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Working Session

Innovative Design and Structural Materials

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Seismic Responses of Variable Stiffness Semi-Active Control System

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Summary

This paper presents one type of new structural system with the variable stiffness semi-active control. The system for reducing earthquake response is composed of the control devices and the energy dissipation devices jointed to diagonal braces. According to the feedback structural information, the control devices open or close the energy dissipation devices. When the energy dissipation devices are closed, the braces work as the structural members have certain stiffness and absorb the seismic energy; When the energy dissipation devices are opened, the braces stiffness becomes zero and the absorbed seismic energy is dropped. Usually the semi-active control system uses the structural displacement as control information. The basic theory, the mathematical model, the seismic analysis of the structures, one example of the frame-shear wall structure and some conclusions are given in this paper.

1. Introduction

In the basic equation of motion of structures, the mass, damper and stiffness are three basic influence parameters for structural seismic responses. If any one is changed, the structural responses will be changed. Various structural control approaches including passive control, active control, semi-active control and hybrid control can modify these dynamic parameters in a desirable manner, all kinds of control devices have been successfully applied in many engineering structures to reduce earthquake responses.

Variable stiffness control is able to switch dynamically the stiffness of some components in a structure from a certain constant to zero, thereby modify the natural period of the structure to avoid the main seismic period, moreover, the controlled variable stiffness components can generate the hysteresis loops under earthquake, thus, a lot of seismic energy is dissipated and the seismic responses of controlled structure is reduced greatly. Semi-active control devices that typically have extremely low power requirements can achieve the variable stiffness control.

In this paper, a new type of variable stiffness semi-active control technique is presented. The system of seismic response reduction is composed of control devices and energy dissipation devices. According to the feedback information of structural displacement the control devices switch off or switch on the control valve of the energy dissipation devices jointed to diagonal braces and main structure. When the structure reaches the maximum displacement position and begins moving to the opposite direction, the energy dissipation devices are opened by the control devices and the stiffness of braces becomes zero. The absorbed seismic energy by the braces is dropped into the energy dissipation devices, otherwise the energy dissipation devices are closed, and the braces as structural components provide the certain stiffness and absorb the seismic energy. Because the structure goes through return point two times in one period, thus, the structure is able to modify vibration property and drops seismic energy two times in every period.



2. The basic theory and mathematical model

The schematic of energy dissipation system is shown in Fig.1. The variable stiffness semi-active control system of reducing seismic response is fitted on the structural interstory as shown in Fig.2.

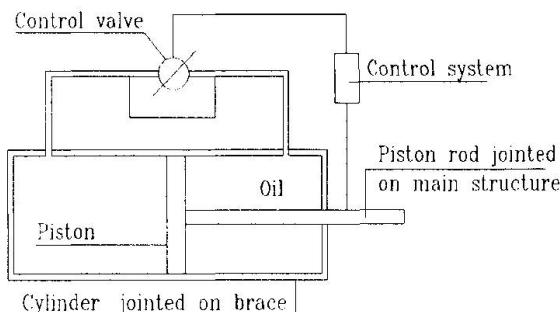


Fig.1. Schematic of energy dissipation system

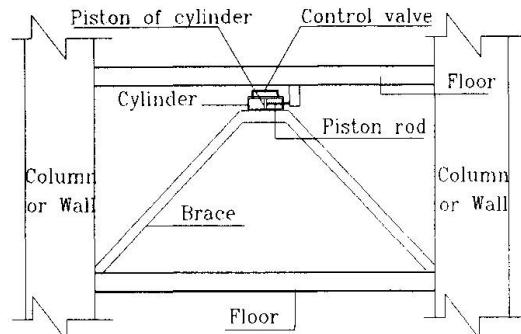


Fig.2. Scheme of structural system with variable stiffness semi-active control

The energy dissipation system is composed of the cylinder and the relative oil -way with the control valve that is controlled by the control system to close or open. The cylinder body is rigidly connected to the upper end of the braces which bottom is jointed on the floor, the piston rod of the cylinder is rigidly connected to the upper floor. When the control valve is closed, there is not the relative movement between the piston and the cylinder body. The braces go through same interstory displacement as the main structure, and provide certain lateral stiffness and restoring force absorb the seismic energy. When the control valve is opened, the relative movement between the piston and the cylinder body will be generated and the braces whose structural lateral stiffness becomes zero return to the original position. The seismic energy absorbed by the braces is dropped by the cylinder.

The control system is composed of a computer and relative information transmitters. Based on the information of the structural incremental displacement or velocity, the control devices can discern the structural point of return at which the sign of velocity or incremental displacement is changed, and open the control valve if the floor is at the point of return or close the control valve if the floor is not at the point of return.

The restoring force model of controlled braces is shown in Fig.3. The points A and C are the point of return, the lines OA and BC are the process of energy absorption of the braces in which the braces remain original lateral stiffness being equal to the rate of line, the lines AB and CD are the state of energy dissipation of the braces at which the lateral stiffness of the braces is changed to zero.

As stated above, the restoring force of the controlled braces is:

$$\begin{cases} \Delta x_i \cdot \Delta x_{i+1} < 0, & F = 0 \\ \Delta x_i \cdot \Delta x_{i+1} \geq 0, & F = K_d(x_i - x_0) \end{cases} \quad (1)$$

where, Δx_i , Δx_{i+1} are respectively the incremental displacement relative to the ground at the i th and $(i+1)$ th time moment, x_i, x_0 are respectively the relative displacement at the i th time moment and at the previous time moment at which the structure is at the point of return, k_d is the original lateral stiffness of braces, F is the restoring force of controlled braces.

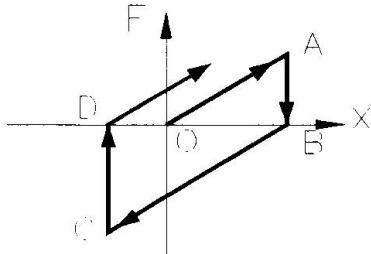


Fig.3.Restoring force model of controlled brace

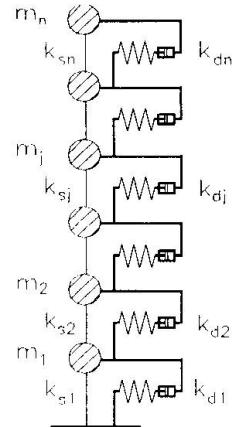


Fig.4.Calculation schematic of the controlled structure

3. Structural seismic analysis

The Calculation schematic used for seismic analysis of a multiple-story structure system with the variable stiffness semi-active control is shown in Fig.4. The springs with the energy dissipation device connected on interstory are the modelling of controllable reducing response system and can cooperate with the main structure under earthquake.

The basic equation of motion of the controlled structural system is:

$$\begin{cases} [M]\{X''\} + [C]\{X'\} + [K_s]\{X\} + \{F_d\} = -[M]\{I\}x_g'' \\ \{F_d\} = [K_d]\{X\} \end{cases} \quad (2)$$

In which, $[M]$ is the mass matrix, $[C]$ is the damper matrix and $[C] = \alpha[M] + \beta[K]$, $[K_s]$ is the lateral stiffness matrix of the main structure, $[K_d]$ and $\{F_d\}$ are respectively the lateral stiffness matrix and the generated force vector of the reducing response devices, $\{X\}$, $\{X'\}$, $\{X''\}$ is respectively the relative displacement vector, the relative velocity vector, the relative acceleration vector, x_g'' is the acceleration of the earthquake ground motion.

It is convenient for finding solution by using numerical analysis method to change Eq.(2) into the half incremental equation, as follows:

$$\begin{cases} [M]\{X''\}_i + [C]\{X'\}_i + [K_s]\{\Delta X\}_i + \{F_s\}_{i-1} + \{F_d\}_i = -[M]\{I\}x_{g,i}'' \\ \{F_d\}_i = \{F_d\}_{i-1} + [K_d]_i \{\Delta X\}_i \end{cases} \quad (3)$$

where i and $(i-1)$ are the number of the time moment (Δt). The structural seismic responses at i time moment can be found by Eq.(3). Taking these responses as the initial data, the solution at next moment is able to obtain.

The control condition shown in Eq.(4) that must be checked at every moment, relevantly, the



stiffness matrixes, the story shear vectors and the restoring force vectors must be modified by the control condition.

$$\begin{cases} \Delta x_{i,j} \cdot \Delta x_{i+1,j} < 0, V_{i,d,j} = 0 \\ \Delta x_{i,j} \cdot \Delta x_{i+1,j} \geq 0, V_{i,d,j} = V_{i-1,d,j} + k_{d,j} \cdot \Delta x_{i,j} \end{cases} \quad (4)$$

where $\Delta x_{i,j}$, $\Delta x_{i+1,j}$ are incremental j th floor displacement relative to the ground at the i th and $(i+1)$ th time moment respectively, $V_{i,d,j}$, $V_{i-1,d,j}$ are the shears of the reducing response devices at the i th and $(i-1)$ th time moment in j th story respectively, $k_{d,j}$ is the j th story lateral stiffness of the reducing response devices.

4. Numerical Example

Taking a 16 story R.C. frame-shear wall structure with the variable stiffness semi-active control system for the numerical example, the structural plane and elevation are shown in Fig.5., the main structural characteristic parameters are shown in table 1.

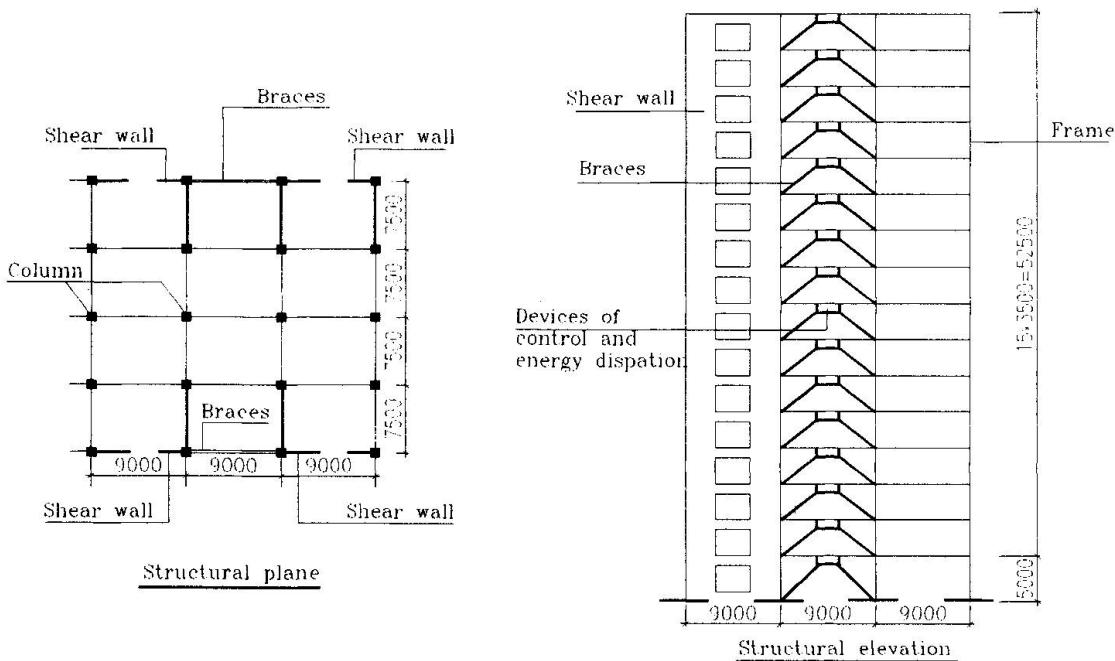


Fig.5. Structural plane and elevation

Table 1. main structural characteristic parameters

Floor No.	Column section(cm)	Wall thickness(cm)	Mass (t.sec. ² /m)	Elastic module of concrete(t/m ²)	Story height (m)
1	90×90	25	93.3	3250000	5
2-8	90×90	25	93.3	3250000	3.5
9-15	80×80	18	93.3	3250000	3.5
16	80×80	18	77.8	3250000	3.5

The horizontal seismic analysis is made in the transverse direction. The EL centro(in 1940,N-S) earthquake is used for the seismic ground motion whose peak acceleration is adjusted to 200gal and the time moment is 0.02sec. The lateral story stiffness of the controlled braces is r times the lateral story stiffness of the frames in main structure, r is called the rate of stiffness written as follows:

$$r = \frac{k_{d,j}}{k_{f,j}} \quad (5)$$

where, $k_{d,j}$ and $k_{f,j}$ are respectively the lateral stiffness of the braces and the frames, j is number of floors. The structure is considered to be linear-elastic in the analysis.

Based on the analysis results, the maximum interstory displacements and maximum story shears of the structure are respectively shown in Fig.6. and Fig.7. Four curves given in above Figs. indicate respectively the structural response results in no control ($r=1$) and control($r=0.5, 1.0, 1.5$) state.

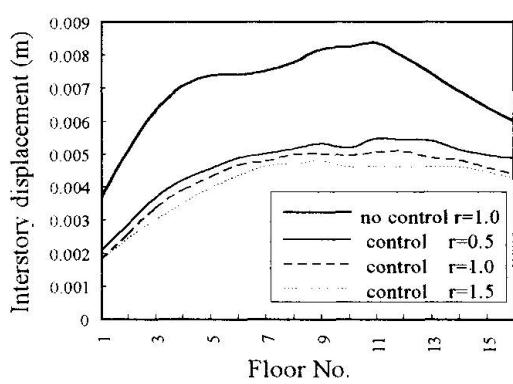


Fig.6. Maximum interstory displacement comparison

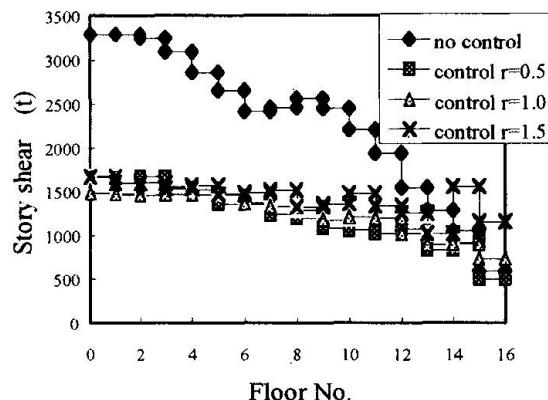


Fig.7. Maximum story shear comparison

From the Figs., the efficiency of reducing seismic responses by means of the variable stiffness semi-active control system is obvious and is different with the different rate of stiffness(r), as a general rule, the reduction of response increases with increment of rate of stiffness. The maximum interstory displacement is reduced by 51% of the uncontrolled response, the maximum story shear is reduced by 60%. The maximum interstory displacement angle is about 1/700 and indicates that the controlled structure remains the elastic state.

To study effect of reducing response of such cases that the controlled braces are fitted on some stories and other stories have no braces, the main structure remains unchanged, but the braces ($r=1.0$) are only fitted on 1-3 story in case 1 and on 1,9,16 story in case 2. The seismic analysis is made for two cases, the analysis results about the maximum interstory displacements and maximum story shears are respectively shown in Fig.8 and Fig.9 in which the results of every story control (1-16) are given for comparison.

From Fig.8 and Fig.9, the difference of three cases is not big, particaly the difference of story shears is small, so the notional design including notional select of brace stiffness and notional select of stories in which the braces are fitted can get the better economical effect.

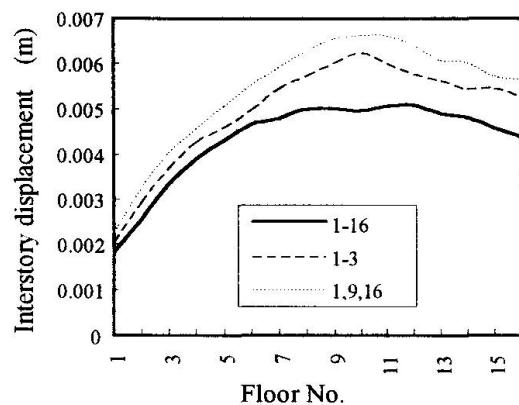


Fig.8. Maximum interstory displacement comparison

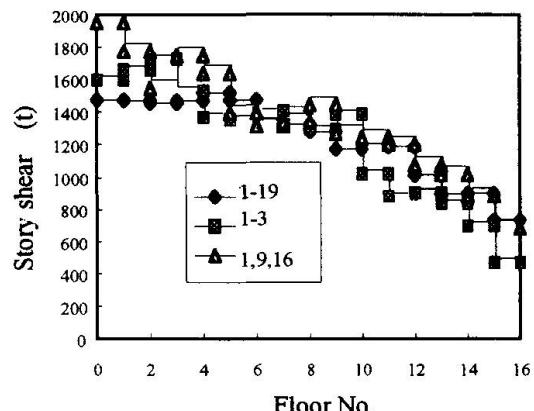


Fig.9. Maximum story shear comparison

5. Conclusions

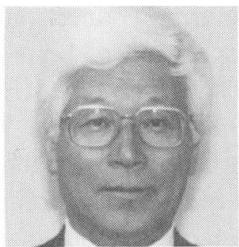
1. The variable stiffness semi-active control system is effective to reduce seismic response of controlled structures, the controlled structures can remain elastic state under strong earthquake.
2. The lateral stiffness of the controlled braces has obvious influence on the effect of reducing seismic response, it is necessary in design to select notional rate of stiffness.
3. The effect of reducing response of such case that the controlled braces are fitted on some stories and other stories have no braces is satisfactory.
4. The restoring force model of controlled braces and the analysis procedure of controlled structure can be used for the seismic analysis of this kind of structures.

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Investigation on the Statical Characteristics of Beam-to-Column Connections

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Summary

The statical characteristics of WBFW (web bolted and flange welded) type beam-to-column connections were experimentally investigated. The specimens with H-shaped columns or RHS columns were tested. The main test parameters are the disposition of bolts at the bolted web connection and the section of column members. From this test, the basic statical characteristics, such as ultimate bending strength eMu , plastic deformation ability, failure mode, are obtained. The results are summarized that α value ($=eMu/Mp$ Mp :full plastic moment of the connected beam) and deformability of specimens are influenced by the bending strength of bolted web connection and by wide-to-thickness ratio of column flange.

1. Introduction

In Japan beam-to-column connections of small or medium size steel building frames are fabricated at shop by welding. On the other hand WBFW type beam-to-column connections, which are executed at the building site, are commonly used at high rise building frames. Up to now, at the design of beam connections it is customly assumed that the flange connected parts carry the whole bending moment and the web connected part carries the whole shear force. But the depth of beams are getting large, web connected parts should be carry some bending moment in addition to shear force. But the stress transfer capability of the bolted web connections is not so large because such bolted connections are commonly single shear connections. In addition the section of columns are usually square hollow section, and in such case the bending stress transfer at the web connection is considerably restricted. From such view points a series of experimental study on the WBFW type beam-to-column connections were executed in order to investigate the effect of the bending strength of the bolted web connection on the overall performance of the beam-to-column connection.



2. Test Specimens and Test Setup

The objective of this test was mainly the investigation of the influence of the bending strength of the bolted web connections to the maximum bending strength and the deformation capacity of the whole beam-to-column connections. The main experimental parameters are cross sections of column and disposition of bolts at the web connected part. The specimens using H-shaped columns were cantilever type (H series). Horizontal cyclic loads were applied to the tip of the beam. The specimens using RHS columns were T-Shaped type (B series). The test setup of this test series is shown schematically in Fig.1. Horizontal cyclic loads were applied to the top of the RHS column. Those specimens were cyclically loaded under displacement $\pm 2c\delta p$, $\pm 4c\delta p$, $\pm 6c\delta p$ and $\pm 8c\delta p$, where $c\delta p$ is the calculated elastic displacement corresponding to the plastic moment at the end of the beam. A total of fifteen nearly 1/2 scale model specimens with different type of column, different cross-sections of beam, and different bolt disposition at the bolted web connection were prepared, as summarized in Table 1. The character p, g, e, n and m in Table.1 are corresponding to the length and so on shown in Fig.2. Bolts were F10T class high strength bolts and they were tightened by nut rotation method. The connection details of B series specimens are illustrated in Fig.3.

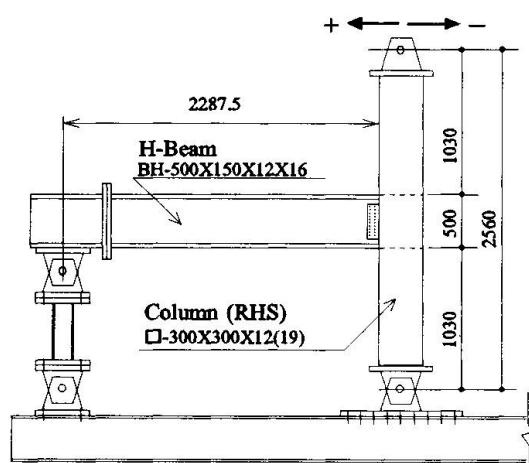


Fig.1 Test Setup (RHS Column)

bolts and they were tightened by nut rotation method. The connection details of B series specimens are illustrated in Fig.3.

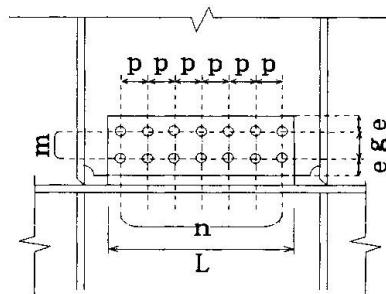


Fig.2 bolted web connection details

Table 1 Test Specimens

Specimen	Column	Beam	Gasket Plate	H.T.B	p g e n m					
					tg (mm)	L (mm)	(mm)	(mm)	(mm)	
H-1-1	BH-400x350x16x16	BH-500x150x12x16	19	375	M12	35	35	30	10	1
H-1-2	BH-400x350x16x16	BH-500x150x12x16	19	375	M12	35	35	30	10	2
H-1-3	BH-400x350x16x16	BH-500x150x12x16	19	375	M12	35	35	30	10	3
H-2-1	BH-400x350x16x16	BH-500x100x16x16	22	380	M16	50	50	40	7	1
H-2-2	BH-400x350x16x16	BH-500x100x16x16	22	380	M16	50	50	40	7	2
H-2-3	BH-400x350x16x16	BH-500x100x16x16	22	380	M16	50	50	40	7	3
H-3-1	BH-400x350x16x16	BH-500x100x12x12	19	360	M20	50	50	30	7	1
H-3-2	BH-400x350x16x16	BH-500x100x12x12	19	360	M20	50	50	30	7	2
B12-1-1	□-300x300x12	BH-500x150x12x16	19	375	M12	35	35	30	10	1
B12-1-2	□-300x300x12	BH-500x150x12x16	19	375	M12	35	35	30	10	2
B19-1-1	□-300x300x19	BH-500x150x12x16	19	375	M12	35	35	30	10	1
B19-1-2	□-300x300x19	BH-500x150x12x16	19	375	M12	35	35	30	10	2
B19-1-S1	□-300x300x19	BH-500x150x12x16	19	375	M12	35	35	30	10x1+4x1	
B19-1-S2	□-300x300x19	BH-500x150x12x16	19	375	M12	35	35	30	4	2
B19-1-0	□-300x300x19	BH-500x150x12x16	19	375	M12	35	35	30	—	—

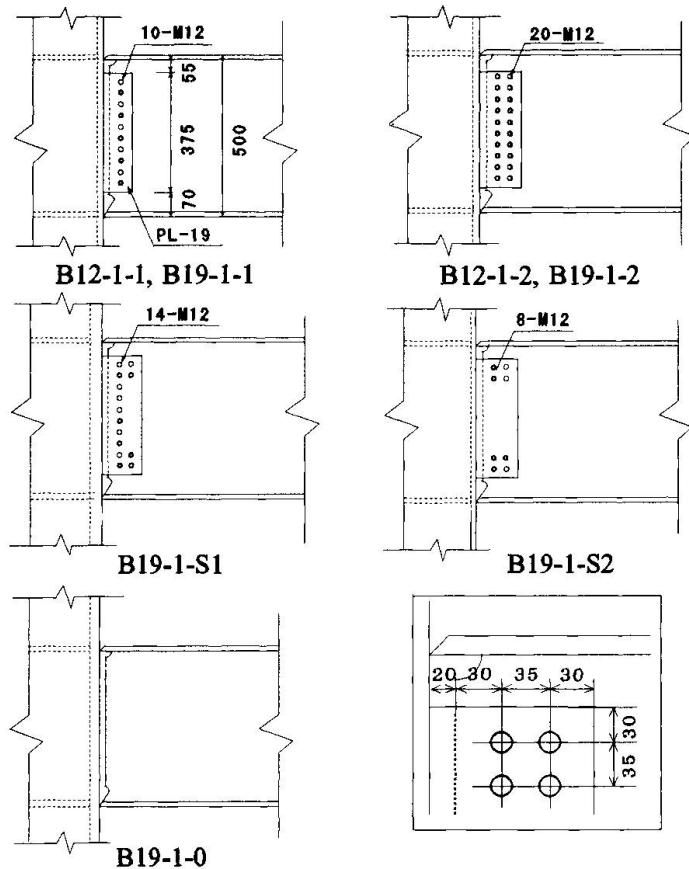


Fig.3 Connection Details (B series)

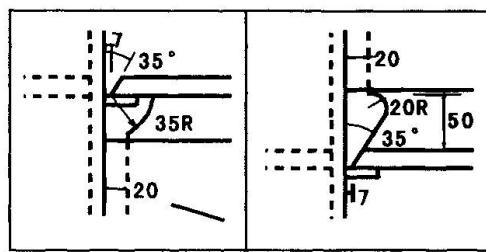


Fig.4 Weld Access Hole Details

Table 2 Mechanical Properties

Specimen	σ_y (t/cm ²)	σ_u (t/cm ²)	EL (%)
H series	PL-12	2.97	4.38
	PL-16	2.67	4.21
	PL-19	2.61	4.18
	PL-22	2.56	4.14
B series	PL-12	2.87	4.25
	PL-16	2.82	4.17
	CO-12	4.05	4.75
	CO-19	4.50	5.10

 σ_y : Yield Strength σ_u : Tensile Strength

EL : Elongation

Table 3 Test Results

Specimen	eMu (tcm)	M_p (tcm)	α	Failure mode	η	η_s
H-1-1	7121	5063	1.41	F. M. 1	27.4	15.2
H-1-2	7426	5063	1.47	F. M. 1	32.6	17.1
H-1-3	7661	5063	1.51	F. M. 1	49.3	28.8
H-2-1	5217	4384	1.19	F. M. 2	8.0	5.3
H-2-2	5664	4384	1.29	F. M. 2	11.9	7.6
H-2-3	6181	4384	1.41	F. M. 2	13.0	8.1
H-3-1	4230	3809	1.11	F. M. 2	12.9	6.7
H-3-2	4606	3809	1.21	F. M. 2	14.9	7.7
B12-1-1	6485	5162	1.26	F. M. 1	12.6	7.1
B12-1-2	6673	5162	1.31	F. M. 1	13.5	8.2
B19-1-1	6701	5162	1.30	F. M. 1	15.3	8.8
B19-1-2	6989	5162	1.35	F. M. 1	18.3	10.4
B19-1-0	6269	5162	1.21	F. M. 2	5.8	3.2
B19-1-S1	7110	5162	1.38	F. M. 3	20.1	10.4
B19-1-S2	6365	5162	1.23	F. M. 3	12.8	6.8

 eMu : maximum bending strength M_p : plastic bending moment (calculated value) α : maximum strength ratio of bending strength (= eMu/M_p) η : cumulative inelastic deformation ratio (based on $M - \theta$ curve) η_s : cumulative inelastic deformation ratio (based on skeleton $M - \theta$ curve)



As for specimen B19-1-0, only the flanges were connected without web connection. Complete penetration groove welds were used to connect the beam flanges to the column flanges in all specimens. The detail of the weld access hole are shown in Fig.4. The material of all beams and H-shaped column were SS400 steel and material of all RHS columns were STKR 400 steel. The mechanical properties of the material of the members used in the specimens are summarized in Table 2.

3. Experimental Results

Each specimen was subjected to cyclically increasing displacements. Cyclic loading was continued until failure occurred at the connection. When all specimens reached the inelastic range, small cracks occurred at the flange of the beam near the weld access hole. At the ultimate stage the fracture of the beam flange, which started from the center of the flange, occurred and developed to overall the flange. Ultimately the local buckling was also observed at the compression side beam flange. The overall fracture of the beam flange at the specimens using the beam with usual cross-section, H-1-1, H-1-2, H-1-3, B12-1-1, B12-1-2, B19-1-1, B19-1-2, B19-1-S1 and B19-1-S2, occurred after sufficient plastic deformation (F.M.1). At the specimen B19-1-S1 and B19-1-S2 web connecting bolts broke off simultaneously. At the H-2 type and H-3 type specimens brittle fracture occurred at the overall flange (F.M.2).

In Table 3 the maximum bending strength at the end of the beam eMu , which corresponds to the horizontal maximum load, the plastic bending moment M_p , maximum strength ratio $\alpha = eMu/M_p$, failure mode, cumulative inelastic deformation ratio η which were computed from the normalized beam end moment (M/M_p) versus the normalized beam end rotation ($\theta/c\theta_p$) curves, cumulative inelastic deformation ratio η_s from the skeleton curves of $M/M_p-\theta/c\theta_p$ relationships are summarized. The plastic rotation at the beam end $c\theta_p$ were calculated based on plastic displacement $c\delta_p$. The skeleton curves of $M/M_p-\theta/c\theta_p$ relationship are shown in Fig.6.

Fig.7 shows the maximum strength ratio α versus the number of web bolt lines m relationship of the specimens using beams with same cross section. As for the specimen without web connection, B19-1-0, the α value was plotted at the number of bolt line $m=0$.

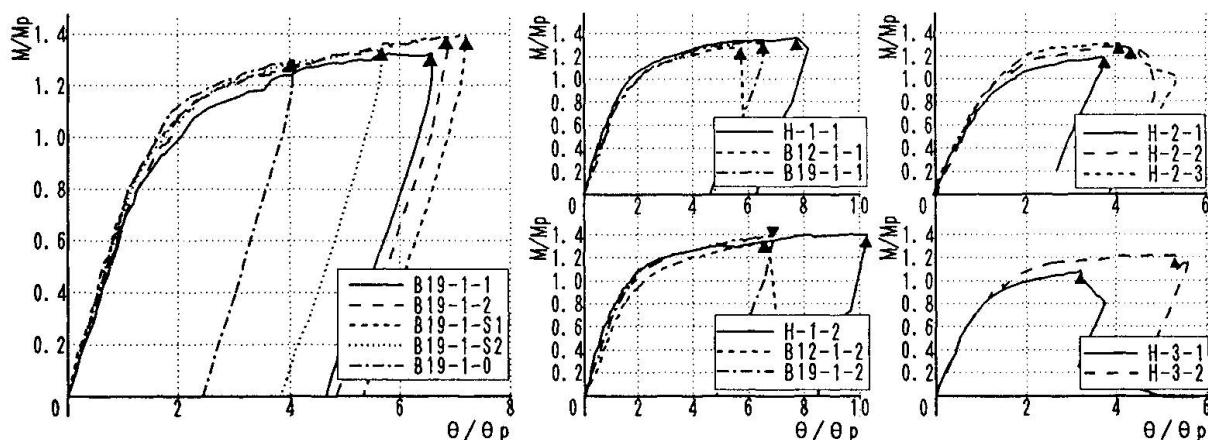


Fig.6 $M/M_p - \theta/\theta_p$

The maximum bending strength of the beam end connection increases with increasing of bolt line at the web connected part. The maximum strength ratio α versus the width-to-thickness ratio of the column flange D/t relationships are shown in Fig.8, where D and t are the width and the thickness of the column. As for the specimens using H-shaped columns the maximum strength ratio were plotted against $D/t=0$. It is observed that the maximum bending strength decreases with increasing of D/t .

The maximum strength ratio α versus the cumulative inelastic deformation ratio η_s relationship about H series specimens are shown in Fig.9. As above mentioned, the fracture of beam flanges of the H-2 and H-3 type specimens occurred in brittle manner, because the cross-section of the beam of those specimens were so special that the section module of the flange is extremely small in comparison with usual beam section. Therefore the value of η_s are very small, nevertheless number of bolt line at the web connected part increases. When the beams with usual cross-section such as that of the H-1 type specimen are used, η_s values are larger than 15 and it increases with increasing of bolt line. Fig.10 shows the $\alpha - \eta_s$ relationship of B series specimens. It is clear that η_s values of B series specimens were considerably small in comparison with those of H series specimens. The main reason of this fact is considered to be the difference of the cross section shape of the column. When the columns are RHS, web connected parts of the beam end connections do not work effectively to transfer the bending stress from web of the beam to column, because the out of plane stiffness of the web connected parts of the columns is very small. The tendency of this phenomenon appears more clearly, when D/t of the RHS column becomes large. From Fig.10 it becomes clear that the plastic deformation ability of the connected beams increases with increasing of the maximum bending strength of the beam end connections and that the maximum bending strength of the beam end connections increases with increasing of the maximum bending strength of the web connected parts. The bolts located near the flange work better than those located center of the web. But it is necessary to connect center part of the web at least one line to get better performance of the beam end connection.

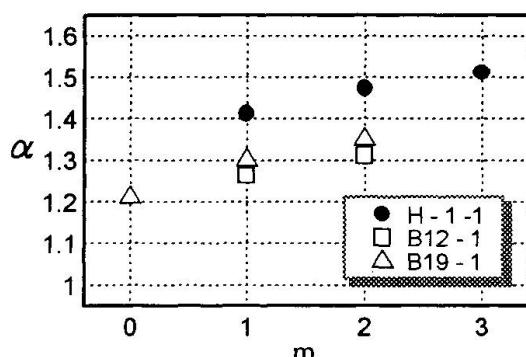


Fig.7 α - m Relationships

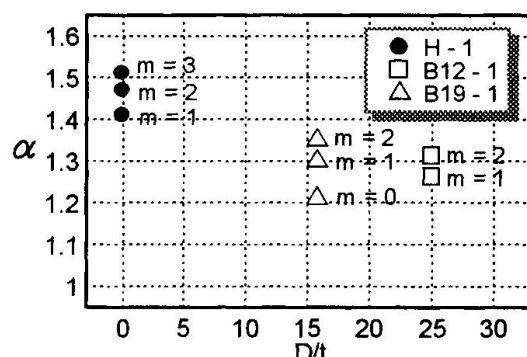


Fig.8 α - D/t Relationships

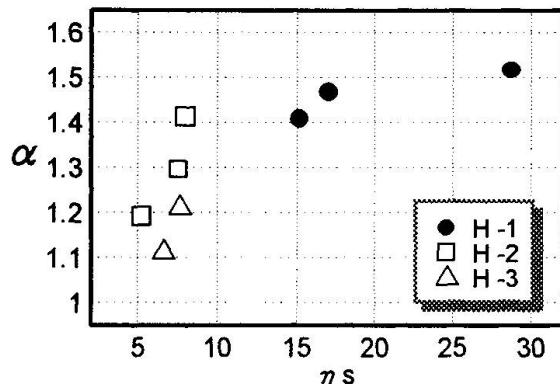


Fig.9 α - η_s Relationships
(H - series)

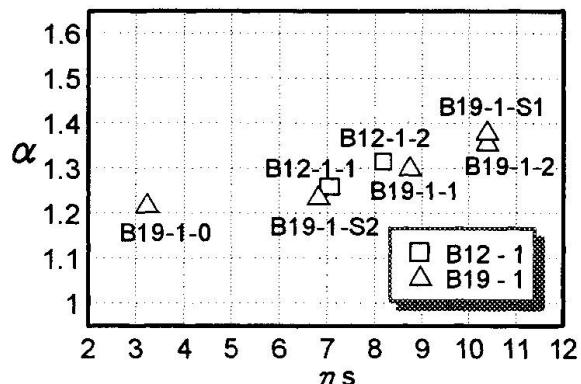


Fig.10 α - η_s Relationships
(B - series)

4. Conclusion

In order to investigate the statical characteristics of WBFW type beam-to-column connections, experimental study was carried out. Main items obtained from this study are summarized as follows.

- 1) The shape of the section of column member has large influence upon the statical characteristics of the beam end connections. In case column members are RHS, the maximum bending strength of the beam end connections are smaller than the case of using H-shaped column.
- 2) The plastic deformation ability increases with increasing of the maximum bending strength of the beam end connections.
- 3) The maximum bending strength of the beam end connections increases with increasing of the bending strength of the bolted web connection.
- 4) At the bolted web connection the bolts located near the flange work better than those located center part of the web. But it is necessary to connect center part of the web.

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Ductile Performance of RC Columns in High-Rise Buildings

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Summary

Confinement of concrete by transverse hoop or steel tube is effective in enhancing the ductility of reinforced concrete columns, especially columns under high axial load. Six specimens were tested under repeated lateral force to investigate seismic behavior of the confined columns. Test results of these specimens verified effectiveness of confinement by steel tube or high-strength hoop with rational configuration. The confined R/C columns exhibited very ductile performance even under axial compression force so high as the axial load ratio was 0.67.

1. Introduction

In Japan design standard for reinforced concrete structures, the upper limit of axial load ratio is recommended as 0.33 for column to ensure ductile seismic performance. However, columns at the lower stories of high-rise buildings usually have to sustain higher compression force when hit by a strong earthquake. It has been widely known that earthquake-resistant performance of the reinforced concrete columns under high axial load is very poor. Therefore, to promote the use of reinforced concrete structures in high-rise buildings located on seismic areas, a practical method is desirable to make the columns subjected to high axial load more ductile.

Confining reinforced concrete column by transverse reinforcements is one of the effective methods to enhance the earthquake-resistant capacity. This paper proposes two confining methods for square concrete columns under high axial load. The proposed methods are: 1) use of steel tube in lieu of conventional hoop or spiral, and 2) use of high-strength hoop, to confine columns. Using steel tube or high-strength hoop as confining material was for applying stronger lateral restraint to larger expansion of concrete in the column subjected to high axial force. Six specimens were tested under earthquake-simulated loading to investigate effectiveness of the proposed methods. This paper describes the experimental results of these tests.

2. Outlines of experiment

Six 250x250x1000 mm prismatic columns were fabricated. Longitudinal bars in each specimen consisted of twelve 13 mm diameter (D13) deformed bars uniformly distributed along the core perimeter. Among six specimens, four columns were confined by square steel tube, and the other

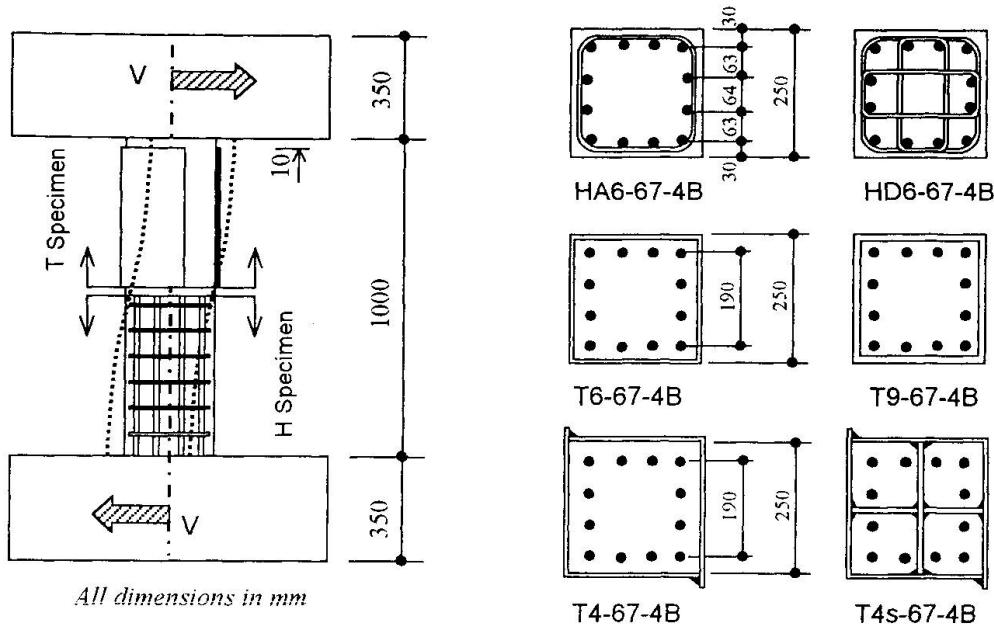


Fig. 1 Details of test specimens

two confined by high-strength hoop having yield strength of 1026 MPa. For specimens confined by steel tubes, spaces of 10 mm were provided between loading stubs and steel tube at both ends of the columns. This was for ensuring that the steel tube provides a confining effect only, rather than a direct resistance to the axial stress due to the applied axial load and bending moment. Fig. 1 and Table 1 show sectional details and properties of the specimens, respectively.

The variables among the four specimens confined by steel tube were wall thickness of tube as well as with or without inner stiffener. Variable for the two specimens confined by hoop was configuration type of the hoop. As shown in Fig. 1, hoop has two types of configuration. One consisted of one perimeter hoop only, and the other was of square perimeter hoop with two rectangular overlapping hoops. The level of applied axial load, expressed in terms of axial load ratio ($=N/A_g f_c$), was 0.67 for all specimens, which is twice the upper axial load ratio recommended in the AIJ standards [1] for reinforced concrete column. Specimens T6-67-4B and T9-67-4B had little different axial load ratios, because the core sectional dimensions of them were different from the other specimens (see Table 1).

Two batches of ready-mixed concrete, with a slump of 180 mm and maximum aggregate size of 20 mm, were used to construct specimens. The first batch was used for making specimens T6-67-4B and T9-67-4B, and the second for the others. The target compressive strength of concrete was 41 MPa, and the concrete cylinder strengths at testing stage are given in Table 1.

All specimens were tested under reversed cyclic lateral load while subjected to constant axial load. After applying axial load with a 5 MN universal testing machine, cyclic lateral load was applied through one 500 kN hydraulic jack to deform specimen in a double curvature pattern. Loading pattern for lateral load was displacement-controlled type with alternating drift reversals. The peak drifts were increased stepwise from 0.005 rad to 0.03 rad with increment of 0.005 rad after three cycles at each drift level. The maximum drift was 0.05 rad for specimen T4-67-4B and T4s-67-4B with one cycle at each drift level after 0.02 rad. For specimen HD6-67-4B, the first half cycle of loading in the positive (push) direction reached drift level of 0.01 rad due to miss of operation. From the subsequent cycle, scheduled loading program was applied. Applied lateral

Table 1 Properties of the test columns

Specimen	f_c' (MPa)	Axial load		Details of transverse reinforcement						
		N (kN)	$\frac{N}{A_g f_c'}$	ρ_h (%)	f_{yh} (MPa)	d_h (mm)	C (mm)	s (mm)	D_c (mm)	Type
T4-67-4B	51.1	2109	0.67	7.02	292	4.3	250	0	250	
T4s-67-4B	47.9	1971	0.67	10.5	292	4.3	125	0	250	Tube
T6-67-4B	40.5	1655	0.72	9.66	303	5.6	238	0	238	
T9-67-4B	40.5	1655	0.76	15.0	296	8.5	232	0	232	
HA6-67-4B	40.5	1655	0.67	2.21	1026	6.4	190	27	215	Hoop
HD6-67-4B	40.5	1655	0.67	2.21	1026	6.4	190	55	215	

Note: f_c' = strength of concrete cylinder,

A_g = gross sectional

area ρ_h = volumetric ratio of hoop or steel tube

f_{yh} = yield stress of transverse steel

d_h = nominal diameter or wall thickness of transverse steel

s = hoop spacing

C = unsupported length of transverse steel

D_c = dimension of confined core concrete

load was measured by calibrated load cell. Lateral deflection of column was recorded by two 100 mm displacement transducers.

3. Observed behavior and test results

Crack patterns of all specimens are shown in Fig. 2 along with hysteretic lateral load V – drift ratio R relationships. Crack patterns of the specimens confined by steel tubes were observed by cutting and removing steel tubes after the tests. The drift ratio R in Fig. 2 is defined as δ/H , where δ is the lateral deflection, and H is the clear height of column. The dashed line shown in the hysteresis loop of specimen HD6-67-4B represents the first half cycle of loading, which was out of the planned loading program.

Specimen T4-67-4B had the thinnest steel tube with width-to-wall thickness ratio $B/t = 60$ as its lateral confining material. The specimen showed rapid strength degradation during cycles of lateral loading at 0.015 rad drift level. During subsequent cycle of loading to the peak drift of 0.02 rad, axial shortening of the column due to high axial load became significant, and the steel tube touched the loading stubs and soon buckled. As the steel tube buckled, the specimen lost its axial and lateral load-carrying capacities. This result implies that steel tube with $B/t = 60$ cannot provide sufficient confinement to make the column ductile under high axial load.

Specimen T4s-67-4B was also confined by square steel tube having $B/t = 60$, but the steel tube was laterally strengthened by two crossed inner stiffeners provided along 250 mm end regions of the column (see Fig. 1). Having the same wall thickness as steel tube, inner stiffener greatly increased the lateral stiffness, hence confinement pressure, of the thin steel tube, and significant lateral expansion of the concrete was not observed. The specimen exhibited very ductile performance and sustained 90 percent of the peak load at large drift level up to 0.045 rad.

Specimen T6-67-4B had steel tube with $B/t = 46$ as its lateral confiner. The specimen showed relatively stable response until the drift ratio $R = 0.02$ rad. During the cycle of loading to the drift ratio of 0.025 rad, steel tube touched loading stub due to accumulation of the axial shortening. However, unlike specimen T4-67-4B, specimen T6-67-4B exhibited higher load-carrying capacity as shown by the dashed line in the V - R hysteresis loop after steel tube had touched the loading stub. Use of thicker steel tube apparently increased not only the confinement pressure, but the

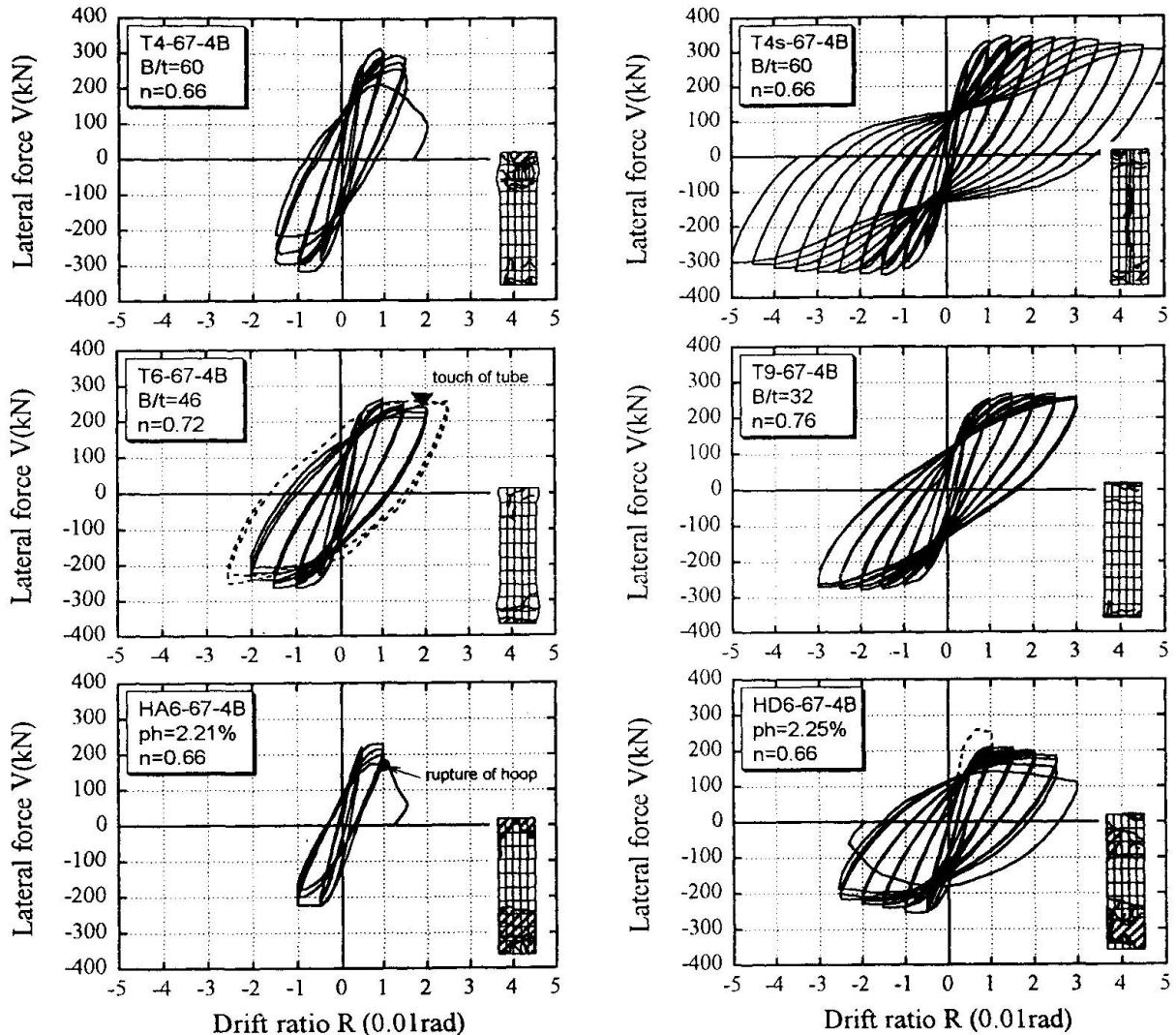


Fig. 2 Hysteretic response of specimens

axial buckling-resistant capacity of steel tube as well.

Specimen T9-67-4B was confined by the thickest steel tube with $B/t = 32$. The specimen showed very ductile behavior until end of test at drift ratio $R = 0.03$ rad. Damage was not observed, and the axial shortening was also very small.

Specimen HA6-67-4B was confined by high-strength conventional hoop. The amount of transverse hoop, expressed in terms of volumetric ratio of the hoop to the core concrete, was 2.21%, which is nearly equal to the maximum amount of hoop effective in resisting shear force [1]. This specimen showed rapid strength degradation when the cover concrete commenced spalling at the drift ratio of 0.01 rad. Cover concrete of the column completely spalled off at the end of loading cycles of 0.01 rad. During the subsequent cycle of loading to $R = 0.015$ rad, transverse hoop was broken, and the specimen lost its sustaining capacity to axial load and failed.

Specimen HD6-67-4B had the same quantity of transverse hoop as specimen HA6-67-4B. The difference between them was hoop configuration as shown in Fig. 1. The specimen HD6-67-4B

Table 2 Experimental and calculated results

Specimen	V_{max} (kN)	R_{max} (10^{-2} rad)	R_u (10^{-2} rad)	M_{exp} (kN-m)	M_{cal} (kN-m)	M_u (kN-m)	M_{ACI} (kN-m)	$\frac{M_{exp}}{M_u}$	$\frac{M_{exp}}{M_{ACI}}$
T4-67-4B	317	0.90	1.52	168	128	155	90	1.08	1.87
T4s-67-4B	338	1.38	4.50	183	147	176	87	1.04	2.10
T6-67-4B	264	1.00	2.01	140	114	142	69	0.99	2.03
T9-67-4B	274	1.50	5.40	149	129	163	62	0.91	2.40
HA6-67-4B	226	0.85	1.05	120	101	121	83	0.99	1.45
HD6-67-4B	254	1.28	2.45	138	101	139*	83	0.99	1.66

Note: V_{max} = maximum experimental lateral force R_{max} = drift ratio corresponding to V_{max}

R_u = drift ratio where the lateral load dropped to 90% of the V_{max}

M_{exp} = experimental ultimate moment

M_{cal} = theoretical ultimate moment [2,3]

M_u = enhanced ultimate moment by Eq. 1

M_{ACI} = the ACI moment [4]

*: $n=0.884$ for core section has been used to calculate the enhanced ultimate moment by Eq. 1

responded in a stable manner and sustained 90 percent of its peak load up to the peak drift of 0.025 rad. The cover concrete completely spalled off at drift ratio $R = 0.01$ rad, but spalling of cover concrete had little negative influence on the seismic performance of the column. This can be attributed to the use of supplementary hoops, which enhanced confinement force of hoop.

4. Ultimate moment capacity and deformation

Experimental moments of all specimens are given in Table 2. The experimental moments were measured as the maximum column end moment at the positive loading within drift ratio of 0.02 rad, including the $N-\delta$ moment. The ultimate drift ratios shown in Table 2 were measured as drift ratio where the lateral force dropped to 90% of the peak load. For specimens whose tests were terminated at drift ratio of 0.03 rad, the ultimate drift ratios were determined by extrapolation of the envelop curves. Theoretical moment M_{cal} and M_{ACI} were obtained by using two stress blocks proposed by authors [2,3] and in the ACI codes [4], respectively. confinement effect of steel tube has been taken into consideration in the stress block proposed by authors.

As is obvious in Table 2, the ACI ultimate moments are 45% to 140% conservative due to ignorance of confinement effect of steel tube. By taking confinement effect of steel tube into consideration, the moments predicted by authors method show good agreement with experimental moments. Experimental moments M_{exp} exceeded theoretical moments M_{cal} by 25% on average mainly because of existence of stiff loading stubs at the ends of column, which would apply extra confinement to the compressed concrete at critical end regions of column. To account for the effect of extra confinement from loading stub, authors have developed an empirical formula for the moment enhancement above the predicted moment M_{cal} , of the form [3]

$$\frac{M_u}{M_{cal}} = \begin{cases} 1.10, & n \leq 0.3 \\ 1.10 + 0.8(n - 0.3)^2, & n > 0.3 \end{cases} \quad (1)$$

where M_u is the ultimate design moment, and n is the axial load ratio. From Table 2, very good agreement can be observed between the experimental moments and ultimate design moments calculated by Eq. 1. The ratio of M_{exp} to M_u has mean value of 1.00 and standard deviation of 0.06. These results show importance of taking confinement effect of steel tube into account when calculating the ultimate capacity of confined concrete columns.



The experimental ultimate drift ratios are plotted in Fig. 3. The linked horizontal line superimposed in Fig. 3 presents the permissible ultimate drift ratio for well-confined concrete columns recommended in the AIJ guideline [5]. When confined by steel tube, columns show higher ultimate deformation as wall thickness of steel tube is thicker. In addition, use of inner stiffener is very effective in enhancing confinement effect of thin steel tube, hence ultimate moment and deformation capacity of the column. On the other hand, hoop configuration provided by one perimeter hoop only cannot make column ductile enough to satisfy the requirement of AIJ guideline. So, to fully utilize advantage of high-strength hoop, hoop should be used with rational configuration.

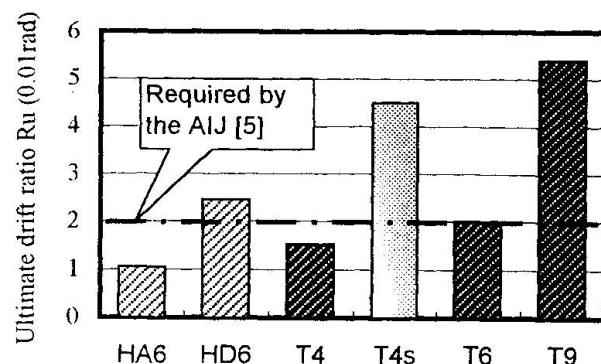


Fig. 3 Ultimate deformation

5. Conclusions

To make concrete column in high-rise buildings more ductile, this paper proposed two confining methods that involve use of square steel tube and high-strength hoop. The following conclusions can be drawn from the tests reported in this paper on the effectiveness of the proposed methods.

- (1) Confinement of concrete by square steel tubes was effective in enhancing the ductility of reinforced concrete columns under high axial load, but too thin steel tube could not provide sufficient confinement to the column because of its weak lateral stiffness. For concrete column under high axial load to have the ultimate drift ratio of 0.02 rad, one should use steel tube with $B/t < 46$ to confine the column. However, if strengthened with inner stiffener, the thin steel tube having $B/t = 60$ could make column to be able to sustain 90% of the peak load at $R = 0.045$ rad.
- (2) One single perimeter hoop cannot prevent column under high axial load from brittle crushing failure, even a large amount of high-strength hoops were used. However, use of supplementary hoop could increase confinement effect of hoop, and the column confined by hoop with rational configuration showed very ductile performance at large deformation up to $R = 0.025$ rad.

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Seismic Behaviour of Steel Encased Reinforced Concrete Columns

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Summary

The earthquake type loading tests of steel encased reinforced concrete (SRC) columns were carried out. The main objectives of this program were to investigate the seismic behavior of SRC columns using high strength longitudinal bars and lateral reinforcing bars and to obtain guideline for its structural design for high-rise buildings.

1. Introduction

The 1995 Hyogoken-Nanbu Earthquake attacked Kobe on January 17, 1995 and brought us the huge destruction. In general, we have thought that steel encased reinforced concrete (SRC) members have more ductility than reinforced concrete ones have. But in this earthquake, a lot of wide flange encased reinforced concrete columns, especially in multiple dwelling houses, received shear failures. To avoid the shear failure, usage of more slender columns is effective. But, in that case, flexural and shear strength will be short due to the small section. Then, high strength longitudinal and lateral reinforcing bars are considered to be used for slender columns to give more flexural and shear strength. We proposed to use the SD490 (yield strength: 490 MPa, ultimate strength: 620 MPa) as the longitudinal bar, and the KSS785 (yield strength: 785 MPa, ultimate strength: 930 MPa) as the lateral reinforcement for SRC columns. However, the superposed strength method in the SRC Standards published by AIJ is not applicable to such high-strength materials. This paper presents empirical results of tests on prototype columns. The main objectives of this test were to determine the seismic behavior of the column using these reinforcing bars and to obtain guidelines for its structural design for high-rise buildings.



2. Beam-column test

2.1 Test Specimen

Seven column specimens were tested under earthquake-type loadings. The section of concrete(35 × 35 cm, specified compressive strength: 35.3 MPa) and that of cross shaped steel(2H-210×80 × 6×16, tensile strength: 490 MPa) are common to all specimens. The variables are:

- 1) shear span to depth ratio($M/QD=2.0$ and 1.29);
- 2) area ratio of longitudinal reinforcement ($Pt=0.47, 0.65$ and 0.74%);
- 3) lateral reinforcement ratio($Pw=0.23\sim0.74\%$); and
- 4) axial load levels (n) with $0.1, 0.3$ and 0.5 , where n is calculated by equation (1) and (2).

$$n = N/N_0 \quad (1)$$

$$N_0 = B \cdot D \cdot c \gamma_u \cdot F_c + A_s \cdot s \sigma_y + A_r \cdot r \sigma_y \quad (2)$$

where: $c \gamma_u = 0.85\sim2.5 \cdot sPc$

$$sPc = sac/BD$$

sac : Total area of compressive steel (cm^2)

N_0 : Ultimate compressive strength (N)

B : Column width (cm)

D : Column depth (cm)

F_c : Specified compressive strength (MPa)

A_s, A_r : Total areas of steel, longitudinal reinforcement (cm^2)

$s\sigma_y, r\sigma_y$: Yield strengths of steel, longitudinal reinforcement (MPa)

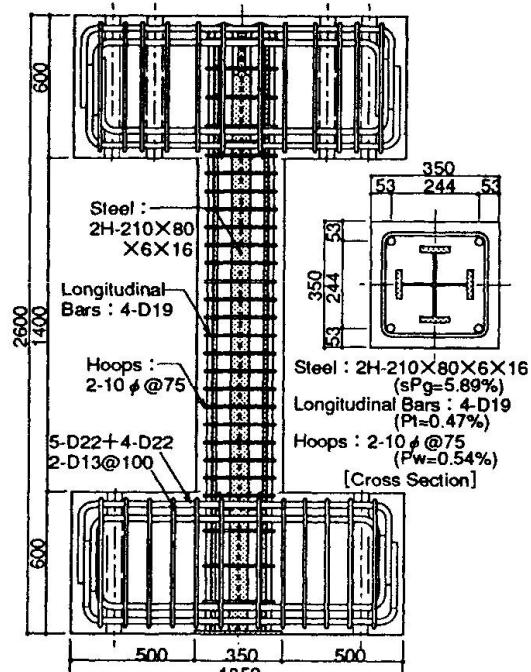


Fig.1 Test Specimen

Table 1 Details of Test Specimens

Specimen		h (mm)	Longitudinal Reinforcement		Lateral Reinforcement		Axial Load Ratio n	Q _{mu} (kN)	Q _{su} (kN)	Q _{su} / Q _{mu}
			Arrangement	Pt (%)	Arrangement	Pw (%)				
M series	1	1400 (M/QD =2.00)	4-D19	0.47	2-10φ@75	0.54	0.3	530	666	1.26
	2						0.5	514	666	1.30
	3		4-D16, 8-D13	0.74	2-10φ@55	0.74	0.3	570	718	1.26
	4						0.1	572	718	1.26
S series	5	900 (M/QD =1.29)	8-D16	0.65	2-6φ@80	0.23	0.3	888	561	0.63
	6				2-6φ@80		0.5	857	561	0.65
	7				2-6φ@40	0.45	0.3	888	712	0.80

h : Inside measurement of column

Pt : area ratio of longitudinal reinforcement

Pw : Lateral reinforcement ratio

Axial load ratio : Ratio of axial load to ultimate compressive strength

Q_{su} : Ultimate shear strength

Q_{mu} = (M_{u1}+M_{u2})/h

M_{u1} : Ultimate flexural strength of upper end

M_{u2} : Ultimate flexural strength of lower end

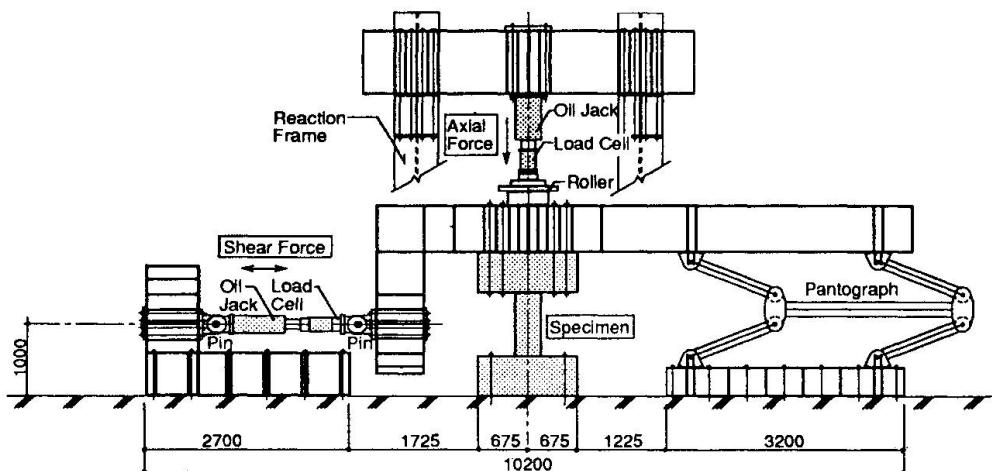


Fig.2 Loading apparatus

Table 2 Mechanical Properties of Concretes

Specimen	fc *1	fc' *1	E _s /fc' (GPa)	ft *1
1~4	35.3	39.0	32.3	3.0
5~7	35.3	38.7	28.5	2.8

fc' : Measured compressive strength of concrete

E_s/fc' : Secant Modulus of concrete at fc'/3

ft : Measured splitting tensile strength

Table 3 Mechanical Properties of Steel

	Thickness (mm)	f _y *1	f _u *1	ε _u (%)
Flange	16	326.0	527.2	27.64
Web	6	372.4	526.4	26.38

f_y : Yield strength of steelf_u : Ultimate strength of steelε_u : Ultimate Strain

Table 4 Mechanical Properties of Steel Bars

Type of Steel Bars	f _y (MPa)	f _u (MPa)	ε _u (%)
Longitudinal Reinf.	Deformed Bar 13mm dia	569.3	699.0
	Deformed Bar 16mm dia	531.9	710.0
	Deformed Bar 19mm dia	566.4	771.8
Lateral Reinf.	High Strength Bar ϕ 6mm	870.4	904.2
	High Strength Bar ϕ 10mm	896.6	1096.9

f_y : Yield strength of steel barsf_u : Ultimate strength of steel barsε_u : Ultimate Strain

The variables and ultimate shear strength of each specimen are as listed in **Table 1**. The full capacities of the specimen 1~4 in M series are determined so as to make them exhibit flexural-type failure mode and those of the specimen 5~7 in S series are determined so as to make them exhibit shear-type failure mode. The mechanical properties of concrete, steel and reinforcing bars are shown in **Table 2,3** and **4**. Cyclic horizontal load is applied to each specimen while the axial load keeping constant. The inflection point is kept at middle height of the column by using testing apparatus, as shown in **Fig.2**.



2.2 Test Results and Discussions

Table 5 gives the test results. **Fig. 3** and **4** show a comparison of measured horizontal load-story drift angle relations and the envelopes of horizontal load-story drift angle relations. The following results can be derived from these table and figures:

M series (specimen 1~4): Specimen 3 shows excellent ductility and lateral load carrying capacity up to the story drift angle exceeding 50×10^{-3} rad. On the other hand, specimen 1,2, and 4 tested under the higher axial compressive load of 30,50% of the ultimate compressive strength behaves in less ductile manner when compared with specimens 3. The failure mode for specimen 3 is flexural failure, and those for all other specimens are flexural compressive failure. The ductility of every specimen is reduced after reaching maximum strength, especially for specimen 2. It seems that this is due to the difference of the axial load levels between specimens. The story drift angles (R_M) at which the maximum lateral loads are sustained vary from $7.0 \sim 12.4 \times 10^{-3}$ rad. and the ultimate story drift angles (R_U) at which 80% of the maximum lateral loads are sustained vary from $12 \sim 50 \times 10^{-3}$ rad. The ratios of the measured ultimate strength (V) to the calculated strength by using superpose method¹⁾ (V_{cal}) vary from 0.91 ~ 0.94. The strain of the concrete at ultimate compressive strength is 1750×10^{-6} and the yield strains of the longitudinal bars are $2647 \sim 2922 \times 10^{-6}$, respectively, and the compressive concrete had crushed before the longitudinal bars yielded. The longitudinal bars maintain the elastic state when the maximum lateral loads are sustained and this might be the reason why the ratios (V/V_{cal}) of the measured ultimate strength to the calculated one is smaller than 1.0. In case that the ultimate strength is calculated by using

Table 5 Test Result

Specimen		N (kN)	N ₀ (kN)	n	Maximum Strength			R_M ($\times 10^{-3}$ rad.)	R_U ($\times 10^{-3}$ rad.)		
					Measured	Calculated	V / V_{cal}				
					V (kN)	V_{cal} (kN)					
M Series	1	2112	7040	0.30	496	530	0.94	10.0	30		
	2	3521	7040	0.50	482	514	0.94	7.0	12		
	3	739	7392	0.10	534	570	0.94	12.4	50		
	4	2217	7392	0.30	521	572	0.91	9.9	40		
S Series	5	2162	7207	0.30	599	561	1.07	5.1	15		
	6	3604	7207	0.50	617	561	1.10	4.2	12		
	7	2162	7207	0.30	651	712	0.91	5.1	30		

N : Axial load

N₀ : Ultimate compressive strength

V : Empirical value of shear force

V_{cal} : Smaller one of the two values of shear forces Q_{mu} and Q_{su} , which are calculated by using superposed method¹⁾

R_M : The story drift angle at which the maximum lateral load was sustained

R_U : The ultimate story drift angle at which 80% of the maximum lateral load was sustained

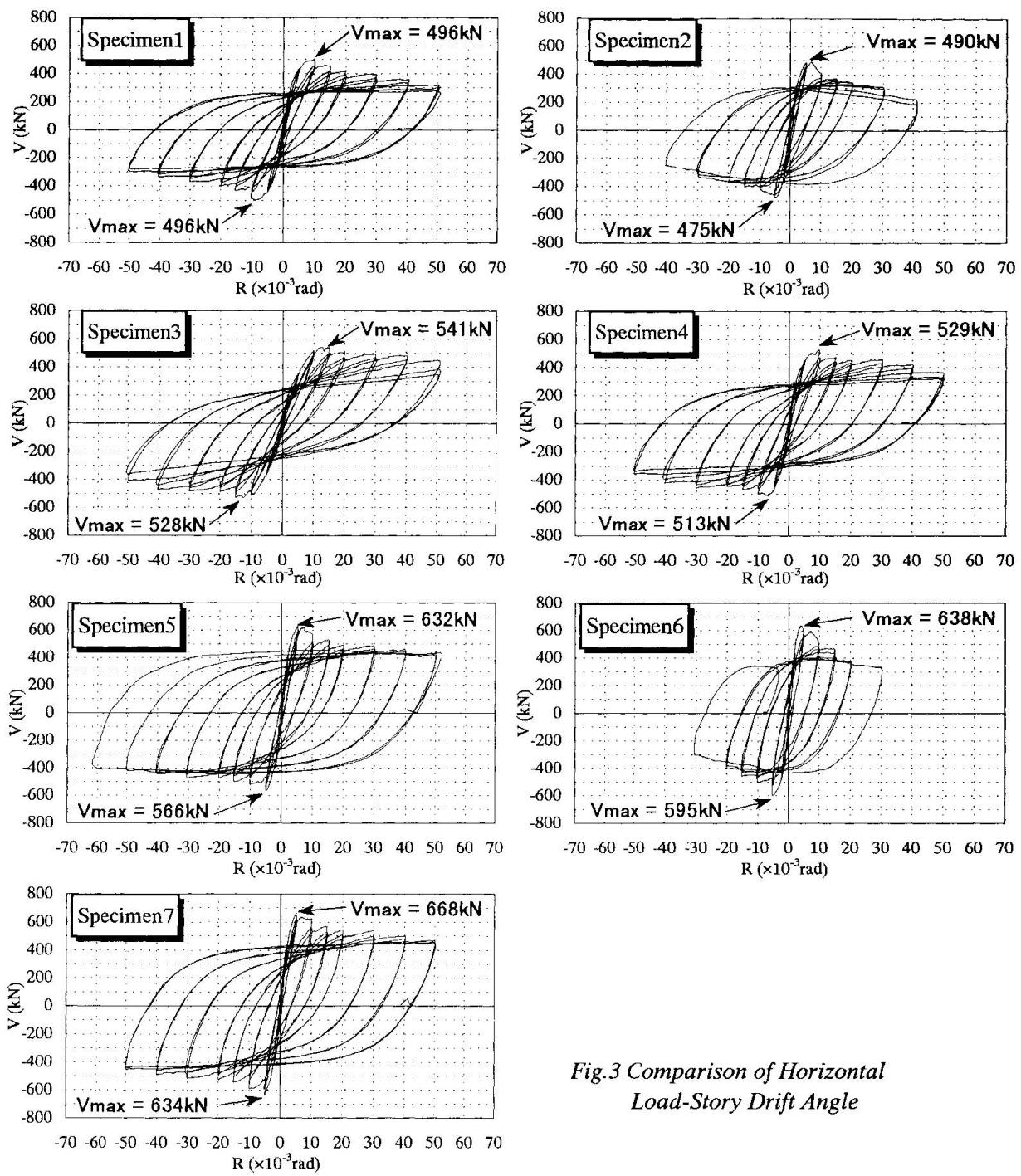


Fig.3 Comparison of Horizontal Load-Story Drift Angle

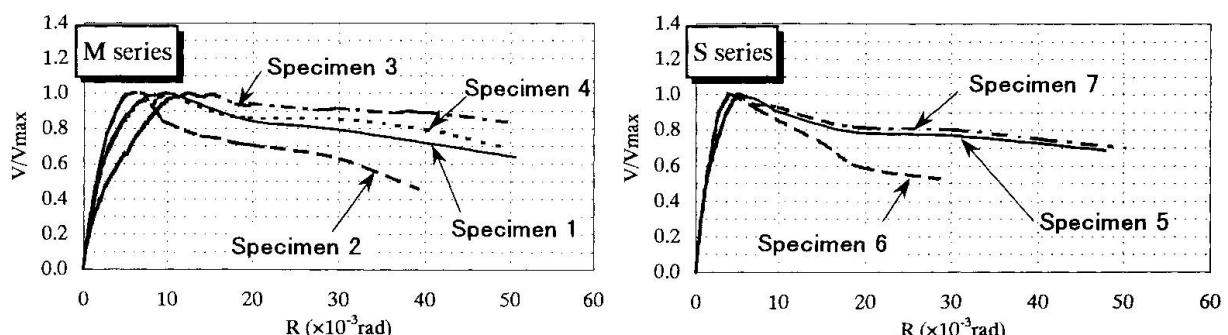


Fig.4 Envelopes of Horizontal Load-Story Drift Angle Curves



the stress of the longitudinal bars when their strains are 1750×10^{-6} , the ratios (V/Vcal) of the measured ultimate strength to the calculated one vary from 1.01 ~ 1.04.

S series (specimen 5~7): The failure mode for each specimen is shear failure. Specimen 5 and 7 show enough ductility and lateral load carrying capacity up to the story drift angle exceeding 5×10^{-2} rad. However, specimen 6 tested under the higher axial compressive load of 50% of the ultimate compressive strength behaves in less ductile manner when compared with specimens 5. It seems that this is due to the difference of the axial load levels between specimens as same as **M series**. The story drift angles (R_M) at which the maximum lateral loads are sustained vary from $4.2 \sim 5.1 \times 10^{-3}$ rad. and the ultimate story drift angles (R_U) at which 80% of the maximum lateral loads are sustained vary from $12 \sim 30 \times 10^{-3}$ rad.. The ratios (V/Vcal) of the measured ultimate strength to the calculated strength of specimen 5 and 6 ($P_w=0.23\%$) are 1.07 and 1.10, respectively, and that of specimen 7 ($P_w=0.45\%$) is 0.91. Therefore, the ratio V/Vcal tends to decrease as lateral reinforcement ratio (P_w) increases. This might be due to the fact that the lateral reinforcing bars maintain the elastic state when the maximum lateral load is sustained.

3. Conclusions

The conclusions obtained from the experimental study are summarized as follows.

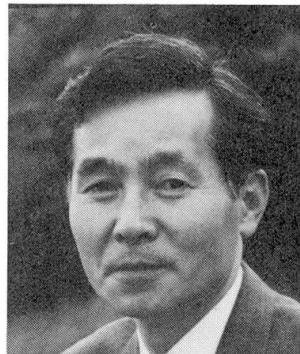
- (1) The SRC columns using high strength longitudinal bars and lateral reinforcing bars have the same sufficient ductility as those of SRC columns using normal strength ones.
- (2) Axial load level influences the flexural ductility of the SRC column.
- (3) In this test, the measured ultimate strength of columns using high strength longitudinal bars and lateral reinforcing bars are a little smaller than the calculated strength according to the SRC Standards¹⁾. To evaluate the ultimate strength of these columns, it is necessary to consider the relation between the strain of the concrete at ultimate compressive strength and the yield strain of the longitudinal bar in case of flexural failure mode, and also the strength balance between concrete and lateral reinforcing bar in case of shear failure mode.

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Ultimate Strength and Collapse Mechanism of Composite Frames under Seismic Loading

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Summary

In seismic design of composite building frames in Japan, the weak beam strong column moment frame system is usually adopted. In the case of composite low rise building frames, the plastic hinges formed at beams and columns show not only the rotational deformation but also the axial elongation. Considering these axial and rotational deformations, the real collapse mechanism is somewhat different from the expected one. In this paper, the ultimate strength and the collapse mechanism of the composite frames under the vertical and horizontal loads are discussed. Using geometric compatibility condition, a recurrence formula is derived between the rotation angles of the columns and beams, and a procedure for obtaining the ultimate strength and the collapse mechanism is shown. The analytical results show that plastic hinges should be formed not only at the beam ends and column bases but also at almost all the lower ends of the columns.

1. Introduction

In seismic design of composite moment frames, the weak beam strong column system is usually adopted. In this case, plastic hinges are supposed to form at all beam ends and column bases. A plastic hinge formed in the composite members shows not only the rotational deformation but also the axial deformation. The effect of this axial deformation at the plastic hinges on the collapse mechanism is important, but only a few researches have been made^{1,2,4)}. In this paper, the ultimate strength and the collapse mechanism of the composite frames are discussed, considering both the rotational and axial deformations at the plastic hinges. A recurrence formula is obtained between the rotation angles of the members. Using the formula, a procedure to obtain the ultimate strength and the collapse mechanism is shown, and some numerical analyses are carried out.

2. Ultimate Strength and Plastic Deformation of Composite Members

Fig.1 shows a composite member having a rectangular concrete cross section with an ideal I-section steel. For simplicity, the cross section is idealized into three point model such as shown in the figure. Considering rigid perfect plastic stress-strain relationship for both concrete and steel, such as shown in Fig.2, the yield condition of the idealized model cross section can be obtained as a hexagonal shape under the axial force (N) and bending moment (M), such as shown in Fig.3(a)^{3,4)}. The associated flow rule shows that the ratio between axial deformation e and rotation ω is constant when the generalized stress (N,M) moves on each line of the yield locus, such as



shown in Fig.3(b).

$$\begin{aligned}
 e &= -(D/3)\omega & \text{for } -sN_o \leq N \leq_c N_o/3 \\
 e &= 0 & \text{for } cN_o \leq N \leq 2cN_o/3 \\
 e &= (D/3)\omega & \text{for } 2cN_o \leq N \leq_c N_o + sN_o
 \end{aligned}$$

where $s_N_o = 2s\sigma_o A_f$ and $cN_o = c\sigma_o BD$. If the axial load level is low ($-sN_o \leq N \leq cN_o/3$), axial elongation occurs when the plastic hinge rotates, such as shown in Fig.3(c).

The yield condition of this range is

$$f = N - (sN_o / sM_o)M + sN_o = 0$$

Rate of energy dissipation D can be obtained as a simple form as follows:

$$D = N \cdot e + M \cdot \omega = s M_o \cdot \omega$$

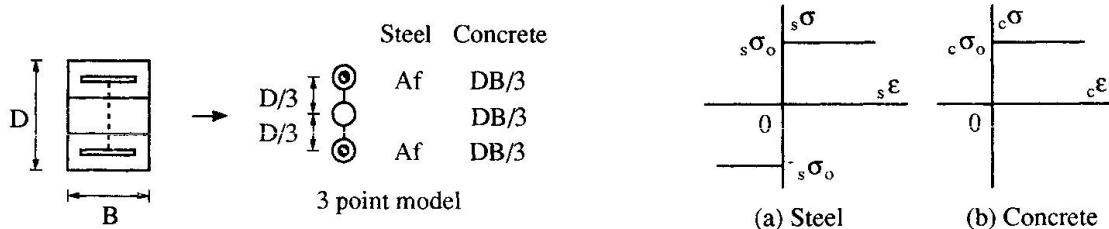


Fig.1 Model of composite cross section

Fig.2 Stress strain relations

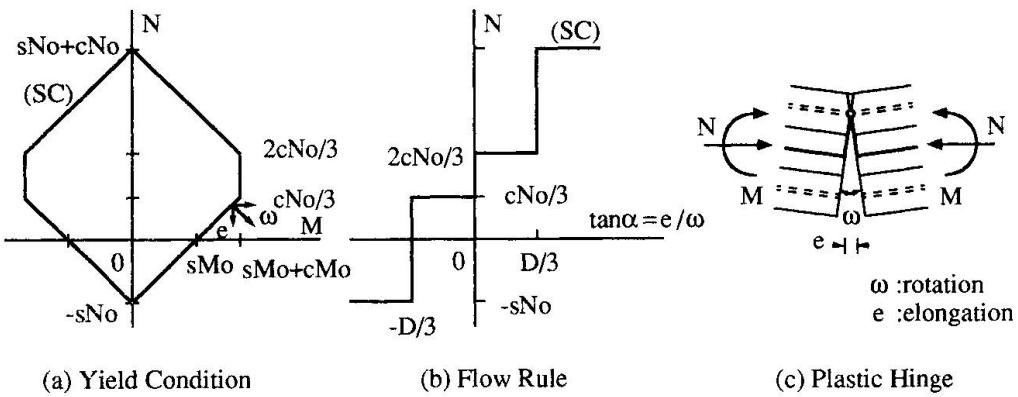


Fig.3 Yield condition and associated flow rule

3. Ultimate Strength and Collapse Mechanism of Composite Frames

3.1 Design of Composite Frames

The composite frames subjected to vertical and horizontal forces are designed to collapse by forming plastic hinges at all beam ends and column bases, using simple plastic analysis. To realize the weak beam and strong column frames, the total amount of plastic moment of the columns is 1.2 times as large as that of beams gathering at each beam to column joint. The vertical and horizontal forces are applied at each beam to column joint such as shown in Fig.4. The horizontal force is specified in conformance with the building standard law of Japan. The base shear coefficient is assumed to 0.3 for the ultimate strength stage, in this case. For simplicity, all the column concrete cross sections are the same ($D_c \times D_c$) and also all the beam cross sections are the same ($D_b / 2 \times D_b$) such as shown in Fig.5. The required plastic moment capacity of each member $s M_o$ is adjusted by the cross sectional area of the steel I section. The plastic moment of the outer column is assumed to be 0.6 times to that of the inner column.

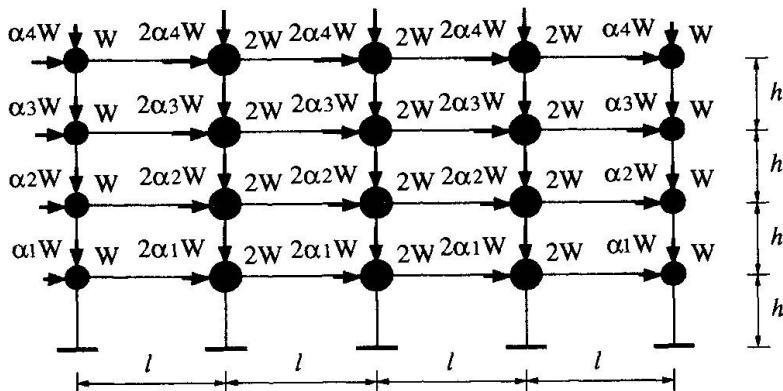


Fig.4 Composite frames subjected to vertical and horizontal loads

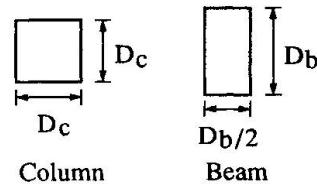


Fig.5 Cross section

3.2 Rotation Angle of the Members

The plastic hinge shows not only rotation but also elongation when the axial load level is low. According to the axial deformation at the plastic hinges, elongation of the beam or the columns occurs. Fig.6(a) shows a possible collapse mechanism of a 2-span 3-storied frame. The plastic hinges are supposed to be formed at all beam ends and column bases, according to the simple plastic analysis. But this collapse mode (collapse mechanism A) shows the additional plastic hinges formed at some lower ends of the columns, due to the axial elongation of the beams. And elongation of the column plastic hinges brings the rotational deformation to the beams. On the basis of geometric compatibility condition, the rotation angle of the columns $\omega_c(i, j)$ can be expressed as follows :

$$\omega_c(2, 3) = \omega_c(3, 3) = \omega_c(1, 3)$$

$$\omega_c(1, j) = [1 - (2D_b / 3hK_r)]\omega_c(1, j+1)$$

$$\omega_c(2, j) = [1 - (2D_b / 3hK_l)]\omega_c(2, j+1) + [\omega_c(1, j+1) - \omega_c(1, j)]/K_l$$

$$\omega_c(3, j) = [1 - (2D_b / 3hK_l)]\omega_c(3, j+1) + \sum_{i=1}^2 [\omega_c(i, j+1) - \omega_c(i, j)]/K_l$$

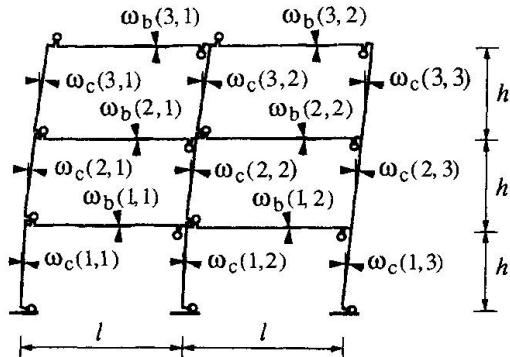
where $K_l = 1 + (D_b / 3h)[1 - (2D_c / 3l)]$ and for $j=1, 2$.

And the rotation angle of the beams $\omega_b(i, j)$ can be expressed as follows:

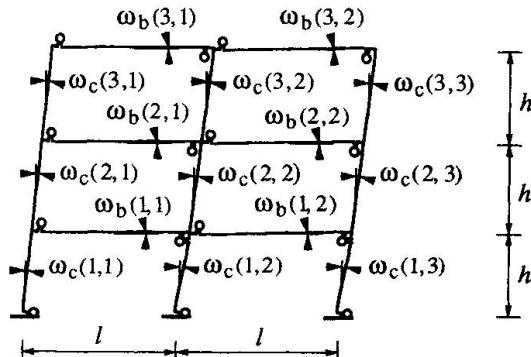
$$\omega_b(i, j) = (D_c / l)[\omega_c(i, j) - \omega_c(i, j-1)]$$

for $i=1, 2, 3$ and $j=1, 2$.

Fig.6(b) shows another possible collapse mechanism (collapse mechanism B) of the same frame.



(a) Collapse mechanism A



(b) Collapse mechanism B

Fig.6 Collapse mechanism of 2-span 3-storied composite frame



The left hand side column forms a plastic hinge only at the column base. In this case, additional plastic hinges are formed at the upper end of another first-story columns. Using geometric compatibility condition, the rotation angle of the columns and beams can be expressed by the following equations:

$$\omega_c(1,2) = \omega_c(1,3) = [1 + (2D_b/3hK_r)]\omega_c(1,1)$$

$$\omega_c(2,j) = \omega_c(1,1)$$

$$\omega_c(3,j) = \omega_c(1,1)$$

$$\omega_b(i,j) = 4D_c D_b / (9hK_r)$$

where $K_r = 1 - (4D_c D_b / 9h)$ and for $i=1,2,3$ and $j=1,2$.

Using the rotation angle of the column having a plastic hinge only at the column base, $\omega_c(1,1)$ in this case and $\omega_c(1,3)$ in the former case, the rotation angles of another columns and beams can be expressed by the recurrence formula shown above. The recurrence formula can be obtained for any composite frames designed according to the conditions shown in section 3.1.

3.3 Ultimate Load and Collapse Mechanism

The ultimate load and the collapse mechanism of the frames (from 2-span 2-storied to 6-span 6-storied frames) will be discussed in this section. Assuming the position of a column with a plastic hinge only at the column base, rotation angles of another columns and beams can be expressed, using the recurrence formula. The ultimate load factor λ associated with an assumed collapse mechanism can be obtained using the principle of virtual work as follows:

$$\begin{aligned} & \lambda \sum_{j=1}^{m+1} \sum_{i=1}^n \alpha(i,j) W(i,j) u(i,j) - \sum_{j=1}^{m+1} \sum_{i=1}^n W(i,j) v(i,j) \\ &= \sum_{j=1}^{m+1} \sum_{i=1}^n \{ \xi(i,j,l) c \omega_o(i,j,l) + \xi(i,j,u) c \omega_o(i,j,u) \}_{sc} M_o(i,j) \\ &+ \sum_{j=1}^m \sum_{i=1}^n \{ \xi(i,j,l) b \omega_o(i,j,l) + \xi(i,j,r) b \omega_o(i,j,r) \}_{sb} M_o(i,j) \end{aligned}$$

where $\alpha(i,j)$ shows the ratio of the lateral load to the vertical load $W(i,j)$ and $u(i,j)$ and $v(i,j)$ are horizontal and vertical displacements at the (i,j) th joint. $c \omega_o(i,j,l)$ and $c \omega_o(i,j,u)$ show the rotation of the plastic hinges at the lower end (l) and the upper end (u) of the (i,j) th column, and $b \omega_o(i,j,l)$ and $b \omega_o(i,j,r)$ the rotation of the plastic hinges at the left side (l) and the right side (r) of the (i,j) th beam. $\xi=1$ is for active plastic hinge and $\xi=0$ without plastic hinge. Changing the position of the column with plastic hinge only at its base in turn from left to right, the corresponding ultimate load factor can be obtained. According to the upper bound theorem of plasticity, the minimum load factor obtained is the real collapse load factor and the associated mechanism is the real collapse mechanism. Table 1 shows the collapse load factors calculated. $\lambda=1$ shows the load factor obtained using simple plastic analysis. As the number of stories increases, the collapse load factor decreases, and as the number of spans increases, the collapse load factor increases in a gentle manner, because the effect of the axial force of the beams on the bending capacity is small. Table 2 shows the position of the column forming plastic hinge only at the column base. The position of the column is usually at the right part of the frame, and as the number of stories increases and as the number of spans decreases, the position moves to the right end. So almost all the additional plastic hinges are formed at the lower end of the columns. Fig.7 shows an example of the collapse mechanism. The column having a plastic hinge only at the column base is the 2nd column from the right ($j=6$). The bar graph shows the ratio of the rotation angle of the plastic hinges formed. In the upper part of the frame, rotation of plastic hinges at the lower end of the columns is very small. In the lower part of the frame, rotation of plastic hinges at the beam ends or the column bases decreases as the number of spans (j) decreases but rotation increases at the

Table 1 Collapse load factor λ

number of stories	number of spans	2	3	4	5	6
2		1.123	1.126	1.131	1.136	1.141
3		1.082	1.084	1.086	1.089	1.090
4		1.065	1.067	1.069	1.071	1.072
5		1.053	1.055	1.056	1.058	1.059
6		1.047	1.046	1.047	1.049	1.050

Table 2 Position of the column forming plastic hinge only at the column base (j)

number of stories	number of spans	2	3	4	5	6
2		j=2	3	4	5	5
3		3	4	5	5	6
4		3	4	5	5	6
5		3	4	5	6	6
6		3	4	5	6	6

lower end of the columns as j decreases. Fig.7 shows not only the result of the 6-span 6-storied frame but also the information of another frames. For example, the collapse mechanism of the 4-span 4-storied frame can be shown by the dotted region in Fig.7. On the basis of the results shown in Table 1, the collapse mechanism and the rotation angles of the plastic hinges of the composite frames ranging from 2-span 2-storied to 6-span 6-storied can be obtained.

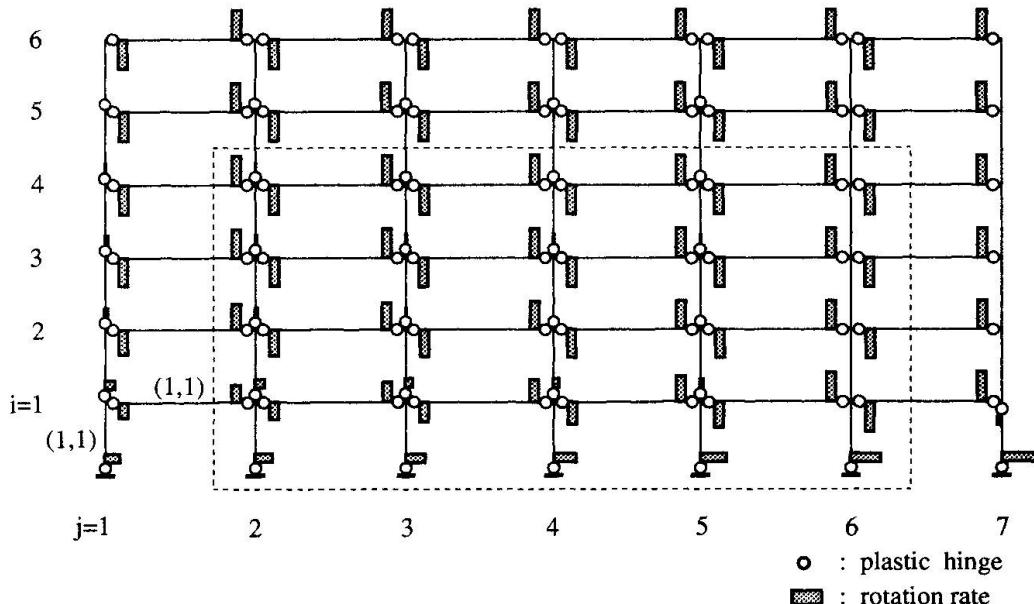


Fig.7 Collapse mechanism and rotation angle of plastic hinges

4. Conclusions

This paper presented a method for obtaining the ultimate strength and the collapse mechanism of the composite frames subjected to seismic force, considering the yield condition of the composite



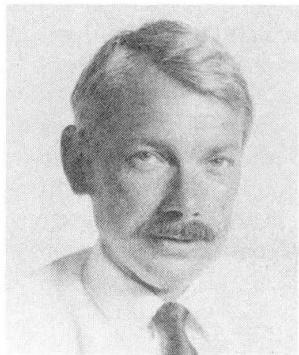
members and the associated flow rule. A recurrence formula is derived between the rotation angles of the plastic hinges formed at the beams and columns. Using the formula, the collapse mechanism of the composite frames composed of strong columns and weak beams is clarified, in the case of the low rise buildings. The real collapse mechanism shows that the plastic hinges are formed not only at the beam ends and column bases but also at almost all the lower end of the columns. The distribution of rotation at the plastic hinges changing from hinge to hinge, is also clarified.

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Advanced Composites for Earthquake Resistant Structures

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Peter Head has played a key role in research and design development leading to the introduction of advanced composites in civil and structural engineering. Notable successes are the world's first all composite footbridge, first advanced composite road bridge and one of the first all composite multi-storey buildings. He was awarded the Royal Academy of Engineering's Silver Medal in 1995 for an outstanding personal contribution to British Industry.

Summary

Lightweight high-strength advanced composite materials are now being used to retrofit and strengthen many earthquake-prone structures in Japan and USA. Little attention has been paid to the role these materials could play in the design of a new generation of building and bridge structures which have much greater resistance to earthquakes. The high strength to weight ratio of fibre reinforced polymers makes them ideally suited to this role and development work in new forms of composite structure carried out over the last ten years points the way forward. The paper examines this work in detail and shows the new forms of structure that could be adopted in the future to provide more cost effective and reliable solutions.

1. Introduction

Advanced composite fibre reinforced polymer materials are now being introduced into a wide range of civil and structural engineering applications. In Japan these include concrete reinforcement and prestressing using carbon fibres, ground anchors using carbon and aramid fibres and rock bolts using glass fibres. Perhaps the largest application area is the retrofit strengthening of existing reinforced concrete structures to improve their resistance to earthquake damage. The high strength and stiffness characteristics of the fibres, when bonded to and wrapped around concrete columns and beams has been found to greatly increase the ductability of the reinforced concrete in the rapidly changing stress cycles experienced during earthquakes.

The stresses generated in earthquakes are a direct function of the mass and dynamic frequency response of the structure. Considerable design development has taken place in the last thirty years to refine the way in which steel and concrete are used in tall buildings and bridges to improve earthquake resistance and further developments are reported at this Conference.

However most findings are reported with no recognition of the important role that advanced composite materials could play in this design area in the future. Research work carried out in the United Kingdom over the last 15 years has demonstrated new structural forms that can be used to build lightweight advanced composite bridges and building structures.

The paper presents detailed information on these systems and then indicates using some design studies how they might be applied in the future to achieve reliable but effective solutions for major structures in earthquake prone areas. The paper concludes by suggesting areas of research for the future.



2. Advanced Composite Materials and Building Systems

Advanced composite materials are fibre reinforced polymer structural sections in which the continuous fibres have high strength and stiffness and are orientated in the direction of maximum stress (Fig.1). Most sections currently used in construction are pultruded and typical properties for different fibre types are shown in Table 1. Important physical characteristics are the high strength to weight ratio compared with steel and concrete which makes the materials more attractive as the structure span and height increases. Designs, particularly when using glass reinforcement, are governed by stiffness rather than strength, because of the high strength to stiffness ratio. Hence in actual designs we find that the low structural weight reduces cyclical dynamic earthquake forces in these structures and there is also a large reserve of strength and strain capability before rupture. Although the materials are inherently brittle in tensile tests, well designed structures exhibit much improved earthquake resistance. The method of construction and form of connection are, however, critical.

Fibre Reinforcement		Glass	Carbon	Aramid	High Tensile Steel Wire
Fibre Fraction	%wt	80	72	67	-
Relative Density	ρ	2.31	1.57 ¹	1.36	7.86
Tensile Strength τ	GPa	0.95	1.61	1.59	1.82
Tensile Modulus ϵ	GPa	50	136	64.3	200
Specific Strength	τ/ρ	44	105	119	24
Specific Modulus	ϵ/ρ	21.6	86.6	47.3	25.4

Table 1: Properties of composites with high proportions of unidirectional fibre reinforcement



Fig. 1 Pultruded section showing glass reinforcement

The Advanced Composite Construction System (ACCS) was conceived and developed by Maunsell to try to meet the growing demand for a lightweight modular system for constructing buildings and bridges which would reduce the labour costs for assembly and would provide lower capital and operating costs. Other systems may come along in the future but this proven one enables new forms of bridge and building to be illustrated. The system that was conceived and patented in 1982 is a cellular system of components (Fig 2) which can be formed around an insulating core.

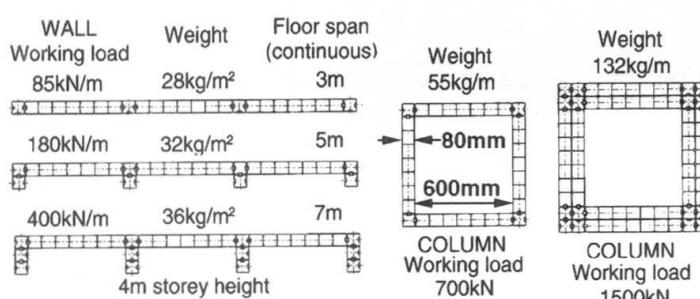


Fig. 2 ACCS components

The unique method of connection allows any two components to be brought together, side by side, with epoxy adhesive applied to one face, and by sliding the toggle connector into the groove which holds the components together in accurate alignment while the adhesive cures. In site applications, heaters can be used to achieve controlled and rapid cure.

The first Research Programme undertaken on structures built from this System was a LINK programme carried out at the University of Surrey between 1991-93 in which two major beams, each 18 metres long, were built (Fig 3) and subjected to a wide range of long and short term static and dynamic load tests (2), including full flexural tests to

Walls, columns, beams, bridge girders, floors and roofs can be built using just three primary components pultruded from glass fibres and a suitable thermoset resin and insulating core materials. Structures can be designed using a fully developed limit state design method which is consistent with British and European building codes (1) (Fig 3).

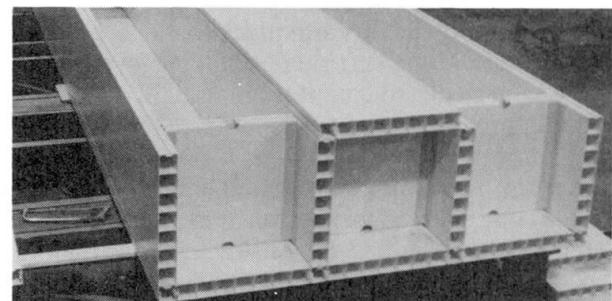


Fig. 3 LINK beam being assembled at the works

destruction. Small components were also subjected to a full range of accelerated weathering, fatigue, fire and strain aging tests.

3. Bridge Decks

ACCS has been used to build a variety of different bridge decks. The beam shown in Fig 3 has been used for the design of footbridges with spans up to 17 metres (Fig 4) and road bridges with spans up to 12 metres (to be constructed in 1999). The weight of the beams is 40% of an all steel design and less than 15% of a steel composite solution with a concrete deck. Hence in addition to offering excellent durability, the lightweight structures offer much reduced vertical and horizontal earthquake forces generated in support structures.



Fig. 4 Parson's Bridge, UK

Fig. 4 Parson's Bridge, UK

Designs are also being progressed in which the advanced composite materials are used to form an aerodynamic enclosure and torsional box connected to a lightweight steel space frame truss (Fig 5). This system is called *SPACES* and offers construction (3) and earthquake resistant advantages also. The weight can be significantly less than a traditional truss bridge deck.

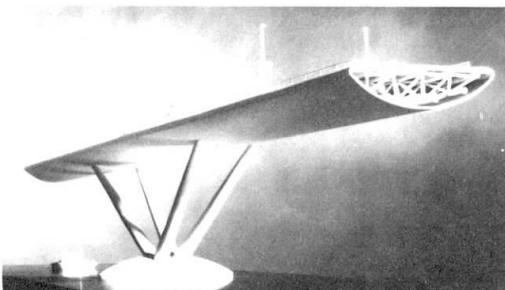


Fig. 5 Poole Harbour bridge design, UK

4. Building Structures

4.1 Beam, Column, Floor, Wall and Roof Sections

Figures 2 and 6 show typical configurations of ACCS used in beam and slab floors, load bearing walls and columns. Table 2 shows some comparative weights of typical steel, concrete, composite and brick structures which provide similar load carrying and other functional characteristics. It can be seen that very substantial weight reductions can be achieved using advanced composite materials and these can have major advantages in reducing foundation costs and improving earthquake resistance in tall buildings. Earthquake resistance also derives from the high strength to weight ratio and lower elastic stiffness. A two storey office building was built in Bristol in 1992 using this System and no other materials. It has been used successfully ever since (Fig 7).

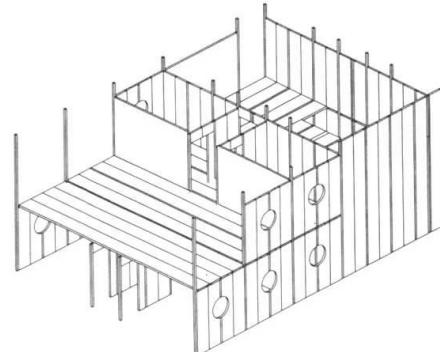


Fig. 6 Typical configuration of ACCS uses in buildings



Fig. 7 Second Severn Crossing site office

Typical Walls	ACCS Wall Brick Cavity Wall Reinforced Concrete Wall	= 0.3 Kn/m ² = 2.0 Kn/m ² = 2.2 Kn/m ²
Typical Floors (5 metre span)	ACCS Floor Reinforced concrete floor Lightweight concrete/metal deck floor	= 0.35 Kn/m ² = 3.3 Kn/m ² = 2.2 Kn/m ²

Table 2: Comparative weights of different materials

4.2 Thermal Insulation

ACCS sections 80mm thick with a polyurethane foam core have a thermal insulation value of $U = 0.35 \text{ W/m}^2\text{K}$. This meets UK requirements for domestic buildings and offices. Buildings can be



fitted with double glazed windows with pultruded GRP window frames and this then provides a largely integral glass structure with excellent thermal insulation and uniform temperature expansion characteristics. The GRP sections overcome the cold-bridge effect and buildings have been found to be comfortable and have low running costs. Window seals maintain their integrity because of the lack of differential expansion.

4.3 Fire Performance

ACCS sections manufactured from glass fibre and isophthalic polyester resins, with a polyurethane foam core, have been tested in a whole range of fire tests (Fig 8). The high glass content in each pultruded section means that heat is not conducted away from the fire source, or through the panel thickness. Also glass has a relatively high melting temperature and the cellular configuration of the panels maintains load carrying integrity for long periods of time. Panels meet BS 476 Part 7 Class 1 surface spread of flame test requirements and can provide up to 60 minutes fire resistance as a wall or floor section in a BS 476 Part 21 Test. Surface coatings can be used to achieve Class 0 spread of flame performance and to further inhibit smoke emission. If smoke is a design criterion, alternative resins are available for pultrusion such as phenolics and methacrylates.

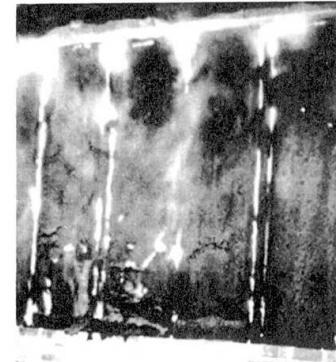


Fig. 8 ACCS wall panel test to BS 476 Part 21

4.4 Durability and Long Term Performance

Well designed Advanced Composite building structures such as ACCS can be expected to have low maintenance costs because of the inherent excellent durability of the materials. They are resistant to UV deterioration, moisture ingress and chemical attack because of the high glass content, careful manufacture and choice of resin. The technological advances made by those developing ACCS over the last ten years has led to a quality of manufacture and accuracy of fit that has previously been unavailable in the building industry. This enables water tightness and integrity to be achieved consistently with new modular forms of construction.

4.5 Environmental Considerations

There is a worldwide realisation of the damage that a rapidly rising energy-consuming population can do to the ecology and environmental balance of the planet. There is general agreement that future technology developments will need to meet criteria such as biological sustainability, minimum use of energy and raw materials that will probably be set internationally.

Advanced composite materials are being seen to have benefits compared with traditional materials in many of these areas, particularly in their low energy consumption during manufacture, construction and subsequent building operation (4). Although there are environmental problems associated with material manufacture, particularly resins, a number of far sighted manufacturers are already improving this aspect significantly. Glass reinforced plastics require relatively low energy during manufacture compared with metal structures, have much lower thermal conductivity and are more durable.

4.6 Electrical Properties and Service Integration

Electrical 'field free' properties are inherent in an ACCS building structure which is a major advantage in an age when buildings are becoming a support framework for large complex computer networks, whether it be the office or home. The cellular structure also enables services to be channelled out of sight. There is an inherent labyrinth of service channels in the panels of a building. If a water service leaks in an unseen location there is no danger of corrosion and structural deterioration.

4.7 Acoustic Insulation

Tests have shown that attenuation of airborne noise through a standard ACCS panel is much better than simple calculations based on the weight would suggest and is typically 25dB Leq. Structural borne noise is a significant design matter because of the lightweight structure. Special floor tiles with a foam underlay, or suspended floor structures are needed where stringent noise transmission requirements are present.

4.8 Blast and Dynamic Load Resistance

The integrity and flexibility of bonded monocoque advanced composite building structures makes them uniquely resistant to bomb and other dynamic loading such as gas explosions. This has been proven in tests carried out for the use of the materials in blast walls for offshore structures (5). Advanced composites have become important materials in offshore construction over the last five years to reduce topside weights and to improve accidental blast and fire resistance. This points the way for the use of the materials in tall buildings.

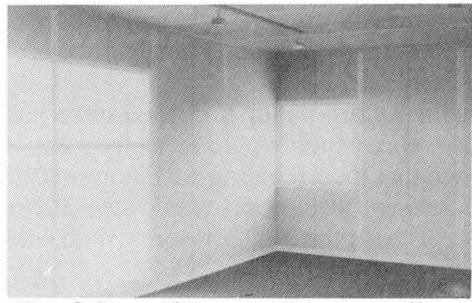


Fig. 9 Second Severn Crossing office conference room

5. Forms of Structure and Connections

A new form of building structure has been developed using ACCS in which the walls and floors are bonded together to form an integral monocoque structure (Fig 6). Vertical ACCS panels in the walls, with suitable openings for doors and windows, form the vertical box carrying members and floors span between them and act as stiffening diaphragms to the vertical box structure. Floors can be ribbed slabs or a beam and slab form if the spans are large.

Wall and floor components can be delivered flat packed and are light enough to be handled by small cranes or robots. Connections are made by bonding as described earlier and load continuity between ends of sections is achieved by bonded lapping sections, rather like bolted splice plates where bolts are replaced with adhesive. Connection integrity can be further enhanced with local *in situ* carbon fibre wrapping.

There is no reason why very tall multi-storey buildings cannot be built this way in the future. Such structures will have major advantages in earthquake prone areas and will enable a completely new approach to building safety to be achieved in these situations. Fig 9 shows the inside of an ACCS office building.

Design work carried out to date has shown that buildings up to ten storeys high can be built using the form of structure shown in Fig 6, and excellent resistance to even extreme earthquakes can be achieved. Also foundation loads are much lower and so construction on poor ground can become more attractive and cracking caused by differential settlement is eliminated.

The architectural possibilities of this form of building structure are only just beginning to be investigated. The combination of ACCS pultrusions, curved mouldings and glass provide exciting opportunities for new structural forms.

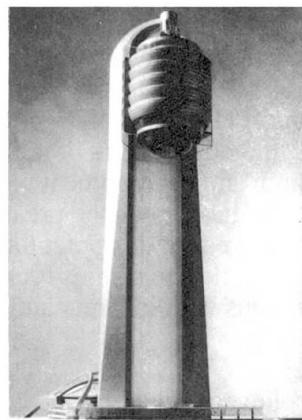


Fig. 10 Globoroma Tower

Fig 10 shows a proposed tower structure called Globoroma in which the top viewing platforms and entertainment centre are constructed using bonded advanced composite materials. The lightweight materials enable such new forms of structure to be considered, whereas previously with traditional materials dynamic oscillation would have been a near insoluble problem. A small mass damper is all that is needed to ensure the comfort of users at the top of the 190m high tower.

6. Extending The Limits of Tall Buildings

An important question that needs to be addressed is cost. Advanced composite materials are generally more expensive materials than steel and concrete but can form cost effective complete structures when advantages of lightness and ease of construction are fully utilised. When incorporated in taller buildings the advantages of weight reduction become greater and greater. Since even two storey buildings can be attractive from a cost point of view, when mass produced, it is certain that their use in tall buildings of



the future will grow rapidly and most particularly in earthquake prone areas. Here both vertical and horizontal acceleration effects will be accommodated more easily.

Obayashi Corporation proposed some years ago a building structure called Aeropolis 2001 (Fig 11) in Tokyo Bay which would be a city complex rising 2000m into the sky. A desk study by the author has shown that incorporating FRP composite floor systems into the 500 storeys instead of the lightweight concrete and steel floors would save 30% of the main column steel weight, a staggering 270,000 tonnes of steel. Also the lighter floors would substantially reduce the design problems under earthquake loading.

Lightweight materials can also be used to build extra storeys on top of existing buildings, thereby gaining significant additional revenue from a developer's existing land.

Building designers can also consider putting an increasing proportion of advanced composites into a building at a greater height from the ground. A building could therefore start with traditional heavy steel and concrete at ground level and move to the use of lighter advanced composite floors at the middle levels and then change to an all advanced composite bonded monocoque form at the top. This change in material could also be reflected in the architecture to bring more curved aerodynamic shapes at the higher levels, where wind speeds are greater.



Fig. 11 Aeropolis 2001

7. Conclusions

Sophisticated design solutions are now being adopted in steel and concrete to produce cost effective tall buildings and bridges in earthquake prone areas. These include large dynamic dampers and the introduction of plastic hinges in steel beams.

More cost effective solutions may lie in the incorporation of advanced composite materials into bridge decks, the upper floors of tall buildings and in the complete structures of buildings up to ten storeys. Research and development work in the United Kingdom and the construction of a number of prototype structures has shown the great potential of these materials. Areas of research and development that are needed to realise this potential in other countries are testing for earthquake resistance and fire resistance; design studies for overall costing; national testing for compliance with Building Standards; discussions with Insurers and Statutory Authorities and production of building and construction standards.

It is believed that advanced composite materials will gain significant applications in building construction in earthquake prone areas of the world within the next 10 years.

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Use of I-Beams in Slim Floor Construction

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Summary

The use of I-beam in Slim floor instead of the Universal column (UC) or Top Hat sections is the focus of this study. The investigation is limited to the design of simply supported beam under uniformly distributed load using limit state approach. The design formulation has the provision for both top and bottom flange plates. The results indicate the suitability of using such beams in Slim floor construction especially for structures subjected to heavier loads like those in warehouses, bridges etc. The cost competitiveness has also been established despite two times increase in steel consumption as compared to RCC structure.

1. Introduction

Slim floor construction is a special type of composite construction, where the beam is contained within the depth of the floor slab. The success of this type of construction using top hat sections as beam elements in Nordic countries, especially in Sweden in the last decade, has resulted in the adaptation of this type of construction in UK tailored to suit their requirement, using UC sections and bottom flange plates, supporting either the profile deck or precast slab both having concrete topping (1).

The major advantages of this type of construction lies in its ability to cover large column free areas, reduce building height, reduce overall weight of the structure, better seismic and fire resistance, faster construction, unhindered passage for service lines, elimination of the use of shuttering and props for the slab-beam system and better cost competitiveness despite increase in consumption of steel.

The work carried out till date has been limited to the use of top hat sections and UC sections. The suitability of using I-beams in place of UC section or top hat section in Slim floor construction was felt to be important as I-beams are the most commonly available beam section in developing countries. This study aims at introducing I-beams in Slim floor construction.



2. Structural Configuration and Design

The simply supported Slim floor beam (Fig.1) considered here consists of a rolled I-beam having a bottom flange plate supporting a deep profile deck topped with concrete. The beam can also have a top flange plate to increase its efficiency and strength. The RCC topping can be either of normal weight concrete(NWC) or light weight concrete(LWC). Shear stud connectors welded to the top flange of the I-beam ensures composite action.

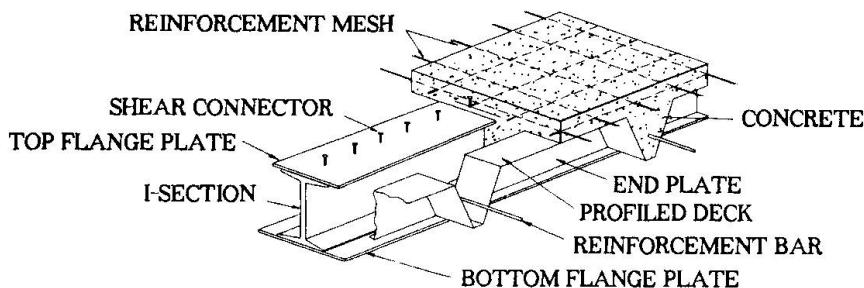


Fig.1 Schematic View Of A Slim Floor Beam

2.1. Basis of Design

Limit state approach has been followed for design. Factored load was used for strength design and working load for serviceability limits. In the absence of any uniform design guidelines in different countries, the provisions of the British codes (2,3) in general and those of Indian codes (4,5) in particular have been followed.

Only plastic or compact rolled beam sections in conjunction with rectangular stress block have been used for design. To ensure non-occurrence of irreversible deformations, the steel stress was limited to design stress and the concrete stress to 0.5 times the cube strength. The design steel stress may be either the yield stress or its reduced value depending on the code of practice to be followed.

2.2. Steps in Design

In line with the sequence of construction that is followed, the design was carried out for both the construction stage and the composite stage loads.

Construction stage

The bottom flange plate was designed for biaxial state of stress caused by the deck loading and overall bending of the beam. Von-Mises Yield criteria (6) was used to obtain the moment capacity.

The combined steel section was designed considering the effect of lateral transverse buckling (LTB) caused due to the top flange being unrestrained (2).



During construction stage, the out of balance load on the beam caused by one side being fully loaded (concrete poured) results in torsion in the section. This was treated in a simplified manner by replacing these forces by equal and opposite horizontal transverse forces in the flanges in equilibrium with the torsion caused by the out of balance load.

The unity factor condition was checked for the combined stresses caused by biaxial bending. The beam was also checked for LTB in case of bending moment caused by the total construction load on the beam.

Composite stage

The effective breadth of concrete compression flange was taken as $\frac{1}{4}$ of the span but was limited to centre to centre distance of the beam. The modular ratio was taken as 10 and 15 for NWC and LWC respectively. The plastic moment capacity which depends on the degree of shear connection (assumed as 40%) was obtained in terms of the resistance of various elements of the beam by rearranging the stress diagram in a manner similar to that explained in British code (3).

Transverse reinforcement was provided to enable the concrete flange to transfer the longitudinal forces at ultimate limit state into the slab without splitting of concrete (3).

Serviceability Limit Check

All critical serviceability stresses (3) in concrete and steel were checked using cracked section properties. Deflection checks (3) were carried out for both construction stage and composite stage. Since most of these beams were quite long, the calculated natural frequency of vibration, based on the total dead load plus 10% imposed load, was limited to 4 Hz.

2.3. Computer Programme

A programme in FORTRAN77 has been developed for the automated design of Slim floor beams having both top and bottom flange plates. The input data consists of a trial I-beam section, flange plate dimensions, span, loads and details about the profile deck and material properties. The output from the programme consists of the most optimum I-beam section, the final flange plate dimensions, number of shear connectors and the thickness of RCC topping.

3. Results and Discussion

A few design examples with only bottom flange plate configuration have been worked out to study the various aspects of using I-beams in Slim floor construction. The I-beams and plates used have been restricted to those manufactured by Steel Authority of India Ltd.

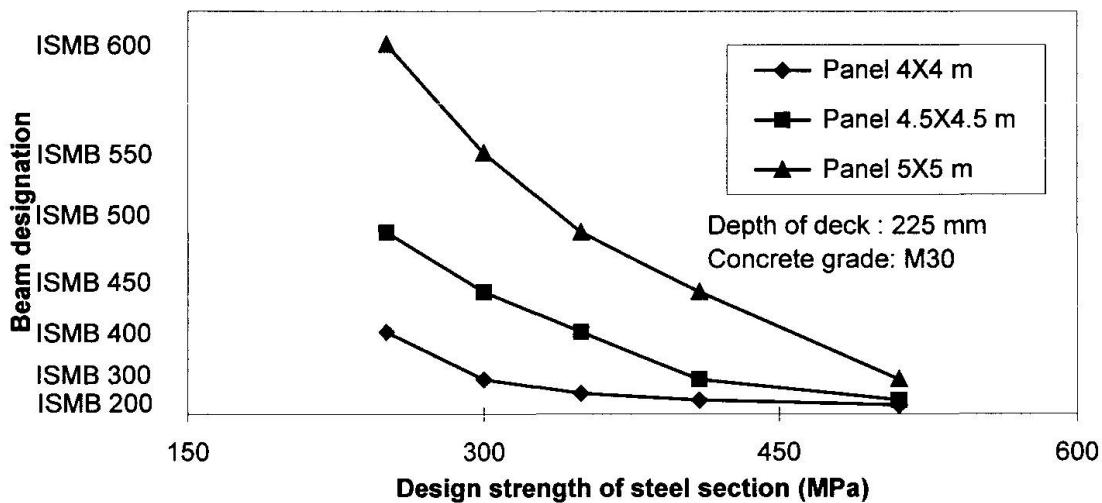
Example 1 : A Slim floor beam for a 4.5 m x 4.5 m panel having imposed load of 6.0 kN and LWC (grade M30) topping have been designed using I-beam and compared with that using UC section as shown in Table 1. In both the cases yield strength of steel was 510 MPa, bottom flange plate thickness was 16 mm and steel decking was 210 mm deep. The smaller depth for UC section using British code can be attributed to higher distribution of material in its flanges



making it a more efficient section as compared to the I-beams. The higher load factor values as prescribed in Indian code, have led to even greater overall and section depth.

Item	British Code Load Factor		Indian Code Load Factor
	UC Section	I-Section	I-Section
Overall depth (mm)	295	310	335
Section depth (mm)	176.8	225	250

Example 2 : A comparative study of the effect of the steel strength on the size of I-beams is depicted in Fig.2. As expected, the size of I-beams decreases with the increase in the design stress of the material. Moreover, it is also noticed that as the steel strength increases, the effect



of panel dimension on the size of beam reduces in an asymptotic manner indicating greater moment capacity of beams with higher design strengths.

Example 3 : A seven storied commercial building having 400 sq.mt. area in each floor has been

Items	Type of structure			
	RCC		Slim floor	
	Quantity	Total (Rs)	Quantity	Total (Rs)
Reinforcement bars	66.53 T	11,84,378	15.95 T	2,84,052
SAIL-MA (Y S 410MPa)	-	-	86.24 T	21,99,120
IS: 513 sheet (Y S 250MPa)	-	-	45.01 T	11,20,749
Plastering wall	4018 M ²	1,86,354	2774.1 M ²	1,27,786
Plastering ceiling	2800 M ²	1,53,748	-	-
Brick work	416.15 M ³	6,10,076	344.4 M ³	5,04,890
Concreting	621.74 M ³	22,63,134	585 M ³	21,19,400
Shuttering	2800 m ²	3,39,780	-	-
False ceiling	2800 m ²	12,60,000	-	-
Interest	5 months	4,50,650	-	-
Total cost		64,48,120		63,55,997

designed as conventional RCC construction and also as Slim floor construction for cost comparison studies. Occupational and partition loads have been assumed as 4.0 kN/m^2 and 1.0 kN/m^2 . The RCC design has been carried out using limit state approach (7). The cost analysis shown in Table 2 is based on the current market price in India (Rs. 38=1US \$). The cost of both the structures are practically same. However, Slim floor construction has the added advantage of the building being available earlier for occupation due to faster construction and the salvage value of steel. The increase in steel consumption in case of Slim floor construction using I-beams is 2.2 times that of RCC construction as against about 4 times with UC-section (8).

Example 4 : Table 3 depicts the maximum imposed load a particular beam section can withstand for a fixed span. In all the cases the flange plate thickness was 16 mm, the grade of concrete was M30 and the yield strength of steel was 350 MPa with 210 mm deep steel decking. It is evident

Panel width (m)	Span (m)		
	4.0	5.0	6.0
	Maximum Imposed load (kN/m^2)		
4.0	22 (ISMB 225)	50 (ISMB 450)	> 50 (ISMB 550)
5.0	38 (ISMB 250)	> 50 (ISMB 500)	> 50 (ISMB 600)
6.0	40 (ISMB 350)	> 50 (ISMB 550)	--

that upto a certain span the design is governed by the construction load and is not sensitive to the imposed load.

Example 5 : Keeping in mind the requirement of reducing the dead load due to concrete, two sizes of profile decks of depth 225 mm and 290 mm were considered in the design along with I-beams in Slim floor construction. The results are presented in Table 4 for LWC which shows that after a particular panel size, the Slim floor construction with deeper deck is advantageous. Same trend has also been observed using NWC.

Panel size	225 mm Deck			290 mm Deck		
	Depth of section (mm)	Plate thickness (mm)	Overall depth (mm)	Depth of section (mm)	Plate thickness (mm)	Overall depth (mm)
3.0 m X 3.0 m	200	8	295	225	8	360
4.0 m X 4.0 m	250	12	335	250	8	360
4.5 m X 4.5 m	400	24	485	300	22	385
5.0 m X 5.0 m	500	20	585	500	10	585
5.5 m X 5.5 m	600	8	685	550	28	635



4. Conclusion

The analysis of the preceding examples brings out certain important features of Slim floor construction using I-beams .

1. It is cost competitive as compared to RCC construction.
2. The use of high strength beams for longer spans are more economical as compared to ordinary structural steel beams with yield strength of 250 kN/mm^2 .
3. The ability of the Slim floor beams to support heavier imposed loads makes them suitable for use in warehouses, bridge decks etc. Conversely, for a particular imposed load condition, it is possible to cover longer span.

As the present study was restricted to simply supported beam under uniformly distributed load, it may be worthwhile to develop design and construction technique for primary and secondary beam system to cover even larger column free areas. For extending the Slim floor construction concept to bridge decks, study of dynamic behaviour of such construction needs to be carried out.

Acknowledgement

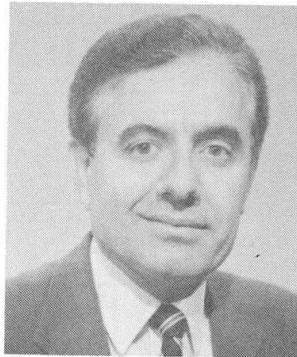
The present work was carried out at the Application Engineering Centre of Steel Authority of India Limited as a part of its continuing programme on increase steel usage in building structures. The authors gratefully acknowledge the support extended by the R&D Centre for Iron and Steel, Steel Authority of India Limited.

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Serviceability of Buildings Subjected to Seismic Excitations

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Summary

The response of a vibrating building, due to earthquake or wind, is greatly influenced by the soil-structure interaction. This interaction modifies the building resonant frequencies and affects its serviceability relative to the human occupant. In this paper, the serviceability is measured in terms of "human comfort" which is expressed as absorbed power (rate of energy dissipation) through a biomechanical model placed at a given floor in the building. This single value encompasses the characteristics of the structure, the soil, the human, and the dynamic excitation. The latter is represented by its power spectrum whose parameters were evaluated by using nonlinear regression on available earthquake spectra. The results indicate that absorbed power differentiates between comfort levels at different floors, and that the damping in the structure as well as the soil foundation and power spectrum characteristics have significant impact on building serviceability.

1. Serviceability of buildings to vibration

In addition to being functional, a building must have structural integrity and be serviceable relative to the human user. The serviceability of a building in a vibrational environment has been the subject of numerous studies and recommendations, Chang and Robertson(3), and Chen(4). The recommended criteria have tended to specify acceleration, velocity or displacement limits. For example, it is generally accepted that accelerations of the order of $0.5\%g$ - $1\%g$ are perceptible, $1\%g$ - $5\%g$ are annoying, and $>15\%g$ are disturbing and may be intolerable. Some recommendations for residential buildings, Chang(3) also couple the amplitude of vibration with a corresponding frequency thus leading to human comfort limit curves which are a function of frequency. However, it has been shown by Farah (7) that human comfort levels can be evaluated in terms of the absorbed power through a biomechanical model. This single value incorporates the characteristics of the human, soil-structure system and the seismic excitation.

Earthquake excitation characteristics coupled with suitable site conditions can lead to a large amplification of the structural response of a building. Such a situation arose with the 1985 Mexico earthquake where resonance was set up in buildings due to the soft Tacubaya Clays near the surface of the lake bed in Mexico City, Abiss(1), resulting in extensive damage.

In this work, the seismic excitation is represented by its power spectrum, the building is modeled as a multi-degree-of-freedom system, the soil foundation is expressed in terms of translation and rocking motions resulting in 2DOF possessing mass, stiffness and damping elements, and the human is represented by a 3DOF model simulating a standing human.



2. System modeling

The computation of the building serviceability requires the modeling of the human, the building, the soil-structure interaction, and the seismic excitation.

2.1 Biomechanical model

The human body is highly sensitive to vibration, especially in the standing position. A suitable biomechanical model of a standing human in the fore and aft mode was developed by Farah(6) and is shown in Fig. 1. The parameters of the model were obtained by fitting its frequency response to available experimental data. Based on this model, the absorbed power for the thresholds of perception, annoyance and intolerance in residential buildings are 1.33×10^{-4} W, 8×10^{-4} W and 1.7W respectively. Note that there is great variation in human response to vibration among individuals, and thus the response of the biomechanical model used in this work should be considered only as being a reasonable representation. The biomechanical model has three resonant frequencies, 0.58, 11.10, and 17.00 Hz. Note that the fundamental frequency lies within the range of the fundamental frequencies of tall and medium-height buildings.

2.2 Building model

The building is represented as a multi-degree-of-freedom (MDOF) lumped-parameter system consisting of masses, springs, and viscous damping elements as shown in Fig. 2. The structure has as many degrees-of-freedom as it has stories. The mass and stiffness matrices are first determined, then empirical techniques such as Biggs' method(2) are used to generate a damping matrix for the building based on assumed critical damping ratios.

2.3 Soil-structure interaction model

During an earthquake, the response of a building is greatly influenced by the flexibility of the soil foundation and its interaction with the structure. This interaction modifies the resonant frequencies as well as the amplitude of the structural vibrations. The soil-structure interaction impedances are represented by equivalent springs and dashpots to simulate the soil stiffness and damping. The virtual mass of the soil is calculated by a suitable formula and added to the mass of the base of the building. Generally the soil foundation characteristics are frequency dependent, however, for engineering applications these parameters can be treated as being frequency independent, Tsai(9). Only the translation and rotation (rocking) motions of the

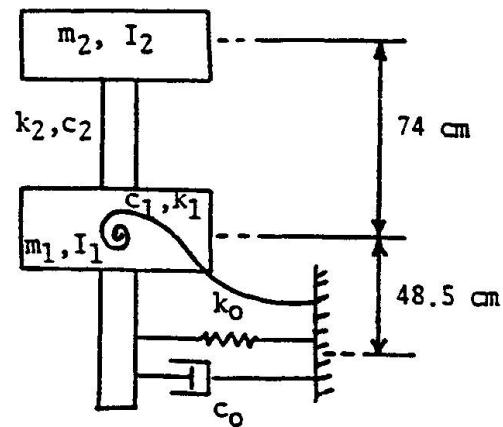


Fig.1 Three-degree model of standing man in the fore and aft mode

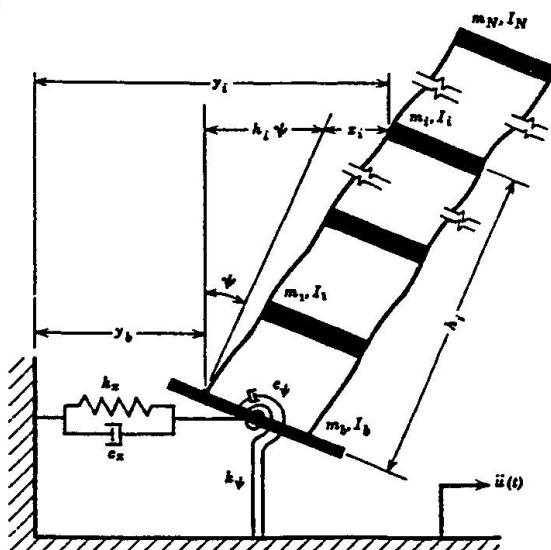


Fig.2 Configuration of flexible-base structure for horizontal seismic excitation (after Tsai)

foundation are considered in this work. This results in a soil-structure system with 2DOF greater than the number of DOF of the structure by itself. The values of the frequency-independent parameters are computed based on formulae developed with the soil being treated as an elastic half-space, Clough and Penzien(5). The soil properties involved in these calculations are the shear velocity, mass density of the soil, and the radius of the rigid disk representing the foundation half-space. The latter is usually taken as half the width of the building.

2.4 Seismic excitation model

Earthquake ground motions are usually treated as stochastic processes. While the simplest model represents ground accelerations as a white noise process with a constant power spectral density, accelerograms from earthquakes indicate that the spectral amplitudes of seismic energy are frequency dependent. Commonly used models of acceleration power spectra are those due to Kanai-Tajimi and Clough-Penzien.

The power spectra used in this study were obtained by fitting, in a least squares sense, available power spectra from et al) the 1985 Mexico earthquake to rational functions which are capable of capturing three peaks in the power spectrum function as shown in Fig. 3.

3. Equations of motion

The effect of the soil-structure interaction on the building is to change its structural response. This results from the incorporation of two additional degrees-of-freedom, horizontal translation and rotation, representing the soil foundation. This in turn increases the number of frequencies by two. Thus, there are $N+2$ equations of motion describing the soil-structure system response, where N refers to the number of stories in the building. The resulting equations of motion, based on Fig. 2, can be written in the following partitioned matrix form, Tsai(9):

$$\left[\begin{array}{c|c} [M] & [0] \\ \hline [0] & [m] \end{array} \right] \{ \ddot{v} \} + \left[\begin{array}{c|c} [C] & [C_1] \\ \hline [C_2] & [C_3] \end{array} \right] \{ \ddot{v} \} + \left[\begin{array}{c|c} [k] & [k_1] \\ \hline [k_2] & [k_3] \end{array} \right] \{ v \} = - \ddot{u}(t) \left\{ \begin{array}{c} \{ M \} \\ \frac{m_b}{m_b} \\ 0 \end{array} \right\} \quad (1)$$

where $[m] = \begin{bmatrix} m_b & 0 \\ 0 & I_s \end{bmatrix}$ with m_b being the mass of the base and I_s representing the sum of the mass moments of inertia of the structure and the foundation; $[M]$, $[C]$, and $[K]$ refer to the mass, damping, and stiffness matrices of the structure respectively, and are of size $N \times N$; $[C_1]$, $[C_2]$, $[C_3]$ and $[K_1]$, $[K_2]$, $[K_3]$ are damping and stiffness matrices that couple the structure and the

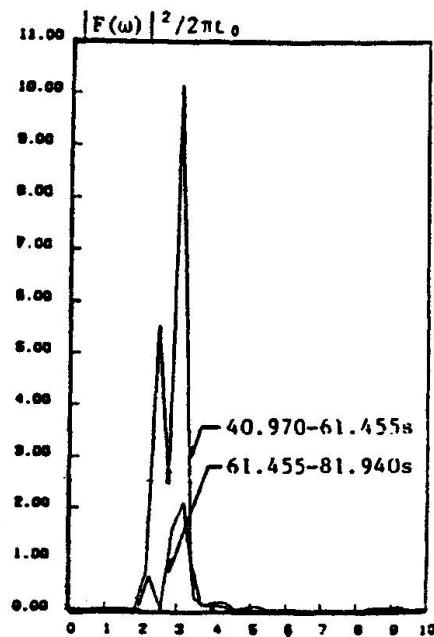


Fig. 3 Acceleration power spectrum of SCT-EW 1985 Mexico earthquake (after Grigoriu et al)



flexible foundation; $\ddot{u}(t)$ is the seismic acceleration and $\{v\} = \begin{Bmatrix} \{y\} \\ \frac{y_b}{\psi} \end{Bmatrix}$.

4. Calculation of absorbed power

Absorbed power is calculated by considering the velocity and the force of interaction, between the human and the floor, at the point of contact. The force and velocity can be related to the displacement of the point of contact in the frequency domain, thus relating the biomechanical model response to that of the floor. The floor response power spectrum, $S_f(\omega)$, can be related to the seismic power spectrum, $S_a(\omega)$, by:

$$S_f(\omega) = |H(i\omega)|^2 S_a(\omega) \quad (2)$$

where $H(i\omega)$ is the complex frequency response function of the floor level where the biomechanical model is located. It can be shown, Farah(7) that the absorbed power, P , through the biomechanical model can be evaluated from the integral:

$$P = -\frac{1}{\pi} \int_0^{\infty} \omega \operatorname{Im}[G(i\omega)] S_f(\omega) d\omega \quad (3)$$

where $G(i\omega)$ is the complex frequency function between the force and displacement at the point of contact between the floor and the biomechanical model. The determination of $H(i\omega)$ requires that Eq. 1 be transformed to the frequency domain and the resulting matrix is inverted in closed form and $H(i\omega)$, for the various floor levels, is then obtained from the inverse matrix. The efficient inversion technique was developed by the author based on the Fadeev-Leverrier method. Note that the normal mode decomposition method cannot be used directly in this case due to the coupling terms in the damping matrix resulting from the soil-structure interaction.

4. Results and discussion

To illustrate the procedure described above, absorbed power values were calculated for the floors of an 8-story building and for various critical damping ratios. The power spectra of the seismic excitation were those of the Sept. 1985 Mexico earthquake as given by Grigoriu et al(8). The response of these buildings was evaluated for three site shear velocities 50m/s, 75m/s, and 400m/s. The shear velocity impacts the values of the damping and stiffness elements of the soil foundation. Note that both the damping and stiffness are proportional to the shear velocity and its square respectively, and that generally, the shear velocity is lower in softer soils. Fig. 4 shows that the response corresponding to the 50m/s is higher than that for the 75m/s velocity, and that the absorbed power for a given floor is larger than that corresponding to a lower floor as would be expected since higher values of absorbed power are associated with a lower comfort

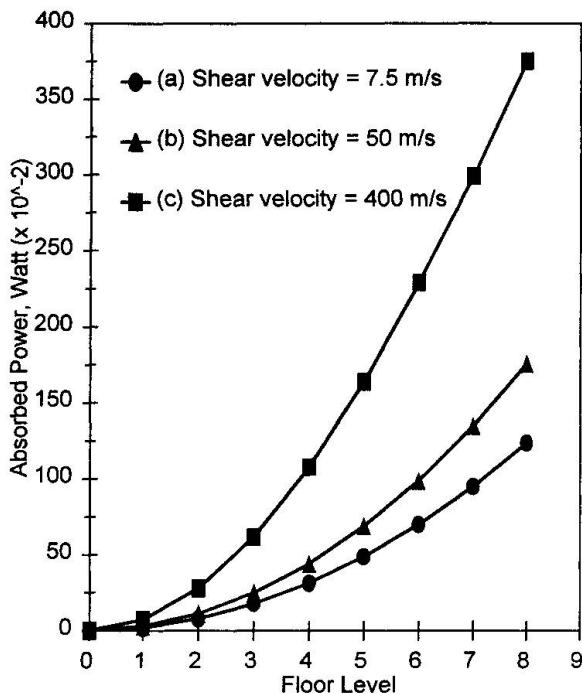


Fig. 4 Absorbed power for an 8-story building with SCT power spectrum

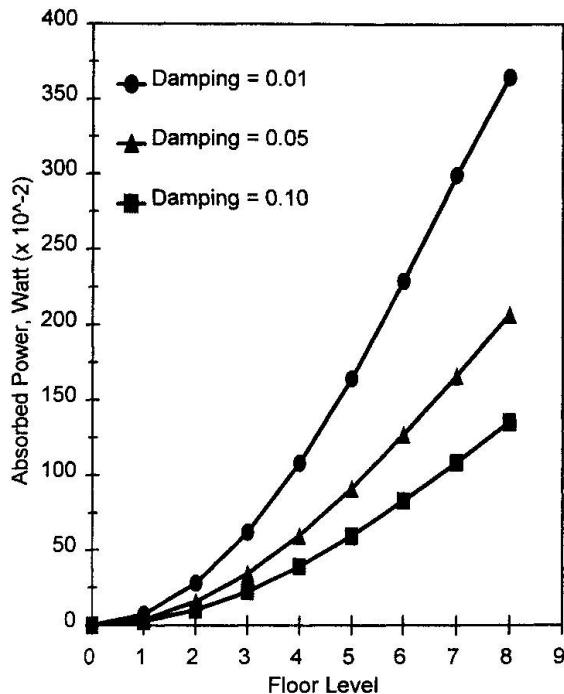


Fig. 5 Effect of damping ratios in structure on absorbed power (shear velocity = 400 m/s)

level. In addition, the response of all floors and for all shear velocities is greater than the level required for the threshold of annoyance which is equal to 8×10^{-4} W. However, the absorbed power values for floors 6, 7 and 8 for the 400m/s shear velocity are even larger than the value of 1.7 W which corresponds to the threshold of intolerance. Such high values of absorbed power are indicative of an unacceptably severe building response such as that which occurred during the Mexico earthquake. It is also important to emphasize that for a large building response, the building behavior will not remain elastic and thus the absorbed power values would be different from those given above. The high absorbed power values for the 400m/s shear velocity were produced due to the introduction of frequencies in the soil-structure system which are very close to those of the seismic power spectrum and the first frequency of the biomechanical model. This situation arose because the fundamental frequency of the soil-structure system is 3.2r/s, that of the biomechanical model is 3.64r/s, and the dominant frequency of the earthquake power spectrum is 3.1r/s. In effect a quasi-resonance behavior was setup with the resulting large response.

The effect of the damping in the structure is very significant when the system is in a resonant vibration state. Fig. 5 shows the absorbed power in the building corresponding to a shear velocity of 400m/s. It is seen that the absorbed power on the eighth floor for a damping ratio of 0.01 results in a value of 3.75 W and the corresponding values for damping ratios of 0.05 and 0.10 are 2.07 W and 1.35 W respectively. However based on other results obtained in this study, the impact of damping in the structure is not as significant if resonant or quasi-resonant conditions are not generated.



5. Conclusions and recommendation

It has been shown that absorbed power can be a good indicator of the level of human response to building vibration subjected to seismic excitations, and has the capacity to distinguish human comfort at different floor levels. The damping in the structure is mainly important in reducing the response in a resonant or quasi-resonant vibrational environment. The properties of the soil foundation greatly influences the building behavior, with softer soils, associated with lower seismic shear velocities, generally producing higher absorbed power values except for situations where higher shear velocities generate resonant conditions. The ability to estimate the absorbed power should be invaluable to the engineer in achieving a serviceable building design.

Although this study has demonstrated the potential of absorbed power as a criterion for assessing buildings serviceability to earthquakes, further research is recommended in the following areas:

1. Development of a human model suitable for determining absorbed power under simultaneous horizontal and vertical vibration.
2. Calibration of absorbed power to various earthquake magnitudes and frequency characteristics associated with various site conditions.
3. Determination of the impact of nonlinear behavior of structures on absorbed power and therefore its serviceability.
4. Assessment of the effect of external damping devices such as viscoelastic dampers on the serviceability and safety of structures.

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Zarate - Brazo Largo Bridges: Stay Cable Damages and Rehabilitation

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Summary

The paper describes the reasons and basis for a structure related design basis for rehabilitation which is using the modern load resistance factor design. This design basis is formulated such that it depends on all available information regarding the loading conditions, results from material tests, observations of the degree of deterioration and an in-depth knowledge of the structure in question. The main results of a sensitivity analysis for the roadway load model are presented and the assessment of the stay cable capacity explained.

1 Introduction

The Zárate - Brazo Largo bridges are part of a roadway and railway infrastructure project in Argentina providing a crossing of the national road No 12 over the two main branches of the Paraná river - Paraná de Las Palmas and Paraná Guazú - northwest of the river delta close to the town of Zárate. The two bridges comprise cable stayed steel girder main bridges with spans of 110-330-110 m and a total of about 16 km concrete approach viaducts for railway and roadway traffic.

The bridges were constructed during the years 1972-1977, based on an alternative bid design submitted by the Joint Venture Techint - Albano, and opened to roadway traffic in 1977 and railway traffic in 1978 - the Guazú bridge is shown in fig 1.

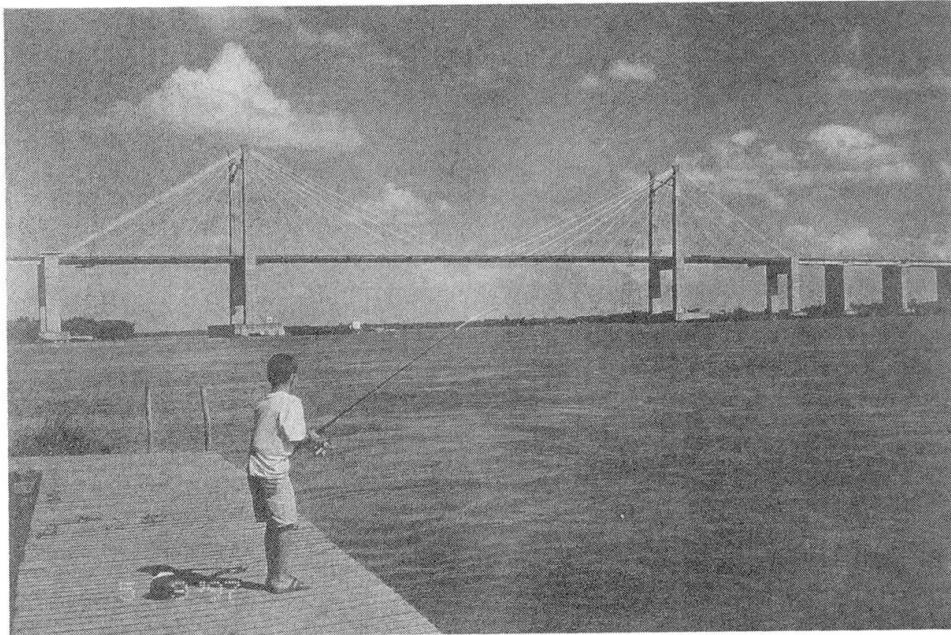


Fig. 1: *Photograph of the Guazú bridge*

Since the opening of the bridges the traffic volume and traffic pattern has changed. Due to the continuous development of the mercantile requirements for transportation as well as the use of new and heavier trucks. Since Argentina entered into the Common Market of South America, the MERCOSUR, late in 1991, the national road No 12 constitutes an important part of the infrastructure on an inter-American scale and its condition is a matter of highest importance.

Stay cable 7C of the bridge across Paraná Guazú ruptured on 20 November 1996. The stay cable failed close to the bottom socket at the deck level. The cable was removed from the top anchorage and put on the bridge deck. It showed that almost all of the 121 wires \varnothing 7 mm failed at about 200 mm outside of the bottom socket with severe signs of corrosion and fatigue-like ruptures in the wires.

As an emergency action, the bridges were closed to traffic on 25 November 1996, and the consortium Albano - DyCASA - Freyssinet (ADF) was entrusted by Dirección Nacional de Vialidad, DNV, the national highway authority of Argentina, with the immediate replacement of stay 7C, using the Freyssinet mono strand stay type.

2 Bridge Assessment and Rehabilitation Approach

The design and assessment of ordinary structures under normal conditions is appropriately accommodated by codes of practice and regulations. This is because the codes and regulations have been formulated and calibrated specifically to ensure that the most commonly built structures under normal conditions are both economic and sufficiently safe.

For the design and assessment of structures which are unique, e.g. by proportion, concept, material or condition, codes and regulations cannot be expected to yield structures which are appropriate in terms of economy and safety. It is therefore common practice for such

structures to formulate and calibrate a design basis which is specific. Such a design basis can be understood as a specific code of practice for the design and assessment of that structure.

2.1 The rehabilitation design basis

The Zárate-Brazo Largo bridges are unique what concerns the condition of the deteriorated stays. The existing codes and regulations are thus not adequate for use as basis for a safety assessment and for rehabilitation design. Therefore, in order to ensure that the bridges are efficiently rehabilitated to a condition where they maybe accepted for safe continued use, and possibly even upgraded in terms of traffic loading, it is necessary to formulate and calibrate a rehabilitation basis specific for this bridge type, its observed state and the desired use of the bridges in the future.

To ensure the compatibility between this specific rehabilitation design basis and generally accepted design basis, such as the Eurocodes, the ISO codes and the AASHTO codes, the safety format for the rehabilitation design basis is the well-known load and resistance factor design (LRFD).

In general, the basis for the calibration of the safety factors and load combination factors is the application of modern reliability methods in accordance with the principles described in the background documents for the Eurocodes and the ISO codes, see [1].

2.2 Updating of the rehabilitation design basis

The rehabilitation design basis is formulated such that it can be modified for changes in the

- assumptions regarding past and future road and rail traffic
- results from material tests from dismantled stays
- measurements of stay forces
- inspections of damaged wires strengthening and replacement of stays.

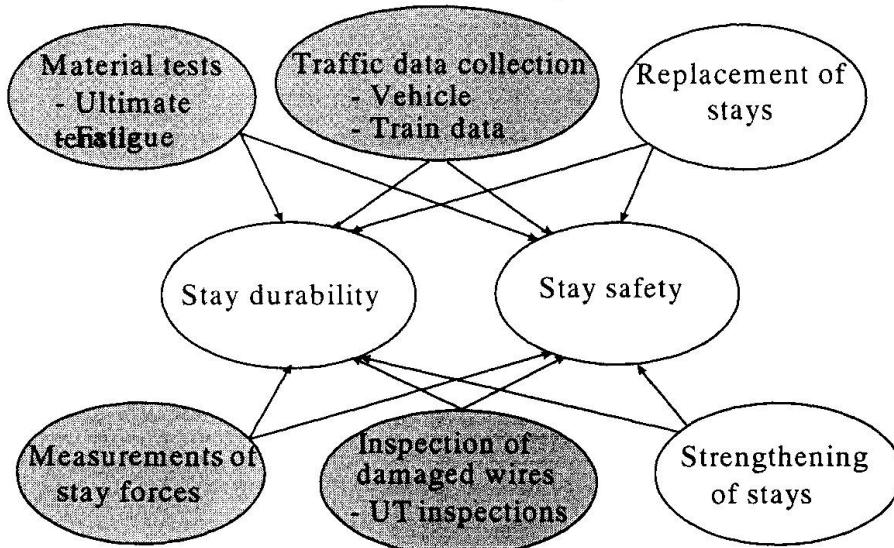


Fig. 2

Illustration of the rehabilitation actions considered in establishing the rehabilitation design basis for the Zárate-Brazo Largo bridges.



In fig 2 the rehabilitation actions considered for the rehabilitation of the Zárate-Brazo Largo bridges are illustrated. The grey toned "knowledge collecting" actions are those which continue changing during the course of the assessment and for which the rehabilitation design basis has been especially designed to accommodate.

The design basis for the rehabilitation of the Zárate-Brazo Largo Bridges presented in the following summarises the relevant information on loads and materials for the different stages of the rehabilitation of the bridges.

3 Roadway and Railway Loads

Probabilistic models have been formulated for the roadway and the railway loads acting on the bridges. With regard to the roadway load model basis has been taken in the theoretical framework formulated for the development of the design basis for the Great Belt East Bridge, see [2]. The railway load model has been formulated on basis of the model developed for the design of the Great Belt West bridge [3].

The roadway load model has been calibrated to the extend possible on the basis of traffic observations from the Zárate-Brazo Largo bridges. Some experience for European traffic has been used as a supplement when necessary.

The railway load model has been based on information regarding the actual traffic and traffic restrictions made available by DNV.

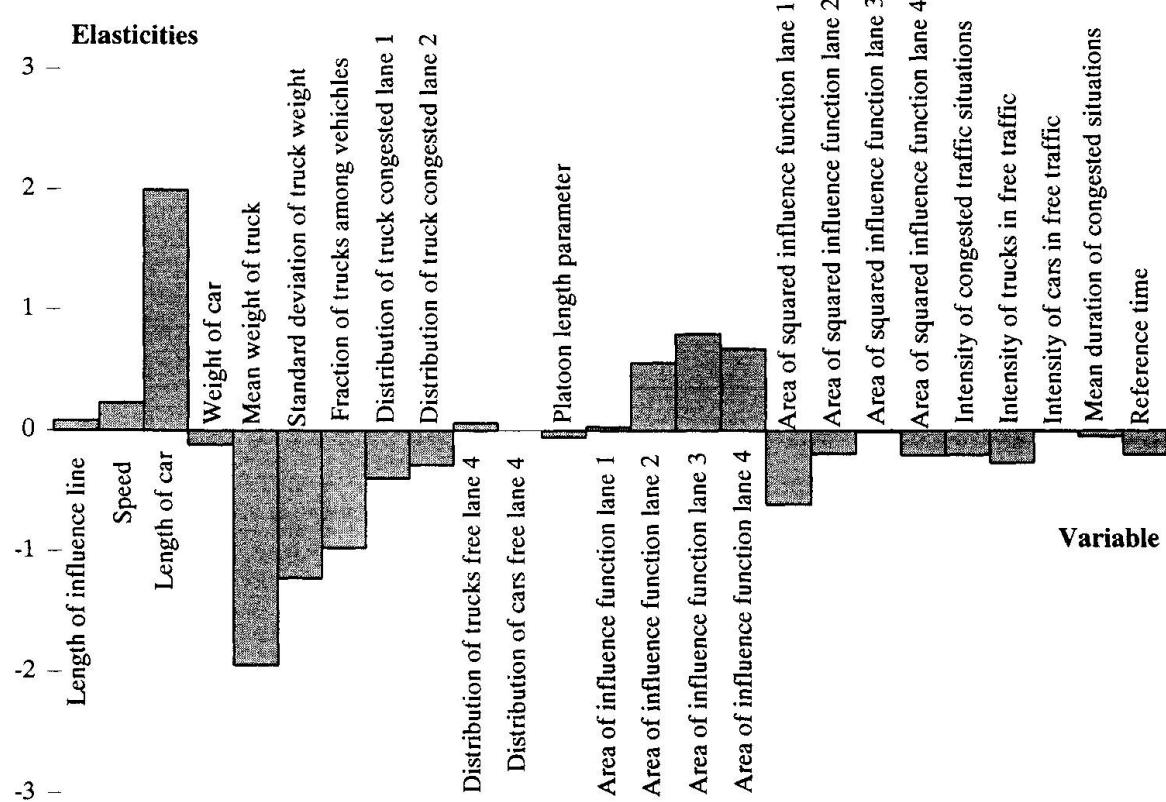


Fig. 3 *Sensitivity analysis of the roadway load model.*

Using the probabilistic load models together with influence lines for a number of selected load effects (including stay forces, road girder load effect and pylon load effect), the statistical distribution of the maximum load effect during one year has been determined. By defining the characteristic load as the 98 percentile of this distribution, an equivalent uniformly distributed load (EUDL) has been derived which yields the same load effect as the real load.

The results of a sensitivity analysis regarding the influence of the different input to the roadway load model is shown in fig 3.

Taking basis in the same requirements to safety as have been suggested in the back ground documents to both the Eurocodes and the ISO codes, the safety factors for the permanent and the variable loading together with the combination factors for vehicle and train loads have been derived on the basis of probabilistic principles.

4 Condition Assessment

To estimate the strength of a parallel wire cable it is necessary to determine the number of broken wires in the cable. The number of broken wires can be estimated by the use of UT-inspection. However, the UT-inspection will not be able to detect all broken wires with probability one. Furthermore, the inspection may indicate that intact wires are broken. To estimate the reliability of the inspection method it is necessary to determine the probability of detecting a wire which is broken and the probability of detecting that an intact wire is broken.

5 Assessment of Stay Cable Capacity

The resistance safety factors for the stay cables have been derived with basis in a probabilistic model of the time varying strength of parallel wire cables subject to fatigue deterioration. The model incorporates all available information regarding the material characteristics of the wires as obtained through testing under ultimate tensile as well as fatigue loading. The ultrasonic testing, together with the limited testing of cable 7C and two further cables, constitute the most essential data input to the current safety evaluation.

The model for the safety factor for the stay strength allows for a differentiation of safety factor in accordance with the intended service life of the considered stay. Furthermore the safety factors are given for the stays individually, taking specifically into account the damage condition and the loading on the individual stays.

Using the model, it is possible to derive safety factors corresponding to an interim period of the bridge (rehabilitation period) which is shorter compared to the situation where a normal service life is considered. Furthermore, the model also yields safety factors to be used in the design of stay cable replacements.

The computation of the residual life is based on a method proposed in [4]. The first step in the computation of the residual life is the estimation of the life distribution of a single wire. The number of cycles, t , to failure of a given wire subject to a stress range, Δs , is given by a Weibull distribution,

$$F_T(t, \Delta s) = 1 - \exp\left(-\left[\frac{\Delta s}{r_c}\right]^\alpha \left[\frac{t}{K}\right]^{\frac{\alpha}{m}}\right), \quad \text{where } \alpha, m, K,$$

and c are unknown parameters, n is the number of sections of the test specimen with different properties and A_0 is the cross sectional area of the wire. The parameters m and K are also the parameters in the SN-curve which is given by $t = K\Delta s^{-m}$.

A large parallel wire cable with an infinite number of wires is considered. It can be shown that even though the number of wires is assumed to be infinite the results are in general valid for cables with a finite number of wires. The failure times of the intact wires given a certain level of mean stress and stress range are assumed to be identically and independently distributed. Also, the initial load on the cable is small enough to cause no static wire failure.

During service one wire breaks after the other due to accumulated fatigue damage. The wire with smallest failure time fails first in each state of the system. There is an immediate load redistribution after failure of a wire without dynamic effects during stress redistribution. The residual static strength of the individual wires is not influenced by the fatigue damage which might be accumulated. Finally, it is assumed that there is no dependency between the static strength of a given wire and the fatigue resistance of the wire.

The effect of corrosion is taken into account by determining a set of material parameters specifically for corroded wires. Further, corrosion implies that the length of wire where the material parameters can be assumed to be constant becomes small. Therefore, the parameter r_c which depends on n (the number of parts with different material parameters) depends on whether the wire is corroded.

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Suitability of Recycled Aggregate Concrete in High-Rise Construction

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Summary

This paper presents the results of an experimental investigation carried out on the suitability of recycled aggregate concrete for the construction of high rise structures. The strength and workability of conventional natural aggregate concrete obtained from high grade cements (43 grade & 53 grade) is compared with recycled aggregate concrete obtained from high grade cements. Also, it is shown that, the shortcomings of recycled aggregate concrete in strength and workability can be overcome by the use of superplasticizers.

1. Experimental Work

In this experimental programme the strength parameters and workability of three different concretes (with different w/c ratios to cater the needs of workability) were compared. Viz -

- a. concrete with conventional natural aggregates (w/c = 0.45)
- b. concrete with recycled aggregates (w/c = 0.55)
- c. concrete with recycled aggregates and known dosage of superplasticizers (w/c = 0.45)

The tests were conducted on a mix of proportion 1:2:4. Zentrament Super BV superplasticizer (lignosulphonate based) was used with a dosage of 0.7%.

2. Experimental Results

Table 1 gives the strength parameters (compressive strength, tensile strength and flexural strength) of different categories of concretes produced from 43 grade & 53 grade cements.

Table 2 gives the workability of different categories of concretes produced from 43 grade & 53 grade cements. The workability of 43 grade & 53 grade concretes are almost same.



Table - 1 Results of strength parameters.

Particulars of concrete	Conventional natural aggregate concrete produced from		Recycled aggregate concrete produced from		Recycled aggregate concrete with superplasticizer produced from	
	43 grade	53 grade	43 grade	53 grade	43 grade	53 grade
Comp. strength (MPa)						
7 days	16.92	19.93	12.00	13.64	22.71	24.00
28 days	18.92	22.66	16.44	18.66	24.79	32.88
2 months	29.77	35.50	28.53	32.44	32.08	36.79
Tensile strength (MPa)						
7 days	1.95	3.27	1.79	2.19	2.40	3.96
28 days	2.70	3.82	2.06	3.13	3.97	4.52
2 months	4.38	4.78	4.07	3.90	5.45	5.27
Flexural strength (MPa)						
7 days	4.64	5.36	2.96	3.76	5.04	5.96
28 days	5.40	5.80	5.04	4.04	7.00	7.44
2 months	5.84	8.52	5.24	7.36	8.92	9.36

Table 2 - Results of Workability

Particulars of Concrete	w/c ratio	Slump (mm)	C.F.	V.B. Degree (sec)	% Flow
Conventional natural aggregate concrete	0.45	0	0.87	20	4
	0.60	10	0.90	12	13
	0.70	15	0.92	6	20
	0.80	20	0.94	4	60
	0.90	Collapse	0.97	2	80
Recycled aggregate concrete	0.45	0	0.84	25	2
	0.60	0	0.86	14	9
	0.70	12	0.90	8	15
	0.80	18	0.92	7	52
	0.90	Collapse	0.95	3	71
Recycled aggregate concrete with superplasticizer	0.45	0	0.89	18	5
	0.60	8	0.93	10	14
	0.70	14	0.94	5	21
	0.80	Collapse	0.96	4	60
	0.90	Collapse	0.98	2	80

3. Conclusions

The following conclusions can be drawn -

- The compressive strength, tensile strength and flexural strength of conventional natural aggregate concrete is more than that of recycled aggregate concrete. But the strength of recycled aggregate concrete with superplasticizer show a distinct rise in the strength compared to the conventional natural aggregate concrete. Thus the offset in the strength properties of recycled aggregate concrete can be overcome by using a right dosage of superplasticizer.
- The strength properties are distinctly greater for concrete produced from 53 grade cement as compared to concrete produced from 43 grade cement.
- The workability is distinctly more for recycled aggregate concrete with superplasticizer.
- Hence recycled aggregate concrete can be effectively used in the construction of high rise buildings, especially near the vicinity where recycled aggregates are available in plenty.

Reconstruction of a Department Store Damaged in the 1995 Earthquake

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Summary

Daimaru-Kobe's Main Building had been expanded in 4-phases since 1925, and had 7 stories above ground, 1 basement (partially 2), and the total floor area of about 34,000 m². The Main Building along with its adjoining West and South Annex buildings, and the 38-ban Annex were loved by many Kobe citizens as a symbol for Kobe-Motomachi. However, the Southern Hyogo Earthquake occurred on January 17, 1995 and destroyed most of its above ground levels, and a fast restoration was keenly desired.

In spite of a reconstruction plan for the entire building, we decided to make use of most of the substructure portion, which did not suffer major damages, for restoration with shorter construction period and more economical construction cost.

1. Basic Principles of the Restoration Plan

We established the following principles, hoping this project to lead the restoration plan for the energetic and historical Motomachi streets.

1. Make a comeback by shortest possible construction period.
2. Try to work out with the least possible construction cost.
3. To seek an ideal department store prospecting future needs and necessities.
4. To restore streets integrated with historical architectures.

2. Structural Solutions

For the above 1 & 2, reduce the costly and time-consuming underground work by exploiting the external wall and foundation of the Main Building's underground portion that were luckily undamaged.

The aboveground new building was designed to be of rigid steel frame to enjoy its merit, i.e. shorter construction period. The steel frame was rooted to the underground by erecting major columns from B1 floor to ensure earthquake resisting performance of the entire building. Horizontal force will be resisted aboveground by the rigid steel frame and underground by giving sufficient net effective rigidity and strength to the 1F floor slab so that the most part of the shear force be transferred to the underground exterior walls and shear walls. What is intended here is to reduce the stress transferred from the steel column legs to the old columns and beams by decreasing the burden, upon the B1 structure, of the horizontal force from the rigid steel frame.

For the above 3., spans were reviewed. The span is to be about the double length of the existing 6- to 7-meter(approx.) spans, so that square-sales space with visibility can be created.

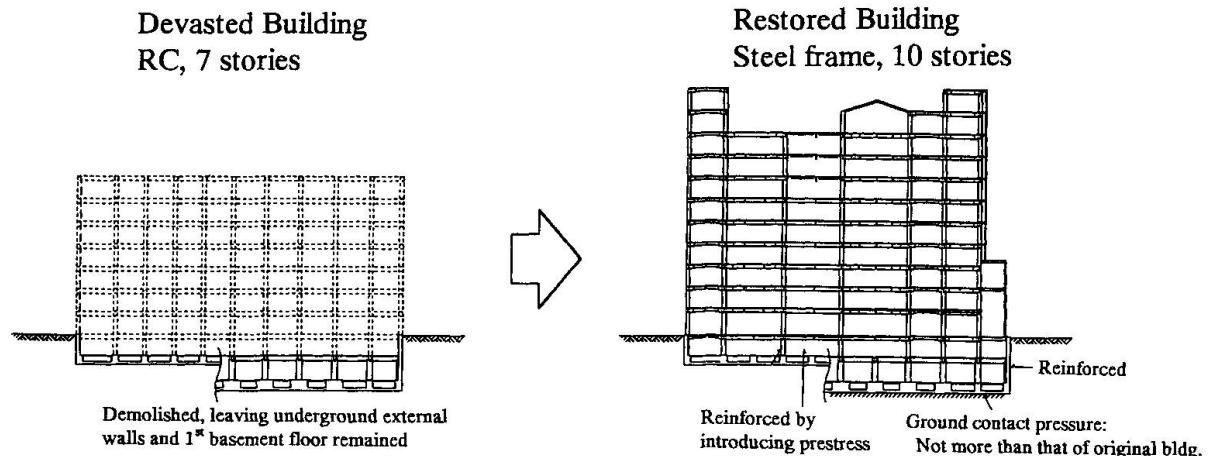
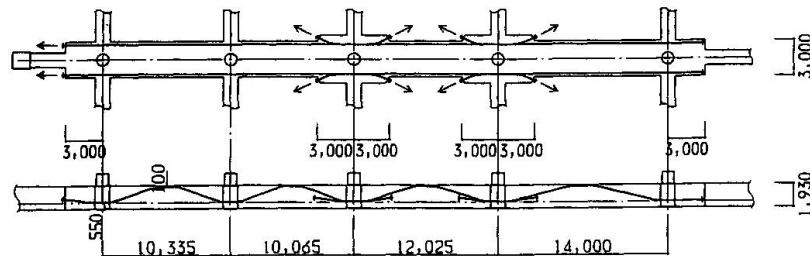
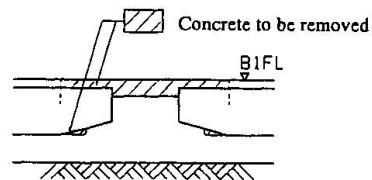


Fig.1 Outline of Restoration Plan

With the enlargement of the span of the aboveground structure, the span of the foundation beams (spread foundations) had to be doubled to about 12 m. This involved the great increase of stress due to subgrade reaction. Further, it was difficult to reinforce lower sides of the foundation beams, which are in tensile zone at their ends. For this, the width of the foundation beams were increased and upper reinforcing bars were sufficiently installed to ensure ultimate strength, and then prestressing was introduced to prevent its cracking and ensure durability.



Present



Arrang bars at the side of foundation beam and on upper surface of the slab and place concrete.

After reinforcing

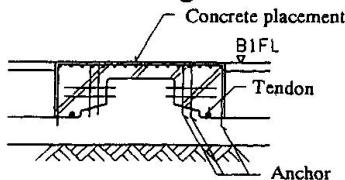


Fig.2 Reinforcement of Foundation Beam by Using Tendons

For the above 4., align the lower levels to the adjoining historical building's height, and set back the higher levels by about 5m. The purposes of these are to pay attention the continuity of the old streetscape skyline.

Setting back the building without decreasing necessary floor area requires increasing the number of floors. The new building has ten aboveground floors, exceeding the original number of seven, with the maximum subgrade reaction being about the same as of the original building by adopting steel frame structural system with light-weight-concrete of the original building.

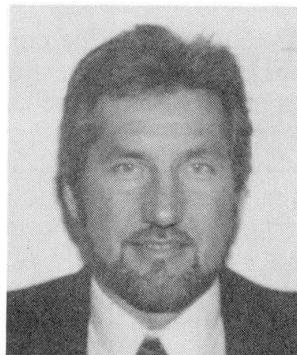
3. Conclusion

The above mentioned solution on structural issues enabled us to realize the principles for the restoration plan, and to complete the construction works with shorter period of 6 to 8 months and construction cost reduction of about 12%, comparing to renovating the entire structure.

Daimaru-Kobe store celebrated its rebirth-grand opening on March 2, 1997, and has been receiving many customers along with a good reputation that it has been a torch for Kobe's comeback.

Coating Technology for Maintenance and Architectural Applications

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Summary

Moisture-cured polyurethane coatings are widely used in maintenance and new construction applications because they offer durable protection. When compared with other coatings, polyurethane systems rank among the lowest in life cycle costs. Polyurethane coatings also provide chemical and abrasion resistance, low-temperature curing and recoatability and surface tolerance application. Of added significance, the solvent based formulations are commercially available to meet VOC emission limits of 2.8 lbs./gal.

Moisture-Cured Polyurethane Technology

Polyurethane moisture-cured coatings are classified as Type II urethanes under the ASTM D-16 Specification. They are single package compositions that cure by the reaction of residual isocyanate groups with atmospheric moisture. The reaction of these products with atmospheric water in the field is a two stage process. The water and isocyanate groups first produce the unstable carbamic acid, which immediately dissociates to form an amine and carbon dioxide. The carbon dioxide evaporates from the film, and the amine reacts with a second isocyanate group, yielding a polyurea. In a two-component polyurethane coating, the polyisocyanate reacts with a resin having a reactive hydrogen, which can be acrylic, polyester or polyether. The paint film resulting from this reaction is the foundation of the performance properties of polyurethanes: durability, corrosion and chemical resistance, and color and gloss retention. Moisture-cured polyurethane coatings are increasing in popularity due to the wide range of application advantages that some manufacturers are claiming:

- Can be applied on cold damp surfaces
- At temperatures below freezing
- No dew point restriction
- Adhesion to bare metal, marginally prepared surfaces and existing coatings
- Year round application season
- Excellent recoatability
- Single component - no mixing/measuring
- Excellent chemical resistance

Moisture-cured polyurethane coatings are applied independent of weather (temperature, humidity and dew point), are quick drying and surface tolerant. Moisture cured surface tolerant polyurethanes can be applied as a two coat or a three coat system using various preparation techniques. Zinc rich primers are preferred when blast cleaning is used to prepare the surface, varying from SSPC-SP 6 "Commercial Grade" to SSPC-SP 10 "Near White," depending upon service conditions. In addition, moisture cured polyurethane based primers perform well even when using lower level of zinc or zinc free



pigmentation. Coatings containing micaceous iron oxide provide excellent adhesion and durability. When overcoating tightly adhered rust remaining from hand/ power tool cleaning, it may be desirable to treat the rusted surface with a clear moisture-cured polyurethane penetrating sealer to help seal and reinforce the substrate surface.

Pigments commonly used in moisture-cured urethanes are titanium dioxide, mica, talc and micaceous iron oxide (natural or synthetic). During application of moisture-cured urethane primers and intermediate coats pigmented with micaceous iron oxide, mica and talcs, platelets align themselves more or less parallel to the substrate in closely packed layers. After the coating has cured, the overlapping pigment particles form a dense barrier against moisture and other corrosion promoters. The pigment layers also reinforce the coating, relieve stresses within the paint film and improve intercoat adhesion.

New Construction, Maintenance, and Overcoat Painting

Shop Painting: Moisture-cured polyurethane zinc rich primers are growing in use for fabrication shops due to their greater tolerance for higher film builds without mud cracking, easy touch-up in the field and they can now pass the Class B slip coefficient rating for fayed surfaces. **Maintenance Painting:** Both moisture-cured zinc rich primer and micaceous iron oxide and aluminum pigmented primer systems are utilized for blast cleaned surfaces. **Overcoat Painting:** Moisture-cured polyurethane coatings are very popular for overcoat application because of their excellent adhesion to marginally prepared surfaces, rust and existing coats. They are very tolerant of job side conditions that include low temperature variation and surface moisture.

APPLIED OVER NEAR WHITE BLASTED STEEL (SSPC-SP 10)		
TEST	Zinc Rich Moisture Cured Urethane Primer/MIOX Filled Intermediate Coat/Moisture Cured Urethane Aliphatic Topcoat	Moisture Cured Urethane Primer with MIOX and Aluminum/MIOX Filled Intermediate Coat/Moisture Cured Urethane Aliphatic Topcoat
Salt Spray ASTM B-117 (8000 hours)	Moderate rust and 1/8" creepage and few medium blisters at score line	Moderate rust, scattered medium blisters and 1/8" creepage at score line
Prohesion (8000 hours) 1 hour wet, 1 hour dry	Moderate rust, medium dense blisters and 1/8" creepage at score line	Moderate rust, dense medium blisters and 1/8" creepage at score line
Humidity Chamber (7000 hours)	No Effect	No Effect
Chemical Resistance (250 hours) 37% HCl 50% H ₂ SO ₄ 10% CH ₃ COOH 50% NaOH	No Effect No Effect No Effect No Effect	No Effect No Effect No Effect No Effect
APPLIED OVER MARGINALLY PREPARED STEEL		
TEST	Moisture Cured Urethane Primer with MIOX and Aluminum/MIOX Filled Intermediate Coat/Moisture Cured Urethane Aliphatic Topcoat	
Salt Spray ASTM B-117 (6000 hours)	Moderate rust, dense medium blister formation around score line and 1/8" creepage at score line	
Prohesion (6000 hours) 1 hour wet, 1 hour dry	Heavy rust, medium to large dense blister formation around score line	
Chemical Resistance (250 hours)	Same as above	
APPLIED OVER MARGINALLY PREPARED STEEL		
TEST	Moisture Cured Urethane Penetrating Sealer/MIOX Filled Intermediate Coat/Moisture cured Urethane Aliphatic Topcoat	
Salt Spray ASTM B-117 (2800 hours)	Slight rust along score line, 1/16" creepage at score line	
Prohesion (1500 hours) 1 hour wet, 1 hour dry	Slight rust and 1/16" creepage at score line	

Figure 1. Performance Tests for Moisture Cured Polyurethane Systems