THE BEHAVIOUR OF BEAMS IN REINFORCED CONCRETE FRAMES UNDER THE COMBINED ACTION OF VERTICAL AND HORIZONTAL LOADINGS

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1. Introduction

The philosophy of structural design of weak-beam, strong-column type of frames has been widely adopted for seismically design of reinforced concrete buildings during the latest decade. Little informations have, however, been obtained on the combined effects of working vertical load and cyclic horizontal inertia force on such type of frames, though some investigations without the definite philosophy of such type of ductile frames, have been reported[1]-[4].

Experimental study is being continued in our laboratory to have the various informations concerning the behaviours of such type of frames both during and after earthquakes. Two of the study results have been presented in Ref. 5 and 6 on the vertical load carrying capacity of beams in reinforced concrete frames with the experience of reversed horizontal loading and on the stability of the columns of multi-story frames with the experience of horizontal loading respectively.

When a weak-beam, strong-column type of reinforced concrete frame is subjected to a series of reversed horizontal loading cycles along with a previously applied constant working vertical load on its beams, the two extreme ends of the beams gradually lose their rotational stiffness. The beams then gradually turn to behave like simply supported ones, by decreasing the fixed end moments and also by increasing the beam center moment. These phenomena result into the gradual increase of the vertical deflection along with the beam during the reversed horizontal loading. Now the practical problems that may arise due to this increase of vertical deflection along with the beam, are to have to use an uneven and deflected face of floors and roofs, causing ponding of water on roofs which may result in accelerated deterioration. And also for instance, the door and window panels may not be closed or opened properly during emergency etc.[7]. The possibility of these phenomena may be overcome by providing enough shear walls for the buildings to reduce the maximum interstory deflection during reversed horizontal loading.

There are some research examples of behaviour of beams in frames under combined action of vertical and horizontal loading. Ernst[9] reported that the midspan deflection of beams were increased by 70 % to 140 % after the first yield of a critical section in reinforced concrete portal frame specimens, subjected to working vertical as well as cyclic lateral loads. Yamada[10] found also the gradual increase of vertical deflection along steel-concrete composite beams subjected to alternately repeated cyclic loading with incremental deformation amplitude under constant vertical load. Though the structures tested by him were steel portal frames, the slabs which were being sustained by the steel beams, were made of reinforced concrete.

The aspect of this paper is to observe both analytically and experimentally the behaviour of beams in reinforced concrete frames under the combined action of vertical and horizontal loadings. In particular, focus is placed on the continuous progress of the vertical deflection along the length of the beams during reversed horizontal loading cycles, with working vertical load on the beams applied. The vertical load carrying capacity of these beams with the experience of horizontal loading is also verified[11]. The abstract of this paper was already reported in Ref. 9 and 10.

First a symplified analytical model, which has been developed to explain the phenomena of the above mentioned continuous progress of the vertical deflection of the beams, is presented.

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2. Derivation of Analytical Model

An analytical procedure is developed to suit the experimental study to be described in the following chapters, which deals with a series of statically one degree indeterminate, single-bay, single-story reinforced concrete frames, with two mechanical hinges at the bottoms of their both columns.

Based on the following assumptions, the vertical deflection of beam center under combined horizontal and vertical loading is going to be calculated.

1) The basic assumptions being considered are, (a) a plane section of any member remains plane at any stage of loading, (b) there does not also occur any shear deformation of joints, and (c) the members are axially rigid.

2) The members of the frames are devided transversely into desired numbers to form a linear mesh of finite elements, which are connected to each other on their both ends as shown in Fig. 1 for the model specimen, which was divided at an interval of 1 cm. The bending moment, curvature, deformation etc. along the entire length of an element will take the corresponding values obtained for its center. The loads may only be applied at the centers of the elements.

3) The tri-linear moment-curvature \((M-\Phi)\) relations for the sections of the elements of the members for various axial force levels, \(N\) may be obtained by the method as explained in Ref.5. To suit the experimental study, for the beams, the portion between the crack and the yield moments of these curves may be taken to be represented by the straight line joining the cracking point, corresponding to the axial force resulting from the effect of the working vertical load, and the yield point corresponding to the maximum axial force developed in the beam when it yields under the combined loadings. On the other hand, for the columns, it may be done by joining the cracking and yield points corresponding to the minimum and the maximum axial force which occur in the columns under the combined loadings. The effect of the slipping-out of the tensile reinforcement may be considered by increasing the yield values of curvature, which is calculated from that given in the R.C. Code of AIJ for the corresponding values of rotations, Fig 2 shows the way how to determine these yield curvatures from the corresponding yield rotations. The value of \(\Phi_R\) in Fig. 2 (c) takes the form, \(R_e/R_o=\Phi_R=(0.043+1.64 Np_t+0.043 A/D+0.33 Np_t(d/D)^2)\). In this study, this effect has been considered for the beams only. For the relatively strong columns, which are being used in the experimental study, it has been checked analytically that there are almost no effects on the responses, whether the slipping-out effects of the tensile reinforcement of columns are considered or not. Thus the calculated moment-curvature relations for different members, are shown in Fig.3, for the specimens of Fig 9.

4) Fig.4 shows the degrading stiffness properties, considered to trace the paths in the \(M-\Phi\) diagram for the cyclic loadings. To trace the curves for the loading portions, it is assumed that either it aims towards the experienced maximum moment-curvature point, or just traces on the envelope curve. For the unloading portions, when the experienced maximum moment lies between the crack and the yield moments, it traces such that the residual curvature would be about the one-third of the difference.
between the cracking curvature and the ever experienced maximum curvature\textsuperscript{25}. Thus obtaining the curvatures at the centers of the elements along the length of the members, the member deflections can be calculated.

5) Under reversed horizontal load, for higher amplitudes, the bending moments at different critical sections of the beam reach their yielding values, resulting in the formation of yield hinges. These yield hinges are formed alternately at the both end sections of the beam. It may be considered that there occurs a flow of curvatures in these regions over a certain length along the beam axis. For simplicity of calculation, for this study, this length of the yield hinges was considered to be \((d_a/2+d_s)\) which is a little modified form of that reported in Ref. 5. Table 1 shows the values of the yield hinge length, proposed by different authors, calculated for the model specimens being used in the experimental study\textsuperscript{25}. After yielding of these sections, the plastic curvatures \(\phi_p\), constant over these lengths, are added to the curvature distributions of the elements in these regions, at the starting of yielding of those critical sections for any instants of loadings. Then the added plastic curvatures remain active as long as the bending moments of the respective elements do not change their signs.

6) For the statically indeterminate structures, the values of bending moment for the elements along the members may be calculated by considering the geometrical compatibilities of the frames. In this study the only one geometrical compatibility is to be considered for the one degree indeterminate structures, which was done in such a way that the vertical deflection of the beam-column joint of the right column becomes zero. Also the magnitude of the plastic curvatures along the yield hinge length may be calculated in the same way to fulfill the geometrical compatibility, providing that the bending moment at that critical section takes its yield value.

7) The qualitative deflected shapes of the model specimen at the peak and at the end of the positive horizontal loading cycle are given in Fig. 5 (a) and 5 (b) respectively. From the geometry of these figures the following equations can be reduced for both the cases, with the sign of \(\theta\) taken clockwise,

\[
\delta_{el} = \mp \theta \cdot \frac{L}{2} \tag{1}
\]

\[
\delta_{es} = (\theta \pm \theta' - \theta) \cdot \frac{L}{2} \tag{2}
\]

\[
\delta_{ce} = (R - \theta) \cdot \frac{L}{4} \tag{3}
\]

\[
\delta_{ca} = (R - \theta') \cdot \frac{L}{4} \tag{4}
\]

[Fig. 3 Calculated Moment-Curvature Relations]

[Fig. 4 Degrading Stiffness Properties of Moment-Curvature Relations for Analytical Model]

Table 1 The Comparison of Equivalent Yield Hinge Length Proposed by Different Authors\textsuperscript{32}

<table>
<thead>
<tr>
<th>Proposer</th>
<th>Equivalent Yield Hinge Length, (L_e)</th>
<th>Calculated (Lu) (cm) for the Model Specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baker</td>
<td>(2(0.8k+0.048c))</td>
<td>21.0</td>
</tr>
<tr>
<td>Malik</td>
<td>(2(0.5d+0.05c))</td>
<td>19.6</td>
</tr>
<tr>
<td>Sawyer</td>
<td>(2(0.25d+0.075c))</td>
<td>15.4</td>
</tr>
<tr>
<td>Authors</td>
<td>(d^2+0.8d_{0}) (End)</td>
<td>18.2</td>
</tr>
<tr>
<td></td>
<td>(2d_{0}) (Center)</td>
<td>26.6</td>
</tr>
</tbody>
</table>

(a) At the Peak of Horizontal Loading

(b) At the End of Horizontal Loading

[Fig. 5 The Geometry of Deflected Shape of the Specimens for Horizontal Loading]
where, $\theta$, $\theta'$, $\theta''$ and $R$ are the deflection angles as shown in the figures, and $\delta_{BL}$, $\delta_{BR}$, $\delta_{CL}$, and $\delta_{CR}$ are the member deflections of the left half-length beam, right half-length beam, left column and right column respectively.

By using the Eq.s (1)–(4) the interstory deflection angle $R$ and the vertical deflection of the beam center $\delta_v$ become

$$R = -\left(\delta_{BL} - \delta_{BR} + 2\delta_{CL} + 2\delta_{CR}\right) \frac{1}{L} \quad \text{(5)}$$

$$\delta_v = -\left(\delta_{BL} - \delta_{BR} - 2\delta_{CL} + 2\delta_{CR}\right) \frac{1}{2L} \quad \text{(6)}$$

The qualitative bending moment and curvature diagrams of the frame under different stages of loadings calculated by the above analytical method are shown in Fig. 6 (a) and 6 (b). From these figures, it can be observed that the residual moment and curvature diagrams at the end of positive and negative loading cycles are not the same.

3. Preparation for Experimental Study

An interior frame of a six-story, single-bay reinforced concrete building was selected to be the prototype for the specimens tested for this study. Fig. 7 shows the plan and the elevation of the frame, which was designed according to the R.C. Code of AIJ. It was considered that the columns of...
this frame would carry the one-third of the horizontal inertia force produced in its span during earthquake, while the rest two-third would be taken up by the end walls provided in that building. The section properties of the Rth (roof) and the 4th floor beams, according to the results of the above design, are given in Table 2. Table 2 also shows the amount of working vertical load to be carried by the beams of the respective floors.

The beams of the model specimens were provided with the calculated amount of reinforcement ratios as well as the working vertical load as shown in Table 2. The tensile reinforcement ratios throughout the length of the beams for both the upper and lower positions in the cross section were selected to be the same as those which had come out from the design for the upper position at the end section of the respective beams of the prototype frame. Thus the resulting ratio, \( \gamma \) of compressive reinforcement to the tensile reinforcement along the beams of the models became unity. This was done for the simplicity of the construction of the reinforcement skeleton, which is nowadays being used in commercial cases. The hysteresis loops, calculated by the above mentioned analytical method are shown in Fig. 8, in terms of the horizontal load, \( P \) versus the interstory deflection angle, \( R \) for a typical 4th floor model specimen with three different values of \( \gamma \), (1.0, 0.66 and 0.33) for its beam. It can be seen by comparing these three figures that the tensile reinforcement at the both end sections of all the three beams yields at about 0.005 radian of interstory deflections, but for the cases with the values of \( \gamma \) other than unity, there appears one more yield point which corresponds to the yielding of the tensile reinforcement at the lower layer of the sections at a distance of one-fourth span length from the both ends of the beams, where the reinforcement ratio is changed for its bending up from the lower layer to the upper. Table 3 shows the progress of the vertical deflection of beam center for the above three cases with respect to the interstory deflection angle. In this Table it can be seen clearly that the beam center deflection increases continuously with the progress of the deflection for all the three cases, showing almost no reasonable effects of the variation of the value of \( \gamma \). For the above calculations, the material properties, the horizontal loading program and the working vertical load (33%) on these beams were considered to be the same for all the three cases.

4. Experimental Study

4.1 Test Specimens

Two typical one-fifth scaled model specimens BS-41 H and BL-61 H were supposed to represent the Rth and 4th floor beams of the above mentioned prototype frame without the floor slabs. As the tensile reinforcement ratios at the center of the beams in the case of models, compared to those of the prototype, were rounded down from 0.52% to 0.37% and rounded up from 0.54% to 0.84% in the cases of Rth and 4th floor beams respectively, the amounts of the working vertical load were also changed almost in the same ratios as shown in Table 4. Thus the level of the vertical loads happened to be about 30% of the calculated ultimate capacity for vertical loads of the beams without the experience of horizontal loading. For the reason of this increased level of working vertical load, the web reinforcement ratios were also made double to the designed ones to ensure sufficient ductility. The both types of reinforcement ratios in the columns of these specimens were provided sufficiently to ensure the weak-beam, strong-column mechanism.

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Table 3: The Values of Beam Center Deflection, \( \delta_v \), Increasing Along with the Interstory Deflection Angle, \( R \), for Different Values of \( \gamma \)

<table>
<thead>
<tr>
<th>( R ) (rad.)</th>
<th>( \delta_v ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( r=1.00 )</td>
<td>( r=0.66 )</td>
</tr>
<tr>
<td>0.0025</td>
<td>1.45 (1.40)</td>
</tr>
<tr>
<td>0.0050</td>
<td>1.80** (1.81)</td>
</tr>
<tr>
<td>0.0100</td>
<td>3.05 (3.29)</td>
</tr>
<tr>
<td>0.0200</td>
<td>5.50 (5.36)</td>
</tr>
</tbody>
</table>

( ) Values Corresponding to Residual Interstory Deflection
* Upper Reinforcement Yield
** Lower Reinforcement Yield

Table 4: The Variation of Different Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Beam</th>
<th>Column</th>
<th>Working Vertical Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>End Center</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BS-41H</td>
<td>0.37</td>
<td>0.37</td>
<td>600</td>
</tr>
<tr>
<td>BS-41V</td>
<td>-</td>
<td>-</td>
<td>2.49</td>
</tr>
<tr>
<td>BL-61H</td>
<td>0.84</td>
<td>0.84</td>
<td>1.070</td>
</tr>
<tr>
<td>BS-61H</td>
<td>1.450</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>BS-61V</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Stirrups & Hinges

The same values for both tensile and compressive reinforcement

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In this study the number of two different types of specimens totaled five. Two of which were the Rth story floor beam frames and the rest three were the 4th story floor beam frames. Fig. 9 shows the overall dimension and the reinforcement of all the specimens. For simplicity of construction, in the case of the 4th floor, the portions of the columns developed above the beam level were not included. To observe the effects of higher level of working vertical load, the specimen BS-61 H was tested under an increased level of 45% of its ultimate vertical capacity without the experience of horizontal loading.

The mechanical properties of the materials used for the construction of the models are shown in Table 5 (a) and 5 (b).

4.2 Loading and Measurement

Three specimens with their numbers ending with the alphabet 'H' (eg. BS-41 H) were tested under displacement controlled reversed horizontal load, during which there was a previously applied constant two points working vertical load (W) on the beam as explained above. At the end of the last cycles of horizontal loading, the vertical load on the beam was increased up to the failure of the beam in terms of three gradually increasing repeated vertical loading cycles in the case of 'BS' series only. On the other hand the same type of specimens in 'BL' series are being used in the long term (creep) test under the same working vertical load, to get the time-dependent informations of these beams of the frames with the experience of reversed horizontal loading. However the time-dependent informations are not included in the scope of this paper. In this study, the working axial force in the columns is not
included, as it has very negligible effects on these types of strong columns. The other two specimens with the alphabet V in their numbers were tested only under three gradually increasing repeated two points vertical loading cycles. Fig. 10 shows the setup for the test as well as the loading and the measuring apparatus. Both the positive and the negative horizontal loadings were applied on the outer surface of the columns at the beam axis level, while the two points vertical load was applied at the one-third points on the beams. The vertical loading apparatus was designed in such a way that the top of the specimens could move freely in its plane. Oil jacks were used to apply both the types of loadings.

Seven Displacement Transducers (D. T.) were used along the length of the beam to measure the vertical deflection of the beam while six more were used to read the two lateral (horizontal and transverse) directional deflections of both the columns. Twenty Wire Strain Gages (W. S. G.) were used on concrete to measure the either side surface strains at ten different points on the beam and columns. Twenty more embedded W. S. G. were used on the longitudinal reinforcement of the beams of 4 th story floor beam specimens only. In addition to these, Contact Gages (C. G.) were used at six places on the beam and beam-column joints to measure the crack width at the different levels of horizontal and vertical loading. However, all the experimented results has not been included in this paper.

Fig. 11 shows the program for the reversed horizontal loading which is the same as that used in Ref. 5 and 6.

5. Experimented and Calculated Results and Discussions

From the test results of all the five specimens, it was observed that the specimens of both the Rth floor and the 4 th floor beams show almost similar behaviour under different loadings. Here, in this paper, focus is going to be placed on the 4 th floor beam specimens, because the qualitative test results of the Rth floor beam specimens are more or less similar to their corresponding ones of 4 th floor beam specimens.

5.1 The Crack Progress Observed on Beams During Horizontal Loading

The crack patterns of BL-61H under working vertical load and at the end of the 2nd, 4th, 6th and 8th (last) cycles of horizontal loading are given in Fig. 12 (a). It was observed in the cases of all the specimens that the working vertical loading caused a few hair cracks on beam just under the two points of vertical load and or at the two extreme ends. With the application of reversed horizontal loading, the flexural cracks began to appear at the interval of 7~10 cm, starting from the bottom surface in the middle-third portion of the beam. As the interstory deflection angle was increased, these cracks began to extend deeper and wider inside the beam as new cracks appeared and spread on both sides along the length of the beam. On the other hand, on the top surface there appeared almost no new cracks except those at and very near the two extreme ends of the beam.

If the figures of cracks of both BL-61H and BS-61H at the end of the 8th cycle of horizontal loading in Fig. 12 (a) and 12 (b) are compared, it will be found that there are almost no significant effects of the increased level of working

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vertical load on the crack patterns.

5.2 The Hysteresis Loops of Horizontal Load Versus Interstory Deflection Angle.

The experimented and calculated load-deflection \((P \cdot \delta_n)\) curves for reversed horizontal loading test of two different specimens BS-41H and BS-61H are given in Fig. 13 (a) and 13 (b), and those for BL-61H was already given in Fig. 8 (a). Up to the interstory deflection angle of 0.5 percent, the hysteresis loops were spindle shaped for all the specimens tested under horizontal loading. Then as the loading was increased beyond the deflection angle of 0.5 percent, the shape of the loops appeared to be of inverted 'S' type one. It was observed by comparing the hysteresis loops of BL-61H and BS-61H that there were almost no remarkable effects of the increased level of working vertical
load on the pattern of the loops but the required level of horizontal load for determined interstory deflection was decreased in the case of BS-61H. It can be seen from the comparison of these figures that the calculated curves resemble well enough the experimented ones. Thus the analytical method seems to be quite reliable for the prediction of the hysteresis loops under the combined loadings.

5.3 The Increase of Vertical Deflection of Beam During Horizontal Loading

Fig. 14 shows the vertical deflection progress at the center of the beam ($\delta_v$) during reversed horizontal loading test (BS-61H). As it is seen in Fig. 14, it was not possible to keep the working vertical load on the beam at its constant level during the horizontal loading tests. The vertical load, which was decreasing with the increase of the vertical deflection of the beams, was being readjusted at the end of every horizontal loading cycle. Fig. 15 (a) and 15 (b)

![Diagram of deflected shapes of beams under combined loadings](image)

(a) At the Peak of Horizontal Loading Cycles  (b) At the End of Horizontal Loading Cycles

**Fig. 15** Deflected Shapes of Beams under Combined Loadings (BL-61H)

![Graph of interstory deflection angle](image)

**Fig. 16** Experimented and Calculated Values of Interstory Deflection Angle, $R$ Versus the Beam Half-length Deformation Angle $\theta + \dot{\theta}$, at the Peak and at the End of Horizontal Loading Cycles
shows the deflected shapes of the beam of BL-61H at the peak and at the end of reversed horizontal loading cycles respectively. At it is seen from these three figures, it was observed that the vertical deflections along the length of the beams were increasing continuously during the reversed horizontal loading tests in the cases of all the three specimens. Moreover, it can be seen by the comparison between Fig. 15 (a) and 15 (b) that the vertical deflections along the beams were not decreasing when the horizontal loading was withdrawn at the end of every positive and negative loading cycles. This increase of the vertical deflection of beams was more for the higher level of applied working vertical load during the initial stages of horizontal loading.

The experimented as well as the calculated values of the interstory deflection angle, $R$ versus the beam half-length deformation angle, $\theta + \theta'$ at the peak and at the end of every cycle of positive and negative horizontal loading of all the three specimens are given in Fig. 16. In this figure, the way of the progress of vertical deflection of beam centers during horizontal loading can be observed clearly. It is also seen that the calculated values obtained by the analytical method described above, lies between 80 % - 95 % of the experimented values for both at the peaks and at the ends of the horizontal loading cycles. Thus the method seems to be quite able to predict the progress of the vertical deflection of beam center under the combined loadings.

5.4 The Vertical Load Carrying Capacity of Beams

Experimented and calculated load-deflection ($W$-$\delta_v$) curves under vertical loading of all the specimens are given in Fig. 17, in which the vertical load carrying capacity of the specimens with or without the experience of horizontal loading can be compared. The straight line portions of the curves extended horizontally are the effects of the increased vertical deflection during horizontal loading. The actual behaviour of this portion for BS-61H was already given in Fig. 14. Though the specimen BS-61H was carrying the working vertical load of 45 % of its ultimate capacity, the ultimate vertical load carrying capacity of this specimen with the experience of horizontal loading was about 95 % of that of BS-61V, which was tested without the experience of any horizontal loading. On the other hand, at the end of horizontal loading, the stiffness against vertical loading of BS-61H compared to BS-61V was decreased to a considerable extent. This phenomenon was also observed between BS-41H and BS-41V.

The crack patterns of BS-61H and BS-61V for ultimate vertical load are given in the last two figures in Fig. 12 (b).

Table 6 shows the comparison between experimented and calculated ultimate vertical load carrying capacities of the four specimens of 'BS' series. The calculated ultimate vertical load carrying capacity of these specimens were obtained by considering the collapse mechanism of the beams by forming simultaneous yield hinges at their both ends and also under the vertical loading points. From the experimented results it is seen that the specimens with the
experience of horizontal loading also show nearly the same ultimate vertical load carrying capacity as that for their similar specimens without the experience of horizontal loading. This means that the reversed horizontal loadings have little effects on the ultimate vertical load carrying capacity of the beams in seismic-resistant ductile reinforced concrete frames even if the maximum interstory deflection angle caused by horizontal loading reaches the great value of about 2.0 percent.

From Table 6 and Fig. 17, it is seen that the analytical method can fairly predict the ultimate vertical load carrying capacity and also the beam center deflections under vertical loading for beams in frames with or without the experience of reversed horizontal loading.

Table 7 shows the calculated values of shear capacities of the frame members and the column faces at beam ends. The values were obtained from the shear strength equations as described in Ref. 5. It can be seen from Table 7 that shear is not a critical factor, except that shear cracks may occur in the beams. The calculated shear in the columns for the ultimate vertical load is only about 1.0 ton, which is not a critical factor also.

6. Conclusions

Based on the study reported herein, the following conclusions may be made.

1) An analytical method has been developed to predict the behaviour of single-bay, single-story reinforced concrete frames under the combined effects of vertical and horizontal loading. This method includes the assumptions of finite element theory, the equivalent yield curvature, the plastic curvature of any section and also the hysteretic properties in the moment-curvature relation.

2) The 1/5th scaled single-bay, five reinforced concrete frame specimens have been tested to study the behaviour of the beams of frames under combined action of vertical and horizontal loadings. Two of the specimens were considered to represent the roof floor beam, while the rest three were to the 4th floor beam of a single-bay, six-story building, designed according to the R.C. Code of AIJ. With these specimens, the effects of the combined vertical and horizontal loads on the behaviour of the frames have been observed. And also the effects of the increased level of the working vertical load during the combined loading have also been investigated.

3) It has been found from the experimental results that the reversed horizontal loading produces a considerable continuous progress of the vertical deflection along the length of beams, under working vertical load on the beam. It was also observed that the deflections become even greater at the ends of the horizontal loading cycles than those at the peaks of them.

4) The experimented results have been compared with those found from the calculations, using the above mentioned analytical method. It has been found that this method is quite reliable for the prediction of the hysteresis loops and also the gradual increase of vertical deflection at the center of the beams during the horizontal loading cycles, while there is a previously applied constant working vertical load on the beams.

5) Even when the beams are carrying a considerable amount of working vertical load, the reversed horizontal loading does not significantly affect the ultimate vertical load carrying capacity of the beams of seismic resistant ductile frames as long as the previously applied maximum interstory deflection angle caused by reversed horizontal loading is less than 2.0 percent.

6) Further study is needed to get the general conclusions by conducting experimental studies on beams of frame with various horizontal loading programs as well as with the different patterns of improved reinforcement against the increase of vertical deflection of the beams.
7. Acknowledgements

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文献6において、水平荷重をうけたり降伏型フレームの曲げ柱は鉛直荷重に対する抵抗力を著しく低下することを示した。本研究では、より降伏型フレームの他の問題点であると考えられる、はりそのものの鉛直荷重に対する抵抗性能を考慮したものである。特に、本論文は、水平荷重載荷時および除荷後のはりの鉛直たわみ性状を解析的、かつ実験的に明らかにすることを目的としている。水平荷重載荷経験をもつはりの時間依存変形については、現在、実験実施中であり、その関連に関しては、別報で結果を報告する予定である。まず、はりの鉛直たわみ性状を予測するための解析手法を開発したので、そのことについてのべる。この解析法は次の仮定に基づいて作成したものである。

1) 部材断面の平面保持等は考慮する。
2) フレーム各部材を材軸に直交分割した要素を考える（図-1）
3) 要素にトリニアン型の曲げモーメント-曲率曲線を考える（図-3）。ただし、はり端部、柱端部も荷重変化に伴い、軸力が変動するが、変動幅が小さいので、それぞれ単一の曲げモーメント-曲率曲線を仮定する。
4) 曲げモーメント-曲率曲線（包絡線）において、降伏耐力はε関数で求めるが、降伏時曲率の値は降伏剛性低下率に関する文献12の提案式に基づいて求める。
5) 曲げモーメント-曲率曲線の曲力時および再加荷時のルールは図-4に示すとおりでスリップモデルに仮定する。
6) 要素のどれかが降伏しぼむに達すると、要素近傍にある一定幅の降伏域が発生して、塑性流れが生じるものとする。（図-6のフレームのように1次不静定の場合は降伏域が1か所の場合、適合条件から塑性流れの大きさが定まるが2か所できると断面をとる）
以上の仮定に基づいて解析を行う場合、図-1のフレームの変形を図-5のように考えると、層間に変形角とはり中央たわみが式（5）（6）によって求めることが期待できる。
このような解析法の有効性を確かめるために以下に示す実験的研究を実行した。まず、そのための準備として図-7に示す建物を考え、試験体の設計に入ることが解釈されると、相対的に大きな地震力をうけるとされる中低層建物を対象とし、6層建物を想定した。そしてそれの屋根床板と4階床板をとりあげた。ただし、試験体では製作を容易にし、かつ基本である理由で、全断面通過筋ることとしたので、上記解析法を用いて、はり断面の複数比γの影響を示した。図-8は本実験で用いた標準試験体をもとに、配筋を変えた場合に、水平荷重-変位関係がどう変化するかを示したものである。一定鉛直荷重のレベルは設計荷重としている。同図より、各ケースで崩壊メカニズムは異なるもので、表-3より、鉛直たわみの増加傾向はほとんど変わらないことがわかる。

以上により、実験では、はりは通過筋に統一している。また試験体では、このほかに、次のような単純化を許している。Ⅰ）単体の1/5の大きいで、床スラブはつけない。Ⅱ）はり両側隣接柱は下部半断面のみとする。また、柱の頂部への鉛直荷重（軸力）は加えない。Ⅲ）は主筋および補筋は基準とする。そして表-4に示すように、試験体は全部で5体である。屋根床板が2体、4階床板が3体である。4階のそれは鉛直荷重レベルの影響をみると4体となっている。試験体用に用いた材料の強度等は表-5のとおりである。また加力測定の様子を図-10に示す。加力プログラムとしては文献5と同様のものを用いた。測定としては、変位、ひずみ、ひび割れ幅など詳細な検討を行ったものであるが、本報では結果の詳細は省略する。さて、以下に実験結果を述べるが、屋根床板と4階床板とで、その傾向にありずかしかったので、4階床板を中心に説明を行う。
図12(a)はBL-61 H試験体について、一定レベルの鉛直荷重のみを作用させている場合に比べ、水平荷重が加わってゆくと、ひび割れ模様がいったん変化してゆくかを示したものである。水平荷重が加わってゆくと、鉛直荷重は一定のままである。ひび割れの進展も著しく、本数がふえてゆくことがわかる。
図-13 は BL-41 H、BS-61 H 両試験体について、水平加力時の荷重-変形履歴曲線を示したものである。図には、2 章の解析法に基づいた計算線も示してある。基本的には、実験線と計算線の傾向は一致しており、本解析法は、鉛直荷重作用下におけるフレームの水平荷重-変形履歴曲線をある程度予測できることを示している。

図-14 は BS-61 H 試験体について、はり中央での鉛直たわみが、水平荷重下で変化してゆく様子を示したものである。また図-15 は BL-61 H 試験体について、はり全体のたわみ変化を示したものである。以上の図から明らかなように各繰り返し段階での最大水平荷重時とその段階での除荷時とで、鉛直たわみはあまり変らない。図-16 は 2 章の解析法に基づいた計算値と水平荷重載荷を行った試験体の実験値とを比較したものである。横軸には各繰り返し段階での層間変形角を、縦軸には鉛直たわみをはりスパン長の半分で除した値を示している。最大水平荷重時とその除荷時の両場合を比較しているが、いずれも、実験値と計算値の傾向は一致しており、本解析法は鉛直たわみ増大現象を予測するのに、有効なものであることがわかる。

最後に、水平荷重載荷経験をもつはりと、経験をもたないはりの鉛直耐力を検討した。図-17 は、鉛直荷重を終局時まであけていった場合のはり中央のたわみ変化の状況を示している。また、表-6 は試験体の鉛直耐力について、実験値と計算値を比較したものである。計算値は、はり両端、加力 2 点に降伏ヒンジを仮定して求めた崩壊時荷重である。図-17、表-6 から明らかのように、水平荷重載荷は、はりの鉛直耐力にあまり影響を与えないことがわかる。このたびの実験では、与えた最大水平変形角は 0.02 rad であったが、その程度までであれば、問題がないことを示している。また、いずれも、計算値ともほぼ一致している。表-7 は、フレーム各部材のせん断耐力計算値を示したものである。せん断力に対しては、かなりの余裕がある。