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Autor: Matsuo, Akira / Nakamura, Yuji / Salib, Rafeek W.
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Seismic Behavior in Steel Structures of Weak Connections

Comportement sismique de structures présentant des assemblages faibles

Erdbebenverhalten von Stahlbauten mit schwachen Verbindungen

Akira MATSUO
Dr. Eng.
Hiroshima University
Higashi-Hiroshima, Japan

Akira Matsuo, born in 1947, received his Ph.D. at Hiroshima University, Japan, in 1985. He has been involved in design and research of steel structures for 25 years.

Yuji NAKAMURA
Dr. Eng.
Hiroshima University
Higashi-Hiroshima, Japan

Yuji Nakamura, born in 1936, got his Ph.D. at MIT 1968. He worked on the design of steel structures at NKK for 30 years. Now at Hiroshima University, he does a variety of research on steel structures.

Rafeek W. SALIB
Dr. Eng.
Suez Canal University
Port Said, Egypt

Rafeek W.Salib, born in 1958, received his Dr. Ing. at Hiroshima Univ., Japan, 1993. He has been involved in steel structures research work since 1983. Now he is a staff member in Suez Canal Univ., Port Said, Egypt.

SUMMARY

Simple formulation of panel moment-shear deformation for steel beam-to-column connections is presented. Dynamic analyses on low-rise steel frames of the weak column-strong beam are made changing the shear strength of the connection panels. According to the numerical results for the recorded earthquakes, as the shear strength of the connection panel decreases, the plastic deformation of each column decreases and may reach a level of less than half the strong panel cases.

RÉSUMÉ

Une formule simple est, proposée pour exprimer la déformation due au cisaillement dans des assemblages poutres-colonnes. Une analyse dynamique de cadres métalliques avec des colonnes faibles et des poutres fortes est effectuée en faisant varier la résistance au cisaillement des parois. Selon les résultats numériques enregistrés lors de séismes, lorsque la résistance au cisaillement de la paroi décroît, la déformation plastique de chaque colonne décroît et devient inférieure à la moitié de celle constatée pour des parois fortes.

ZUSAMMENFASSUNG

Der Bericht präsentiert zunächst eine einfache Formulierung der Beziehungen von Fachwerkfeldmoment und Scherungsdeformation bei Stahlträgern / Säulenverbindungen. Anschließend wird eine dynamische Analyse niedriger Stahlkonstruktionen mit schwachen Säulen-starken Trägerverbindungen durchgeführt. Die Ergebnisse für die verzeichneten seismischen Wellen legen nahe, dass die plastische Deformation der Säulen mit abnehmender Scherfestigkeit der Fachwerkfeld-Verbindungen zurückgeht.

1.INTRODUCTION

In the frame analyses deformation of the beam-to-column connection must be considered to know the exact behavior of the steel buildings. Many experimental results suggest that the strengths of the H-shaped beam-to-column connections are scattered in a wide range. Authers presented the formulations of the maximum strength for several types of connections through limit analysis[4]. In this paper these formulations are simplified for practical use. Then the formulation of the monotonic and cyclic curves of panel moment ${}_pM$ and shear deformation γ relation of the connection is presented. In Japan about 95% of steel buildings are low-rise. Based upon the Building Law in Japan most of them are designed according to the allowable stress design and the ultimate collapse form is not checked. In such a frame earthquake damage is sometimes concentrated at the weakest story. In this paper the dynamic analyses of 4 and 5-storied frames are made changing the shear strength of the connections to know the seismic performance against several recorded earthquakes.

2.RESTORING FORCE CHARACTERISTICS OF CONNECTIONS

2.1 The maximum strength

Authers presented the maximum strength of a standard type and a non-scallop type of an H-shaped steel beam-to-column connection for the interior column(X-type) and the exterior column(T-type)[5]. Eq.1 is the strength for the standard type.

$${}_pM_u=\tau_u V_p+2\pi\sigma_{ywb}t_{wb}a(a+b)/\sqrt{3}+4a(M_{pbf}+M_{pcf})/b+5.2(1+a/b)(M_{ubf}+M_{ucf}) \quad , \qquad (1)$$

where V_p, τ_u:the volume and the maximum shear stress of the panel plate, M_{pbf}, M_{pcf}, M_{ubf}, M_{ucf}:the plastic and ultimate bending moment of the beam and the column flange, σ_{ywb}, t_{wb}:yield stress and thickness of the beam web, a:the rigid length at the joint of the flanges and $b=\sqrt{2\sqrt{3}(M_{pbf}+M_{pcf}+1.3M_{ubf}+1.3M_{ucf})/\pi\sigma_{ywb}t_{wb}}$ according to the minimum condition. The first term shows the ultimate panel moment(shear strength) of the panel plate and the following terms the amount of the bending moment directly transmitted through the intersections of the beam and the column flange ${}_FM$, which looks as if the beam and the column reinforced the shear panel. This equation is applied to the 255 combinations of a practically used beam and column H-section(JIS SS400) and ${}_FM$ for each combination is plotted in Fig.1, where ${}_pM_y=\tau_y V_p$. ${}_FM/{}_pM_y$ mainly depends upon the geometrical conditions of subassemblages, as ${}_FM$ and ${}_pM_y$ contain the same material properties. Solid line shows the best fit curve Eq.2 in terms of panel yield ratio R_{py}.

$${}_FM/{}_pM_y=0.135/R_{py}+0.098, \qquad (2)$$

where $R_{py}={}_pM_y/\sum M_{my}$, $\sum M_{my}$: the total of the yield moment of the beams or columns jointed to the connection, whichever the smaller. A small difference of ${}_FM$ is found between a standard type and a non-scallop type and between X-type and T-type for the practical range(R_{py}>0.5) of T-type[5]. General expression for the maximum strength of the beam-to-column connections of various materials is given as follows using Eq.2.

$${}_pM_u=\tau_u V_p+{}_FM$$
$$=(1/\alpha+0.135/R_{py}+0.098){}_pM_y, \qquad (3)$$

where α: yield ratio of steel. Figs.2 show two groups of experimental and predicted maximum

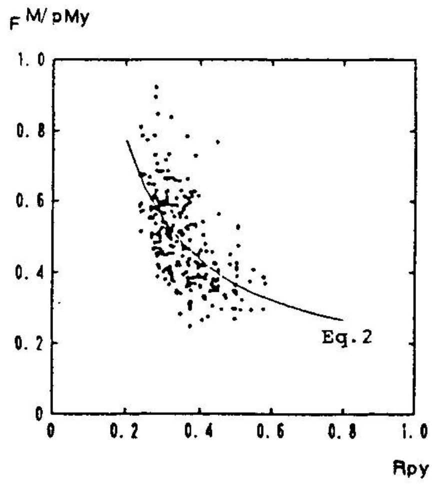


Fig.1 ${}_FM$ for practical sections (X-type with scallop) strengths of the connections depending upon α. Fig.2a contains the data of $0.59<\alpha<0.67$ and Fig.2b $0.67\leq\alpha<0.75$. Each prediction is made using the average value of α.

2.2 Panel moment and shear deformation relation

Monotonic panel moment ${}_pM$ - shear deformation γ relation of the connection is illustrated in Fig.3, where ${}_pM_{st}$, γ_y, γ_{st} and γ_u are given by Eqs.4 and 5 based on a tension coupon test. Cyclic curves can be established by connecting the positive and negative monotonic curves. Figs.4 and 5 show predicted and experimental ${}_pM$ - γ curves of various types of the connections.

$$
\begin{aligned}
{}_pM_{st} &= \tau_y V_p + {}_FM \\
&= (0.135/R_{py} + 1.098)\,{}_pM_y \qquad (4)
\end{aligned}
$$

$$
\begin{aligned}
&\gamma_y = \tau_y/G, \quad \gamma_{st} = \gamma_y + \sqrt{3}(\varepsilon_{st} - \varepsilon_y), \\
&\gamma_u = \gamma_y + \sqrt{3}(\varepsilon_u - \varepsilon_y) \qquad (5)
\end{aligned}
$$

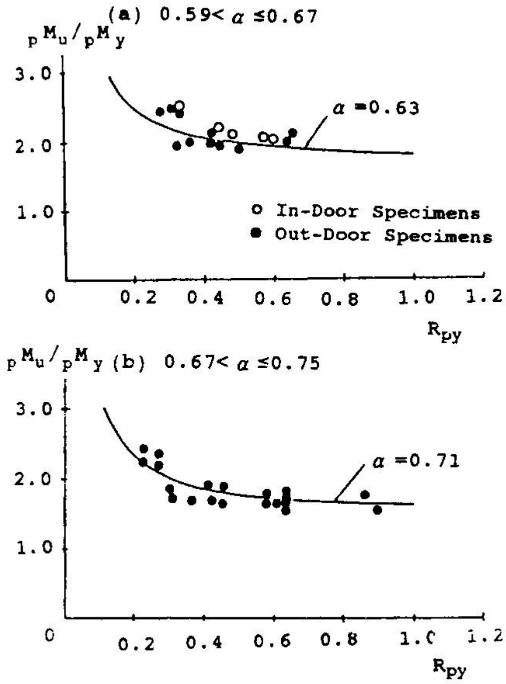


Fig.2 Predicted maximum strength

3.DYNAMIC PERFORMANCE OF LOW-RISE STEEL FRAMES

3.1 Frames and earthquakes

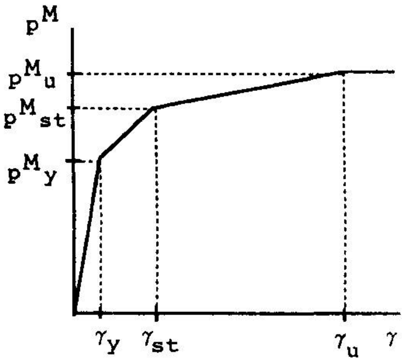
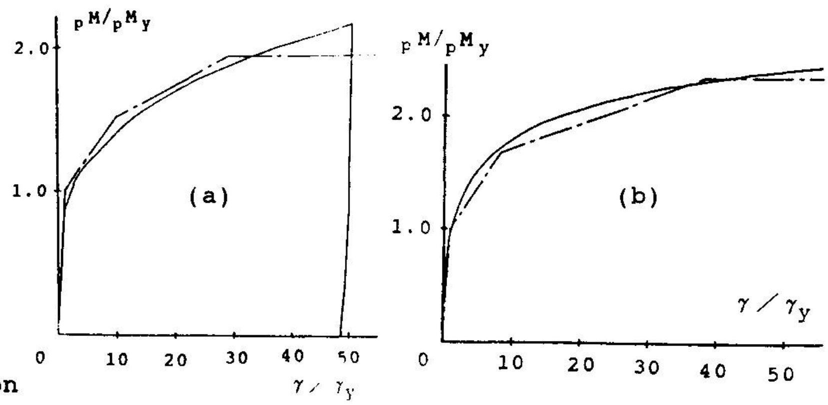


Fig.3 Assumed ${}_pM$-γ relation



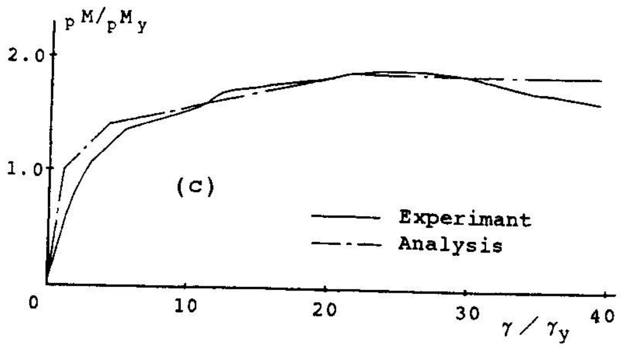


Fig.4 Predicted ${}_pM$-γ relations

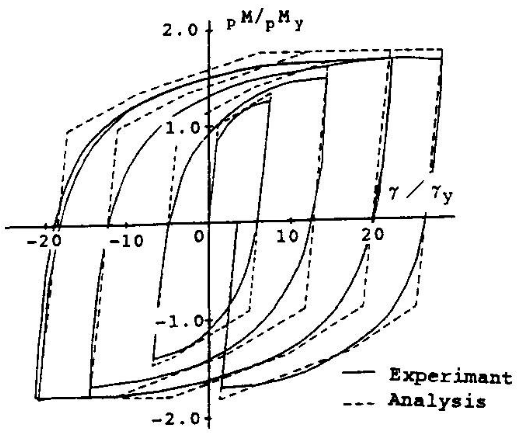


Fig.5 Predicted ${}_pM$-γ relations

For the first time, the sectional properties of the beam and the column for a 4-story 3-bay frame and a 5-story 3-bay frame are determined according to the Japanese aseismic design code(Route 2). A 5-story frame is named "A-frame" and a 4-story frame "B-frame". The sectional properties of the beam in A-frame are increased to 1.2~1.4 times as much as the minimum required value considering the composite effect of the slab concrete. As an analytical model the subassemblages are illustrated in Fig.6 consisting of the interior columns and the connecting beams cut off at their center. The section sizes of the beams and the columns are shown in Tab.1. The shear plate in H-shaped beam-to-column connection is usually reinforced through doubler plates so as to satisfy the allowable atress condition. However, the thickness of the doubler plates depends upon the existing shear stress, nominal thickness of the plate and intension of the designers. Various combinations of strengths(R_{py}=0.5,0.7,0.9,1.2) of the panel plate for each story are selected. The naming rule of the frame is illustrated in Fig.7. The first natural period for these frames are listed in Tab.2.

The frame analytical method is based on the hybrid type complementary energy's principle, where the bi-linear stress-strain relation(E_{st}/E=1/100) is assumed for a flexural member and $10\gamma_y$ for γ_{st} and $40\gamma_y$ for γ_u in $_pM - \gamma$ relation for the panel plate. Newmark's β method(β=0.25, Δt=1/400sec) is used for numerical integration in dynamic analyses. Two earthquakes recorded in Japan(Miyagi Oki Earthquake 1978.6--MIYAGI NS max acc 350,400gal and Tokachi Oki Earthquake 1968.4--HACHINOHE NS max acc 350gal) are adopted.

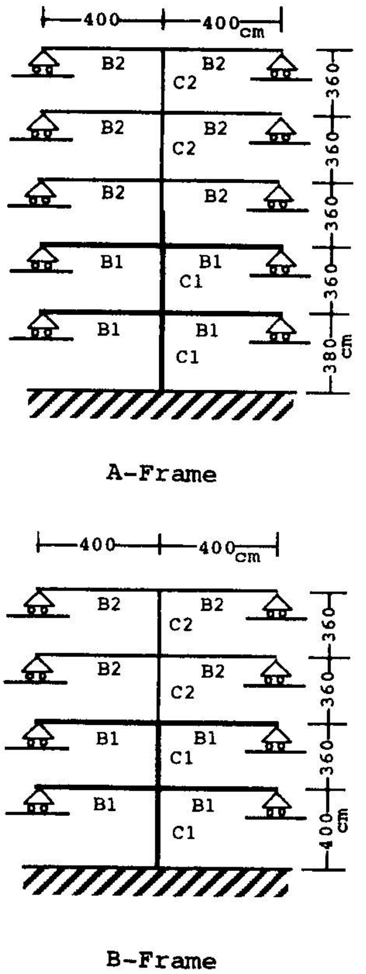


Fig.6 Analytical model

	A-Frame	B-Frame
C1	H-400×408×21×21	H-388×402×15×15
B1	H-582×300×12×17	H-506×201×11×19
C2	H-350×357×19×19	H-440×300×11×18
B2	H-606×201×12×20	H-496×199×9×14

Tab.1 Sections of the beams and the columns

Specimen	T(sec)	Specimen	T(sec)
A-11112	1.120	A-33333	1.069
A-11222	1.108	A-33444	1.062
A-11333	1.100	A-44222	1.077
A-22112	1.100	A-44333	1.059
A-22222	1.088	A-44444	1.052
A-22333	1.108	B-1112	0.969
A-22444	1.073	B-2222	0.944
A-33112	1.088	B-3333	0.929
A-33222	1.077	B-4444	0.920

Tab.2 Natural period of the frames

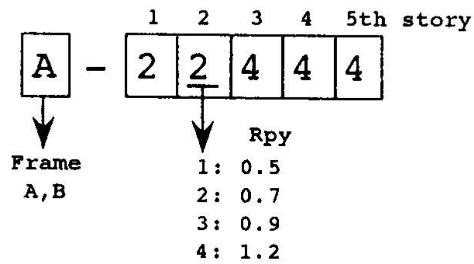


Fig.7 Naming rule of the frames

3.2 Analytical results

Ratio of the energy absorbed by the beams, columns and the connection panels is illustrated in Fig.8. Most of the energy is absorbed by the beams and columns in case of the strong panel A-44444, B-4444(R_{py}>1.0). Around 30% of total energy is shared by the panels in case of A-33333, B-3333(R_{py}=0.9) and more than half in case of $R_{py} \leqq 0.7$. Figs.9a and 10a are the maximum story drift angle R for the frame of equal R_{py} for each story. Figs.9b,c and 10b,c are the total deformation η of the panel and the column, where η_p=a half of the energy shared by the panel/${}_pM_y \cdot \gamma_y$, η_c=a half of the energy absorbed by the column/$2M_p \cdot \theta_p$ and M_p, θ_p: full plastic moment and corresponding elastic deformation of the column. R and η_c of the 4th and the 1st story in A-44444 and B-4444 subjected to MIYAGI NS are much larger than that of other stories, which suggests the local collapse in these stories. The maximum value of η_c is 6 for A-Frame and 15 for B-Frame. However, when R_{py} decreases, R of the collapsed story becomes smaller and that of other stories larger. This corresponds to decreased η_c in the 1st and 4th story and increased η_p over the whole frame. η_c for $R_{py} \leqq 0.7$ is less than half of that for R_{py}>1.0. Figs.11 and 12 are the numerical results for the frames with panels of different R_{py}. When this difference is small, such as A-22333, dynamic performance is almost the same as that of the corresponding frame of equal R_{py} such as A-22222. However, in case of large difference of R_{py}, such as A-33112, A-44222, R and η_p sometimes become large in spite of small η_c.

4.CONCLUSIONS

1) Strengths and panel moment ${}_pM$ - shear deformation γ relation of the H-shaped steel beam-to-column connection are successfully formulated.
2) In the dynamic analyses of the frames of the weak column and the strong beam, the drift R of the story which collapsed for R_{py}>1.0 becomes smaller, when $R_{py} \leqq 0.7$.
3) The column in the 1st story for R_{py}>1.0 is often damaged so much. The maximum deformation of the column η_c is ranged 6 to 15. However, η_c for $R_{py} \leqq 0.7$ decreases to less than half of that for R_{py}>1.0.
4) In case of much different R_{py} within the frame(A-44222,A-33112), drift R and deformation of the panel η_p become sometimes large.

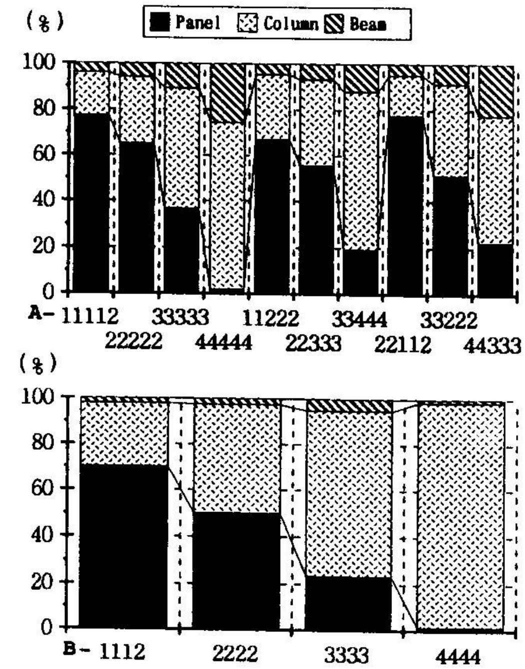


Fig.8 Energy absorbed by each member (MIYAGI 350gal)

REFERENCES

1.KAWANO A. et al., On the effect of shear stiffening of beam-to-column connections in aseismic design of low-rise steel frames. Trans.of AIJ, No.334, 1983.12
2.HASEGAWA T. and YAMANOUCHI H., Required deformation capacity of steel structural members based on damage distribution in steel frames under strong earthquake ground motions. Journal of Structural Eng. Trans.of AIJ, No.460, 1994.6
3.INOUE I. et al., Earthquake response of member plastic deformation of rigid frames with RHS columns. technical papers of annual meeting of AIJ, 1994
4.MATSUO A., NAKAMURA Y., SALIB R.W. et al., Formulation of the maximum strength of H-shaped steel beam-to-column connections. Proc. of the 10th WCEE, 1992.7
5.MATSUO A., NAKAMURA Y., SALIB R.W. et al., An evaluation of the strengths and restoring force characteristics of H-shaped steel beam-to-column connections. Journal of Structural Eng. Vol.39B(1993.3)

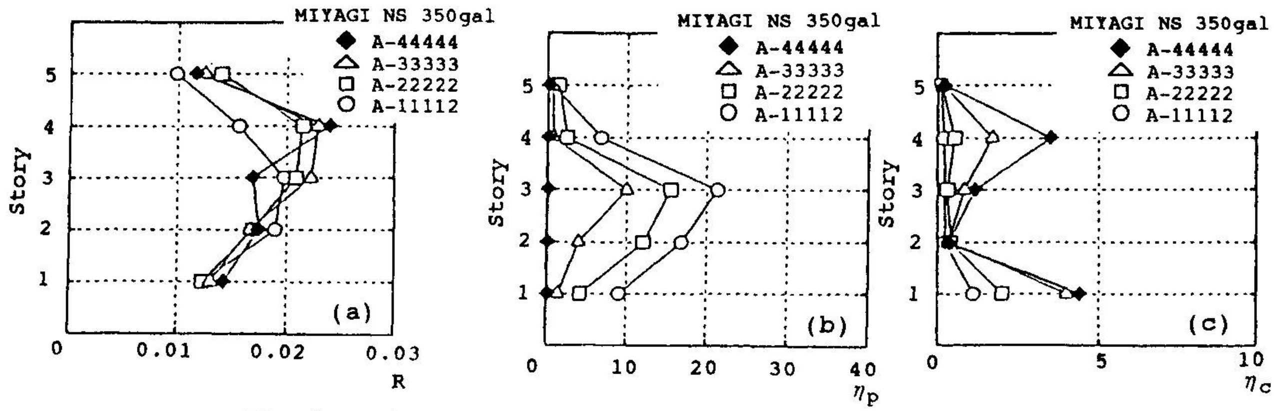


Fig.9 Maximum response of A-frame(MIYAGI 350gal)

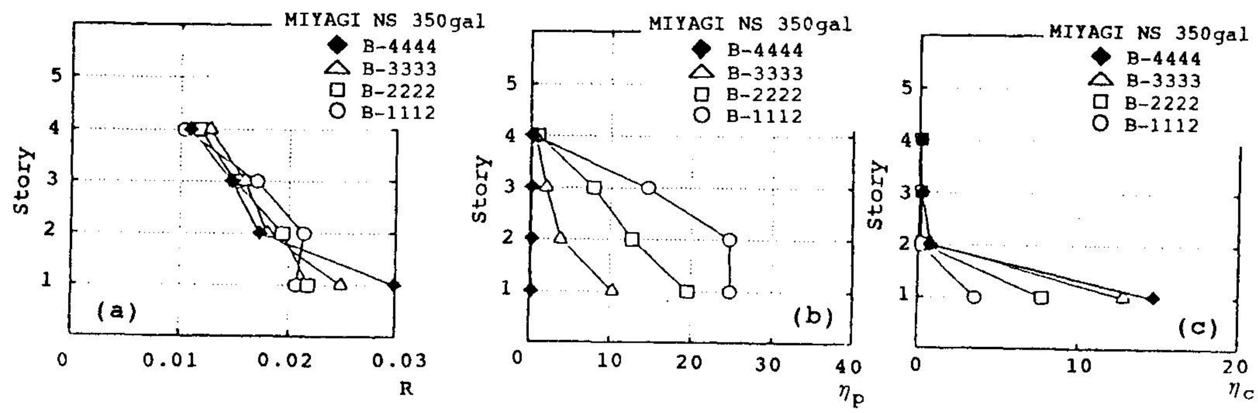


Fig.10 Maximum response of B-frame(MIYAGI 350gal)

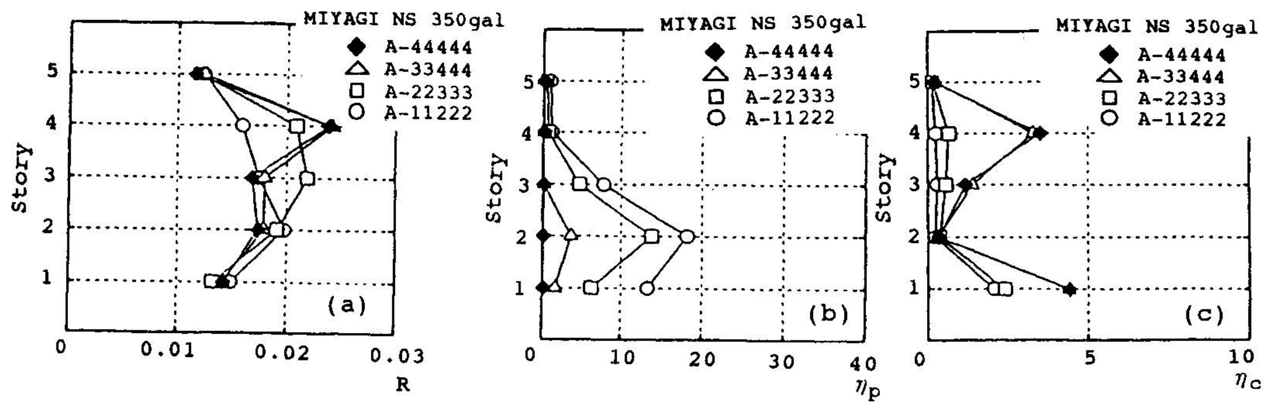


Fig.11 Maximum response of A-frame(MIYAGI 350gal)

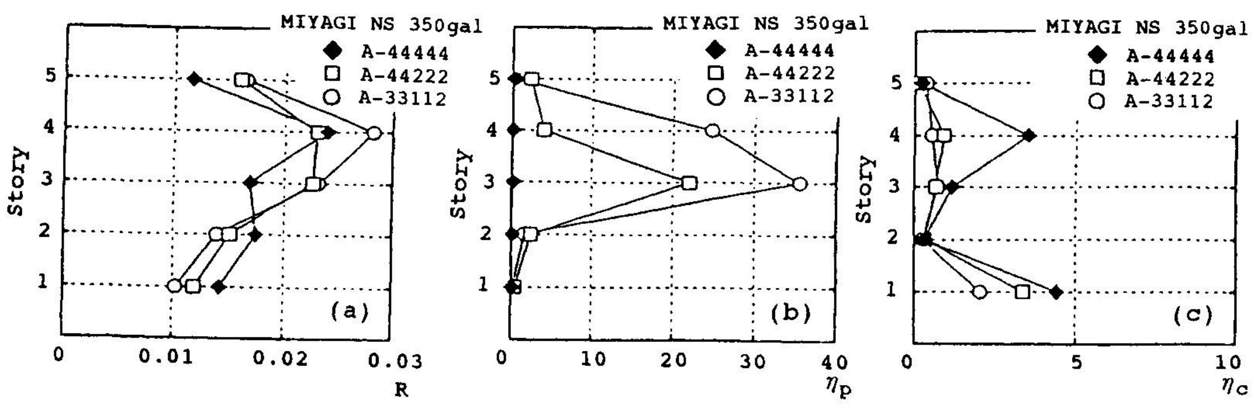


Fig.12 Maximum response of A-frame(MIYAGI 350gal)