A system for reinforcing tall buildings. The building comprises an internal frame comprising a plurality of horizontal floor beams and a plurality of vertical columns extending between adjacent floor beams. The reinforcing system comprises diagonal bracing members connected between vertically spaced floor beams at an angle to both the floor beams and the columns.
S. S. MOTION N°3 (SPRING ACTION) ONLY AT EXTERIOR STUB CONNECTORS

A SIDE HORIZONTAL STRUT (TYP) / 2 seismic force / 3 (SPRING ACTION)

ELASTIC HINGE 1 STUB CONNECTOR / 1

S. S. MOTION N°3 (SPRING ACTION)

location of seismic force due to tributary mass of dead load (typ)

OUTSIDE HORIZONTAL STRUT (TYP)

STUB CONNECTOR

ELASTIC HINGE N°1

MOTION N°3 (SPRING ACTION)

TOTAL MOVEMENT OF STUB CONNECTORS DUE TO SEISMIC FORCE TYPICAL FOR ALL STUB CONNECTORS

MOTION N°3 (SPRING ACTION)
FLOOR  ELASTIC HINGE N°2  ELASTIC HINGE N°2

DIAPHRAGM FLOOR  ELASTIC HINGE N°2  ELASTIC HINGE N°2

FLOATING FLOOR  ELASTIC HINGE N°2  ELASTIC HINGE N°2

FLOATING FLOOR  ELASTIC HINGE N°2  ELASTIC HINGE N°2

DIAPHRAGM FLOOR  ELASTIC HINGE N°2  ELASTIC HINGE N°2

MOTION N°1 SEISMIC ENERGY ABSORBED BY BENDING OF COLUMN BETWEEN ELASTIC HINGES N°1 AT DIAPHRAGM FLOOR DAMPING: SEISMIC ENERGY ABSORBED BY THE WORK REQID TO ROTATE ELASTIC HINGES N°1 AT FLOATING FLOOR

FIG. 6
Ties at 45° on all four faces of the column. As an alternate, we could use [ ] or [ ] at 45°.

Figure No. 3
GENERALIZED 30 STORY CONCRETE BUILDING

FIG. 7
FIG. 13

Beam A

Beam B

Beam C

FIG. 7B
HORIZONTAL DESIGN RESPONSE SPECTRA
SCALED TO 1g HORIZONTAL GROUND ACCELERATION

SPECTRE DE REPONSE HORIZONTALE DE L'U.S.N.R.C.
(REGULATORY GUIDE 1.60)

FIG. 8
\[ \approx 75 \text{ cm} \]
\[ \approx 2 - 6'' \]

Fig. 18
The outside vertical bracing system.

DIA. FL. = DIAPHRAGM FLOOR
FLOAT. FL. = FLOATING FLOOR

FIG. 21
STUDS COMPOSITE SECTION (TYP)

ADDITIONAL STUB CONNECTOR

The outside vertical bracing system

FIG. 22
EXOSKELETON SYSTEM FOR REINFORCING TALL BUILDINGS

INTRODUCTION

[0001] The author presents a new vertical bracing system, placed outside the building envelope, for concrete or steel framed buildings which provides excellent resistance to seismic forces in high rise buildings at a cost expected to be lower than conventional framing systems.

[0002] The proposed system is a combination of simple standard bolted connections (shear connections) for the building frame and vertical bracing located approximately 2'/6" (=75 cm) outside the face of the building framing. The framing supporting the vertical loads (building column and beams) is separate from the lateral load bracing (outside vertical bracing).

[0003] The outside vertical bracing system has excellent ductility under elastic load reversals (i.e. wind and moderate seismic forces) due to the elastic hinges developed between the outside bracing and the building framing.

[0004] Under inelastic load reversals (i.e. severe seismic forces) the elastic hinges have the potential for becoming plastic, thus providing even greater ductility. The elastic and plastic hinges act as structural fuses protecting the framing.

[0005] Steel high rise buildings which must be designed for horizontal loads (seismic forces or wind forces) require moment connections or vertical bracings. For very tall buildings, the trend today is to use field welded framing where large assemblies, which form the exterior of the building, are fabricated in the shop then are shipped to the site where they are field welded. The use of field welded framing and very large built-up members (especially box sections) requires more material, takes more time, and makes the building more expensive. In addition it sometimes makes the building more flexible, creating problems with the outside curtain wall, glass panels, etc.

[0006] Description of Framing System

[0007] Following is a description of the proposed system of outside vertical bracing. The vertical bracing mounted outside the face of the building framing consists of diagonal bracing, and horizontal and vertical struts (see FIGS. 1 and 2). The columns of the standard framing of the building will carry the vertical forces due to overturning from horizontal forces (wind or seismic). Therefore, the construction of the building can proceed in the same manner as framing with standard connections (shear connections: angles shop welded to beams and field bolted to columns). As the building progresses in height, another team can install the outside vertical bracing (diagonal bracing and horizontal and vertical struts only). Thus the vertical bracing lags behind the main steel framing and does not interfere with the regular schedule of steel erection for standard connection framing. Because field welding is minimized and the framing of the building requires rolled shapes only, the construction time is reduced and so the price of the building is reduced.

[0008] Diaphragm Floors

[0009] The floors attached by stub connectors to the outside horizontal strut are held in place by the bracing and are designed as diaphragm floors.

[0010] All the columns (interior and exterior) between two diaphragm floors have about the same absolute lateral stiffness, K=Px. As a result, the seismic forces transferred by all the columns are about equal. The diaphragm floor collects tributary seismic forces from the floors above and below and also resists the seismic forces due to its own dead load (concrete, partitions, steel framing, curtain wall, etc.). The diaphragm floor transfers the seismic forces to the outside horizontal strut through the stub connectors, as previously described.

[0011] Stub Connectors

[0012] The stub connectors attaching the outside vertical bracing to the building act like damping springs or structural fuses which absorb energy due to seismic forces. They deliver forces due to overturning of the building by bending and shear to the building columns and transfer the horizontal forces by bending about their weak axis. One can “tune-up” this damping system by designing the stub connectors to be more rigid or more elastic and to yield at a certain value of lateral forces. After an earthquake, all the stub connectors can be inspected and the damaged ones replaced. Therefore, when detailing the stub connectors, it is necessary to provide for possible replacement by using bolted plates in addition to shop welding. The “tune-up” of the stub connectors can be done using a model of the building.

[0013] Sub-Frames

[0014] Due to the action of elastic hinges, the building frame is divided into two sub-frames. Sub-frame 1 consists of all of the diaphragm floors and the outside vertical bracing which are connected through stub connectors. Sub-frame 2 consists of all of the diaphragm floors and all of the columns of the building (interior and exterior). Sub-frame 1 and sub-frame 2 are connected with elastic hinges which can develop into “plastic hinges”, insuring ductility in response to a wide range of load intensities.

[0015] Damping Action of Framing System

[0016] Due to the elastic hinges, located at all the floors, and sub-frames, there is a substantial damping action which will absorb a large quantity of seismic energy. In order to increase the damping, mild carbon steel ASTM A-53 Grade B (Fv=25 KSI to Fv=35 KSI) can be used for stub connectors (Elastic Hinge N 1) and for the column splice (Elastic Hinge n 3). See FIG. 4.

[0017] In addition, there is damping due to the different modes of deflection and to the different fundamental elastic periods of vibration between the columns (interior and exterior) and the outside vertical bracing, in the direction under consideration. The damping provided by the outside vertical bracing must be established by extensive laboratory testing on a model.

[0018] Building codes use the factor K which defines the ductility of the system framing and also reflects the redundancy of the system (framing).

[0019] For highly ductile and redundant frames which are the rigid moment frames, the code identifies a value of K=0.67. The code does not allow smaller values for K.
For systems which are more "brittle" and do not have large inelastic deflections, the value of K is very large (K=1.33) because these systems cannot reduce the vibrations and the resulting deflections caused by the earthquake.

The system of outside vertical bracing is a system which fits between a "brittle" system and a highly ductile and redundant system due to the elastic hinges, plastic hinges and the sub-frames.

We propose a formula for K as follows:

\[ K = 0.67 + \frac{0.33}{n} \]

In this formula, "n" is the number of stories between the diaphragm floors. The true value of K must be determined by modeling.

To quantify the proposed system, the computer program HERCULE, created by the French Control Office SOCOTEC-VISPA, was used to model a thirty story building 10' floor heights, with three 15' bays in each direction, see Supplemental I for more details. Three cases were studied and compared:

A. The horizontal and vertical load framing systems in the same plane as the building exterior.

B. The outside lateral load bracing placed approximately 26' (≈75 cm) outside the building envelope and linked to the interior structure by rigid connections on the extension of the main cantilever beams.

C. Configuration identical with Case B, with the addition of anti-seismic springs between the two structures.

Computer analysis indicates that deformations are quite similar in Cases A and B due to the great rigidity of the cantilever beams. However, the maximum deflections are reduced by 2.5 times in Case C due to the springs or elastic hinges. The sway of the inner structure increased from 7 to 16 cm in Case C over the results in Cases A and B, but still remains within acceptable limits for a building 300' high, FIG. 1A.

In addition, a 60% to 70% decrease in the axial stress in the columns and main bracing members was realized. The corner columns of the inner structure had a 75% decrease in axial stress, but was accompanied by a parallel increase in the maximum flexural moments. The increase in flexural moments was forseen due to the loading arrangements and can be mitigated by introducing hinges at the bottom of the columns instead of fixing them. Finally, the reactions in the anti-seismic springs varies from 5 to 12T (max.) which is in the range of springs commonly used.

Interior Diagonal Bracing

The interior diagonal bracing is made up of steel members running diagonally between the corners of the floor modules.

To achieve greater flexibility for architectural aspects, these diagonal members can extend one or multiple floors in height. These members consist of a wide flange section at the building center for narrow structures or at ¼ or ¼, etc. panels for wider and/or taller buildings.

Interior bracing can be 2 or 4 floors in height and should complete the standard triangle shape:

\[ \text{or} \]

Note: Exterior bracing can be any of the above configurations.

FIGS. 3A and 3B

System Advantages

The framing system described in this paper has the following advantages over conventional high rise design:

1. The lateral framing is approximately 26' (≈75 cm) outside the building exterior so that multiple floors (three or four maximum) can be braced together thus eliminating many connections.

2. The structural steel frame and exterior bracing can be bolted moment connections for greater economy.

3. The design of all but the elastic hinge stub connectors can be performed with desk top calculators.

4. The framing has a built in self-damp system due to the action of the stub connectors which connect the building framing with the outside vertical bracing.

5. The vertical bracing is outside the building so it can be easily inspected for damage after an earthquake; repair work is also simplified.

6. The bracing system lends itself to retrofitting existing concrete or steel buildings which do not meet present seismic codes.

7. The framing system could allow for very tall structures up to approximately 115 floors.

Work-in-Progress

The author is in the process of evaluating, by computer modeling, three additional and different cases for the exterior bracing systems. All three cases have the same geometric characteristics and all incorporate diaphragm floor connections at the top and bottom of each exterior brace, the difference is in the relative strength of the two support systems. The two systems are described below:

System A:

This framing systems consists of the exterior building columns, the two diagonals of the triangular bracing and the exterior floor beams to which the triangular bracing is attached.

System B:

This framing system consists of the two horizontal and two vertical members of the outside bracing as well as the same component of the diagonal outside braces which it shares with System A.
By varying the relative strengths of the two systems, the author can evaluate the most effective use of the bracing. The three cases under consideration are:

1. **Case 1:** System A resists 90% of the load, System B resists 10% of the load.

2. **Case 2:** System A resists 50% of the load, System B resists 50% of the load.

3. **Case 3:** System A resists 5% of the load, System B resists 95% of the load.

See FIGS. 1 through 5 for an illustration of these cases.

We studied and compared 3 cases:

1. **Case A:** Both structures on the same plane surface of the facade acting integrally.
2. **Case B:** The structure for horizontal stability placed 80 cm front of the inner structure and linked to this one by hinged connections on the extension of the main cantilever beams.
3. **Case C:** Identical to Case B with addition of 14 anti-seismic springs on each facade as shown on FIG. 11.

We considered:

- **A.** The interior structure for the vertical leading schedule (FIG. 9) with beam data as follows:
  
  \[
  A = 135 \text{ cm}^2, \quad S = 2500 \text{ cm}^3, \quad I = 67500 \text{ cm}^4
  \]

- **B.** The exterior structure for horizontal stability (FIG. 10) with columns identical to the inner structure and cross bars changing section every 6 levels:

<table>
<thead>
<tr>
<th>Level</th>
<th>25 to 30</th>
<th>28 to 30</th>
<th>31 to 34</th>
<th>35 to 37</th>
<th>38 to 40</th>
<th>41 to 43</th>
<th>44 to 46</th>
<th>47 to 49</th>
<th>50 to 52</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (cm²)</td>
<td>60</td>
<td>120</td>
<td>240</td>
<td>480</td>
<td>960</td>
<td>1920</td>
<td>3840</td>
<td>7680</td>
<td>15360</td>
</tr>
<tr>
<td>S (cm³)</td>
<td>120</td>
<td>240</td>
<td>480</td>
<td>960</td>
<td>1920</td>
<td>3840</td>
<td>7680</td>
<td>15360</td>
<td></td>
</tr>
<tr>
<td>I (cm⁴)</td>
<td>480</td>
<td>960</td>
<td>1920</td>
<td>3840</td>
<td>7680</td>
<td>15360</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Oct. 30, 2003
and for the most loaded bracings a decrease of:

- 234 t in Case A
- 226 t in Case B (−4%)
- 80 t in Case C (−66%)

On FIG. 15, we have shown the axial stresses and the flexure moments for the corner columns of the inner structure.

We observe for the most loaded columns a decrease of:

- 365 t in Case A
- 240 t in Case B (−35%)
- 99 t in Case C (−73%)

accompanied by a parallel increase of the maximum flexure moments. The increase of the flexural moments is compatible with the presence of the axial loads due to the loading schedule and this increase can be limited by introducing hinges at the column bottoms on foundation instead of the fixed ends considered in the study.

Finally on FIG. 16, we have indicated the value of the reactions in the antisismic springs (varying from 5 T to 11.6 T maximum values in agreement with the mechanical characteristics of the springs commonly used in practice.

We have tried on FIGS. 17 to 19 to sketch three architectural solutions among others possible.

In conclusion, it is the author’s belief that the use of the outside bracing for tall buildings will achieve economy of materials and down the construction time.

Supplement II—Exterior Bracing for Reinforced Concrete Structures

Framing of structures in a seismic area must be able to resist the earthquake without collapsing and with minimum damages.

A steel framing is well suited for earthquake because it is flexible and can absorb the energy from the earthquake.

After displacement, the steel framing comes back in almost the same initial position.

Concrete framings, because they are more rigid (brittle framings), have a disadvantage in absorbing the energy from earthquake and regarding the displacement.

Present building codes require for rigid ductile concrete frames a special design and detailing which provides sufficient reinforcement to ensure ductile frame behavior.

The trouble is that once the concrete is poured, all this tremendous amount of reinforcement is "locked" in the mass of concrete which is brittle.

When the earthquake acts on the building, the concrete is "activated" first and the concrete framing has a brittle behavior.

When the level of this brittle behavior is exceeded and the concrete has suffered excessive (rotations, displacement, cracks), then and only then is the steel reinforcement "activated" to full capacity.

It is my conclusion from all the data in respect to effects of earthquake on concrete buildings that there is no harmonious response from concrete and reinforcement in resisting the earthquake due to the following behavior:

When the earthquake acts on the reinforced concrete building, the concrete is “activated” first. The reinforcement is “locked” in the mass of concrete. Therefore, we have a brittle behavior which can cause very severe damages or collapse of the building.

Usually the reinforced concrete building has a concrete shear wall which must resist all of the earthquake force and a rigid frame which is the second line of defense.

The rigid frames (the second line of defense) must resist the portion of the earthquake which is absorbed by the rigid frames proportional to their absolute lateral stiffness versus the absolute lateral stiffness of the concrete shear walls. However, the code requires that the rigid frame absorbs minimum 25% of the total earthquake force.

Therefore, the present approach for the reinforced concrete building to resist seismic forces starts with a rigid frame and shear walls, which essentially is a brittle structure.

The follows a theoretical approach which, using a tremendous series of assumptions in the design and detailing of the reinforced concrete, tries to transform the brittle structure into a rigid ductile concrete frame.

In the last few years a new theoretical approach was developed: the dynamic inelastic design approach. This design starts from the same concrete framing, which is a brittle structure. Then, using this inelastic design approach, the brittle structure is transformed into a framing with yielding at certain locations, which means the structure is behaving like an elastic system capable of developing yielding in selected locations. The inelastic dynamic analysis approach is based also on a tremendous series of assumptions and techniques.

Let’s list some of the series of assumptions and techniques:

No. 1: Earthquake Accelerogram—This is the model of the motion of the ground at certain locations where the structure is built. This model can be selected as site-dependent or site-independent ground motions. We know that the site for a structure vs. a simple and is not completely known. Therefore, the ground motions are very complex and variable.

No. 2: The Dynamic Inelastic (Response History) Analysis of a structure is a step-by-step investigation, in very small time increments, of the response of a given structure to an earthquake accelerogram selected as mentioned in No. 1 above. The results of this dynamic inelastic (response history) analysis are therefore dependent on the earthquake accelerogram and how accurate is that earthquake accelerogram. (This is like a chain reaction.)

No. 3: Selection of a Hypothetical “Maximum Credible” Earthquake—This is the most severe ground motion which we assume will ever happen at the site of the structure. In case we have an earthquake which is more severe than the “maximum credible” earthquake selected, the dynamic inelastic analysis is not correct anymore.
No. 4: Selection of Inelasticity Only on the Horizontal Elements (yielding of beams at certain “hinge” locations)—This selection is coupled with definite limits on the corresponding “inelastic deformations.”

In my opinion, this item can be the subject of two theoretical research, but I don’t think we can relate it to the real world of reinforced concrete structures.

I want to make the following analogy:

Elevators, which are moving vehicles to transport people within a building, are designed with a factor of safety of F.S. = 5.0 (the wire ropes and the supporting frame).

During an earthquake, the reinforced concrete building starts shaking and moving and becomes practically an “elevator” carrying the people in the building. The ultimate strength design for reinforced concrete provides a factor of safety of F.S. = 1.7−1.8.

This factor is already a small factor of safety for a “moving vehicle.” To assume that inelasticity will occur only at selected locations (horizontal elements which are beams) means to further reduce this factor of safety because we narrow down the behavior of the framing to an assumed theoretical pattern.

I think this is not acceptable from the point of view of safety of the people who are in the building. I also think this is not in accordance with the practice in the construction industry for the simple reason that we cannot push the real frame of reinforced concrete all the way to a condition when the real yielding and hinges will occur. This yielding and hinges conditions is only a hypothetical condition and we really don’t know how the concrete building will behave, no matter how much computer time and how many iterations we use.

No. 5: For the Inelastic Dynamic Analysis, the columns and shear walls are kept elastic throughout their seismic response.

It is my feeling that we can use such an approach when we are dealing with wind loads, which are loads with a “mild” behavior and intensity.

In my opinion, it is wrong to apply this “mild” behavior and intensity to earthquake forces and to assume that columns and shear walls will stay “elastic” during earthquakes, when they are brittle members to begin with.

No. 6: The Dynamic Inelastic Analysis is greatly affected by the magnitude of the percentage of critical damping assumed. In selecting the percentage of critical damping, we have to make again another assumption.

I will stop here the long list of items included in the series of assumptions and techniques used in the present approaches for the design of reinforced concrete structures for earthquake forces. I consider that the present approaches are very theoretical and unrealistic. Therefore, we can have very severe damages or collapse of the reinforced concrete framing during a very severe earthquake. These present approaches can be summarized as follows: Starting with a brittle structure and then using a very theoretical, complex and special design and special detailing, this brittle structure is transformed into a ductile frame behavior or into an inelastic frame with confined inelasticity only to the horizontal elements (yielding of beams at certain “hinge” locations).

In order to provide a reinforced concrete structure which can withstand satisfactorily and more realistic the earthquake, I am proposing a system which is a combination of reinforced concrete rigid frame and vertical steel bracing located outside the face of the reinforced concrete building (see FIGS. 6A through 7).

In this proposed system, the reinforced concrete rigid frames are divorced from the outside vertical steel bracings, thus permitting the two framings to be installed independent from one another, using conventional methods of construction (rigid frames for reinforced concrete, bolted connection for the outside vertical steel bracings).

The outside vertical steel bracings is the third line of defense (life saver).

The proposed system has the following advantages:

(1) Because the outside vertical steel bracings resist a major part of earthquake force during the “elastic behavior” of the hybrid system, and because the outside vertical steel bracings must resist all the earthquake forces at the “failure stage,” the reinforced concrete rigid frames are not very congested with reinforcement, and detailing and construction are simplified.

(2) When the earthquake acts on the building, both systems (the reinforced concrete rigid frames and the outside vertical steel bracings) are activated and act in a harmonious response.

The reason for this harmonious response is due to the fact that we took out from the concrete a major part of reinforcement which was locked in the mass of brittle concrete and then we converted this reinforcement into outside vertical steel bracings which act independently.

Thus, when the earthquake acts, the reinforced concrete rigid frames and the outside vertical steel bracings act simultaneously and resist part of the earthquake forces proportional to their absolute lateral stiffness. This advantage is one of the most important advantages.

(3) After an earthquake, a reinforced concrete building has cracks and minor and major damages. It is very hard to evaluate if the building can resist another severe earthquake.

Repairs done to a reinforced concrete framing are very questionable as far as their strength and efficiency for the next earthquake.

With the proposed method, after an earthquake the outside vertical steel bracings can be inspected and parts which are damaged can be replaced or reinforced.

The reinforced concrete frames can also be repaired. The strength and the efficiency of the reinforced concrete building is guaranteed by the outside vertical steel bracings during the next earthquake. With the outside vertical steel bracings we don’t gamble: we have a third line of defense (life saver).

A reinforced concrete building cannot provide this insurance.
[0144] (4) The system has a good behavior resisting the earthquake because the combination of reinforced concrete rigid frames and the outside vertical steel bracings is improving the overall damping of the concrete building, due to stub connectors.

[0145] (5) The design can be done by desk calculators. Accuracy in designing the reinforced concrete rigid frames is no longer very critical, since any excess earthquake force can be easily absorbed by the outside vertical steel bracings.

[0146] (6) The outside vertical steel bracings provide a better environment for the people living in the reinforced concrete buildings because people have a feeling of comfort, knowing they have a better chance to survive a severe earthquake.

[0147] Regarding Advantages No. 1 and No. 2, I want to mention and clarify the “elastic behavior,” the “failure stage,” and the distribution of earthquake forces between reinforced concrete rigid frames and outside vertical steel bracings based on their absolute lateral stiffnesses.

[0148] In order to clarify these items, I will make a reference to a reinforced concrete building analyzed by John J. Driskell and H. C. from John J. Driskell & Associates, California, and U.C.L.A. extension (ACI) seminar, in December 1966.

[0149] The building has 30 stories, is 366 feet in height and an in plan is 80 feet by 180 feet.

[0150] On the outside of this concrete building, I am providing the vertical steel bracings.

[0151] The configuration in plan of the outside vertical steel bracings is shown in FIGS. 6A through 7.

[0152] For the outside vertical steel bracings, I am studying two alternates:

[0153] Alternate No. 1—All members of outside vertical steel bracings (columns, diagonals and struts) are W 14x730. This is a conservative solution.

[0154] Alternate No. 2—The members of outside vertical steel bracings are as follows:

[0155] Columns W 14x314

[0156] Diagonals W 14x87

[0157] Struts W 14x87

[0158] This is an economical solution.

Alternate No. 1

[0159] In order to find the distribution of forces, we must find the absolute lateral stiffness of the reinforced concrete frame and outside vertical steel bracings.

[0160] The Reinforced Concrete Frame

[0161] By applying a force H=250.Kips at the roof, we can calculate the total displacement of the roof relative to the base:

\[ \Delta_{\text{roof}} = \frac{H}{K_{\text{frame}}} = \frac{250 \text{ Kips}}{8.60 \text{ inches}} = \frac{250}{8.60} \text{ Kips/inch} \]

[0162] where the summation is for all members, columns and beams, from first story to the roof. We get:

\[ \Delta_{\text{roof}} = 8.60 \text{ inches} \]

[0163] and the absolute lateral stiffness:

\[ K_{\text{frame}} = \frac{H}{\Delta_{\text{roof}}} = \frac{250 \text{ Kips}}{8.60 \text{ inches}} = \frac{250}{8.60} \text{ Kips/inch} \]

[0164] per one reinforced concrete frame.

[0165] The Outside Vertical Steel Bracings

[0166] By applying a force H=250.Kips at the roof, we can calculate the total displacement of the roof relative to the base:

\[ \Delta_{\text{roof}} = \sum \frac{K_{\text{vertical bracings}}}{E_{A}} \Delta_{i} \]

[0167] where the summation is for all members, (columns, diagonals and struts) from the first story to the roof. We get:

\[ \Delta_{\text{roof}} = 0.93 \text{ inches} \]

[0168] and the absolute lateral stiffness:

\[ K_{\text{vertical bracings}} = \frac{H}{\Delta_{\text{roof}}} = \frac{250 \text{ Kips}}{0.93 \text{ inches}} = \frac{250}{0.93} \text{ Kips/inch} \]

[0169] per one vertical steel bracing. Based on these stiffnesses, the earthquake forces will be resisted by the outside vertical steel bracings and the reinforced concrete frames. About 75% of earthquake forces will be resisted by outside vertical steel bracings and about 25% will be resisted by the concrete rigid frames during the “elastic behavior” based on absolute lateral stiffness.

[0170] The UBC Code also requires that the vertical steel bracings must resist the total earthquake force. I call this criteria the “failure stage.”

[0171] In this Alternate No. 1, the outside vertical steel bracings are overdesigned. Because of this overdesign, during the “elastic behavior,” the outside vertical steel bracings absorb a tremendous percentage—75%—of the earthquake force.

[0172] During the “failure stage,” the outside vertical steel bracings which resist all the earthquake force are stressed to less than half capacity because the members are overdesigned.

[0173] This alternate is not an economical solution.
Alternate No. 2

Following the same procedure as in Alternate No. 1, we get:

The Reinforced Concrete Frame

\[ k_{\text{Frame}} = 29 \text{ Kips/inch} \]

per one reinforced concrete frame.

This is the same value as in Alternate No. 1 because we did not change the frame.

The Outside Vertical Steel Bracings

\[ k_{\text{bracings}} = 77 \text{ Kips/inch} \]

per one vertical steel bracing.

This is a smaller value than in Alternate No. 1 because the sizes of steel members are smaller.

Based on these absolute lateral stiﬀnesses, the outside vertical steel bracings resist about 50% of the earthquake forces and the reinforced concrete frames resist about 50% of the earthquake forces during the “elastic behavior.”

During the “failure stage,” the outside vertical steel bracings must resist the total earthquake forces. In Alternate No. 2, the steel members are stressed close to their capacity during the “failure stage” because the steel members have a smaller size than in Alternate No. 1.

This is an economical design. The outside vertical steel bracings add about 10% to the cost of the building, which is compensated by a simpler reinforcement for the concrete rigid frames.

Additional Comments

Concrete floors must be designed as a diaphragm to transfer horizontal earthquake forces to the outside vertical steel bracings. We must design for shear (this may require the exterior bays to have a thicker slab) and for top and bottom chords.

Because of outside vertical steel bracings, all members must be designed for 1.25 times-earthquake forces (U.B.C. requirement for braced frames).

We must design the stub connectors which connect the reinforced concrete building to the outside vertical steel bracings.

It is my belief that the reinforced concrete columns must have at the first story only diagonal ties at 45 degrees in both directions on each face of the column. These ties at 45 degrees are more efficient than a spiral reinforcement (see FIG. 6C). This system of ties at 45 degrees can be tested in the laboratory to find out how efficient they are.

In conclusion, this system of outside vertical steel bracings is especially important in countries like U.S.A., Turkey, Yugoslavia, Iran, South American and China where the majority of the buildings are built with reinforced concrete because of economical reasons.

The system I am proposing can make the life of the reinforced concrete buildings longer and save the life of the people living and working in these buildings.

Interior Diagonal Bracing—Reinforced Concrete Structures

The interior diagonal bracing for reinforced concrete structures is similar to that described previously for steel structures (see FIGS. 7A, 7B).

The outside vertical bracing system has excellent ductility under elastic load reversals (i.e., wind and moderate seismic forces). This property is inherent in the proposed system due to the elastic hinges (i.e., soft connections).

Elastic Hinge No. 1 is stub connector Nos. 1, 2, 3, and 4.

Elastic Hinge No. 2 is the point of contact between the column (interior or exterior column) and the diaphragm floor and the floating floor.

Elastic Hinge No. 3 is the column splice (flange plates and web plate).

(For location of Elastic Hinges No. 1, Soft Connections, see FIG. 1 through FIG. 3.

Under inelastic load reversals (i.e., severe seismic forces), the elastic hinges have the potential of becoming plastic hinges, thus providing an excellent ductility. The elastic hinges and the plastic hinges act as structural fuses protecting the framing.

Another characteristic of the system is its ability to produce effective damping action. This results from the division of the framing of the building into two sub-frames.

Sub-frame No. 1 consists of all of the diaphragm floors and the outside vertical bracing. The diaphragm floors and the outside vertical bracing are connected through stub connectors Nos. 1, 2, 3 and 4.

Sub-frame No. 2 consists of all of the floors supported between the diaphragm floors (floating floors) and all of the columns of the building (interior and exterior). Sub-frames Nos. 1 and 2 are connected with elastic hinges which can develop into plastic hinges, insuring ductility in response to a wide range of load intensities.

As a result of the design (interconnection) of Subframes Nos. 1 and 2, wind and seismic forces are transferred from the building to the foundations, or vice versa, in a series of three motions.

Motion No. 1—the bending of all the columns supported between the diaphragm floors, transfers these forces from the floating floors to the diaphragm floors. This motion includes the rotations of Elastic Hinges Nos. 2 and 3 located on all the columns (interior and exterior).
[0203] Motion No. 2—the bending of stub connectors Nos. 1, 2, 3 and 4 (Elastic Hinge No. 1) about the weak axis Y-Y, transfers the horizontal forces from the diaphragm floors to the outside vertical bracing (outside horizontal strut).

[0204] As these wind and seismic forces are transferred down to the foundations, Motion No. 3 occurs. This consists of the bending of stub connectors Nos. 1 and 4 (Elastic Hinges No. 1) about their strong axis X-X to transfer the overturning of the outside vertical bracing to the exterior columns D and G. These three motions impart a soft behavior to the whole framing and result in a smaller base shear.

Key Words


BRIEF DESCRIPTION OF THE DRAWINGS

[0206] FIG. 1 Generalized Structure
[0207] FIG. 1A Building Drift
[0208] FIG. 2 Framing Detail—Exterior Wall
[0209] FIG. 3 Member Loading—Seismic Case
[0210] FIG. 3A Exterior Elevation
[0211] FIG. 3B Interior Bracing—Sections
[0212] FIG. 4 Isometric Bracing model of the whole framing and outside vertical bracing.
[0213] FIG. 5 Deflected shape of the column (interior or exterior) between diaphragm floors, under seismic or wind forces.
[0214] FIG. 6A Isometric elevation outside vertical steel bracings connected to a concrete building.
[0215] FIG. 6B Plan of Concrete Building with Steel Exterior Bracing
[0216] FIG. 6C Ties at 45 degrees in the reinforced concrete columns at 1st story only.

[0217] FIG. 7 Reinforced Concrete Building with Steel Exterior Bracing
[0218] FIG. 7A Reinforced Concrete Building with Steel Bracing
[0219] FIG. 7B Concrete Building—Section at Interior Bracing
[0220] FIG. 8 Spectrum of Horizontal Response
[0221] FIG. 9 Interior Structure
[0222] FIG. 10 Stability Structure
[0223] FIG. 11 Location of Anti-Seismic Springs
[0224] FIG. 12 Deflection of Stability Structure
[0225] FIG. 13 Deflection of the Interior Structure
[0226] FIG. 14 Axial Forces in Tons on Columns and Bracings of Stability Structure
[0227] FIG. 15 Axial Forces in Tons and Moments in Tons at corner columns of Interior Structures
[0228] FIG. 16 Stresses in Springs (Tons)
[0229] FIG. 17 Exterior Elevation 1
[0230] FIG. 18 Exterior Elevation 2
[0231] FIG. 19 Exterior Elevation 3
[0232] FIG. 20 Is a partial perspective view of a building employing a bracing system of the present invention.
[0233] FIG. 21 Is a close up view of the bracing system of FIG. 20.
[0234] FIG. 22 Is a top plan view of the building of FIG. 20.

I claim:

1. A system for reinforcing tall buildings comprising an internal frame comprising a plurality of horizontal floor beams and a plurality of vertical columns extending between adjacent floor beams, the system comprising diagonal bracing members connected between vertically spaced floor beams at an angle to both the floor beams and the columns.

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