

EVALUATION METHOD OF SEISMIC CAPABILITY FOR MOMENT-RESISTING FRAMES OF REINFORCED CONCRETE STRUCTURE-REACTION MODELLING OF MEMBER DISPLACEMENT

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ABSTRACT. Story drift of structure is an important indication of structural damage subjected earthquake excitation. It is thus more reasonable to include base shear and story displacement volume in the assessment of seismic resistant capability of structure. The purpose of this study is to apply lateral displacement reaction model of structural member established through experiments as a base and put in use the derived relation between member deformation and story displacement of structure, integrated with the influence of dynamic $p-\Delta$ effect and the required consideration for the structure's functionality, to evaluate earthquake proof capability of moment-resisting frames of reinforced concrete structure. This proposed evaluation method is indicated by the collapse ground acceleration and the extreme volume of story drift. To test the correctness and simplicity of this proposed method, example analysis can be employed and compared the proposed method with elastic static earthquake proof method, nonlinear dynamic analysis method and capacity spectrum method. The analysis outcome shows that the analysis results of proposed method are the nearest to the results of dynamic analysis, and this method is also the simplest and most accurate than the others.

AMS Classification: 62N05

Keywords: yielding of member, ultimate deformation capacity, bar deformation of structure, story drift of structural system, collapse ground acceleration, ultimate story drift capacity

1. Introduction

Located on Circum-Pacific Seismic Zone, Taiwan has to experience several earthquakes every year. Especially, many of the strong earthquakes have brought many losses of life and property damage in recent years. Thus, the assessment of seismic resistant capacity of buildings is helpful for us to discover underlying safety problems and take effective measures to reinforce or dismantle the dangerous structures, in an attempt to minimize the damage caused by earthquakes. This is the priority in preventing earthquake-caused damages. However, the reaction of buildings to earthquakes is very complicated. Besides, the prediction of earthquake characteristics, the factors such as all nonlinear behaviors and mutual-restraining reactions, there are still numerous man-made variation involved. It is difficult to exhaustively assess seismic resistant capability of structure and estimate utility fixed number of years of building once again. But, it is a good approach if we can come up with an appropriate and simple evaluation procedure under the balance of construction cost and evaluation cost.

Arcade building is a unique and widely used construction in Taiwan. The seismic resistant capability of this kind structure is inestimable. During Chi-Chi earthquake of Taiwan, 1999, many buildings of this style were damaged due to lack of earthquake proof capability, causing considerable casualties [6]. In the meantime, there are three major ways to assess seismic resistant capability of this kind building: (1) Elastic static earthquake proof method [1] is frequently used to check the safety of structure under a specific earthquake; (2) Non-linear Increase Spectrum Analysis devised by Professor Sheu [8] [9] [10]; (3) Capacity Spectrum Method specified in ATC-40 and FEMA-273 [2]. The above three methods consider the earthquake response behaviors as elastic line. Despite these three methods unite practicable and simple features, they have the following flaws: (1) These three methods have to be processed inner stress analysis of building by means of structural analysis software suite; (2) The dynamic $p-\Delta$ effect that may cause collapsing is not included; (3) Except Capacity Spectrum Method, the relation between collapse ground acceleration and the structure's functionality is not considered, but the analysis result of Capacity Spectrum Method may not be correct [4]. Thus, it is not appropriate to only use strength of member in evaluating damage degree; (4) The random variation of structural parameters is not considered; (5) The assessment of the required field data is complicated. Additionally, the common used analysis method- structural nonlinear analysis method is mostly based on sectional point-to-point (or column tip) simulation method. The discretization of the section will lead to excessively simplify the hysteresis loop of structural member. Thus, the analysis result is not viable and unable to reflect actual dynamic reaction subject the excitation of earthquake. In order to improve the above weaknesses, this study aims to investigate the structural response characteristics of the moment-resisting frames of reinforced concrete structure, is presently common used buildings (under 6 stories) in Taiwan. Furthermore, a simple and reasonable evaluation method, based on the reaction modeling of member displacement, is proposed in this paper. This proposed method employs member-to-member simulation method to acquire an estimated value from sampling tested values by means of statistical and regression analysis. This value will be used on member displacement model, to simplify the process of modelization. In executing this analysis method, engineers just refer to the relation equations of member

displacement of plane frame and story drift of the structural system, dynamic $p-\Delta$ effect and the structural functional demand table to achieve estimated data and proposal of the weak points in need of reinforcement that can be carried out quickly and economically. This proposed method does not need to apply structural analysis software suites; it can be conducted through simple hand calculation.

2. Evaluation method of member displacement model

The objective of seismic resistant assessment is to compute the maximum seismic resistant capability provided with building before the functions of building are completely lost. This evaluation is to verify whether the earthquake proof of building is greater than the required volume of building or not. The assessment of structural seismic resistant capability and demand is based on the parameters such as structural strength, ductility, drift, and etc. These parameters are reflected on the relation of force and displacement reaction of structure subjected to external force. Thus, the main procedure of assessing seismic resistance of building is to assess the force and displacement volume curve, represents seismic resistant capability and structural response characteristics of buildings. In a reinforced concrete moment-resisting frame system, it relies on the rigid joint of beam-column connectors to resist overturning moment resulted from seismic force. The following computation method, proposed in this paper, is conducted on the structural characteristics of low-story and mid-story reinforced concrete building in Taiwan.

2.1. Member Displacements and Computation Method. The frames of this structural type like a cantilever beam and result in a shear deformation model when this kind building taken lateral force. The main cause of frames' lateral displacement quantities are: deflection deformation capacity of beam (Δ_b), axial force of column, bending deformation capacity, and shear deformation capacity ($\Delta_{cf} + \Delta_{cs}$) of column, deformation capacity of column-beam connectors (the links between beam and column) and base deformation capacity.

To simplify computation procedure of lateral displacement of frame, reduce the analytical complexity, and define the application of this method, the following hypotheses are thus proposed:

- (1) Disregarding the axial deformation effect on lateral displacement.
- (2) The frames are based on rigid base of foundation, and there is no base deformation.
- (3) In the analysis of frames, this method is hypothesized that column-beam connectors are rigid, and rigid-zone effect is excluded.
- (4) Suppose the inflection points of column and beam are situated at the center of them that are the occurrence of deformation. If the two sides of the node differ in their spans, the average of these two spans is taken as the beam length and the upper and lower spans of the column must be taken the same length, too. This can be taken the center of the top span as a reference point, and then develop gradually downward story. If the inertia moment of upper and bottom columns are not equal, then taken the average value.

Given the above conditions, the causes of the frames, lateral deformation are only deflection deformation capacity of beam (Δ_b) and shear deformation capacity of column (Δ_{cs}) as shown in Figure 1.

2.1.1. Established the relationship of nodal deformation and lateral force based on member deformation. In order to explain how to establish the relations of nodal displacement and lateral force of frame with member deformation model, single-span-single-story frames are adopted, as example shown in Figure 1; where the yielding of beams and columns and the ultimate displacement utilize member deformation model [11]. The yield of the beam and the ultimate displacement rotation capacity of element are expressed as follows:

$$(1) \quad \theta_y = \phi_y \frac{L_s}{3} + 0.0025 + \alpha_{sl} \frac{0.25 \varepsilon_y d_b f_y}{(d - d') \sqrt{f'_c}}$$

Where :

L_s : the length of shear span;

ϕ_y : yield curvature;

α_{sl} : pull out parameter of main reinforcement;

(if main reinforcement is pulled out by bond let $\alpha = 1$, otherwise let $\alpha = 0$)

$\varepsilon_y = f_y / E_s$;

$(d - d')$: the distance from the centroid of the compressive strength of concrete to the centroid of the tension steel;

d_b : the diameter of main reinforcement;

f_y, f'_c : The yield strength of main reinforcement and compressive strength of concrete respectively;

ϕ_y : yield curvature of flexural model; (The semi-empirical formula is used to evaluate the yield curvature, $\phi_y = 1.7(\varepsilon_y / h_b)$ for beam, $\phi_y = 2.12(\varepsilon_y / h_c)$ for column.)

The ultimate displacement of column rotation capacity of member is expressed as follows [11]:

$$(2) \quad \begin{aligned} \theta_u(\%) &= \alpha_{st} \alpha_{cyc} \left[1 + \frac{\alpha_{sl}}{2.3} \right] \left[1 - \frac{\alpha_{wall}}{3} \right] [0.2^v] \left[\frac{\max(0.01, \frac{\rho' f'_y}{f'_c})}{\max(0.01, \frac{\rho f_y}{f'_c})} f'_c \right]^{0.275} \\ &\times \left(\frac{L_s}{h} \right)^{0.45} 1.1^{(100 \alpha_{sx} \frac{f_{yh}}{f'_c})} (1.3^{(100 \rho_d)}) \end{aligned}$$

Where :

α_{st} : classificatory coefficient of steel;

α_{sys} : monotonic loading $\alpha_{sys} = 1$, repeated loading $\alpha_{sys} = 0.6$;

α_{sl} : pull out parameter, slip let $\alpha_{sl} = 1$, otherwise let $\alpha_{sl} = 0$;

α_{wall} : Shear wall is set to be 1, 0 is for beam and column;

$v = N/A_g f'_c$: ratio of axial load; (axial force)/(cross section of column $\times$ compressive strength of concrete)

$\rho' f'_y$: ratio of the area of compressive steel and yield strength of steel;

ρf_y : ratio of the area of tension steel and yield strength of steel;

α : the effect confined concrete parameter;

$$\alpha = \left(1 - \frac{s_h}{2b_c}\right) \left(1 - \frac{s_h}{2h_c}\right) \left(1 - \frac{\sum b_i^2}{6b_c h_c}\right)$$

Where :

b_c, h_c : width of confined core measured to outside of hoops;

s_h : spacing of hoop;

b_i : spacing of longitudinal main reinforcement.

$L_s/h (= M/Vh = L/2h)$: ratio of shear span; (L : the height of column or length of bar; h : depth of column; when $L/2h > 3.5$, flexure failure of column under high axial load; when $L/2h < 3.5$, shear failure of column under high axial load.)

$\rho_{sx} = A_{sx}/(b_w s_h)$: lateral reinforcement ratio, paralleled the loading direction;

Where :

A_{sx} : area of lateral reinforcement paralleled the loading direction;

b_w : waist width of gross section.

f_{yh} : yield strength of lateral reinforcement;

ρ_d : diagonal reinforcement ratio of diagonal direction.

Equation (1) and (2) can be calculated with geometric measures and features of material strength and force and displacement relation of beam and column are shown in bilinear mode, as Figure 2 shown. A waiting established relation of displacement and lateral force of frames and nodes, as shown in Figure 3, where point A is the yield volume point of each node of frame, Point B is the ultimate volume point of each node of the frame, and the rigidity of frame before yield can be estimated with the following equation [12]:

$$(3) \quad EI_{eff}/EI_{g,tr} = (0.4 + p/(2.4p_b)) \leq 0.9$$

Where :

$EI_{g,tr}$: transformed section of reinforced concrete without crack (equivalent pure concrete section);

p : axial pressure;

p_b : axial pressure of balanced steel ratio to induce destruction.

The predictive value of this equation is closer to tested values [12] than the other suggestion values from different design codes. Thus, this formula is adopted in this research to reasonably estimate the stiffness of member just yielding.

2.1.2. *Computation of yield capacity for each node of frame (F_Y, Δ_Y).*

Among the frame nodes, when yield point occurs on one of the beams or columns that cross at the node, the node starts yielding. Meanwhile, the shear force capacity of structural story is the shear volume of the yield node (F_Y), and its relative lateral deformation is the yield displacement volume of the node (Δ_Y). The yield volume of frame nodes (F_Y lateral yield capacity of beam-column of sub-structure, Δ_Y deformation yield capacity of beam-column of sub-structure) can be calculated by comparing the relative nodal shear volume (F_{BY} lateral yield capacity of beam-column of sub-structure is referred to beam reaching yield point, F_{CY} lateral yield capacity of beam-column of sub-structure is referred to column reaching yield point) as the yield of the beams and columns at the nodes occurs. F_Y is the smaller one of F_{BY} and F_{CY} . And, the relation of elastic lateral force (F) of frame nodes and drift (Δ) can be obtained through elastic analysis. Calculations of yielding drift volume (Δ_Y) can be expressed as follows:

$$\Delta = (\Delta_b + \Delta_c) = \frac{FH^2}{6E} \left(\frac{L}{2I_b} + \frac{H}{I_c} \right)$$

That is,

$$(4) \quad \Delta = F \left(\frac{1}{\frac{12EI_b}{H^2L}} + \frac{1}{\frac{6EI_c}{H^3}} \right) = F \left(\frac{1}{K_b} + \frac{1}{K_c} \right)$$

2.1.3. *Computation of ultimate capacity for each node of frame (F_U, Δ_U).*

It is defined in this paper that if any beam or column is damaged, the node is damaged; as a result, the ultimate volume of nodes (F_U lateral ultimate capacity of beam-column sub-structure, Δ_U deformation ultimate capacity of beam-column sub-structure) is controlled by the ultimate displacement volume of the beams or columns (Δ_{bu} Nodal lateral displacement component induced by destruction of

beam, Δ_{cu} Nodal lateral displacement component induced by destruction of column). The computing method of the ultimate displacement volume after yield is processed by the equivalent elastic stiffness method, as shown in figure 4 and the formula is expressed as follows:

$$(5) \quad K_{sec} = \frac{F_y(1 + \nu\mu - \nu)}{\Delta_u} = \frac{K\Delta_y(1 + \nu\mu - \nu)}{\Delta_u} = K \frac{(1 + \nu\mu - \nu)}{\mu}$$

Where:

ν : ratio of stiffness of member before and after yield;

μ : ductility capacity of member (Δ_u/Δ_y).

And let the transformed stiffness coefficient as $S = \frac{(1 + \nu\mu - \nu)}{\mu}$, therefore,

$$(6) \quad K_{sec} = KS$$

After the process of equivalent elasticity, the ultimate volume of nodes can be calculated with elasticity theory, which follows the same method to calculate the former yield volume of the node.

2.2. Established the relationship of story drift and story shear force. In the moment-resisting frames system, the stiffness values of lateral displacement of each story are provided by the lateral stiffness of all the nodes of the same story. The harmonious relations of nodal displacement in the stories are established according to the assumption of rigid floor; therefore, lateral stiffness of stories can be obtained by paralleling the nodal stiffness values of stories. With the nodal force balance, the lateral stiffness values of each node can be achieved by paralleling the softness values of the beams and columns that cross at nodes. Therefore, in order to acquire the stiffness values of lateral displacement among the frames, the stiffness values during the occurrence of yield and ultimate displacement at each node are need to be individually calculated in advance, and next, computed the stiffness values during the occurrence of yield and ultimate displacement of each story. For explaining the assessment methods of multi-story, a typical frame of two-story-two-span are taken as example, as shown in Figure 5, to describe this computation method of each analysis step.

2.2.1. Structural analysis modelling. According to the assumption of the inflection point of beams and columns that occurred at the center of the member, the frames are vertically separated into internal nodes and external nodes, and horizontally separated them into top-story, middle-story, and bottom-story, as shown in Figure 5. The computation mode of the middle-story is shown as Figure 6 and Figure 7; and the top-story and the bottom-story are processed with the same method.

2.2.2. Computation method for unit deformation capacity of member. Calculate the yield of each member and the ultimate displacement capacity by geometric measures and material. With the product of the distances between the

displacement capacity and the member inflection point, the volumes of yield drift and ultimate drift of each member can be figured out.

2.2.3. Computation method for deformation capacity of node. Compare the values of drift volume of beams and columns that cross at the nodes, and take the minimum value as the representation of the drift volume of the nodes; yield displacement volume and ultimate displacement volume of each node can be determined by this way.

2.2.4. Computation method for lateral yield capacity. The lateral drift and lateral force relational expressions of the internal and external nodes for each story within the elastic limit can be calculated with elastic analysis. The computing example of the internal nodes is shown in Figure 7; the external node can be calculated in the same way, and four different relational expressions of nodes at most for symmetric frames merely need to be calculated. The analysis results are shown as follows:

- (1) The elastic lateral displacement component of the internal and external nodes of top-story of sub-structure.

Lateral deformation component of internal node:

$$(7) \quad \Delta_i = \frac{V_i h^2}{24E} \left[\frac{L}{2I_b} + \frac{h}{I_c} \right]$$

Lateral deformation component of external node:

$$(8) \quad \Delta_e = \frac{V_e h^2}{24E} \left[\frac{L}{I_b} + \frac{h}{I_c} \right]$$

- (2) The elastic lateral displacement component of the internal and external nodes of middle-story of sub-structure.

Lateral deformation component of internal node:

$$(9) \quad \Delta_i = \frac{V_i h^2}{12E} \left[\frac{L}{I_b} + \frac{h}{I_c} \right]$$

Lateral deformation component of external node:

$$(10) \quad \Delta_e = \frac{V_e h^2}{6E} \left[\frac{L}{I_b} + \frac{h}{2I_c} \right]$$

- (3) The elastic lateral displacement summation of the columns of bottom-story of sub-structure:

$$(11) \quad \Delta = \frac{V_b \left(\frac{h}{2}\right)^3}{3(3I_c)E}$$

2.2.5. General formulation of nodal force and displacement. The general formula of the nodal force and displacement relations during elastic stage is shown as follows:

$$(12) \quad \Delta = V \left[\frac{1}{K_b} + \frac{1}{K_c} \right]$$

On the other aspect, the general formula of nodal force and displacement relations of the nodes during total transformative stage are shown as follows:

$$(13) \quad \Delta = V \left[\frac{1}{S_i K_b} + \frac{1}{S_j K_c} \right]$$

Where:

S_i and S_j are defined as 1 for elastic status or other value for after-yielding status respectively. There are different combinations for various force members, so far, the capacity curves of forces and displacements at nodes have been established finally.

2.2.6. Established the capacity curve of story shear and displacement. The shear and displacement capacity curves of story nodal area A, B, C, shown in Figure 6, can be established according to the above mentioned methods with the harmonious relations of each nodal displacement of the stories through stiffness paralleling procedure, shown in Figure 8. Top-story capacity curve and bottom-story capacity curve can be established by the same method.

2.2.7. Evaluation seismic resistant capability. The maximum story yield shear capacity of each story, V_{ui} should be transformed into the relative yield base shear, V_{bi} from the above section. The distributive law should follow the mass distribution of each story to allot the maximum yield shear capacity to base shear of each story.

$$(14) \quad V_{ui} = \frac{\sum_{i=1}^N m_i}{\sum_{i=1}^N m_i} V_b \Rightarrow V_{bi} = \frac{\sum_{i=1}^N m_i}{\sum_{i=1}^N m_i} V_{ui}$$

Where:

N : summation of number of floors;

m_i : mass of the i^{th} floors.

The minimum relative value of base shear of each story, V_{bi} is the elastic seismic resistant capability of the frames. On the premise of controlling inelastic drift response of buildings, the reduction coefficient of earthquake-induced force should be adjusted follows the earthquake proof design code of building and it should be calculated by story drift capacity and drift capacity of story yield during assessment. Therefore, the relative ground acceleration capacity can be calculated on the condition of the ratio of yield base shear to allowable ductility, which is called "collapse ground acceleration" of building.

3. Case study

To assert the evaluation method derived in this study, a typical used Reinforced Concrete building of the cantilever-walkway style located in Chi-Chi, Nantou, Taiwan is assessed the seismic resistant capacity with elastic static earthquake proof

method, nonlinear increment spectrum analysis method, capacity spectrum method and the assessment of drift in this study and verified with nonlinear dynamic analysis program GENDYN 2001 [13] and IDARC 2D [14].

3.1. Structural data for case study. The building, taken as an example, is a typical accommodation unit whose ground floor is applied as shops. The elevation of its structure faces the street, shown in Figure 9, the structural measures of beams and columns, and the distribution of the reinforced bars, as shown in Table. 1. The designed strength of reinforced bars is $f_y = 2,800 \text{ kgf/cm}^2$; but the real strength of reinforced bars is $f_y = 2,100 \text{ kgf/cm}^2$, the designed strength of concrete is $f'_c = 140 \text{ kgf/cm}^2$, and the foundation is gravel base. The dead load of the roof and each story is $2,400 \text{ kgf/m}^3$, and live load of the roof and each story is 120 kgf/m^2 and 200 kgf/m^2 respectively.

3.2. Evaluation results. The evaluations of seismic resistant capability from different assessments (shown as collapse ground acceleration): The collapse ground acceleration derived from elastic static earthquake proof method is 0.147 g ; capacity spectrum method ATC-40 is 0.249 g . Proposals in this study: (1) Without regarding the damage limit and $p-\Delta$ effect: 0.375 g (the story drift of the weakest story is 22 cm); (2) Regarding the damage limit (story drift ratio = 4%) but without regarding the $p-\Delta$ effect is 0.25 g (the story drift volume of the weakest story is 14 cm); (3) Regarding the damage limit and $p-\Delta$ effect is 0.18 g (the story drift volume of the weakest story is 13 cm).

3.2.1. The results from nonlinear dynamic analysis program :

- (1) A dynamic analysis with and without $p-\Delta$ effect are processed with the multifunctional program of dynamic analysis (Gendyn 2001) [13] for force and displacement mode of elasticity and plasticity and the shear truss model. The analysis results are shown that: (1) Without $p-\Delta$ effect, the ground acceleration is 0.385 g for each floor that reaches the ultimate drift volume; under the same conditions of analysis, the estimated value in this study is 0.375 g . (2) With $p-\Delta$ effect, the ground acceleration is 0.18 g for each floor that reaches allowable ultimate drift capacity; under the same conditions of analysis, the estimated value in this study is also 0.18 g .
- (2) With IDARC 2D, the dynamic analysis program [14], fiber model of member, multi-linear hysteresis loop rules of material and spread plasticity model are adopted to assess the collapse analysis of this example. The results are that collapse ground acceleration is 0.19 g , but the estimated value in this study is 0.18 g ; the maximum story drift ratio is 3.1% . The analysis results are shown in Figure 10.

3.3. Discussion. Compared the analysis results with the actual seismic resistant capacity of this example, 0.2 g [10], the results of the proposed method and the other three methods are within reasonable scope. But, the analysis results of proposed method are nearest to the analysis results of nonlinear dynamic analysis. These results indicate that the proposed method and steps are most reasonable and convenient than the other methods. The allowable value of relative ultimate drift capacity of story can be adopted to assess seismic resistant capability of building.

4. Conclusion

The assessment of earthquake proof capability of existed building is a complex problem; it must have plentiful actual statistical data and experience as backing. The analysis results of proposed method indicate that they approach to nonlinear dynamic analysis results and it is a simple and convenient method. This method, has been proven and compared with the common used evaluation methods, shows that this proposed method is provided with the correctness, simplification and economic benefits. Therefore, it can be applied to assess seismic resistant capability of existed buildings, and it is also suitable for the design verification of functional design procedures. Furthermore, it can also be extended to reliable assessment in the future to extensively apply to evaluate earthquake proof of existed and newly constructed buildings.

The properties and probability distribution database of the geometric measures of member and the characteristics of material strength are established through wide investigations that conform to local quality. Engineers merely need to observe the geometric measures of the structure to assess the earthquake-drift response capacity of the reinforced concrete moment-resisting frames regarding parameter variation of structures through numerical simulation process, which can not only omit a great deal of detailed investigation, but also rapidly offer reasonable values of earthquake proof capability.

Acknowledgment

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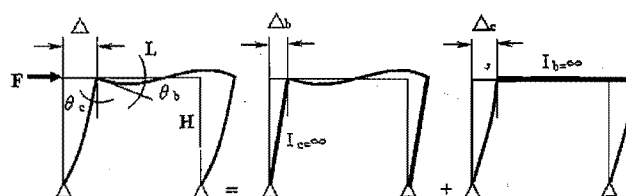
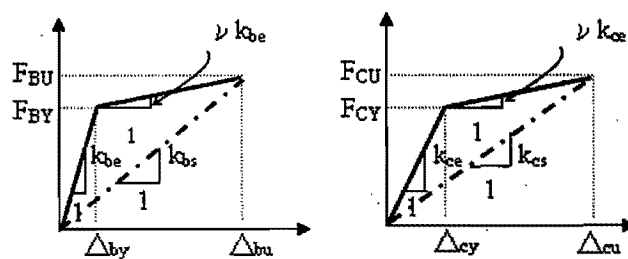


FIGURE 1. Deformation relation to nodes and elements [5]

FIGURE 2. Bilinear model for beam (Δ_b, V_b) and column (Δ_c, V_c) element

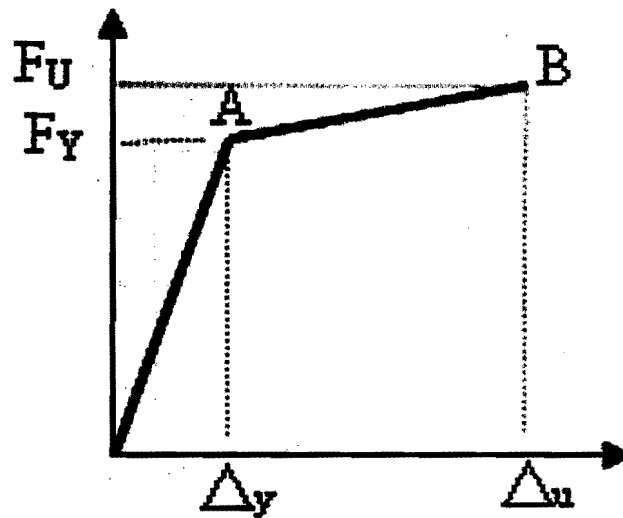
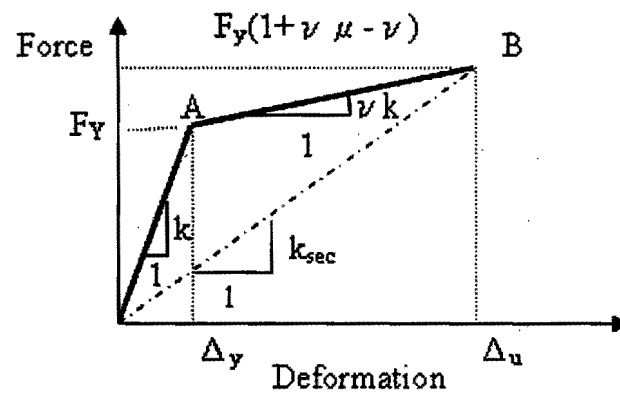


FIGURE 3. Bilinear model for nodes to be established from Figure:2

FIGURE 4. Definition of equivalent elastic stiffness k_{sec}

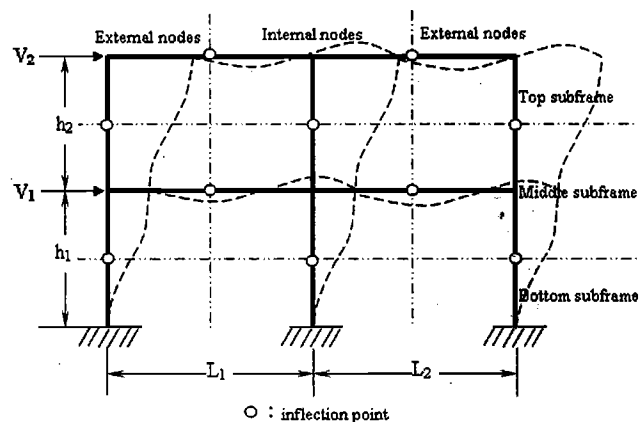


FIGURE 5. Typical frame for explaining assessment method

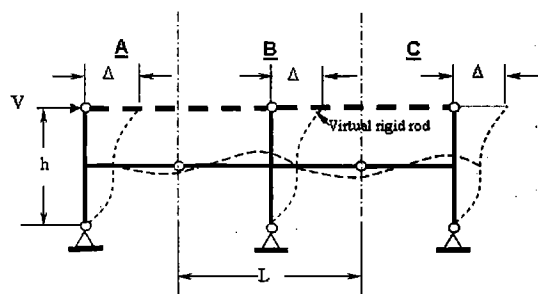


FIGURE 6. Compatible displacement mode of the relationship between nodes of the middle-story in Figure:5

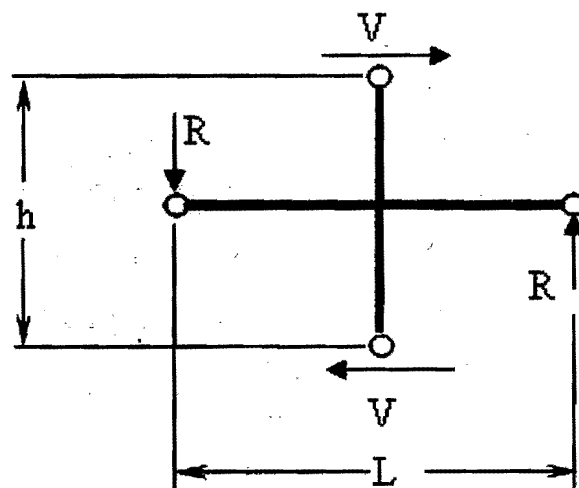


FIGURE 7. Explanation for the calculation of deformation of the internal node in Figure:6 [5]

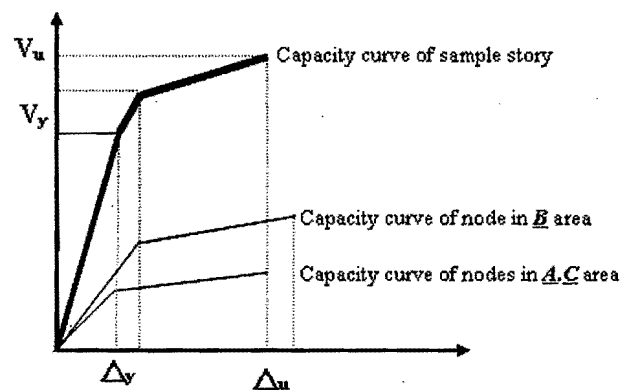


FIGURE 8. Story shear force versus displacement curves of Figure:6

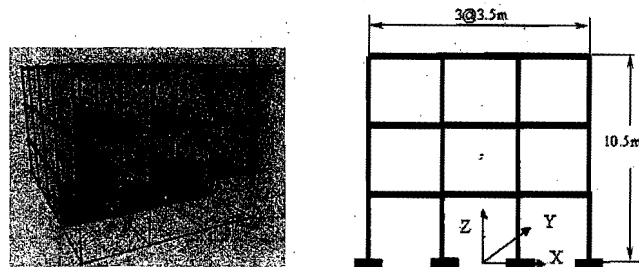


FIGURE 9. Building example for case study

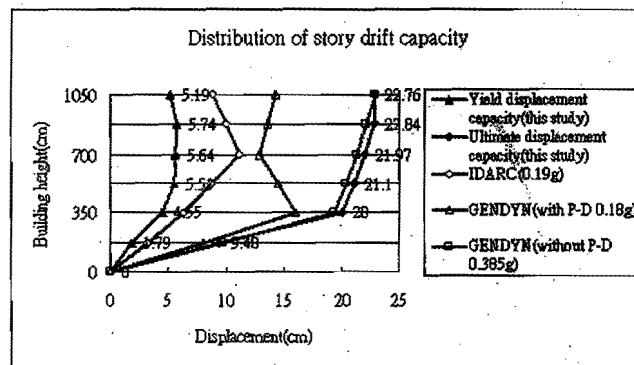


FIGURE 10. Distribution of story drifts capacity estimated by different method.

Table.1 : Reinforcement detailing for case study

Element type	size(cm)	Longitudinal steel	Transverse Steel (real)	Transverse Steel (designed)
Slab(t)	20	—	—	—
R.C.Wall	20	—	—	—
Beam(b×h)	30×40	top: 3-#6 2-#6 4-#6	#3 @ 30	#3 @ 15
		bot: 2-#6 3-#6 2-#6		
Exterior column(x×y)	25×40	6-#8	#3 @ 40	#3 @ 20
Interior column(x×y)	20×40	6-#8	#3 @ 40	#3 @ 20

FIGURE 11

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