance of multi-bolt sliding connections upon dynamically applied cyclic loading. Among others, the post-earthquake resistance and the possible effects of having a larger contact area (higher temperature rises) would require further examination.

ACKNOWLEDGMENTS

This study was supported by the National Sciences and Engineering Research Council of Canada, the Government of Quebec (Fonds FCAR) and the University of British Columbia. The authors wish to thank the following organizations for their generous assistance in providing material for the experiments: Empire Iron Works Ltd., Delta, B.C., and Coast Steel Fabricators Ltd., Port Coquitlam, B.C., (structural steel and bolts), Spae-Naur Inc., Kitchener, Ont., (samples of disc spring washers) and Haynes International, Inc,. Kohomo, Illinois (cobalt-base alloy plate material). Thanks also have to be directed to Harald Kullmann, graduate student, who designed and built the full scale testing frame used in this study and to Dr. Pak Ko, from the Institute for Mechanical Engineering, Tribology and Mechanics, of the National Research Council Canada, Vancouver, B.C., for his valuable comments and suggestions on the tribological aspect of the studied connection.

REFERENCES

Aristizabal-Ochoa, J.D., 1986, "Disposable Knee Bracing: Improvement in Seismic Design of Steel Frames", ASCE, Journal of Structural Engineering, **112**, July, p. 1544-1552.

Elsesser, E., 1986, "A Survey of Seismic Structural Systems and Design Implications", ATC-17, Proceedings of a Seminar and Workshop on Base Isolation and Passive Energy Dissipation, San Francisco, Ca, March, p. 51-62.

FitzGerald, T.F., Anagnos, T., Goodson, M. and Zsutty, T., 1989, "Slotted Bolted Connections in Aseismic Design for Concentrically Braced Connections", Earthquake Spectra, **5**, May, p. 383-391.

Foutch, D.A., Goel, S.C. and Roeder, C.W., 1987, "Seismic Testing of Full-Scale Steel Building - Part I", ASCE, Journal of Structural Engineering, **113**, November, p. 2111-2129.

Grigorian, C.E., Yang, T.-S. and Popov, E.P., 1992, "Slotted Bolted Connection Energy Dissipators", Report no UCB/EERC-92-10, College of Engineering, University of California, Berkeley, Ca., July.

Pall, A.S. and Marsh, C., 1982, "Response of Friction Damped Braced Frames", ASCE, Journal of the Structural Division, **108**, June, p. 1313-1323.

Popov, E.P. and Black, R.G., 1981, " Steel Struts Under Severe Cyclic Loadings", ASCE, Journal of the Structural Engineering Division, **107**, September, p. 1857-1881.

Roeder, C.W. and Popov, E.P., 1978, "Eccentrically Braced Steel Frames for Earthquakes", ASCE, Journal of the Structural Division, **104**, March, p. 391-412.

Roik, K., Dorka, U. and Dechent P., 1988, "Vibration Control of Structures Under Earthquake Loading by Three-Stage Friction-Grip Elements", Earthquake Engineering and Structural Dynamics, **16**, p. 501-521.

SEAOC, 1990, "Recommended Lateral Force Requirements and Commentary", Seismology Committee - Structural Engineers Association of California (SEAOC), Sacramento, Ca.

Seki, M., Katsumata, H., Uchida, H. and Takeda, T., 1988, "Study on Earthquake response of Two-Storied Steel Frame with Y-Shaped Braces", Proceedings of Ninth World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan, August, **IV**, p. 65-70.

Rabinowicz, E., 1965, "Friction and Wear of Materials", John Wiley and Sons, Inc. New York, N.Y.

Whittaker, A.S., Bertero, V.V., Aktan, H.M. and Giacchetti, R., 1989, "Seismic Response of a DMRSF Retroffited with Friction-Slip Devices", Proceedings of the 1989 EERI Annual Meeting, San Francisco, Ca.

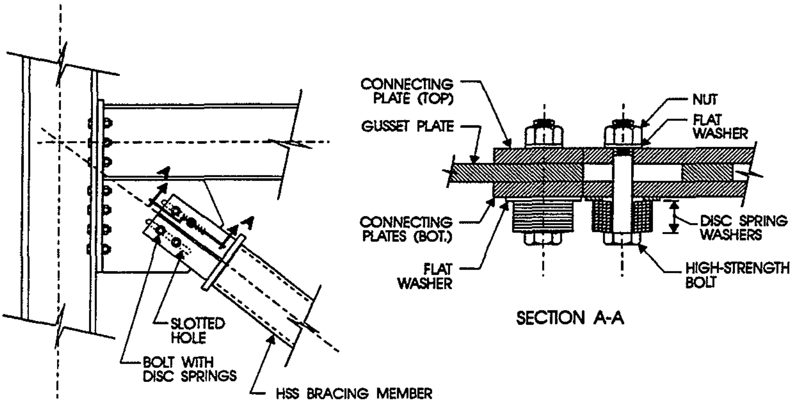


Fig. 1. Proposed FCBF system.

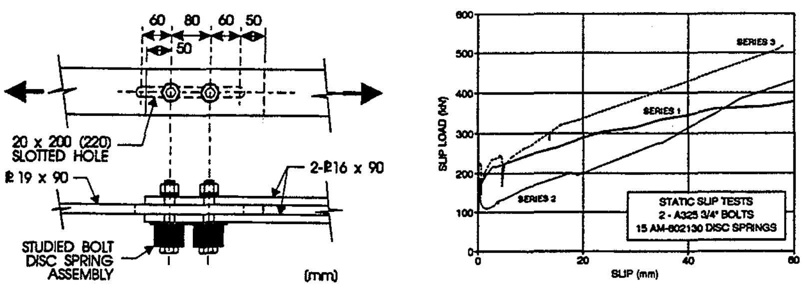


Fig. 2. Static slip tests: a) sample, b) typical response.

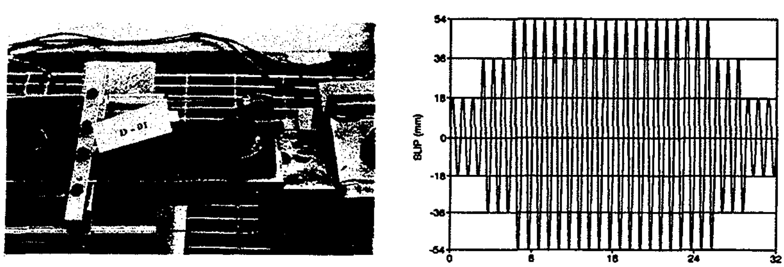


Fig. 3. Dynamic cyclic tests: a) specimen before testing, b) loading history.

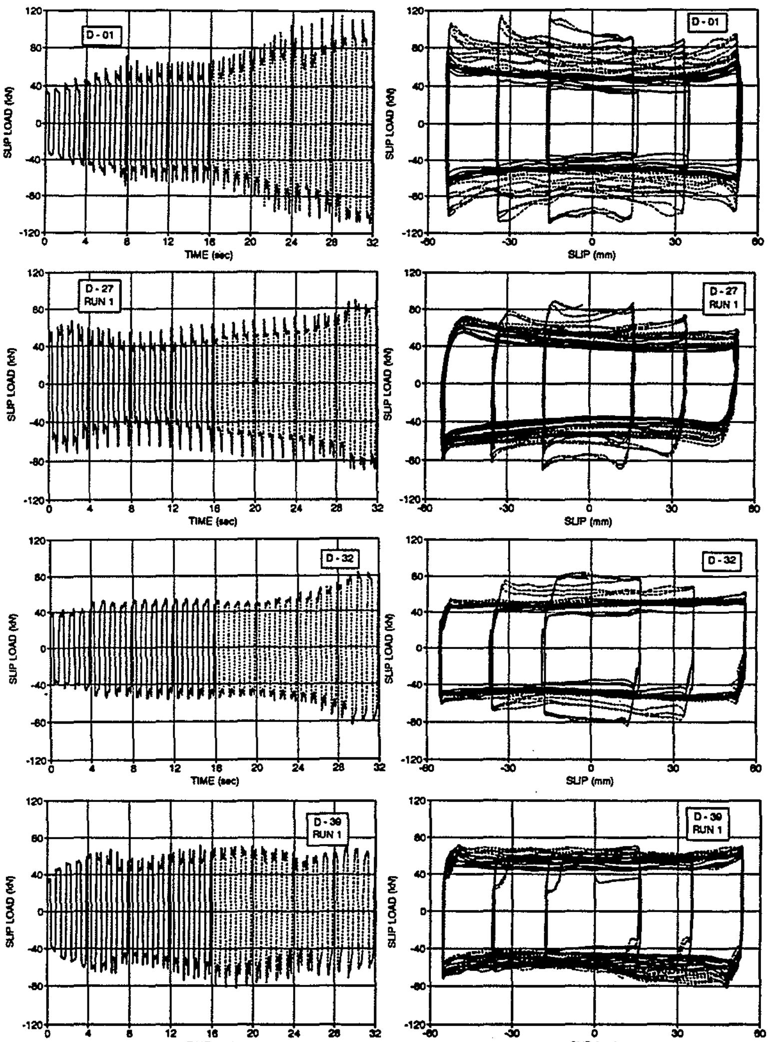


Fig. 4. Dynamic cyclic tests: slip load time history and slip load to relative slip relationships under 1.0 Hz cyclic loading.

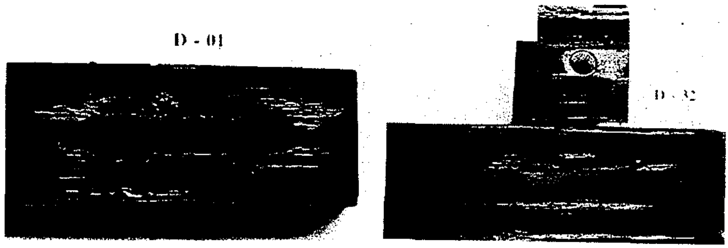


Fig. 5. Dynamic cyclic tests: interior (gusset) plate after testing a) specimen D-01, b) specimen D-32.

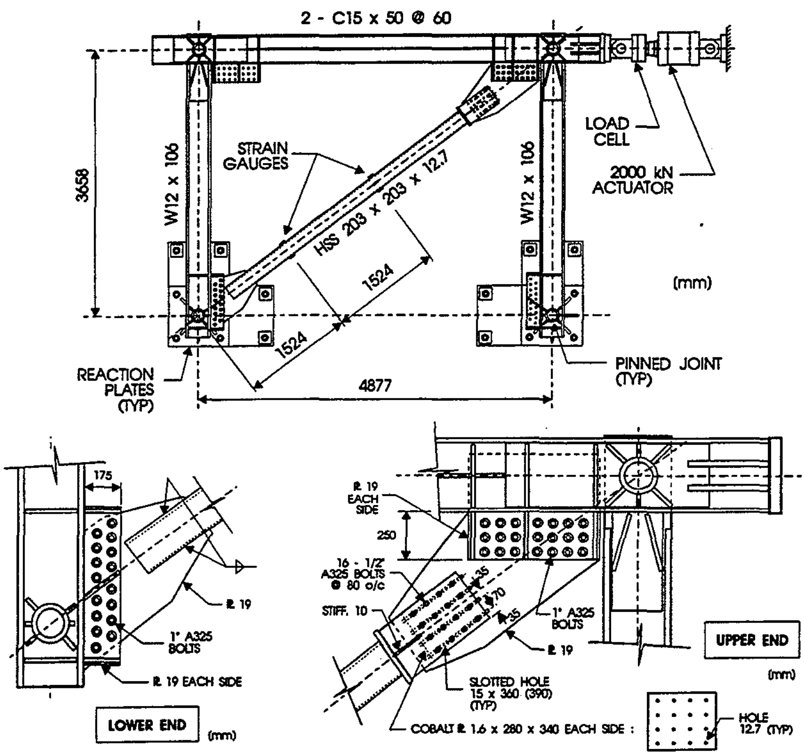


Fig. 6. Full scale braced frame testing: details.

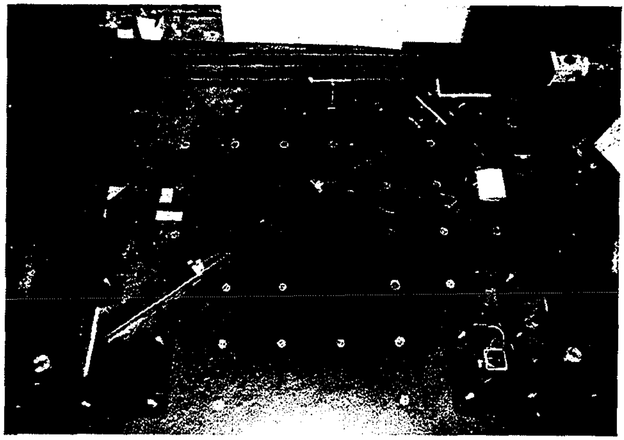


Fig. 7. Full scale braced frame testing: testing setup.

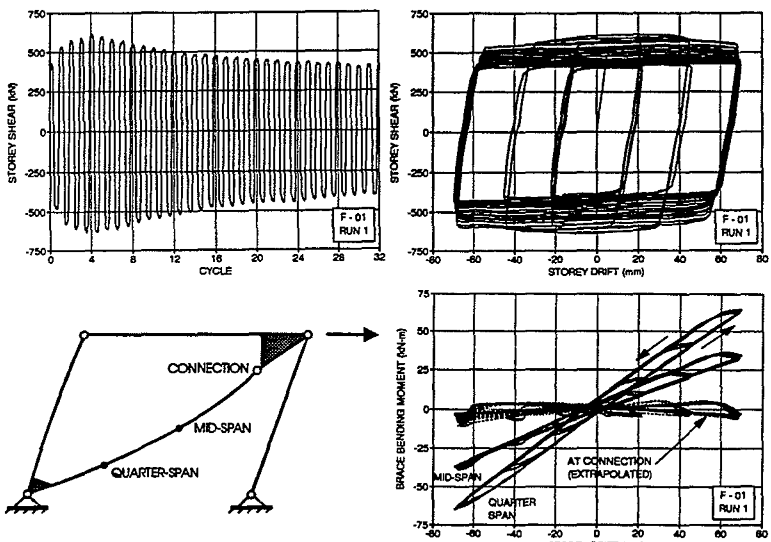


Fig. 8. Full scale braced frame: a) storey shear to storey drift relationship, b) bending moments in the bracing member.

THE WORLD TRADE CENTER BOMBING OF 26 FEBRUARY 1993
A BRIEF CASE HISTORY COLUMN STABILITY AND DAMAGE

Katherine E. Hill, Leslie E. Robertson, Saw-Teen See
Leslie E. Robertson Associates
New York, New York

The FBI tagged it as *"... the largest by weight and by damage of any improvised explosive device that we've seen since the inception of forensic explosive identification ... and that's since about 1925."*

This bomb was detonated immediately against the south wall of Tower 1 of the World Trade Center with the clear intent to bring down the tower, but the 110 stories of steel and concrete shook off the resulting blast as though it were a trivial event. The impact on the building was so slight that the instrumentation package, located high in Tower 1, was not even triggered by the blast, while the dynamic effects of a moderate wind storm a few days later were recorded. The Twin Towers may have survived the most vicious assault to date with their structure virtually unscathed, but the devastation outside of the towers was considerable.

Within the below-grade caverns surrounding the footprint of the towers, all that remained of a portion of the parking garage was a crater of nearly unbelievable proportions. Steel columns once braced by the closely-spaced parking levels stood naked as high as sixty-eight feet (21 m) without definable lateral support. Two levels of reinforced concrete slabs, generally eleven inches (280 mm) thick, lay in a twisted mass atop the refrigeration equipment below. At the crater's edge, the slabs had sheared free of their supporting columns, settling several feet to form "ski jumps" into the hole. Elsewhere, multi-ton portions of concrete were literally dangling from reinforcing steel.

The surrealistic tangle of concrete and steel, interspersed with imploded automobiles and snaking conduits, was molded by the halogen and tungsten lighting into an incoherent mass more believable as science fiction than as the reality of the World Trade Center.

The bomb was detonated at 12:18 pm on Friday, 26 February. By 2:00 pm LERA had received a telephone call from Port Authority requesting the presence of our engineers for what was then believed to be "a transformer room fire at the World Trade Center". Three engineers and one of our Partners boarded a police car for a wild ride to the site. It soon became apparent that the event was far more serious than a transformer fire, bringing two more Partners to the site.

BACKGROUND

While most of us associate the World Trade Center with the twin 110-story towers, it should be remembered that the immediate project also includes a 22-story hotel and four low-rise buildings (Figure 1). Located below the sixteen acre plaza are a Concourse Level with shops and restaurants, and six sub-grade levels (B1 through B6) used for parking, storage, services and mechanical rooms (Figure 2).

All of the floor framing both inside and outside the footprint of the towers is carried on steel wide-flange columns. The Plaza and Concourse levels consist of concrete slabs on profiled metal deck, composite with steel beams and girders. The B1 through B5 levels below are typically 11 inch (280 mm) thick concrete flat slabs with 4 inch (100 mm) deep drop panels. The B6 level is a slab-on-ground. Steel shear heads connect the concrete flat slab construction to the steel columns.

In the immediate vicinity of the blast, the B3 and B4 levels had been left open to provide the needed vertical clearance for the refrigeration equipment at the B5 level below. This equipment provides primary service for the entire World Trade Center project. The B1 and B2 levels above were used for building services and for parking, with original design live loads of 100 psf (4.8 kPa) and 50 psf (2.4 kPa), respectively. In addition, the B1 and B2 slabs act as horizontal diaphragms, providing two levels of bracing for the 6-story high slurry wall that surrounds the World Trade Center site and keeps the Hudson River at bay. It is this requirement that explains the use of such thick concrete slabs.

At the B5 level, the refrigeration machines rest on 24 inch (610 mm) thick concrete platforms. To prevent the transmission of vibrations to the surrounding building, these platforms are isolated from the remainder of the B5 level, which is of relatively light, steel-framed construction. The platforms are supported on concrete columns independent from the rest of the building. In addition, the B5 level is only a partial floor, with minimal connection to the building's lateral system. For these reasons, lateral bracing for the steel columns had been neglected at the B5 level during the course of the original design.

The columns in the area of the blast consist typically of two lifts. The heavier section comprising the lower lift was designed assuming lateral support at the B2 and B6 levels, giving it an unbraced length of about 42 feet (12.8 m). The upper lift is considerably lighter, spanning 16 feet (4.9 m) from the Concourse to B1, and 10 feet (3.1 m) from B1 to B2.

The site of the explosion has been prone to disaster, subjected to two fires in the same location during the construction phase. The first was of modest proportions, but was followed by a more severe fire in late 1971. The intensity of this latter fire raised concerns about the integrity of the steel shear heads connecting the B1 slab to the steel columns. The questionable shear heads were left buried in the drop panels, and our office designed new steel shear heads that were constructed immediately below the existing concrete work. These assemblies have performed well for more than twenty years.

DAMAGE TO THE TOWERS

Most of the damage to the towers was superficial in nature. Extensive damage to block partition walls destroyed the towers' air locks and enabled the smoke to reach even the highest floors via the stairwells and elevator shafts. Tower 2 suffered no structural damage but Tower 1 was closer to the explosion and did sustain minor localized structural damage (Figure 3). A two-piece steel diagonal weighing about 3100 pounds (13.9 kN) and located just a few feet from the blast was torn loose; half was thrown about forty feet (12 mm) into the tower while the other half rebounded into the crater. An adjacent diagonal was bent extensively; the upper end of its 1-1/4 inch (32 mm) thick plate was sheared off its connection to the column. The tower column between these diagonals developed a hairline crack below the weld at the splice, and its 2-3/4 inch (70 mm) thick plate has a slight bow on the explosion side of the column. Two spandrel members were also bent, and several connections for the filler beams along the inside wall of the tower were badly damaged.

The strength and ductility of the towers' structural systems were sufficient to overcome the intensity of the blast. The localized damage did not affect the overall structural integrity of the towers. This was confirmed by the dynamic response of the tower to a wind storm a few days later; the measured stiffness of the building after the explosion was identical to the stiffness measured prior to the bombing.

How did the Twin Towers escape catastrophic damage? Part of the answer lies in the structural design criteria developed for a project of such an immense scope and innovative nature. Strength-related design factors included resistance to hurricane-strength winds, sabotage, and the impact from a fully-laden intercontinental Boeing 707 aircraft (the largest plane in the air at the time).

The energy input from hurricane winds acting over 110 stories far exceeds the energy input from the impact of a Boeing 707; the energy from the impact of such an aircraft far exceeds the energy input from the explosion. In short, the bomb of 26 February was a small event in the expected life of the World Trade Center towers.

Another explanation for the limited tower damage is the mirror effect. The south wall of Tower 1 (Figure 3) consists of closely spaced columns with heavy spandrels and diagonals.

The steel columns are built-up box sections 32 inches (813 mm) on a side and ten feet (3050 mm) on center; plate thicknesses are up to 5 inches (127 mm). The spandrels and diagonals are also built-up sections of hefty proportions. Approximately 40% of the surface area of the tower wall in the subgrade levels is structural steel.

While much of the block infill wall was destroyed by the explosion, the totality of the construction acted like a mirror, reflecting much of the force of the blast away from the tower and to the south. The concrete slab construction in the garage levels was far more flexible than the wall of the tower, and the pressure wave simply followed the path of least resistance.

DAMAGE TO AREAS OUTSIDE OF THE TOWERS

The extent of the damaged area is shown in Figures 5 and 6. The van carrying the explosives was located immediately adjacent to Tower 1 at level B2. The destruction was greatest at this level, leaving a gaping hole 130 feet (40 m) wide. In addition to the hole, more than 25,000 square feet (2300 m^2) of the B2 level has been or will be demolished due to the extent of the damage.

While suffering less damage than B2, the B1 floor was left with a 55 foot by 85 foot (17 m by 26 m) hole, with another 16,000 square feet (1500 m^2) that has been or is scheduled for demolition.

Above B2, the Concourse had a hole of the order of 20 feet by 20 feet (6 m by 6 m). Above the Concourse, the Plaza slab had a small bulge.

The rubble from the demolished slabs fell on top of the refrigeration equipment, which was in turn supported by the heavy concrete platforms at the B5 level. The pressure vessels of the refrigeration plant and the heavy concrete construction below were able to carry the impact from the falling concrete, though much of the equipment and piping was damaged.

The damage extends over much of the site south of Tower 1. As the shock wave travelled through the B1 and B2 slabs, the restraint provided by the columns caused widespread shear failures and shear cracking at the columns. Such damage extends all the way to the slurry wall on the south edge of the site, 350 feet (107 m) away.

Because of the intrinsic resiliency of the columns supporting the construction outside of the towers, not one column failed from the impact of the explosion, from the tearing-off of the surrounding concrete or from the resulting loss of lateral support. One column, reported to have been bowed, was found to be straight, but several columns had easily-repaired damage to column splices.

In addition to the structural damage, damage to other systems was extensive: countless non-structural walls were reduced to rubble, the conduit banks and piping that had been ceiling mounted were draped everywhere, and vehicles far removed from ground zero were imploded by the force of the shock wave.

LATERALLY UNSUPPORTED COLUMNS

Following the blast, some of the wide flange columns spanned as much as 68 feet (21 m), from the Concourse to B6 levels, without "designed" lateral support. The theoretical *l/r* ratios for these columns reached to as high as 190; based on estimated column loads, the factor of safety under gravity loading was less than one. Thus, the most critical structural issue in the aftermath of the explosion was the need to stabilize those columns outside of the towers that had lost a portion of their lateral support with the collapse of the B1 and B2 floors.

While most of the columns in question supported only the Concourse and Plaza levels above, two of the most critical columns were supporting the 22-story Vista Hotel as well. The hotel had been evacuated in the wake of the blast, thus marginally reducing the live load, but any additional load caused by wind could be enough to buckle the columns. It was essential to provide emergency bracing to prevent the possibility of further collapse.

Time was of the essence for this emergency bracing. The damaged area had to be stabilized before the search for the missing people could continue, before the Federal Bureau of Investigation could initiate their investigation, before our engineers could inspect the damage at the lower levels, and before the contractors could start cleaning out the rubble to reach the primary refrigeration equipment.

Within twelve hours of the blast, we had prepared a schematic design for temporary bracing. Before departing the site in the small hours of Saturday morning, we had met with representatives of Karl Koch Erecting Company (KKE), who later fabricated and erected the temporary bracing, and arrived at a general concurrence of our bracing concept.

KKE suggested the use of TS 6x6 tubular sections (150 mm x 150 mm) for the bracing, and this section was ideal for the job. The bracing detail is shown conceptually in Figure 4. There were no shop connections, and field connections were by welding.

By Sunday afternoon, the first tubing had been delivered to the site and was poised to go into position.

The design and installation of the temporary bracing was broken into two phases, each based on a different set of assumptions. Phase I included all bracing needed to temporarily brace the columns to the extent that the FBI, search crews, inspection teams and construction crews could gain access to the lower levels. Phase II included all of the bracing needed over the long term to carry out the demolition and reconstruction process. Of course, considerable overlap existed between the two phases as access to various areas changed on a day-to-day basis.

The first step in designing the Phase I bracing was to determine as quickly as possible the capacity of the columns and the loads upon them. However, there was a great deal of uncertainty involved on both counts. The instability of the concrete work made it difficult to ascertain the remaining sources of lateral support in the field. The original design criteria for the plaza and subgrade levels could serve only as a starting point for the loads as the uses of these areas had evolved over time and a number of renovations had taken place.

The matter was further complicated by the fact that in the area around the hotel, the columns had been redesigned for three or four different proposed hotel structures. With each new design, transfer girders had been installed to accommodate the latest configuration. The existing hotel structure was then designed by another engineering firm and the available information about the hotel column loads was incomplete.

The capacity of the columns was further called into question because of the possible adverse consequences of the earlier fires. These columns had now been subjected to three devastating events, which gave rise to concern regarding the metallurgical characteristics of the columns.

Given the uncertainties described above in the magnitude of the column loading, we made the conservative assumption that the axial load on the columns was equal to the ASD capacity of each column section. The columns had originally been designed assuming lateral support at the Plaza, Concourse, B1, B2 and B6 levels. The lateral support from the B5 level had been neglected for reasons given earlier. Knowing the size of the column and the distance between floors, the development of column capacities was straightforward.

These ASD loads were then converted to LRFD loads by using 1.2Dead + 1.3Wind and assuming that all of the ASD capacity in excess of the dead load was wind load. This may seem somewhat contrived as not all the columns carried wind load. However, the number of transfer girders added made it difficult to determine which columns carried wind loads, and of what magnitudes. Much of the live load had been reduced by evacuating the areas concerned and the column capacities were sufficiently in excess of the calculated dead loads that the 1.4Dead load case never controlled. Using the wind load case guaranteed that the calculated ultimate load was on the conservative side and eliminated the immediate need to determine which columns carried significant levels of wind load and to what degree. It must be remembered that this work was done in just a few days' time, making elaborate analysis inappropriate.

The LRFD loads, then, were used in conjunction with the present AISC Code to determine the maximum unbraced length permissible for each column. Tempory bracing members were located accordingly.

PHASE I BRACING

For the design of the Phase I bracing, we made the most of all possible sources of lateral support. The damaged slab that surrounded many of the columns could not be counted on in the long run for lateral support, but for the short term it was assumed that as long as the damaged slab was left in place, it would provide a full measure of lateral support.

It was also recognized that even though the B5 Level had not been used as lateral bracing in the original design, the floor does provide considerable lateral support for many of the columns.

The debris from the collapsed B1 and B2 levels had landed on top of the refrigeration equipment and wedged itself around the columns. The tons of debris were obviously providing some measure of lateral support, as the columns were still standing despite having a theoretical factor of safety that was less than one. Perhaps for the first time in the history of building construction, the work designed by a building services engineer was stabilizing the structural system. The debris was located well above the B5 slab level, but its presence made the assumption of lateral support at the B5 level that much more reasonable.

In order to make such assumptions for the Phase I bracing, it was essential that the rubble and damaged slabs not be removed or shifted in any way until the columns had been further stabilized by the additional Phase II bracing.

Of concern was the fact that the PATH railroad system and several subway lines pass through the project. The vibrations produced by the trains carried the potential of dislodging precariously supported debris, some of which was providing essential lateral support for the columns. Port Authority's Engineering Department implemented a vibration monitoring system and organized a carefully-planned set of passages of trains. Fortunately, the levels of vibration associated with the passage of the trains turned out to be lower than other background vibrations. The PATH trains, then, were allowed to operate on the Monday morning immediately following the Friday-noon blast.

Another intangible consideration in the design of the temporary bracing was that the columns were encased or partially encased in concrete. This concrete had been used for fireproofing and had not been detailed to carry vertical load. No quantitative calculation could be made of the additional strength provided by the concrete because some of it had been damaged in the prior fires and by the blast, and significant amounts would have to be removed so that the bracing tubes could be welded directly to the steel columns. Still, the concrete encasement provided additional stiffness even where it did not carry axial load. It was reassuring to know that our calculated buckling loads (based on the steel section alone) were on the conservative side.

For Phase I, the assumptions of lateral support provided by the remaining damaged slabs and by the rubble at B5, combined with the switch from ASD to LRFD, and the use of non-prismatic buckling analysis (to account for the change in column section) made it possible to span the clear height from B5 to B1 without additional bracing. Thus Phase I bracing was confined to installing 9 pairs of tubes to replace support lost at the B1 level. This was the easiest and safest bracing to install at the time because the workers could be lowered through holes cut in the Concourse slab, rather than working from the unstable levels below.

PHASE II BRACING

The Phase II bracing was intended to provide all necessary lateral support to the columns so that the subgrade levels could be restored to their initial condition. This bracing was needed to replace the support provided by the damaged slabs and by the rubble at the B5 level, so that they could be demolished and removed. The additional 15 feet (± 5 m) of unbraced length associated with the loss of lateral support at B5 meant that many columns would need temporary bracing at both the B1 and B2 levels.

A total of about 80 pairs of tubes were called for in Phase II. The logistics of installing this many braces meant that one area would be braced and undergoing demolition before the bracing of another area could begin. The staging of the bracing/shoring/demolition operations became critical to the stability of the columns. LERA worked in close cooperation with the Port Authority's Construction Department and with the contractors to develop various scenarios for this work. Demolition schedules and proposed bracing layouts were evaluted on a day to day basis to ensure that a stable configuration was maintained. Where column bracing was delayed, the damaged slabs were shored in place to make the support they provided more reliable. The bracing calculations were constantly revised as new information arrived from the site and load take-downs were refined.

At this point in time, all of the rubble has been removed from the B5 level and repairs to the refrigeration equipment are moving ahead. Reconstruction of the B2 level is nearly complete and work continues on the bracing and demolition of the B1 level.

IN CONCLUSION

The men and the women of LERA have responded before to crisis situations involving the very lives of important buildings. Still, this catastrophe was in a class of its own. It was wonderful to see the office shift two gears up from its usual hectic pace in order to respond to the needs of the World Trade Center. There is no way that anyone can "buy" such dedication; it comes from the heart, not the pocketbook.

Now, with the worst of the crisis behind us and the work week down from the 70- to 80-hour weeks that followed the bombing, all at LERA have a justifiable level of pride in the level of work accomplished. While carrying on with all of the projects then in our office, we had added the professional services associated with the structural evaluation and the staging, shoring, demolition and reconstruction operations.

Of course, we received invaluable support from the Port Authority and from the several contractors involved in the work. Still, it is a good feeling when you know that you have done a good job ... and that your efforts have made life better for tens of thousands of people.

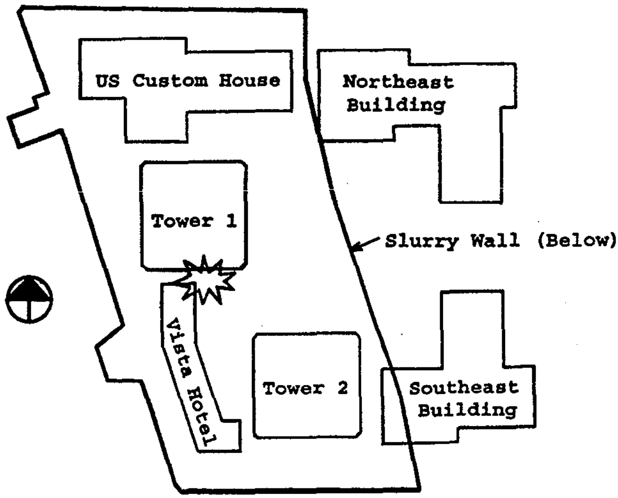


FIGURE 1: Plan view of the World Trade Center Site

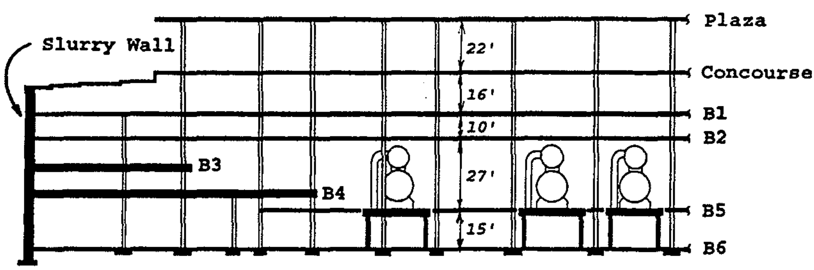


FIGURE 2: Section through sub-grade levels at the site of the explosion

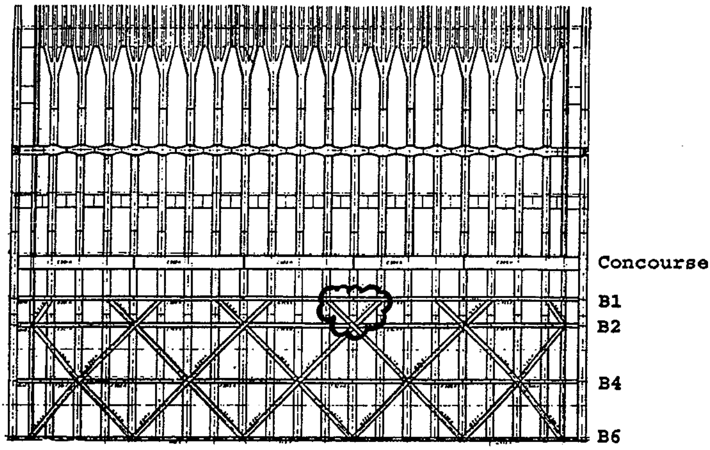


FIGURE 3: South elevation of Tower 1, indicating damaged area

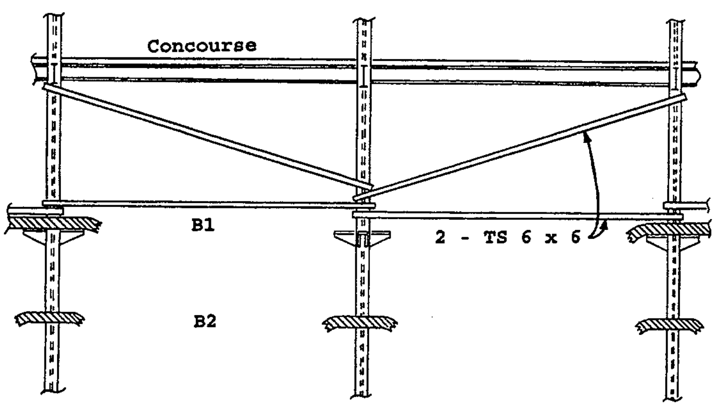


FIGURE 4: Schematic detail of emergency bracing

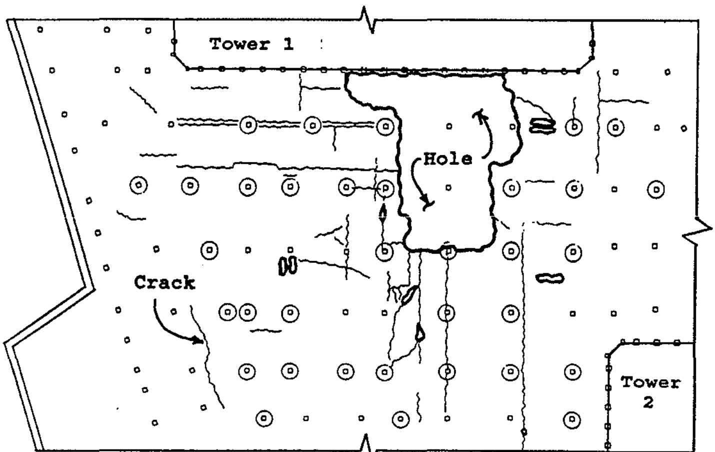


FIGURE 5: Damage at the B1 level

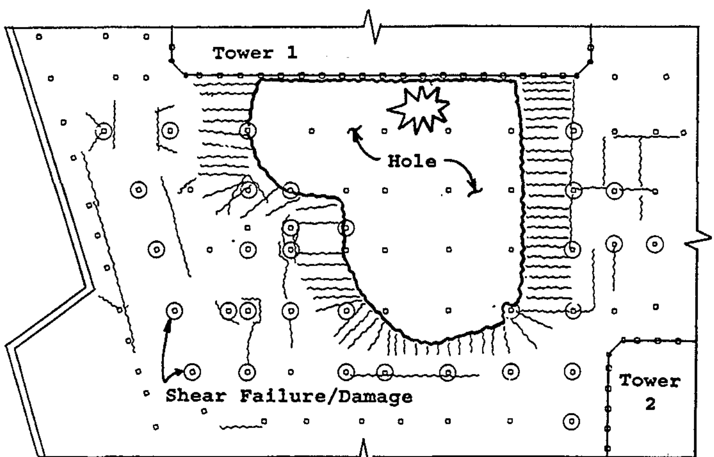


FIGURE 6: Damage at the B2 level

"Is Your Structure Suitably Braced" Panel Discussion

The following pages are a transcription of the Panel Discussion Question & Answer session. Conference participants were asked to submit questions that they would like the panel to address. Prof. Reidar Bjorhovde, University of Pittsburgh served as Moderator.

The panelists were:

Bill Baker,
Skidmore, Owings & Merrill, Chicago

Dick Kaehler,
Computerized Structural Design, Inc.

Russell Q. Bridge,
University of Sydney

Joseph A. Yura,
University of Texas, Austin

* Absent: Theodore V. Galambos
University of Minnesota

Q1: *WHAT UNBRACED LENGTH SHOULD BE USED FOR THE DESIGN OF GIRTS? SOME PEOPLE ARGUE THAT SAG RODS LATERALLY BRACE THE GIRTS. OTHERS ARGUE THAT SINCE THE COMPRESSION FLANGE IS NOT BRACED, THE UNBRACED LENGTH IS THE ENTIRE SPAN.*

YURA: WHERE IS THE SAG ROD?
SOMEONE SAID IT IS IN THE MIDDLE OF THE SPAN. IF IT'S PROPERLY ANCHORED AT THE TOP, ETC., YOU WOULD GENERALLY USE THE UNBRACED LENGTH AS THE DISTANCE BETWEEN THE SAG RODS, AS LONG AS THE BRACING, I WOULD SAY, IS AT LEAST IN THE MIDDLE OR SOMEWHAT TOWARDS THE INSIDE EDGE OF THE GIRT.

Q2: *WHAT IS YOUR RECOMMENDATION ON THE K FACTOR WHEN A SECOND-ORDER ELASTIC ANALYSIS IS PERFORMED? CAN K BE TAKEN AS 1.0 IN ALL THESE CASES?*

BRIDGE: THE USE OF THE K FACTOR = 1.0 IS VERY COMPLEX FOR SAFETY AS YOU PROBABLY REALIZE, AND SOME SPECIFICATIONS ALLOW IT, AND OTHERS WILL STILL USE THE K FACTOR. THE WAY THAT I SEE IT IS THAT IF YOU USE THE K FACTOR = 1.0, IT MEANS THAT YOU HAVE TO TAKE ACCOUNT OF THE OUT-OF-PLUMBNESS OF THE STRUCTURE IN SOME OTHER WAY, AND THERE ARE WAYS OF DOING THIS. ONE WAY IS THE USE OF NOTIONAL HORIZONTAL FORCES, AND YOU CAN CALIBRATE THE NOTIONAL HORIZONTAL FORCE SO THAT YOU GET, USING A BEAM-COLUMN EQUATION, THE SAME COLUMN CURVE THAT YOU WOULD GET IF YOU WOULD USE THE EFFECTIVE LENGTH FACTOR -- SO IT CAN BE DONE. BUT YOU HAVE TO BE VERY CAREFUL. IF YOU CAN'T MIX SPECIFICATIONS, WHATEVER IS IN THE AISC-LRFD, IF THAT SAYS USE THE EFFECTIVE LENGTH, YOU USE IT BECAUSE THE BEAM-COLUMN EQUATIONS HAVE BEEN CALIBRATED FOR THAT. IF YOU ARE USING THE CANADIAN CODE, YOU USE THEIR APPROACH. IN OUR CASE, IN AUSTRALIA, WE HAVE OUR APPROACH, TOO. BUT YOU CAN'T MIX AND MATCH THEM.

YURA: IF YOU ARE USING THE AISC SPECIFICATION, AND YOU ARE USING A SECOND-ORDER ELASTIC STRUCTURE ANALYSIS, YOU MUST USE A K-FACTOR. THE SSRC THIRD EDITION INDICATED THAT YOU CAN DO A K = 1.0. IF YOU CHECK THE 4TH ED., YOU'LL SEE THAT'S BEEN REMOVED AND YOU STILL MUST USE A K FACTOR BECAUSE THE CHECKS THAT WE HAD DONE INDICATED THAT IN HIGH GRAVITY LOAD SITUATIONS, THAT PROCEDURE WILL NOT CUT IT.

Q2: *WHEN IS THE CASE WHERE 2ND ORDER INELASTIC FRAME ANALYSIS SHOULD BE PERFORMED?*

KAEHLER: WHEN YOU CAN GET YOUR HANDS ON A PROGRAM THAT CAN DO THAT FOR YOU, I GUESS.

Q3: *FOR PROF. YURA:*
WE HAVE HEARD THAT YOU ARE PREPARING A BRACING DESIGN MANUAL FOR AISC – WHEN WILL IT BECOME AVAILABLE?

YURA: YES, I AM WORKING ON IT AS HARD AS I CAN AND STILL TRYING TO KEEP A FAMILY. I'M CURRENTLY WAITING ON SOME APPROVAL FOR MOST OF THE TORSIONAL STUFF ON THE BEAMS TO BE CLEARED BY THE TEXAS HIGHWAY DEPARTMENT WHO SPONSORED MUCH OF THE WORK. I AM IN THE PROCESS OF DRAFTING IT, AND IT SHOULD COME OUT BY THE END OF THE SUMMER THROUGH AISC.

Q4: *CONSIDER A BUILDING CONSTRUCTED OF COLUMNS THAT SUPPORT A ROOF ALONE. THE ROOF DECK IS USED TO TRANSFER LATERAL LOADS TO X-BRACING ON THE PERIMETER WALLS. THEREFORE, THE INTERIOR COLUMNS ARE BRACED AND A K = 1.0 SHOULD BE USED. IF THE PERIMETER X-BRACING IS REPLACED WITH PORTAL FRAMES HAVING THE SAME LATERAL LOAD-CARRYING CAPACITIES AS THE X-BRACES, MUST A ΣP ANALYSIS BE USED? IF SO, DO THE INTERIOR COLUMNS HAVE ANY CAPACITY IN THE ΣP ANALYSIS? NOTE: A LEANER COLUMN ANALYSIS WAS PERFORMED TO DETERMINE THE TOTAL LATERIAL LOAD ON THE X-BRACE OR THE PORTAL FRAME.*

BAKER: I'M OF THE OPINION THAT THERE ARE SUCH THINGS AS A BRACED FRAME; YOU CAN GET YOUR LATERAL STABILITY FROM A MOMENT FRAME OR FROM DIAGONAL BRACING. I CAN DESIGN A MOMENT FRAME WHICH HAS THE SAME DRIFT AS A DIAGONAL BRACING AND ONCE YOU GET THIS TRANSLATION THEN YOU HAVE TO ADDRESS STABILITY. THE FACT THAT IT'S GENERALLY IGNORED IN DIAGONAL BRACING IS BECAUSE THE DRIFT IS SMALL. TO MY MIND THEY ARE THE SAME PROBLEM -- YOU HAVE STABILITY FORCES WHICH HAVE TO BE TRANSFERRED TO EITHER THE PERIMETER MOMENT FRAME, IF IT'S A MOMENT FRAME OR THE PERIMETER DIAGONAL BRACING, IF IT'S A BRACING SYSTEM. I SEE THEM AS NO DIFFERENT -- I THINK IT'S ONE PROBLEM.

YURA: I AGREE TOTALLY WITH THAT AND THE SUMMATION OF P CONCEPT IS JUST ANOTHER WAY OF DEALING WITH BRACING. AND SO IF IT'S FLEXURAL FRAME, THAT'S JUST AN EASY WAY TO GET THE PROPER AMOUNT OF STIFFNESS IN THE SYSTEM -- WHEN IT'S DIAGONAL IT'S JUST EASIER TO PUT ON THE BRACE FORCES -- SO BOTH WOULD BE THE SAME.

Q5: *PROF. YURA STATED THAT THE INFLECTION POINT ON A BEAM CANNOT BE COUNTED ON AS A BRACE POINT. COULD MR. YURA PLEASE EXPLAIN. WHAT IS THE UNBRACED LENGTH FOR A BEAM THAT HAS MOMENT REVERSALS?*

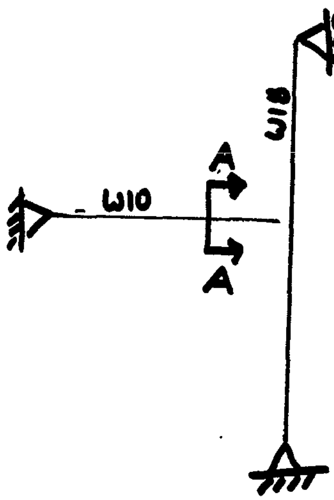
YURA: FIRST, THE INFLECTION POINT -- I'VE GOT TO MAYBE ACTUALLY ASK FOR YOU TO PICTURE SOME CASES -- AND LET'S JUST TAKE THE CASE OF COMPLETE REVERSE MOMENT, LIKE YOU WOULD HAVE IN AN EARTHQUAKE, SO IF THERE IS AN INFLECTION POINT RIGHT IN THE MIDDLE. IF YOU LOOK IN THE AISC OR ANOTHER PUBLICATION YOU WILL FIND C_b FACTORS GIVEN FOR THAT CASE ARE EQUAL TO A VALUE OF 2.3.

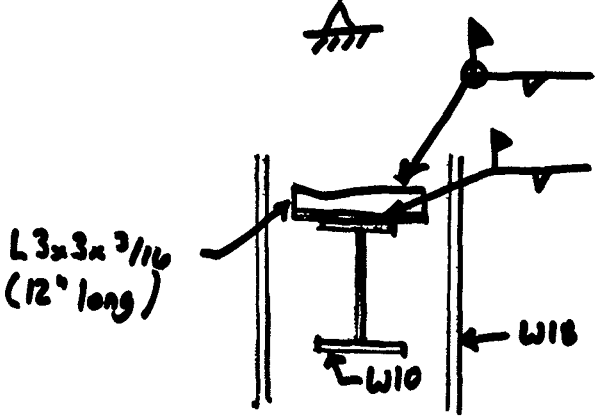
IF YOU ASSUME THE INFLECTION POINT WAS A BRACE POINT OVER THE FULL SPAN, YOU SHOULD USE 1.75 OVER HALF THE SPAN. NOW YOU FIND OUT THAT DOESN'T GIVE THE SAME ANSWER. THE REASON IT DOESN'T GIVE THE SAME ANSWER IS IN THE MIDDLE YOU HAVE A PURE TWIST. THE ONE FLANGE MOVES ONE WAY, THE OTHER FLANGE MOVES THE OTHER WAY. SO THE INFLECTION POINT IN THE MIDDLE OF THE WEB, BY THE WAY, IT DOESN'T MOVE -- BUT THERE'S A PURE TWIST. AND SO THE INFLECTION POINT IS NOT A BRACE POINT AND SO IF YOU WANT TO GO FROM ONE END TO ANOTHER AND PASS AN INFLECTION POINT, YOU HAVE TO FIND SOME SOLUTIONS.

IN THE BRACING MANUAL, THERE ARE CONTAINED SOLUTIONS FOR THOSE CASES -- TYPICALLY LIKE YOU HAVE A TOP-FLANGE COMPLETELY BRACED AND YOU'VE GOT NEGATIVE MOMENTS. WE CAN GIVE C_b FACTORS FOR THAT SITUATION THAT GIVE YOU VERY EXACT SOLUTIONS. AND SO WE HAVE THOSE DONE FOR UPLIFT, WE HAVE FOR ANY MOMENT RATIO AND ANY NUMBER OF BASIC MOMENT DIAGRAM. SO YOU DON'T HAVE TO WORRY ABOUT THE INFLECTION POINT AS A BRACE POINT -- YOU'RE GETTING A SOLUTION THAT'S BASICALLY

EXACT FOR THAT SITUATION. IN GENERAL YOU CANNOT USE AN INFLECTION POINT AS A BRACE POINT.

Q6: *DOES THIS W10 TORSIONALLY BRACE THE W18?*



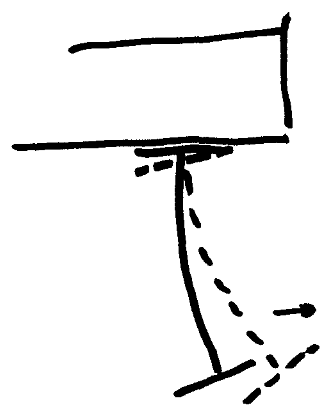


Section A-A

KAEHLER: I THINK THE DETAILS YOU HAVE HERE GIVE YOU A PRETTY GOOD SHOT AT IT -- BUT YOU WOULD HAVE TO AN ACTUAL CALCULATION OF THE STRUCTURAL STIFFNESS OF THE W10 WITH THE VARIOUS DIMENSIONS INVOLVED. BUT BASED ON THE WORK WE'VE DONE, I THINK WE'VE GOT ANGLES SPREADING THE FLEXURE BACK TO BOTH FLANGES -- I THINK YOU'VE GOT A REAL GOOD SHOT AT IT.

YURA: YES, I AGREE WITH THAT -- YOU JUST TAKE THE TORSIONAL BRACING REQUIREMENTS WHICH I GAVE, AND THEN IT'S JUST A MATTER OF EVALUATING THE SPRING STIFFNESS, WHICH IN THIS CASE WOULD BE THE WEAK AXIS BENDING OF THE W SHAPE. BUT I WOULD LIKE TO EMPHASIZE THAT ANGLE IS VERY VERY IMPORTANT. I'LL GIVE YOU AN INSTANCE -- IF YOU TAKE A JOIST AND JUST PUT A SIMPLE SEAT ON THAT W SHAPE AND YOU TRY TO PULL IT OVER, THE PLATE JUST BENDS. AS THE MEMBER TRIES TO TWIST, IF YOU JUST ADD A VERY SHORT PIECE, YOU'RE GOING TO HAVE DISTORTION OF THE CROSS-SECTION -- YOU MUST, IF YOU DON'T WANT TO EVALUATE DISTORTION, YOU HAVE TO PREVENT IT.

Q7: *FOR PROF. YURA -- FOR TORSIONALLY BRACED BEAMS -- SHOULD TIPPING OF WIDE FLANGE BE CHECKED, i.e.,*



YURA: YES, I SEE THE SKETCH, SO I ASSUME THAT THE MEMBER IS NOT ATTACHED TO THE FLANGE, SO THAT THERE COULD BE AN ANSWER TO THE QUESTION ON THAT PREMISE BECAUSE OTHERWISE, IT'S JUST A TORSIONAL BRACING SITUATION. THE TIPPING EFFECT HELPS OUT, NORMALLY WITH REGARD TO THAT PARTICULAR SITUATION, WHICH WAS ALSO, BY THE WAY, TESTED. THE TIPPING EFFECT IN FACT, HELPS WHEN THE MEMBER TRIES TO TWIST, THE ACTUAL FORCE ON THE TOP PUSHES ON THE EXTERIOR OF THE FRAME AND TRIES TO STRAIGHTEN IT OUT. WE'VE DONE TEST ON TWIN BEAMS WHEN WE'VE JUST PUT A WOOD PALLET BETWEEN TWO BEAMS, SO ONLY THE FRICTION IS PREVENTING THE LATERIAL MOVEMENT, AND COMPARED TO PUTTING A KNIFE EDGE AT THAT LOCATION, THE CAPACITY WAS DOUBLED. NOW THE HARD PART OF INCLUDING THAT HELP DEPENDS ON THE ROLL OF THE FLANGE AND HOW MUCH IT CAN ROLL BEFORE YOU CAN REALLY GET THE CONTACT. GOOD WORK HAS BEEN DONE IN GERMANY ON THIS PARTICULAR DETAIL, WHICH GENERALLY PROVES THAT THE MEMBER CAN BUCKLE, BUT YOU HAVE TO WATCH THE ROLL. THIS PARTICULAR ISSUE THAT WE CALL "THE TIPPING EFFECT" IS ACTUALLY A HELP -- NOT A HURT.

Q8: A ROOF IS SUPPORTED BY OPEN WEB STEEL JOIST THAT ARE BRACED WITH CROSS BRIDGING;

1) IS THERE A LIMIT ON THE ANGLE (AS DETERMINED BY JOIST SPACING AND DEPTH) OF THE BRACING WHERE IT BECOMES INEFFECTIVE?

KAEHLER: IT SEEMS THAT THE MORE IMPORTANT THING IS THAT THE BRIDGING HAS GOT TO BE ANCHORED, MORE THAN THE CROSS-BRIDGING ANGLES. I DON'T KNOW THE MAGIC CUT-OFF FOR X-BRACING, BUT IT'S GOING TO BECOME UNECONOMICAL AS THE ANGLES BECOME UNDER 30 DEGREES, I WOULD GUESS.

Q8: 2) IF A WIDE FLANGE SECTION REPLACED A JOIST OVER A COLUMN LINE (SO IT CAN BE PART OF WIND RESISTING SYSTEM), CAN THE CROSS BRIDGING BE USED TO BRACE THE BEAM?

KAEHLER: I DON'T SEE WHY NOT, ALTHOUGH OFTEN THE BEAM MIGHT BE A DIFFERENT DEPTH, BUT IT'S VERY COMMON TO SEE KICKERS FROM THE JOIST LINES DOWN TO OPEN WEB AND SOLID WEB GIRDERS.

YURA: THE BRIDGING DOESN'T KNOW WHETHER IT IS BRACING THE JOIST OR A BEAM.

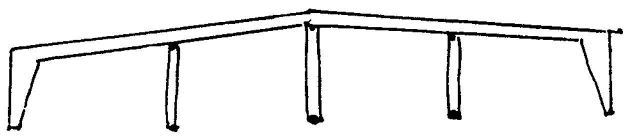
Q8: 3) COULD THE BEAM BE USED AS AN ANCHOR FOR HORIZONTAL BRACING OF THE BOTTOM OF THE JOISTS?

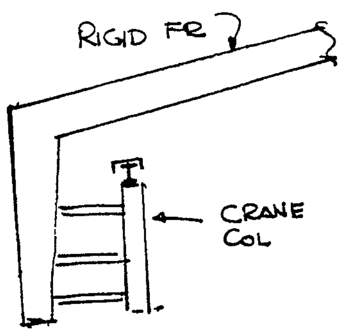
KAEHLER: GENERALLY THE BEAM WON'T BE SIZED FOR THE SORT OF BRACING FORCES THAT ARE INVOLVED, SO THE BRIDGING OUGHT TO BE ANCHORED ELSEWHERE. IF THERE IS ANCHORAGE AT THE END SOMEWHERE, THEN THERE IS NOTHING WRONG WITH TRANSFERRING FORCES ACROSS THE BEAM OR WHEREVER NEED BE, BUT ANCHORAGE IS WHAT'S IMPORTANT FOR MAKING THE JOISTS WORK.

YURA: I THINK THAT WAY I SEE MOST OF THE BRIDGING BEING ANALYZED, IT'S BEING USED AS LATERAL BRACING. ACTUALLY IT'S MORE EFFECTIVE THAN A DIAPHRAGM OR CROSS BRACING, AND THERE YOU DON'T NEED TO ANCHOR IT. YOU'RE JUST TAKING ONE BEAM AND BRACING IT BY ANOTHER. IF YOU APPROACH IT AS A LATERAL BRACE, IT'S GOT TO GO SOMEPLACE, BUT ALL OUR TWIN BEAM STUDIES -- THEY'RE NOT ANCHORED ANY PLACE, AND

THEY WORK FINE. SO I THINK THE BRIDGING SHOULD BE VIEWED MORE AS A TORSIONAL BRACE -- NOT AS A LATERAL BRACE.

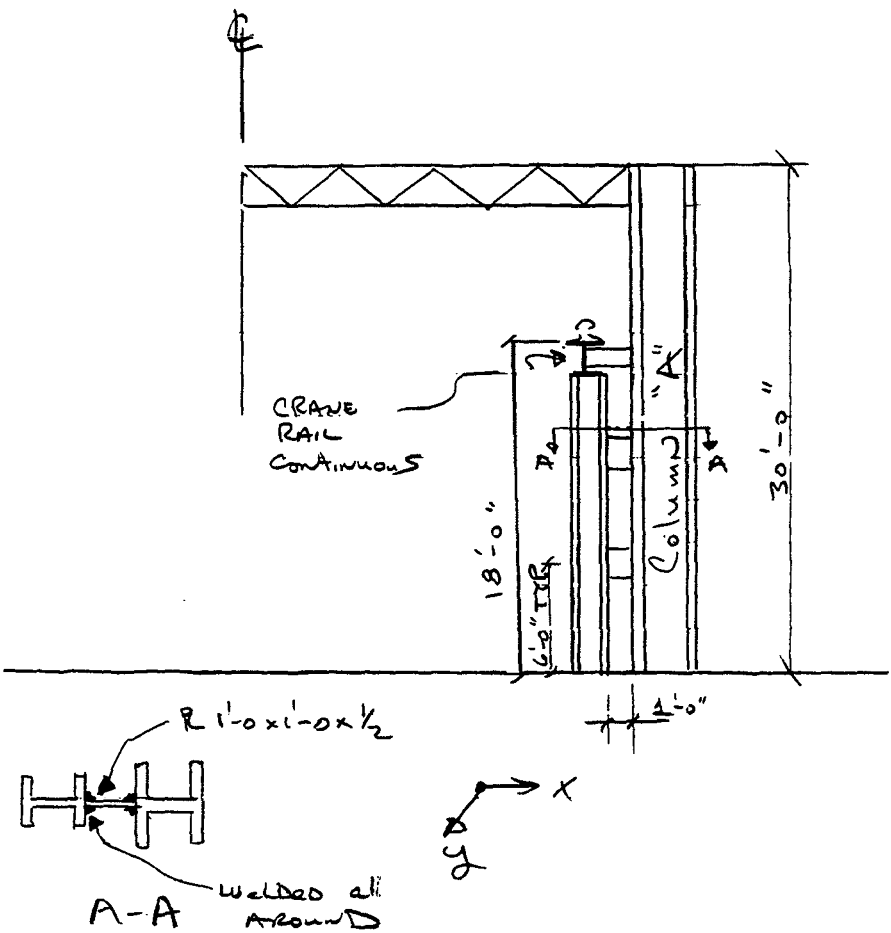
Q9: *FOR PROF YURA:*
CAN WE COUNT ON LEAN ON BRACING FOR THE FOLLOWING:





YURA: IN TERMS OF DESIGN, THE ANSWER WOULD BE YES, BUT YOU HAVE TO WATCH THE SUMMATION BECAUSE YOU HAVE TWO DIFFERENT HEIGHT COLUMNS, AND IF YOU RECALL THAT PROBLEM I DID WHERE THE ONE THING WAS STANDING ON TOP OF EACH OTHER, THE WAY YOU WOULD VIEW IT, IS JUST PUT ON THE P-Δ KIND OF THINGS, AND THEN JUST SEE WHAT THE EQUIVALENT LOAD WOULD BE AT THE TOP.
YOU HAVE TO DO A LITTLE MANIPULATION, BUT THERE IS NOT REASON WHY THAT THING CAN'T LEAN ON THE FRAME.

Q9A: *WHAT EFFECTIVE LENGTH (L_x AND L_y) SHOULD BE USED FOR THIS COLUMN SUPPORTING THE CRANE RAIL?*



KAEHLER: THE COLUMN WOULD NORMALLY BE ORIENTED WITH THE WEB PARALLEL TO THE CRANE RAIL, AND IT WOULD BE OUR PRACTICE NORMALLY TO PUT A CENTRALLY PINNED CONNECTION BETWEEN THE BATTEN AND THE CRANE COLUMN IN THAT SORT OF SITUATION, RATHER THAN TRYING TO DEVELOP MOMENT CONTINUITY TO THE WEB OF THE CRANE COLUMN. SO WE WOULD BE COMFORTABLE TAKING KL_y AS THE DISTANCE BETWEEN THE BATTENS, AND KL_x YOU ARE RELYING SOME KIND OF CRANE BRACING IN THE PLANE OF THE BEAMS. IF YOU WERE COUNTING ON SOMETHING FROM THE BATTENS, IT WOULD BE CERTAINLY WELL OVER 1.0, OTHERWISE K WOULD BE 1.0 TO THE TOP OF THE CRANE RAIL OR TO THE TOP OF THE BRACING HEIGHT.

Q10: *THERE ARE COUNTLESS STRUCTURES IN USE TODAY THAT UTILIZE OUTSET GIRTS (ATTACHED TO OUTER FLANGE ONLY) AS WEAK AXIS BRACING FOR WIDE FLANGE COLUMNS.*

TO MY KNOWLEDGE THERE HAVE BEEN NO DOCUMENTED CASES OF FAILURE OF THIS TYPE OF SYSTEM ATTRIBUTED TO TORSIONAL BUCKLING.

IT HAS BEEN SHOWN THAT FOR EQUAL UNSUPPORTED LENGTHS (WEAK AXIS AND TORSIONAL) THAT TORSIONAL BUCKLING WILL CONTROL ONLY AT VERY SHORT LENGTHS [SALMON & JOHNSON, 3RD. ED.]

THE PROBLEM RAISED BY DR. YURA ON TUESDAY WILL COME INTO PLAY WITH A TORSIONAL UNBRACED LENGTH >> WEAK AXIS UNBRACED LENGTH =>. THIS STILL ASSUMES PURE TORSIONAL BUCKLING.

Q10-1: *IF YOU HAVE ONE FLANGE OF A WIDE FLANGE COLUMN RESTRAINED (EVEN PARTIALLY) FROM ROTATION, CAN PURE TORSIONAL BUCKLING BE ACHIEVED? WHAT ABOUT TORSIONAL RESTRAINT AT COLUMN ENDS => WON'T THAT DECREASE TO THE EQUIVALENT LENGTH OF TORSIONAL BUCKLING?*

YURA: *THE SOLUTIONS WHICH I GAVE WHICH CAME FROM TIMOSHENKO AND GERE(?) ASSUME THAT THERE IS NO TORSIONAL RESTRAINT AND SO THAT WOULD BE A SAFE SOLUTION. OBVIOUSLY IF THE GIRT WAS PROVIDING TORSIONAL RESTRAINT, YOU NOW HAVE A TORSIONAL BRACE, AND YOU WOULD EITHER ADD THAT TO THE SYSTEM WHEN YOU PUT IT TOGETHER. YOU JUST HAVE TO PIECE THE TWO THINGS TOGETHER, BUT CERTAINLY, ANY TORSIONAL RESTRAINT WILL INCREASE THE CAPACITY. MY EXPERIENCE HAS BEEN WITH MOST FRAMES IS THAT THE TORSIONAL BRACING WON'T NECESSARILY CONTROL WHEN I CHECK IT WITH THE GIRTS, IT JUST DEPENDS ON CHECKING THE CAPACITY. WITH THE SHORT ONES, WHEN YOU SAY THAT TORSION SHOULD CONTROL, THE ONE THING YOU NEED TO DO IS WATCH USING AN INELASTIC ANALYSIS FOR THE TORSIONAL LOAD. YOU NEED TO USE A TANGENT MODULUS TO KNOCK THAT DOWN, ALSO. IN MANY INSTANCES, YOU THEN FIND THAT DEFLECTION WILL CONTROL IN THOSE INSTANCES.*

Q11: *A RECENT ARTICLE IN THE MAGAZINE MODERN STEEL CONSTRUCTION DISCUSSED HOW STEEL MATERIAL COSTS ARE DECREASING (RELATIVELY) WHILE CONSTRUCTION LABOR COSTS ARE INCREASING. THE POINT WAS MADE THAT THE ENGINEER SHOULD PAY MORE ATTENTION TO ELIMINATING "UNNECESSARY" STIFFENERS, USING PARTIAL HEIGHT*

STIFFENERS WHEREVER POSSIBLE, USING PARTIAL PENETRATION IN LIEU OF FULL PENETRATION WELDS WHERE POSSIBLE, ETC. IN THIS ATMOSPHERE, I SUGGEST IT WOULD BE PRUDENT TO MORE PRECISELY DEFINE BRACING REQUIREMENTS IN THE AISC SPECIFICATION AND/OR COMMENTARY USING MATERIAL SIMILAR TO THAT GIVEN IN JOE YURA'S PRESENTATIONS. OTHERWISE, THERE WILL BE SOME PRESSURE NOT TO ADD THAT ADDITIONAL BRACE OR STIFFENER. PLEASE COMMENT (STEVE GUNZELMAN)

KAEHLER: THOSE ECONOMICAL PRACTICES THAT YOU MENTIONED -- THEY ARE ECONOMICAL PRACTICES, THEY ALWAYS HAVE BEEN, AND I PERSONALLY ALWAYS HAVE BEEN IN FAVOR OF SEEING MORE GUIDANCE IN THE AISC SPECIFICATIONS AS FAR AS BRACING REQUIREMENTS GO. I WOULD WELCOME THEM PERSONALLY.

YURA: I DON'T THINK YOU COULD DISAGREE WITH THE COMMENT.

Q12: REFERENCE IS MADE TO JOSEPH YURA'S TALK ON "TORSIONAL BRACING REQUIREMENTS IN BEAMS AND COLUMNS". WITH REGARD TO THE PROBLEM OF A PALLET SUPPORTED ON TWO BEAMS, THE PALLET ITSELF WORKED AS A BRACE AS LONG AS THE CROSS-SECTIONAL SHAPE WAS MAINTAINED INTACT VIA USE OF STIFFENERS. MY QUESTION IS ONE OF STIFFENER CONFIGURATION:

A) IF THE PALLET IS, SAY, 12' LONG, WILL ONE, TWO, THREE, ETC., PAIRS OF STIFFENERS DO THE JOB?

YURA: IN THE EXAMPLE PROBLEM - MOST STIFFENERS ARE REQUIRED. THE BEAM WAS UNSTIFFENED, AS A MATTER OF FACT BECAUSE WHEN WE CHECK THE SOLUTION WITH NO STIFFENERS, IF YOU RECALL WHEN I WENT THROUGH IT, I SAID NO STIFFENERS, AND IT WORKED. IF IT DIDN'T, THEN YOU WOULD GO IN AND PUT IN SOME STIFFENERS. NOW THERE, YOU HAVE TO USE SOME JUDGEMENT. YOU ARE UTILIZING THE PALLET OVER 12' LONG BEARING SURFACE OF THE BEAM WEB, YOU HAVE TO USE SOME JUDGEMENT OVER HOW MANY STIFFENERS YOU FEEL COMFORTABLE WITH. TECHNICALLY WE'RE SPREADING THE LOAD OVER 1.5ft, SO ROUGHLY INDICATE THAT A STIFFENER OUGHT TO APPEAR EVERY 3 FT. OR SOMETHING LIKE THAT.

B) WOULD THE STIFFENERS NEED TO BE GRADED ON 1/2 THE BEAM DEPTH, 3/4 THE DEPTH, OR FULL BEAM DEPTH? (I BELIEVE 1/2 AND 3/4 OF BEAM DEPTH WERE SEPARATELY MENTIONED IN THE TALK).

YURA: THE FORMULATIONS WE HAVE WILL ENABLE YOU TO CALCULATE THAT. IF YOU RECALL TODD HELWIG'S PRESENTATION, THERE IS A SLIDE WHICH SHOWS HOW MUCH OF THE DEPTH WHEN WE ARE USING DIAPHRAGMS, AND HOW MUCH OF WEB WAS LEFT UNSTIFFENED, AND IF YOU LEAVE TOO MUCH OF THAT, NO MATTER WHAT YOU DO FOR YOUR DIAPHRAGM, YOU CAN'T GET ENOUGH STIFFNESS. SO THAT INDICATES YOU'VE GOT TO SPREAD IT OUT. MY EXPERIENCE ON ROLLED SECTIONS, U.S. VINTAGE. THAT IS DEPTH-TO-THICKNESS RATIO IS NOT GREATER THAN 6 FT. I GENERALLY FOUND THAT HALF-DEPTH STIFFENERS ARE ALL THAT'S NECESSARY TO DO THE JOB. WHEN YOU START GETTING INTO PLATE GIRDERS, THAT'S NOT TRUE, BECAUSE YOU'RE TALKING ABOUT MUCH MORE SLENDER ELEMENTS, AND THAT DISTORTION CAN BE A LOT MORE LOCAL. I'VE RARELY SEEN AN INSTANCE WHERE I HAVE NEEDED MORE THAN 1/2 DEPTH STIFFENER TO GET THE STIFFNESS UP TO WHERE IT WASN'T A PROBLEM. NOW SOMETHING ELSE IS THE PROBLEM -- REMEMBER WHEN YOU ADD

THOSE STIFFNESSES TOGETHER, IT'S ALWAYS THE WEAKEST ONE -- AND AS SOON AS YOU GET IT PAST THE NEXT WEAKEST ONE, NO ADDITIONAL HELP WILL DO ANYTHING FOR YOU.

Q13: *FOR PROF. YURA*

(1) IN THE TOPIC FUNDAMENTALS OF BEAM BRACING IT WAS SAID THAT AN INFLECTION POINT IS NOT A BRACE POINT. PLEASE EXPLAIN WHY IT ISN'T, EVEN THOUGH THE FLANGE FORCE IS ZERO AND THEREFORE THE BRACING FORCE REQUIRED AND STIFFNESS OF BRACE IS REQUIRED IS ZERO.

(2) ARE GIRTS OUTBOARD OF THE OUTER FLANGE OF A COLUMN ADEQUATE TO BRACE THE WEAK AXIS OF A COLUMN. ASSUME THE GIRT HAS TYPICAL BOLTED CONNECTIONS, AND THAT IT IS A BRACED STRUCTURE.

YURA: AS FAR AS FLEXURAL BUCKLING IS CONCERNED, YES -- YOU DON'T NEED HARDLY ANYTHING FOR FLEXURAL BUCKLING TO DO THE JOB FOR THAT.

13A CONTRIBUTION FROM ONE OF THE FOREIGN SPEAKERS:

COMPRESSION MEMBERS, COMPRESSION FLANGES ALIKE, COMMONLY HAVE MULTIPLE BRACES 2, 3 OR MORE. A REQUIRED SPRING STIFFNESS PREDICTION IS A START, BUT THE REALITY USUALLY IS THAT ONE BRACING SYSTEM SUPPLIES ALL OF THEM. SO A FORCE TO ONE BRACE DEFLECTS ALL BRACED POINTS IN A PREDETERMINED MANNER. THIS BRACING SYSTEM ALSO RESISTS WIND LOAD IN MANY CASES. SO THE WIND MODIFIES THE BRACE RESPONSE.

SIMILARLY, A VERTICAL BRACED BAY (INTERNAL OR EXTERNAL TO THE FRAME IT BRACES) SUPPLIES ALL THE STORY LEVEL SPRINGS AS A SET. AGAIN IT ALSO MOVES WHEN THE WIND BLOWS.

TO OVERCOME THIS EUROCODE 3 ADOPTS THE WINTER "PIN AT BRACE POINT" MODEL. HOPEFULLY THIS ALSO REMOVES THE NEED TO DISTINGUISH INELASTIC AND ELASTIC BEHAVIOR AS THE MEMBER MOMENTS ARE NEGLECTED.

AN INITIAL L/500 IMPERFECTION LEADS TO A CRITERION THAT 2% RESTRAINT (DISTRIBUTED) IS ENOUGH SO LONG AS DEFLECTION DUE TO RESTRAINT FORCES PLUS WIND IS LESS THAN L/2500. OTHERWISE, A LARGE FORCE IS NEEDED FOR MORE FLEXIBLE BRACING (THIS REFERS TO PLAN BRACING).

A SEPARATE CRITERION (ALSO PINNED MODEL) COVERS UNEQUAL FORCES DUE TO UNSYMMETRICAL DEFECTS, ETC. [*VERTICAL BRACING RESISTS AN INITIAL LEAN H/200.]

DO THE AUTHORS CONSIDER THIS APPROACH CAN BE REFINED SIGNIFICANTLY WHILST REMAINING GENERAL AND COVERING REAL BRACING, WIND EFFECTS AND IMPERFECTIONS?

YURA: LET ME PARAPHRASE TO MAKE SURE I GET THE GIST OF THE QUESTION WHICH REALLY RELATES TO THE MAGNITUDE OF THE DEFORMATIONS THAT ARE OCCURRING AND THEIR EFFECT ON BRACING FORCES. IF YOU NOTICE, VIRTUALLY ANYONE WHO DID ANY

CALCULATIONS CLEARLY INDICATED THAT THE BRACE FORCE IS LINEARLY RELATED TO THE MOVEMENT. THE MOVEMENT SHOULD INCLUDE INITIAL OUT-OF-STRAIGHTNESS PLUS ANY MOVEMENTS DO TO DRIFT, ETC. IF THOSE ARE EXCESSIVE, THEN YOU NEED TO COMPENSATE BY HAVING HIGHER BRACE FORCES, OR YOU NEED TO KNOCK DOWN A DEFLECTION. IF YOU HAPPEN TO GET THINGS LIKE A CROOKED COLUMN, WELL DOESN'T THE AISC ASSUME THAT IT IS CROOKED -- SO WHAT DO I NEED TO DO. WELL, IT DEPENDS ON HOW CROOKED. IF IT IS 3X MORE CROOKED THAN THE VALUE THAT WE ASSUME THEN YOU HAVE TO MULTIPLY YOUR BASE FORCE BY 3. AND YOU PUT THAT IN, AND THAT SHOULD HANDLE THE CROOKED COLUMN. SO YOU DON'T HAVE TO STRAIGHTEN IT. BUT BASICALLY YOU HAVE TO UNDERSTAND THAT BRACE FORCES ARE RELATED TO THE MAGNITUDE OF THE AMOUNT OF DRIFT PLUS IMPERFECTIONS, AND JUST ACCOUNT FOR THEM ACCORDINGLY.

BAKER: YOU CAN GET OUT-OF-STRAIGHTNESS FROM GRAVITY LOADS, PATTERN LOADS, ALMOST ANYTHING, SO SOMEHOW YOU DEVELOP A CONCEPT OF WHAT KIND OF SWEEP YOU HAVE IN YOUR SYSTEM, AND THAT GENERATES SHEARS AND MOMENTS WHICH HAVE TO BE HANDLED.

BRIDGE: IN THE EUROCODE, IT STATES THAT YOU CAN CARRY OUT AN ANALYSIS RATHER THAN TAKING THE VALUE, WHICH MEANS THAT YOU HAVE TO CARRY OUT A SECOND ORDER ANALYSIS TAKING INTO ACCOUNT THESE IMPERFECTIONS, AND I THINK THAT CAN BE DIFFICULT BECAUSE PEOPLE AREN'T SURE QUITE WHICH WAY TO PUT THESE IMPERFECTIONS. IT GIVES THE MAGNITUDES, IT GIVES YOU THE TYPES OF IMPERFECTIONS, BUT IT DOESN'T SAY WHICH WAY TO DISTRIBUTE THEM. AND THAT CAUSES SOME GREAT DIFFICULTIES. SO THAT'S ONE AREA. THE OTHER AREA THAT ISN'T COVERED IS THE ONE THAT YOU CALL "LACK OF FIT". AND THIS IS THE QUESTION OF WHEN YOU ACTUALLY CONSTRUCT THE BRACING SYSTEM. SOMETIMES YOU HAVE TO PULL THE BRACE TO THE STRUCTURE, OR PUSH THE STRUCTURE TO THE BRACE. THERE HAS BEEN SOME PRELIMINARY STUDIES DONE ON THAT, AND IN FACT, THE STUDIES SORT OF INDICATE THAT THE "PERFECT FIT" BRACE SEEMS TO BE A REASONABLE SOLUTION. IN OTHER WORDS, THE LACK OF FIT DOESN'T SEEM TO BE THE PROBLEM THAT YOU MIGHT BE AWARE OF. IN THE WORK GOING ON, THE AREA WHERE THE LACK OF FIT SEEMS TO BE THE MOST CONCERN IS FOR THE STOCKY MEMBERS -- IN OTHER WORDS, WHERE YOU ARE TRYING TO BRACE A MEMBER TO GET EXTREMELY HIGH STRENGTH -- YOU ARE TRYING TO PUSH THE STRENGTH FROM THE COLUMN OR THE BEAM RIGHT UP TO ITS FULL CAPACITY. IN THESE CASES, THE BEAM "LACK OF FIT" SEEMS TO BE IMPORTANT IN DETERMINING THE STRENGTH OF THE MEMBER.

QUESTIONS DIRECTLY ADDRESSED TO THE PANEL MEMBERS:

Q: *ROBERT CONLEY*
YOU SAID THAT YOU CAN BRACE A COLUMN IN BUCKLING WITH THE GIRTS -- EVEN IF THE GIRTS ARE OUT OF FLANGE, SO THE GIRTS HAVE TO BE BRACED THEMSELVES TO SOMEWHERE ELSE. IS THE CLADDING ENOUGH SOLUTION TO BRACE THE COLUMN?

YURA: GENERALLY YOU HAVE TO CHECK THE GIRT, AND USUALLY THE CLADDING IS ATTACHED IN A PRESSURE SITUATION. THE CLADDING IS ATTACHED TO THE GIRT, AND THAT'S USUALLY OK, AND THEN EITHER WHEN YOU PUT THE SECTION ON IT, YOU HAVE TO CHECK THE LATERAL STABILITY OF THAT CHANNEL SECTION BECAUSE OF THE TYPE OF FASTENERS, ETC., I PERSONALLY WOULD NOT RELY ON ANY CLADDING TO PROVIDE ANY TORSIONAL RESTRAINT, SO I WOULD USE THE STANDARD LATERAL BUCKLING FORMULA FOR CHANNELS, AND IF THAT

DOESN'T CHECK OUT TO THE SECTIONS, I WOULD PUT SOME SAG RODS OR APPROPRIATE BRACING TO MAKE SURE THAT THE GIRT COULD HANDLE THE PRESSURES AND THE SECTIONS.

Q: JACK PETERSEN, MARTIN & MARTIN

I WAS INTERESTED IF YOU COULD ADDRESS IN WHAT LOAD COMBINATION BRACING FORCES SHOULD BE CONSIDERED WITH – SEISMIC LOAD, LAG LOAD, DEAD LOAD.... FOR EXAMPLE, IN A LARGE ROOF WHERE YOU ARE USING STRUCTURAL STEEL BRACING IN LIEU OF A ROOF DECK, WHAT COMBINATIONS OF FORCES SHOULD BE USED. GENERALLY, THE SAFETY FACTORS. PUT DOWN AS USE 2 X THIS FOR YOUR BRACING FORCE, AND HOW SHOULD BE ADDRESS THIS IN BOTH ASD AND LRFD.

BAKER: GENERALLY FOR STIFFNESS, YOU SHOULD LOOKING AT ULTIMATE LOADS, OR FACTORED LOADS. FOR THE STRENGTH, YOU CAN ACTUALLY USE EITHER SERVICE LOADS OR FACTORED LOADS. IF YOU'RE IN ASD, YOU CAN USE SERVICE BECAUSE YOU GET THIS FACTOR OF SAFETY WHICH COMES IN THE DESIGN OF THE BRACED ELEMENT ITSELF.

Q: PETERSEN --

I WOULD LIKE AN ADDITIONAL COMMENT. IF YOU LOOK AT, FOR EXAMPLE, TRUSS CORES, THEN YOU DEVELOP A BRACING FORCE FOR THE TRUSS CORE. YOU ARE ACCUMULATING THIS ACROSS THE DIAPHRAGM, AND TRYING TO DELIVER IT TO SHEAR WALLS OR PERIMETER BRACING SYSTEM. IF THAT LOAD IS SUBSTANTIAL, SHOULD IT BE COMBINED WITH ALL OTHER LOADS, AND IF SO, SHOULD A REDUCTION FACTOR SIMILAR TO WHAT'S USED WITH SEISMIC LOADS BE CONSIDERED?

YURA: I THINK YOU HAVE TO MAKE SURE YOUR BRACING FORCES WILL GET DOWN INTO THE FOUNDATION, AND SO, I THINK YOU'D USE THAT PARTICULAR LOAD CASE. IF IT'S LRFD, YOU JUST CARRY THAT THROUGH AND MAKE SURE IT CAN WORK OUT. THE PARTICULAR LOAD CASE DEPENDS ON WHETHER IT'S LOCAL BRACING YOU'RE DEALING WITH... ON A PARTICULAR MEMBER, IN FACT EASIEST, JUST WHATEVER THE WORST SITUATION IS, AND IF IT'S OVERALL, JUST MAKE SURE THAT YOUR SYSTEMS CAN CARRY THE FORCES DOWN INTO THE FOUNDATION. SO I THINK THE BRACE FORCES SHOULD BE ADDED TO THE OTHER THINGS TO MAKE SURE THAT THOSE DETAILS CAN TRANSMIT THE FORCES – BECAUSE THEY ARE REAL FORCES.

Q: PETERSEN

ANOTHER COMMENT, WHEN YOU'RE USING METAL-DUCT DIAPHRAGM, AGAIN GOING TO SAFETY FACTORS VS LOAD FACTORS -- THE SDI INFORMATION (OR THE WORKING LOADS) IS DEVELOPED BASED ON 40% (ROUGHLY) OF ULTIMATE LOADS. WHEN YOU'RE COMPUTING STIFFNESS, HOW SHOULD YOU TAKE THAT INTO ACCOUNT VS THE ULTIMATE STRENGTH THAT YOU'RE USING WITH THE BRACING CALCULATIONS.

YURA: IF YOU'RE IN ASD, ALLOWABLE STRESS DESIGN, YOU'RE TRYING TO GET A MOMENT OF INERTIA IN SOME AREA, ETC. AND IN LOADS, YOU'RE ALL CALCULATED ON SERVICE LOADS SO THERE IS NO SAFETY FACTOR THERE. MY RECOMMENDATION IS, ASIDE FROM DOUBLING THE THING FOR INITIAL IMPERFECTIONS, I USE A FACTOR OF SAFETY OF 2.0 IN ASD AS FAR AS THE STIFFNESS. THE FORCES YOU DON'T HAVE TO DO ANYTHING WITH BECAUSE YOU'RE USING ALLOWABLE STRESSES FOR THOSE PARTICULAR THINGS -- SO THAT HAS THE FACTOR

OF SAFETY BUILT IN, BUT THE STIFFNESS IS, AS GREG DEIERLEIN MENTIONED IN HIS LECTURE, IF YOU'RE USING SERVICE LOADS, AND YOU PLUG INTO THOSE STIFFNESS FORMULAS WHERE THE LOAD P, YOU'VE GOT BASICALLY TO FACTOR UP THAT LOAD TO THE STIFFNESS. IN LOAD RESISTANCE FACTOR DESIGN, IT'S ALREADY DONE -- YOU'RE ALREADY AT THE LOAD FACTORED UP, SO IF THAT'S THE FORCE YOU'RE PUTTING IN, EVERYTHING SHOULD BE TAKEN CARE OF.

CLOSING COMMENTS (PROF. REIDAR BJORHOVDE)

IF THERE ARE NO MORE QUESTIONS, I'VE BEEN ASKED BY DON SHERMAN, CHAIRMAN OF THE SSRC TO SIMPLY THANK YOU ALL FOR ATTENDING HERE. FOR ME PERSONALLY, IT HAS BEEN A PLEASURE TO SEE THAT WE ADDRESSING AT THIS BRACING CONFERENCE AN ISSUE OF GREAT CONCERN THROUGH THE PRACTICING PROFESSION. FOR ME AS AN EDUCATOR, PARTICULARLY IMPORTANT, AND PARTICULARLY USEFUL TO SEE THAT EDUCATORS AND ENGINEERS CAN NOW GET TOGETHER AND TALK ABOUT REAL PROBLEMS AND REAL SOLUTIONS. I HOPE IT CAN CONTINUE, BECAUSE THERE ARE A LOT OF QUESTIONS THAT NEVER GET ANSWERED UNLESS WE CAN GET TOGETHER IN A FORM LIKE THIS, AND ADDRESS THEM THROUGH PRACTICAL MEANS, THROUGH CONSIDERATION OF REAL DETAILS, REAL STRUCTURAL SITUATIONS, REAL LOADING CASES.

IT HAS BEEN A BENEFIT TO BE HERE, IN MY OPINION, AND I HOPE YOU ALL FEEL THE SAME WAY. AS YOU HEARD DR. BEEDLE MENTION, NEXT YEAR IS THE 50TH ANNIVERSARY OF THE SSRC, AND WE HAVE A SPECIAL MEETING THAT IS BEING HELD IN BETHLEHEM, PA -- THE HOME OF L.U. AND SSRC IN THE THIRD WEEK OF JUNE. THE PROGRAM WILL BE A LITTLE DIFFERENT THAN HERE. WE WILL HAVE THE USUAL TASK GROUP MEETINGS & PRESENTATIONS, BUT THE TECHNICAL PROGRAM WILL BE A MORE SPECIAL BRIEF LOOK AT WHERE WE HAVE BEEN AND A CLEAR LOOK WHERE WE ARE HEADED IN TERMS OF STABILITY RESEARCH AND DEVELOPMENT AND PRACTICAL APPLICATIONS THEREOF. THE FOLLOWING YEAR, 1995, THE MEETING WILL BE HELD IN KANSAS CITY, MO. AND ALTHOUGH IT HASN'T BEEN DECIDED, WE NEEDED TO SEE WHAT THE PROGRESS AND HOW THE SUCCESS WAS OF THIS PARTICULAR NEW FORMAT UTILIZED HERE IN MILWAUKEE. I AM QUITE CONFIDENT THAT WE WILL INDEED, TRY TO PURSUE A SIMILAR FORMAT WITH A SIMILARLY RELEVANT PRACTICAL PROBLEM AREA THAT WE CAN ADDRESS TO THIS KIND OF A CONFERENCE. SO I AM HOPEFUL THAT YOU WILL PLAN TO COME TO BETHLEHEM NEXT YEAR, AS WELL AS TO KANSAS CITY IN 1995. WE LOOK FORWARD TO SEEING ALL OF YOU THERE.

IT'S BEEN A PLEASURE TO BE HERE. THANKS TO THE PANEL, PROF. BRIDGE, MR. KAEHLER, MR. BAKER, PROF. YURA. AND THANK YOU TO ALL OF YOU WHO HAD QUESTIONS. I HOPE YOU ENJOYED THE CONFERENCE AND FELT IT WAS USEFUL AND MAKE USE OF SOME OF THE THINGS YOU HAVE LEARNED HERE.

STRUCTURAL STABILITY RESEARCH COUNCIL

ATTENDEE ADDRESS LISTING

Wyndham Milwaukee Center
Milwaukee, Wisconsin
April 6-7, 1993

Mr. Ahmad K. Abdelrazaq
Skidmore, Owings & Merrill
224 S. Michigan Ave., Suite 1000
Chicago, IL 60604

Dr. Faisal Al-Mashary
Dept. of Civil Engineering
King Saud University
P. O. Box 800
Riyadh 11421, Saudi Arabia

Mr. Joseph Ales (*)
Dept. of Civil Engineering
University of Texas
Austin, TX 78712-1076

Mr. Edmund R. Aller
NUCOR Corporation
P. O. Box 70
Waterloo, IN 46703

Mr. Donald Allison
Burns & McDonnell Engineering Co.
P. O. Box 419173
Kansas City, MO 64141

Mr. Robert C. Andren
Charles E. Pease Associates
110 W. Higgins Road
Park Ridge, IL 60068

Mr. William F. Baker
Skidmore, Owings & Merrill
224 South Michigan Avenue
Suite 1000
Chicago, IL 60604

Dr. Ever Barbero
West Virginia University
315 Engineering Science Building
Morgantown, WV 26506-6101

Mr. Peter Bartnik (*)
University of Wisconsin-Milwaukee
P. O. Box 784
Milwaukee, WI 53201

Mr. Christopher N. Baudhuin
C.R. Meyer & Sons Company
P. O. Box 2157
895 W. 20th Avenue
Oshkosh, WI 54903

Dr. Lynn S. Beedle
Lehigh University
Structural Stability Research Council
13 E. Packer Avenue
Bethlehem, PA 18015

Prof. Peter C. Birkemoe
Department of Civil Engineering
University of Toronto
35 St. George St., GB 213
Toronto, Canada M5S 1A4

Prof. Reidar Bjorhovde
University of Pittsburgh
Dept. of Civil Engineering
937 Benedum Hall
Pittsburgh, PA 15261

Mr. Gholamali Boolorchi (*)
University of Wisconsin-Milwaukee
P. O. Box 784
Milwaukee, WI 53201

(*) Student

Dr. Russel Q. Bridge
Dept. of Civil Engineering
The University of Sydney
Sydney, NSW 2006, Australia

Mr. Robert F. Brodowski
O'Brien & Gere Engineers, Inc.
5000 Brittonfield Parkway
E. Syracuse, NY 13057

Mr. Mark Broomell
Webber/Smith Associates, Inc.
1921 Wheatland Ave.
Lancaster, PA 17603

Mr. Rod Buchanan
Frankfurt-Short-Bruza
4108 Amon Carter Blvd. #200
Fort Worth, TX 76155

Ms. Liping Cai (*)
Dept. of Civil & Env. Engineering
Rutgers University
Piscataway, NJ 08855

Mr. Steven Carter
Donaldson Company, Inc.
P. O. Box 1299
Minneapolis, MN 55440

Prof. W. F. Chen
School of Engineering
Purdue University
West Lafayette, IN 47907-1284

Mr. Chan Ho Chen
LS Transit Systems
1515 Broad Street
Bloomfield, NJ 07003

Mr. Nelson G. Chenaur
Consulting Engineer
11850 Edgewater Dr. #719
Lakewood, OH 44107

Dr. Mario De Stefano
Istituto di Tecnica delle Costruzioni
P.Le Tecchio, Universite di Napoli
80125 Naples, Italy

Mr. Michael De Sautel
Brander Construction Technology
2357 W. Mason Street
Greenbay, WI 54303

Prof. Antonello De Luca
Instituto di Tecnica delle Costruzioni
P.Le Tecchio, Universita di Napoli
80125 Naples, Italy

Prof. Gregory G. Deierlein
363 Hollister Hall
Cornell University
Ithaca, NY 14853

Mr. Samuel B. Dilcer
Burns and Roe Industrial Services Co.
700 Kinderkamack Road
Oradell, NJ 07649

Mr. Jackson L. Durkee
Consulting Structural Engineer
217 Pine Top Trail
Bethlehem, PA 18017

Mr. Christopher J. Earles (*)
University of Minnesota
425 13th Ave., #1602
Minneapolis, MN 55414

(*) Student

Dr. Mohamed Elgaaly
Civil & Architectural Engrg. Dept.
Drexel University
Philadelphia, PA 19104

Dr. Duane S. Ellifritt
College of Engineering
University of Florida
Gainesville, FL 32611

Dr. Samuel J. Errera
1730 Maple Street
Bethlehem, PA 18017

Dr. Hesham S. Essa
Dept. of Structural Engineering
Faculty of Engineering
Ain Shams University
Abbasia, Cairo, Egypt

Mr. Shu-Jin Fang
Sargent & Lundy
55 E. Monroe Street
Chicago, IL 60603

Dr. David J. Fielding
Consulting Structural Engineer
8306 Shawnee Street
Philadelphia, PA 19118-3907

Mr. Christopher Foley (*)
Marquette University
1515 W. Wisconsin Avenue
Milwaukee, WI 53233

Mr. Richard L. Forest
Frankfurt Short Bruza Assoc.
5701 N. Shartel, Suite 210
Oklahoma City, OK 73118

Mr. Gerard F. Fox
3 Whitehall Blvd.
Garden City, NY 11530

Mr. David D. Frash
Fanning/Howey Associates, Inc.
3750 Priority Way South Dr., #110
Indianapolis, IN 46240

Prof. Yuzuru Fujita
President, The Japan Welding
Engineering Society
1-11 Kanda Sakuma-Cho, Chiyoda-ku
Tokyo, Japan

Prof. Theodore V. Galambos
122 C&ME Building
University of Minnesota
500 Pillsbury Drive SE
Minneapolis, MN 55455-0220

Ms. Juliana E. Garner
Pearson Engineering, Inc.
P. O. Box 128
Benton Harbor, MI 49023

Prof. Al Ghorbanpoor
University of Wisconsin-Milwaukee
C.E. Dept., P.O. Box 784
Milwaukee, WI 53201

Prof. Luis Augusto Godoy
Mechanical & Aerospace Engineering
West Virginia University
Morgantown, WV 26506-6101

(*) Student

Prof. Subhash C. Goel
Dept. of Civil & Environ. Engrg.
The University of Michigan
Ann Arbor, MI 48109

Mr. Anthony E. Goetz
Frankfurt Short Bruza Assoc.
5701 N. Shartel, Suite 210
Oklahoma City, OK 73118

Dr. Arvind V. Goverdhan
Stanley D. Lindsey & Associates
300 Galleria Parkway, Suite 450
Atlanta, GA 30339-3149

Mr. Robert G. Graham
Steadfast Bridges
201 40th St., N.E.
P. O. Box 301
Fort Payne, AL 35967

Mr. Edward A. Grebe
2005 Two Tree Lane
Wauwatosa, WI 53213

Mr. Kenneth E. Grebe
Graef, Anhalt, Schloemer
& Associates, Inc.
345 North 95th Street
Milwaukee, WI 53226

Mr. Stephen X. Gunzelman
Brown & Root Representative
10200 Bellaire Blvd.
Houston, TX 77072-5299

Prof. Jerome F. Hajjar
University of Minnesota
Dept. of Civil & Mineral Engineering
500 Pillsbury Drive, S.E.
Minneapolis, MN 55455-0220

Mr. Dann H. Hall
Bridge Software Development
International, Ltd. (BSDI)
562 Thomas Street, P.O. Box 287
Coopersburg, PA 18036

Mr. Joseph P. Hartman
Corps of Engineers
20 Massachusetts Ave. NW
Washington, DC 20314

Mr. Frank Hatfield
Michigan State University
Civil Engineering Dept.
East Lansing, MI 48824-1226

Mr. Kirk A. Haverland
C. R. Meyer & Sons Company
P.O. Box 2157
895 W. 20th Avenue
Oshkosh, WI 54903

Mr. Todd Helwig (*)
University of Texas
12166 Metric Blvd. #184
Auston, TX 78758

Mr. Paul B. Hofland
Nicol & Hofland Associates, Inc.
1617 Wazee Street, Suite C
Denver, CO 80202

(*) Student

Mr. Michael J. Horne
Graef, Anhalt, Schloemer
& Associates, Inc.
345 North 95th Street
Milwaukee, WI 53226

Prof. Rola L. Idriss
Dept. of Civil, Agricultural and
Geological Engineering
New Mexico State University
Box 30001, Dept. 3CE
Las Cruces, NM 88003-0001

Mr. Jerome S. B. Iffland
Iffland Kavanagh Waterbury, P.C.
1501 Broadway
New York, NY 10036

Mr. John Imsdahl
Donaldson Company, Inc.
1400 West 94th Street
P. O. Box 1299
Minneapolis, MN 55440

Dr. Socrates A. Ioannides
President
Structural Affiliates International, Inc.
2424 Hillsboro Road, Suite 202
Nashville, TN 37212-5316

Dr. Jean-Pierre Jaspart
MSM-Institut du Genie Civil
Quai Banning, 6
B-4000 Liege, Belgium

Mr. James Johnson
Freeland Harris Consulting Engineers
2410 Lexington Financial Center
Lexington, KY 40507

Mr. Steven G. Judd
Martin/Martin-Utah, Inc.
455 E. 400 South St., Suite #400
Salt Lake City, UT 84092

Mr. Richard Kaehler
Computerized Structural Design, Inc.
8989 N. Port Washington Road
Milwaukee, WI 53217

Mr. Gary Karnthaler
Wisconsin Electric Power Co.
231 W. Michigan Avenue
Milwaukee, WI 53201

Dr. Kenneth T. Kavanagh
University of Western Australia
Nedlands, Western Australia
6009 Australia

Prof. Alan R. Kemp
Dept. of Civil Engineering
University of Witwatersrand
P.O. WITS, 2050
Johannesburg, South Africa

Prof. D. J. Laurie Kennedy
Dept. of Civil Engineering
University of Alberta
Edmonton, Alberta
Canada T6G 2G7

Prof. Toshiyuki Kitada
Department of Civil Engineering
Faculty of Engineering
Osaka City University
3-3-138, Sugimoto, Sumiyoshi-ku
Osaka, 558 Japan

(*) Student

Prof. Sritiwat Kitipornchai
The University of Queensland
Dept. of Civil Engineering
St. Lucia, Qld.
Australia 4072

Dr. Laszlo Kollar
Technical University of Budapest
Dept. of Steel Structures
Buegyetem rkp 3-9
Budapest H-1521, Hungary

Mr. Kurt K. Konkol
EXCELL Engineering, Inc.
460 Cambellsport Drive
Cambellsport, WI 53010

Ms. S. A. Korvink
Rand Afrikaans University
Laboratory of Materials
P. O. Box 524
Aucklandpark
2006, South Africa

Mr. Ray Kowalik
Burns & Mcdonnell Engineering Co.
4800 E. 63rd Street
Kansas City, MO 64130

Mr. Peter R. Krallitsch
Peter R. Krallitsch & Assoc., Ltd.
28 E. Jacoson Blvd., Suite 1707
Chicago, IL 60604

Mr. James E. Kreis
Red Mountain Engineers, Inc.
P. O. Box 16115
Santa Fe, NM 87506-6115

Mr. Ronald J. Kroon
Martin Marietta Energy Systems
P. O. Box 2003, Bldg. K1550-C
Oak Ridge, TN 37831-7231

Prof. Shigeru Kuranishi
Dept. of Civil Engineering
Tohoku University
Aoba, Sendai 980
Japan

Dr. James C. LaBelle
Computerized Structural Design, Inc.
8989 N. Port Washington Road
Milwaukee, WI 53217

Mr. Roger A. LaBoube
University of Missouri-Rolla
Associate Director, CCFSS
209 Civil Engineering
Rolla, MO 65401-0249

Dr. Samaan G. Ladkany
University of Nevada, Las Vegas
Dept. of Civil Engineering
4505 Maryland Parkway
Las Vegas, NV 89154

Mr. Stewart Lee
Lane Bishop York Delahay, Inc.
716 South 30th Street
Birmingham, AL 35233

Mr. Stephen E. Long
AVCA Corporation
Consulting Engineers
5855 Monroe Street
Sylvania, OH 43560

(*) Student

Ms. Nadine Love-Filer
C. R. Meyer & Sons Company
P. O. Box 2157
895 W. 20th Avenue
Oshkosh, WI 54903

Dr. LeRoy A. Lutz
Computerized Structural Design, Inc.
8989 N. Port Washington Rd.
Milwaukee, WI 53217

Prof. Murty K.S. Madugula
Department of Civil & Env. Engrg.
University of Windsor
Windsor, Ontario
Canada N9B 3P4

Mr. Michael P. Marinaro
American Consulting Engineers
4165 Millersville Road
Indianapolis, IN 46205

Mr. Peter Marshall
Shell Oil Company
P. O. Box 576
Houston, TX 77001

Mr. Michael D. Martello
Bergmann Associates
1 S. Washington Street
Rochester, NY 14614

Prof. Sheng-Taur Mau
Dept. of Civil & Env. Engrg.
University of Houston
Houston, TX 77204-4791

Prof. Federico M. Mazzolani
Facolta D'Ingegneria
Istituto di Tecnica delle Costruzioni
Universita di Napoli
P.Le Tecchio
80125 Naples, Italy

Mr. David McCloskey
McCloskey-Acevedo
P. O. Box 195361
Hato Rey, PR 00919-5361

Mr. Pat McCormick
Brander Construction Technology
2357 W. Mason Street
Green Bay, WI 54303

Mr. Clarence D. Miller
Chicago Bridge & Iron Technical
Services, Inc.
1501 N. Division Street
Plainfield, IL 60544

Mr. Majid Mobasseri
Clark Dietz, Inc.
1817 S. Neil Street
Champaign, IL 61820

Mr. Thomas E. Montag
Sega, Inc.
P. O. Box 23158
Overland Park, KS 66223

Prof. Nicholas F. Morris
Manhattan College
Civil Engineering Department
Riverdale, NY 10471

(*) Student

Mr. John William Norton
American Buildings Co.
P. O. Box 800
Eufaula, AL 36072-0800

Mr. W. Allan Olson
Arnold & O'Sheridan, Inc.
4129 N. 124th Street
Brookfield, WI 53005

Prof. Tetsuro Ono
Department of Architecture
Nagoya Institute of Technology
Gokiso-Cho, Showa-ku, Nagoya
466, Japan

Mr. John D. Papenfus
Graef Anhalt Schloemer
& Associates, Inc.
345 N. 95th Street
Milwaukee, WI 53226

Mr. Arvind M. Patel
Eichleay Engineers
8240 Gross Point Road
Mortongrove, IL 60053

Mr. Pravin H. Patel
246 Washington Street
Tom's River, NJ 08753

Dr. Dahyabhai R. Patel
Wright-Patterson Air Force Base
39 E. Dixon Avenue
Dayton, OH 45419

Mr. Conrad Paulson
Wiss, Janney, Elstner Associates, Inc.
29 North Wacker Drive, Suite 555
Chicago, IL 60606

Dr. Paul Penserini
Electricite de France
Research and Development Division
1 avenue du general de Gaulle
92141 Clamart, Cedex, France

Mr. Kent D. Pierce
Consulting Structural Engineer
49 Timber Lane
Brownsburg, IN 46112

Prof. Raymond H. Plaut
Virginia Tech
Civil Engineering
Blacksburg, VA 24061-0105

Mr. Todd Poehlman
CPR Associates, Inc.
P. O. Box 3429
De Pere, WI 54115

Prof. Dimos Polyzois
Department of Civil Engineering
University of Manitoba
Winnipeg, Manitoba
Canada R3T 2N2

Mr. Albert P. Popoli
Popoli Engineering, Inc.
920 Cross Lane
Blue Bell, PA 19422

(*) Student

Mr. Alan T. Power
Alan T. Power Structural Engineer
220 S. Rock Blvd. #11
Reno, NV 89502

Mr. R. S. Prabhu
American Buildings Company
#1 State Docks Road
Eufaula, AL 36027

Mr. Thomas Price
NUCOR Corporation
P. O. Box 70
Waterloo, IN 46703

Prof. Helmut Prion
The University of British Columbia
Department of Civil Engineering
2324 Main Mall, Room 2010
Vancouver, BC
Canada V6T 1Z4

Mr. Jeffrey Steven Rabinovitch
University of Alberta, Edmonton
Department of Civil Engineering
220 Civil/Electrical Engrg. Bldg.
Edmonton, Alberta
Canada T6G 2G7

Mr. Rajkumar Rajagopalan
University of Nevada, Las Vegas
Civil & Environmental Engineering
Howard R. Hughes College
of Engineering
4505 Maryland Parkway
Las Vegas, NV 89154-4015

Mr. Richard Redwood
McGill University
817 Sherbrooke St., West
Montreal, Quebec
Canada H3A 2K6

Mr. Jeffrey Reep
Somerville Associates, Inc.
2100 Riverside Drive
Green Bay, Wisconsin 54301

Prof. James M. Ricles
Lehigh University
Structural Stability Research Council
13 E. Packer Avenue
Bethlehem, PA 18015

Mr. James E. Rintala
Marathon
Engineers/Architects/Planners
2323 East Capitol Drive
P.O. Box 8028
Appleton, WI 54913-8028

Mr. Robert J. Riopelle
Riopelle Engineering Sales, Inc.
9406 N. 107th Street
Milwaukee, WI 53224

Mr. Richard B. Roe
Three I Engineering, Inc.
Consulting Engineers/Architects
P.O. Box 6562
2425 W. Indiana St
Evansville, IN 47719

(*) Student

Mr. Steven J. Roloff
Arnold & O'Sheridan, Inc.
2125 N. 124th Street
Brookfield, WI 53005

Mr. Anibal Salgado
McCloskey-Acevedo
P. O. Box 195361
Hato Rey, Puerto Rico 00919-5361

Mr. Walid Salman (*)
Civil Engineering Department
Galbraith Building
University of Toronto
Toronto, Ontario
Canada M5S 1A4

Dr. Charles G. Salmon
University of Wisconsin-Madison
614 South Segoe Road
Madison, WI 53711

Prof. Istvan Satjos
Vorosvari u.27/X/29
Budapest H-1035, Hungary

Mr. Suresh C. Satsangi
Butler Manufacturing Co.
13500 Botts Road
Grandview, MO 64030-2897

Mr. Rolando Saylo
Burns and Roe Industrial
Services Co.
700 Kinderkamack Road
Oradell, NJ 07649

Mr. Michael Schmidt
Arnold & O'Sheridan, Inc.
815 Forward Drive
Madison, WI 53711

Mr. Walter E. Schultz
NUCOR Research & Development
1601 West Omaha, Box 867
Norfolk, NB 68702-0867

Mr. Mark Seng
Hodge Design Associates
329 Main Street, Suite 614
Evansville, IN 47708

Mr. Maurice L. Sharp
R. 1, Box 171AA
Avonmore, PA 15618

Mr. Donald R. Sherman
Dept. of Civil Engineering
University of Wisconsin-Milwaukee
P. O. Box 784
Milwaukee, WI 53201

Mr. G. M. Singhvi
Smith, Hinchman & Grylls
Associates, Inc.
150 W. Jefferson, Suite 100
Detroit, MI 48226

Mr. Robert Sinn
Skidmore, Owings & Merrill
224 S. Michigan Avenue, Suite 1000
Chicago, IL 60604

(*) Student

Mr. Paul D. Smith
Gannett Fleming, Inc.
150 Wood Road
Braintree, MA 02184

Mr. Kamlesh N. Soni
Peter Albrecht Corp.
6250 Industrial Court
Greendale, WI 53129-2432

Mr. Joseph P. Steigerwald
Steigerwald Construction, Inc.
5310 W. State Street
Milwaukee, WI 53208

Dr. Lambert Tall
Florida International University
School of Engineering
Miami, FL 33199

Mr. Katsuhiro Tanaka
Japan Information Processing Service
Company, Ltd.
1-9-1 Edobori, Nishi-ku
Osaka 550, Japan

Mr. J. Colin Taylor
The Steel Construction Institute
Silwood Park, Buckhurst Road
Ascot, Berks SL5 7QN
United Kingdom

Dr. Murray C. Temple
University of Windsor
Faculty of Engineering
Windsor, Ontario
Canada N9B 3P4

Mr. Mathew Tharaniyil (*)
Dept. of Civil Engineering
University of Wisconsin-Milwaukee
P. O. Box 784
Milwaukee, WI 53201

Prof. Robert Tremblay
The University of British Columbia
2324 Main Mall
Vancouver, B.C.
Canada V6T 1Z4

Dr. Tony Tschanz
Skilling Ward Magnusson
Barkshire, Inc.
1301 Fifth Avenue, Suite 3200
Seattle, WA 98101-2699

Dr. G. J. van den Berg
Rand Afrikaans University
Laboratory of Materials
P. O. Box 524
Aucklandpark
2006, South Africa

Mr. Todd L. Vinje
Continental Bridge
8301 State Highway 29N
Alexandria, MN 56308

Prof. S. Vinnakota
Marquette University
1515 W. Wisconsin Avenue
Milwaukee, WI 53233

(*) Student

Prof. Andreas S. Vlahinos
Department of Civil Engineering
University of Colorado at Denver
1200 Larimer Street, NC 3027
Denver, CO 80204

Mr. Jeffery S. Volz
C.T.L.
5420 Old Orchard Road
Skokie, IL 60077-1030

Mr. Marc Voss (*)
Marquette University
1515 W. Wisconsin Avenue
Milwaukee, WI 53233

Mr. Yang Wang (*)
Department of Civil Engineering
University of Colorado at Denver
1200 Larimer Street, NC 3027
Denver, CO 80204

Mr. Dennis Waugh
IIW Engineers & Surveyors
4155 Pennsylvania Ave.
Dubuque, IA 52002

Mr. Stephen M. Way
Bergman Associates
1 S. Washington Street
Rochester, NY 14614

Mr. George Weatherford
Central Soya Co., Inc.
P. O. Box 2507
Fort Wayne, IN 46801-2507

Michael A. West
Computerized Structural Design, Inc.
8989 N. Port Washington Road
Milwaukee, WI 53217

Prof. Donald W. White
School of Civil Engineering
Purdue University
1284 Civil Engineering Building
W. Lafayette, IN 47907

Mr. P. Craig Williams
The Haskell Company
111 Riverside Avenue
Jacksonville, FL 32202-4950

Mr. John M. Yadlosky
HDR Engineering, Inc.
3 Gateway Center
Pittsburgh, PA 15222

Prof. Toshitaka Yamao
Civil & Environmental Engineering
Kumamoto University
Kurokami 2-39-1
Kumamoto 860, Japan

Dr. Chai H. Yoo
Civil Engineering
238 Harbert Engineering Center
Auburn University, AL 36849

Prof. Wei-Wen Yu
University of Missouri-Rolla
Dept. of Civil Engineering
Rolla, MO 65401

(*) Student

Dr. Joseph A. Yura
University of Texas
Dept. of Civil Engineering
Austin, TX 78712

Dr. Ronald D. Ziemian
Dept. of Civil Engineering
Bucknell University
221 Dana Engineering Building
Lewisburg, PA 17837

(*) Student

NAME INDEX

SUBJECT INDEX