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FOREWORD

The 1993 Annual Technical Session of the Structural Stability Research Council was held in the City of Milwaukee, Wisconsin on April 5 and 6. This proved to be a very pleasant venue with good surroundings and eating places. It was the first time the SSRC met in Milwaukee.

Another new venture was a substantial change in the format of the meeting. The presentation of papers for the SSRC meeting took place at Task Group meetings on Monday and at a plenary session on Tuesday morning. The reason for this change was that the traditional half day theme session was expanded to a 1½ day conference with the theme "Is Your Structure Suitably Braced?" Even with the restricted time for the SSRC presentations, a total of 27 presentations were scheduled, with 14 of them in the plenary session. All these papers are contained in these proceedings along with several other papers that were submitted but not presented. The papers were associated with eleven different Task Groups and sincere thanks is due to all the authors for their efforts and participation.

It is not possible to separate the number of participants in the SSRC meeting from the total that also attended the subsequent theme conference. However, the Task group meetings were very well attended and their content was primarily technical in nature. A head count indicated that nearly 100 were in attendance at the plenary session. The combined meetings had over 160 registrants, representing ten different countries. This was the largest attendance ever at an SSRC event. In spite of the initial concerns that the abbreviated SSRC meeting would not be well received, the response of the participants seemed enthusiastic.

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

CH2M Hill

Computerized Structural Design

Graef Anhalt Schloemer Associates

Howard Needles Tammen & Bergendoff

Society of Iron & Steel Fabricators of Wisconsin

Due to the change in format, considerable extra effort was required in organizing and implementing the SSRC meeting. The Program Committee chaired by Clarence Miller did a commendable job in organizing the technical content of the meeting. The Task Group Chairmen were especially cooperative in integrating technical presentations into their meetings. Lesleigh Federinic, SSRC Administrative Secretary, and her assistant Diana Walsh, did their usual excellent job of insuring that everything went smoothly both before and during the meeting. Thanks for the extra effort this year. Our new Associate Director, Jim Ricles, got a good initiation and we appreciate his contribution to the program and arrangements. Also, thanks to the SSRC Director, Lynn Beedle, for his efforts and overseeing all the details that had to be considered. The students from Marquette University and the University of Wisconsin-Milwaukee deserve recognition for their efficient assistance during the conference. Finally a special thanks to Jerry Iffland Chairman of the Finance Committee, for the idea of the new meeting format and for inspiration to others in making this a successful meeting.

Now we can look forward to next year which will be the 50th Anniversary Meeting of the SSRC. This special and exciting event will be held at Lehigh University, June 19-22, 1994. The theme, "SSRC - Link Between Research and Practice" will be addressed by international experts on all of the various stability topics important to SSRC. We hope to come away from this meeting with a good vision of the future of stability research and design.

D. R. Sherman

Donald R. Sherman
Chairman

Milwaukee, Wisconsin
April, 1993

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OTHER RESEARCH

Your Building May be Stronger Than You Think; The Benefits of 3-D Analysis

by Marc Hoit[1] and Duane Ellifritt[2]

Introduction

Structural engineers are always looking for ways to make more efficient use of materials--for finding secondary uses of members that are already there for another purpose. This is the basis for composite construction; the slab is going to be present anyway so why not make it work as part of a floor beam? Another example is steel deck. In floors its primary function is as a concrete form. However, properly designed shear connectors can make it work as part of a composite slab. In roofs it's there to keep out the weather, but its in-plane shear strength can make it work as bracing for frames and other structural elements.

In metal building systems and other conventional one-story commercial and industrial buildings, some form of X-bracing is usually used in the planes of the roof and wall to provide resistance to wind perpendicular to the main frames. This bracing, too, can serve a secondary function that can only be found through a 3-dimensional analysis of the entire structure.

Conventional 2-D Analysis

A common method of bracing 1 story framed buildings against wind perpendicular to the frames is shown in Figure 1. The X bracing may be angles, rods, or cables or in very heavy structures may even be W-sections. In most cases, the assumption is made that these members are incapable of resisting compression without buckling elasticly and therefore are considered to act as tension members only.

When wind blows against the end wall of this building, loads are transferred through the end wall columns, through struts in the roof, and into the bracing system which, with the assumption mentioned above, becomes a simple Pratt truss. For this case a 3 dimensional analysis would be of little value.

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There are certain types of similar structures, however, where a 3-D analysis can be most beneficial.

Hangar Buildings

Buildings which are built for large aircraft maintenance facilities generally have large doors in the end walls, as in Figure 2. In these cases, there can be no end wall columns to transfer wind load, so it is common to build a cantilever door support assembly that transfers the load back to an interior frame by means of an inclined compression strut, as shown in Figure 3. This puts an upward force on the frame to which this inclined strut is attached. An upward deflection of this frame means an inward horizontal movement at the top of the sliding doors. Too much movement may cause damage to the doors.

A typical 2-D analysis, modelling the frame stiffness as springs in-plane, may show very large horizontal deflections at the door tracks as seen in Figure 4. This may cause the designer to believe that a heavier frame is needed, but increasing the frame stiffness comes at the cost of dramatically increasing the weight of that frame. A better solution is to utilize something that is already present anyway--the wind bracing. The beneficial effect of this can only be found by performing a 3-D analysis of the entire building.

3-Dimensional Analysis

A three dimensional analysis will show that the upward force on the second frame is actually restrained by the roof bracing which transfers some of this force to adjacent frames. For a flat roof building, this effect is negligible, but for a gabled frame building it can be significant. Look at the simplified 2-bay model in Figure 5. In Figure 5(a) the upward force is perpendicular to the plane of the diagonal bracing, so the bracing will not resist much load until large deflections take place. In Figure 5(b), the diagonal bracing is already positioned to provide a vertical component of reaction to the applied upward load.

It should be noted that this works because only one frame is loaded. This comes from having the end wall wind reactions carried back to the first interior frame on an inclined member, producing an upward component of force. The same principle would not work if all frames are equally loaded, as in uniform wind uplift on the roof.

It should also be noted that use of this procedure may result in very large forces in the bracing members and their connections. The designer must insure that these members and their connections are adequate for the increased forces from the door headers. The common practice of using a single rod through a hole in the web may not be sufficient for such forces.

Analysis Example - Modeling

As an example, the results of a three-dimensional analysis will be compared to the more conventional two-dimensional analysis for a typical hangar building like the one in Figure 2. The dimensions of the building are shown in Figure 6. The building is ten bays long, symmetrical about a vertical plane through the ridge and has sliding doors in both end walls and a door supporting assembly similar to that of Figure 4.

For the analysis, a finite element program called SSTAN will be used. SSTAN is a three dimensional static analysis program that handles trusses, frames, plates, membranes, shells and solid elements. It can also include P-Delta and non-compression member effects.

In order to simplify the modeling and reduce its size, we will model only the first three frames and use symmetry about the peak. In finite elements, symmetry can be handled by modeling only the symmetric portion of the structure and adjusting the boundary conditions to account for the rest of the structure. This method is exact for linear structures subjected to symmetric loading. Generally, symmetry can not be used in a non-linear analysis. This is because as members yield or change properties, the structure becomes non-symmetric. In this case, the only non-linearity being included is non-compression bracing members. The symmetric loading will give symmetric results even with this non-linearity. Therefore the use of symmetry is correct for this case.

By including only the first three frames in the structure, the assumption is made that most of the load is resisted by these frames. If this were not the case, the entire structure would have had to be used. Even though only the first three frames are being modeled, some method must be used to account for the effect of the rest of the structure. The main effect that needs to be represented is the resistance to deflection offered by the other frames. The typical way to account for portions of a structure is to replace it with simple springs. In a linear example, this method can be exact.

In order to model the rest of the structure with springs, the stiffness value to use for these springs must be determined. The exact way is to model the rest of the structure, then apply a load at each node where there is a connection between the first three frames and this model. The load applied divided by the deflection caused by the load is the stiffness value. In a linear model, this load can be a unit load and the stiffness is an exact representation of the structure. In a non-linear problem, the load must be equal to the real load the structure will see. Clearly the real load is not known and either must be found in a iterative manner or the entire structure must be modeled.

This method is useful when replacing a very complex portion of a structure by an accurate simple representation. This replacement allows us to study the rest of the structure without the complexity. Often, an approximate model of the rest of the structure is used since the difference in the approximate and exact models causes only a very small change in the results of the portion being studied.

In this example, the approximate stiffness is accurate enough since its effect will not change much with more accurate modeling. Therefore, the rest of the structure is modeled by an approximate structure. Fewer nodes and members are used to represent the tapered sections. Only the nodes that connect to the three frames are included. These frames are being loaded about their weak axes so the taper does not effect the moment of inertia. In addition, a unit load is used to calculate the spring stiffness. This is not exact since not all compression members at higher loads will be ignored. In addition, the additional frames do not resist much of the wind load since the bracing transfers most of the load to the first three frames.

Symmetry is also used on the spring evaluation model. Again, only the symmetric portion about the vertical plane along the ridge is used for the spring evaluation. Here, the unit load must be applied as a combination of a symmetric and anti-symmetric part to get the correct displacement. This is because with symmetry, loading is assumed to be the same on the symmetric portion of the structure. In determining the spring values we only want a single load. Using anti-symmetric load method (changing the boundary conditions) we can get this effect. The plot of the structure used to determine the spring stiffness is given in Figure 7.

The three rigid frames on the windward end of the building are modeled fairly accurately. Also included in the model are the purlins connecting the frames and the springs calculated for the rest of the structure. The door support assembly is modeled including the diagonal brace used to transfer load back to the interior frame. Two versions of the analysis model were created. The first simulates a two dimensional analysis. This model does not include any roof bracing. The second model adds the full roof bracing to the model.

The no-roof-bracing model will give exactly the same answers as the simplified model shown in Figure 3. The spring values used in the simplified model can be calculated using a two dimensional model of a single rigid frame. A full Three Dimensional model was actually used for the un-braced frame analysis since the model had to be developed for the braced analysis. Figure 8 shows the structure used. If wind load is applied to the un-braced model very large deflections result as shown in Figure 9. The plot shows deflections in an exaggerated form. However, the dramatic shape can only come from extremely large values. Looking at the bottom of the door support assembly at the peak, note that the horizontal deflection is 70". The vertical deflection is 12". These deflections are clearly unacceptable for a connected door.

A plot of the braced model can be seen in Figure 10. Applying the same wind load that was used on the un-braced model, the results are shown in Figure 11. Here the shape is not distorted. At the same support location, a horizontal deflection of 20" and a vertical deflection of 6" can be seen. The bracing added an enormous amount of stiffness to the structure and helped to distribute the force from the door hanger assembly into adjacent frames.

Conclusions

Any structure is going to behave the way it wants to, regardless of how it is analyzed. A typical two-dimensional analysis, while conservative, may overlook some reserve strength in a structure that can only be revealed in a three-dimensional analysis. This is the case in the example of the aircraft hangar shown. Conventional wind bracing, placed there for alignment during erection and to resist longitudinal wind forces, is in reality performing a third function of distributing the upward force on the first interior frames, due to wind on the hangar door, to adjoining frames. This may require increasing the size of the bracing members found through a two dimensional analysis and improving their connections but at least it is a structural element that is already there. Critical for the gains to occur is having a pitched roof. It is because of this pitch that the bracing offers a vertical stiffness component. Flat roofs do not offer any such advantages.

In summary, a three dimensional analysis does not make a building stronger; it may, however, show that the structure is stronger than you thought it was.

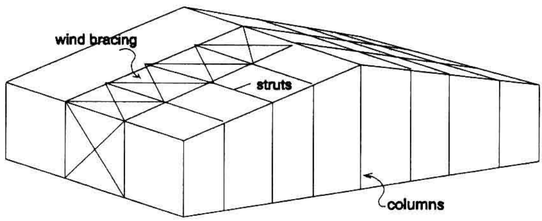


Figure 1 - Conventional Wind Bracing

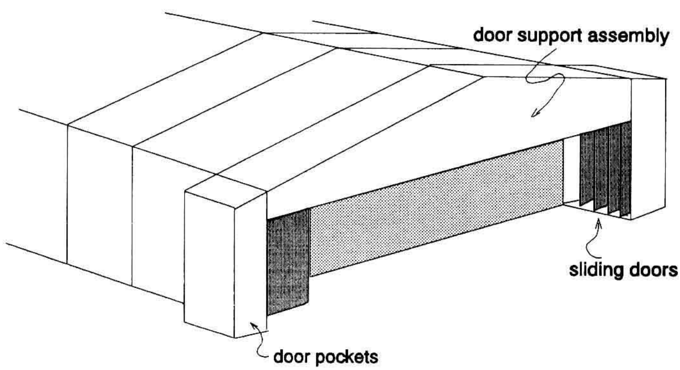


Figure 2 - Typical Hangar

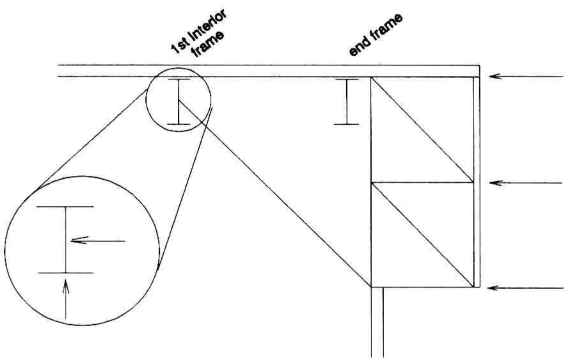


Figure 3 - Wind Loads on Door Support Assembly

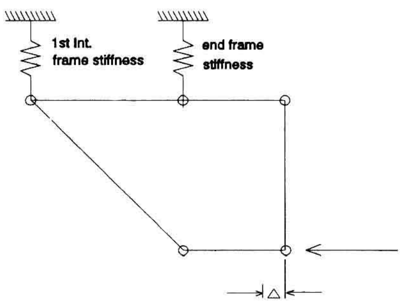
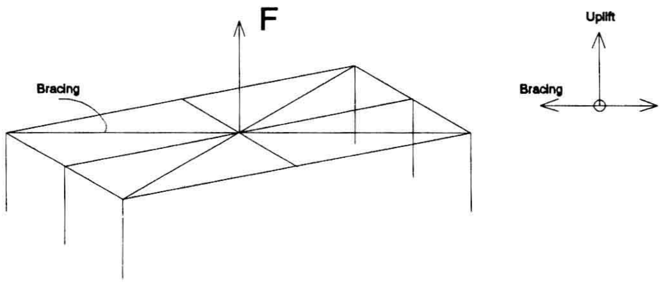
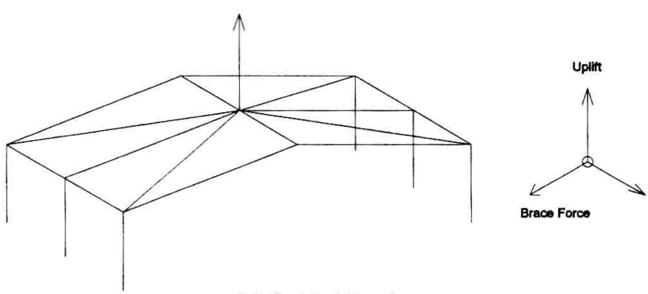


Figure 4 - Schematic of 2-D Deflections In Door Support Assembly



(a) Flat Roof



(b) Gabled Roof

Figure 5 - How Diagonal Bracing Resists Uplift on a Single Frame