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SHEAR WALL EXPERIMENTS AND DESIGN IN JAPAN

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SUMMARY

This paper summarizes the results of recent survey studies on the available experimental data bases and design codes/standards for reinforced concrete (RC) shear wall structures in Japan. Information related to the seismic design of RC reactor buildings and containment structures was emphasized in the survey. The seismic requirements for concrete structures, particularly those related to shear strength design, are outlined. Detailed descriptions are presented on the development of Japanese shear wall equations, design requirements for containment structures, and ductility requirements.

INTRODUCTION

As a result of reviewing Japan Electric Association guideline documents (Ref. [1]** and [2]) and Japanese design standards for nuclear power plants (Ref. [3] and [4]), it was found that the Japanese design practice generally follows the ASME codes in designing components made of metallic materials, e.g., piping and vessels. However, differences between U.S. and Japanese practices have been found regarding the seismic design of concrete structures, particularly in shear wall design criteria.

It is well-known that Japanese firms have invested heavily in large-scale testing of structures and components to improve the seismic design of nuclear, as well as non-nuclear structures, for the last few decades. In fact, U.S. engineers in the nuclear industry have used their test results as it is a source of reliable test data for nuclear power plant structures such as containment structures and pre-stressed components subjected to earthquake loadings. However, the use of test data is rather sporadic, and the availability of the data is largely limited by the language barrier.

This paper is intended to provide U.S. engineers with an overview of the current design practice of RC shear wall structures in some detail. The background studies for key design criteria for reactor buildings and containment structures are described based on the information collected in a recent survey.

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** Reference [1], which contains over 900 pages of technical material, has been translated into English and is now available as NUREG/CR-6241[5].

SURVEY OF THE PAST AND ONGOING SHEAR WALL TESTS

The Japanese seismic design methodology, compared to U.S. design practices, such as the ACI Standards, is based more on empirical formulations utilizing a wealth of available test results. As an example, the results of the correlation study for one of the shear strength equations are shown in Figure 1.

Structural testing under earthquake loadings, in fact, can be considered an "industry" in Japan. The so-called "big five" construction companies and other major construction companies own research institutes which are equipped with large-scale structural testing facilities such as reaction walls, strong floors and shaking tables. These institutes have been producing test results for various structures and components for the last three decades, which serve as the solid technical basis for the current seismic design methods. The amount of structural testing being performed in Japan by construction companies and various institutes probably is beyond the imagination of most U.S. engineers. For instance, for shear wall testing alone, the results for the following number of tests have been reported in the Annual Meeting of the Architectural Institute of Japan (AIJ) for the last five years.

Year	1988	1989	1990	1991	1992
Number of Shear Walls*	130	112	81	79	101

* numbers indicate only test results reported in the annual AIJ meetings.

Among a great number of past shear wall testing projects, a series of large-scale tests of shear walls coordinated by the Nuclear Power Engineering Corporation (NUPEC) may be of particular interest to U.S. engineers in the nuclear industry. In 1980, NUPEC started a series of RC shear wall tests, which now count about 180 specimens, for various parts of RC reactor buildings (Ref. [7]). Table 1 lists the series of test projects conducted so far.

Table 1. Shear Wall Test Projects by NUPEC

Testing Project	No. of Specimens
Small-scale and partial models	31
Whole model (PWR & BWR)	2
Scale effects	5
Dynamic behavior	22
Effect of opening	26
Leakage through cracks	24
Pullout test	69
Total	179

All of these test results have been disseminated in the form of short reports at the Annual Meeting of the AIJ. The authors have reviewed forty five (45) of the AIJ short reports, and the available test results will be summarized in a future report on this subject.

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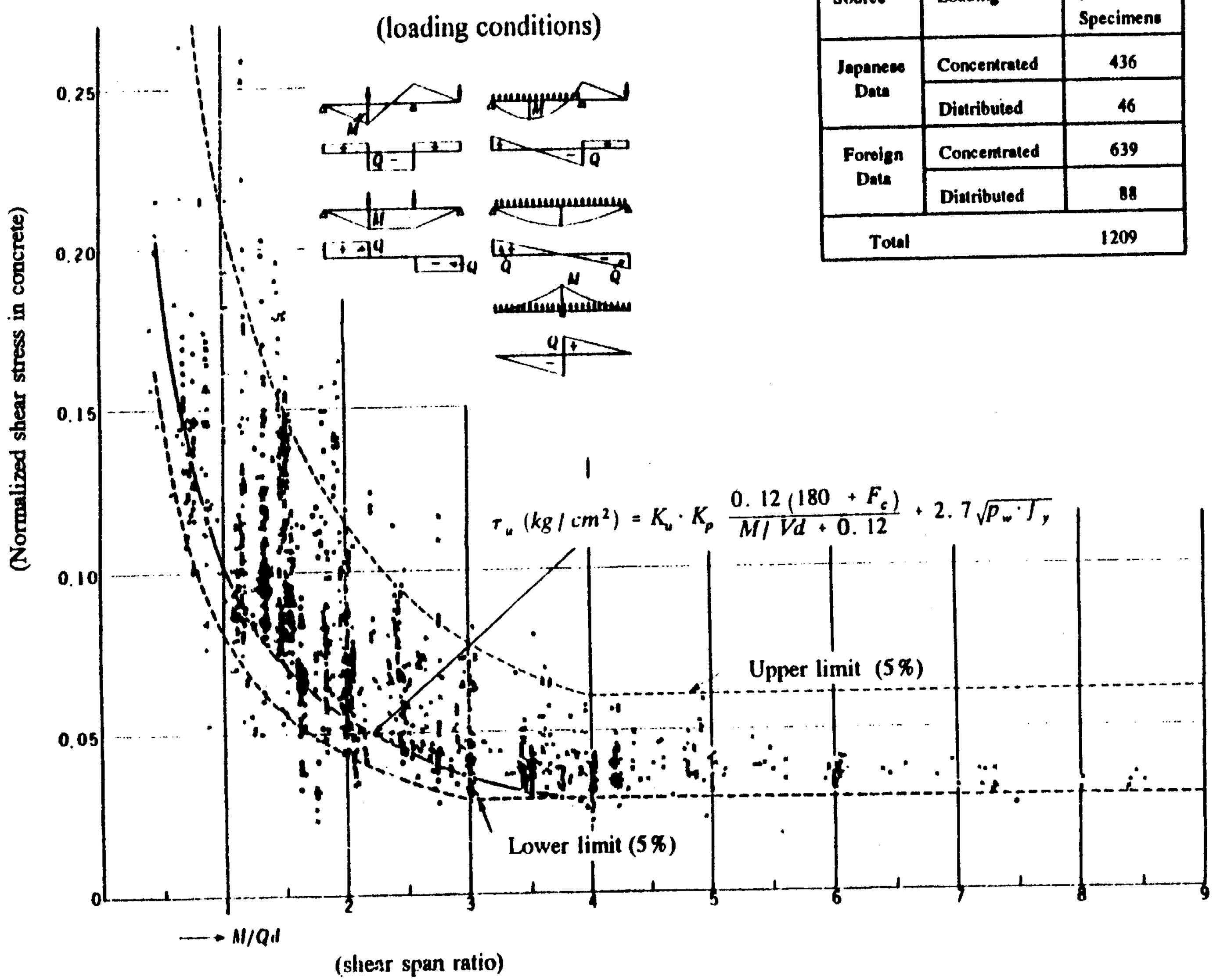


Source	Loading	Number of Specimens
Japanese Data	Concentrated	436
	Distributed	46
Foreign Data	Concentrated	639
	Distributed	88
Total		1209

Fig. 1. Correlation of Shear Strength by Arakawa (Ref. [6]).

For the purpose of explaining the availability of component test data, available test data may be classified according to the following categories:

- raw test data
- tabulated test data
- interpreted test data.

In order for a set of component test data to be classified as "raw data", at least the following information is necessary:

- complete description of model dimensions, reinforcement details, and loading conditions;
- material properties for concrete and rebars;
- at least one load-deformation relationship at a representative location
- description of the failure sequence, e.g., initial cracking, initial yielding, initial shear cracking and the final failure mode, etc.

Several books and institutional reports exist which contain a large amount of "raw data", e.g., reference [8] contains raw data for about 150 shear wall tests. Most of the test results have been published either in the Annual Meeting of AIJ or in the Journal of Concrete Engineering (Concrete Kogaku). The published test results are potentially available in the form of either raw data or tabulated data.

Regarding cylindrical and box-shaped shear wall tests, which may be more relevant to the evaluation of reactor buildings and containment structures, about 100 test results have been published so far (to the writers' knowledge). Most of the test results can be obtained in the form of raw test data in open publications.

The "interpreted data" come in many forms. One of the disadvantages in using such data is the subjectiveness in interpretations and the hidden assumptions, e.g., the selection of "good" test results. Reference [9] contains a large amount of such data, that can be used for the fragility evaluation of various structural components.

Table 2. Allowable Stresses for Concrete (unit: kg/cm^2)

Long-term allowable (D.L. & L.L.)		**Short-term allowable (Seismic Load)**	
Compression	Shear, v_c	Compression	Shear, v_c
$\frac{1}{3}F'_c$	$\min\left(\frac{F'_c}{30}, 5 + \frac{F'_c}{100}\right)$	$\frac{2}{3}F'_c$	$\min\left(\frac{F'_c}{20}, 7.5 + 0.015\,F'_c\right)$

DESIGN REQUIREMENTS FOR SHEAR WALL STRUCTURES

The AIJ standard for RC structures (Ref. [6]), which can be considered as the equivalent of ACI-318, defines the allowable stresses for concrete structures as listed in Table 2. Figure 2 compares the short-term allowable shear stress from Table 2 with available test data. The upper and lower limits of ACI-318 ($3.5\sqrt{f'_c}$ *and* $1.9\sqrt{f'_c}$), are also shown in the figure.

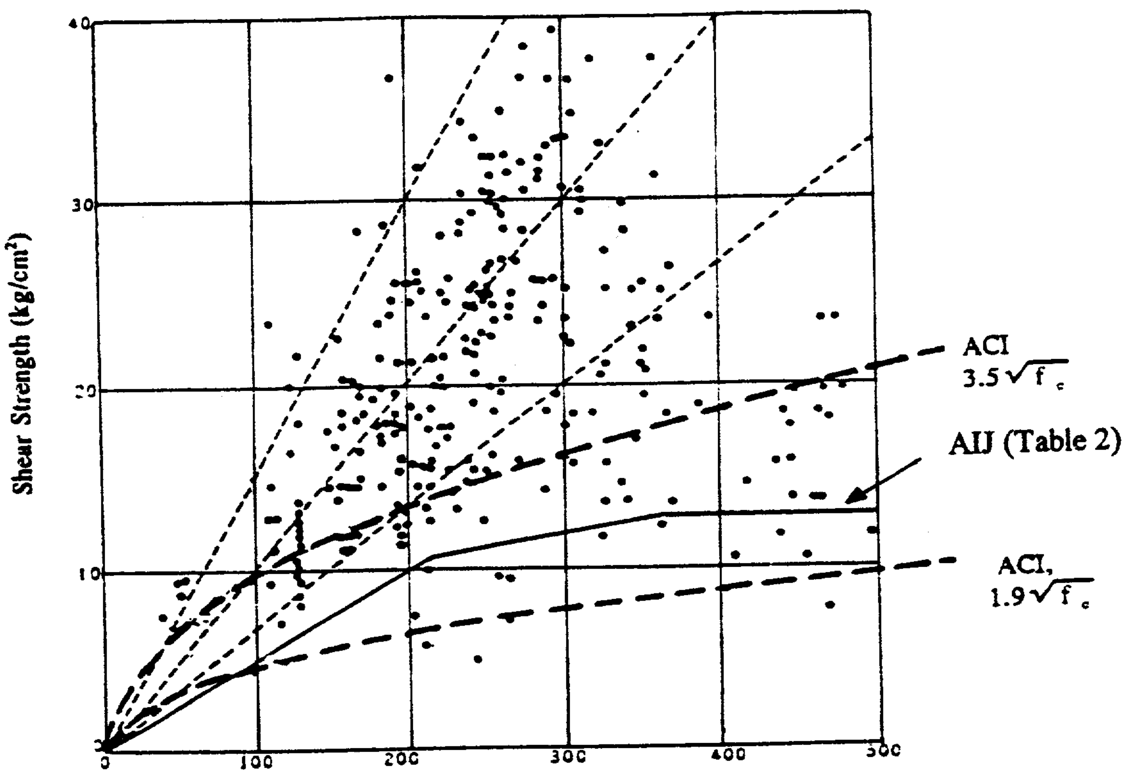


Figure 2. Shear Cracking Stress of Shear Walls Under Lateral Loads (Ref. [6]).

The AIJ recommendations for the design of reactor buildings, which are also described in JEAG4601-1987 (Ref. [1], [5]), are based largely on the foregoing AIJ RC-Standard (Ref. [6]). In the recommendations, the concept of "allowable state" has been introduced, which is similar to the classification of levels A, B, C and D limits (i.e., normal, upset, emergency and faulted) in the ASME code. The classification of the allowable states are summarized in Ref. [21], which also provides additional information on the seismic design requirements for nuclear power plants in Japan. One of the features of this recommendation is that for the extreme design earthquake (referred to as the S_2 earthquake) or for an accident condition, the thermal stress can be neglected.

DEVELOPMENT OF SHEAR WALL EQUATIONS

In the seismic design of reactor buildings, nonlinear dynamic analyses are generally required when considering the response to the S_2 earthquake. Therefore, the shear wall equations are used not only for the component strength evaluation, but also to determine the restoring force characteristics of the nonlinear structural models. The latter requirement necessitates the development of shear wall equations which can predict the ultimate capacity and the nonlinear deformation properties without bias or excessive conservatism. Currently, Hirosawa's equation and the JEAG (or Yoshizaki's) equation are extensively used in the seismic design of reactor buildings. Hirosawa's equation, which is also extensively used for non-nuclear buildings, is also called the modified Arakawa's equation, as it was developed by modifying the shear strength equation for beams and columns by Arakawa (Ref. [6]). Some benchmark studies, which led to the current Japanese shear wall equations, are described below.

<u>Arakawa's Study (1960)</u> The current Japanese design criteria on the shear capacity of R.C. components are based on the studies by Arakawa (Ref. [18], [19]). The foregoing Figure 1 shows a part of the correlation study to develop the following shear strength equation for reinforced concrete beams, which now is called the <u>original</u> Arakawa's equation:

$$v_u(kg/cm^2) = K_u \cdot K_p \frac{0.12\,(180 + F_c)}{M/Vd + 0.12} + 2.7\sqrt{p_w \cdot f_y} \qquad (1)$$

in which

- K_u = reduction factor for scale effect (=0.72 when d>40 cm)
- K_p = $0.82\,p_t^{0.23}$
- p_t = tension steel ratio in percent
- M/Vd = shear span ratio (replaced by 3 when larger than 3)
- p_w = stirrup ratio

The contribution of the reinforcement to the shear strength, v_s or the second term of Eq. 1, was obtained empirically as illustrated in Figure 3. A total of 219 pairs of specimens, for which the shear reinforcement was the only parameter (i.e., one beam is shear reinforced and the other is not), were used to directly determine the contribution by the shear reinforcement as follows:

(Contribution by Shear Reinforcement) =(Shear Strength of Reinforced Beam) - (Shear Strength of Unreinforced Beam).

The results presented in Figure 3 indicate that the truss theory approximation, on which the current ACI equations are based, largely overestimates the shear contribution by steel. The data shown in Figures 1 and 3, in fact, had a major impact on the subsequent design code development in Japan.

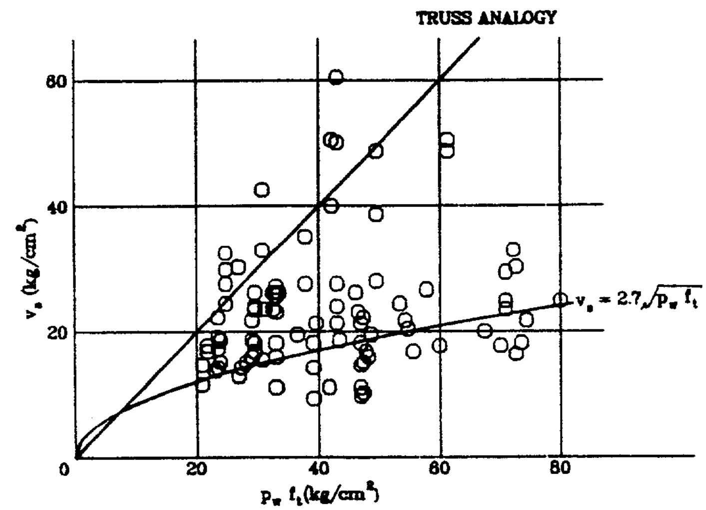
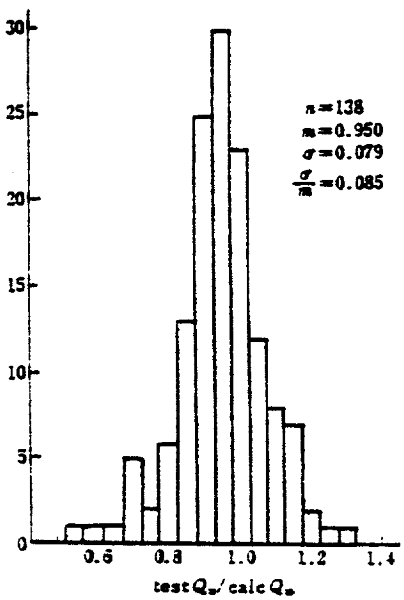
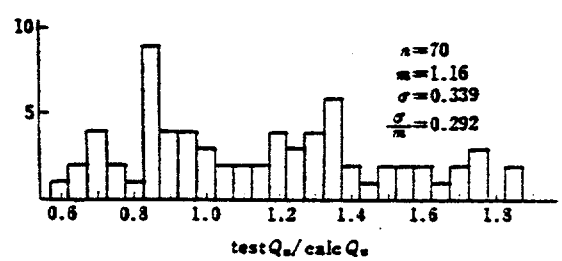


Figure 3. Contribution by Rebars to Shear Capacity (replotted based on the data in Ref.[18], components with $p_w f_t < 20$ are excluded).



(a) AIJ equation (Eq. 1)



(b) ACI equation (Eq. 2)

Fig. 4. Histograms for the Ratios of Tests to Calculated Ultimate Shear Strength of R.C. Beams (Ref. [9]).

The corresponding ACI equation is,

$$V_u(lbs) = V_c + \phi p_w \cdot f_y \cdot bd$$
$$V_c(lbs) = \phi\left(1.9\sqrt{f_c'} + 2500\ p_w \frac{Vd}{M}\right) bd \leq 3.5\sqrt{f_c'}bd \tag{2}$$

Figure 4 shows the results for correlation studies of the above shear strength equations (Ref. [9]). The ACI equations show a much larger scatter compared with the AIJ equation.

Hirosawa's Study on Shear Walls (1975) Hirosawa's equation is extensively used in Japan to calculate the ultimate shear strength of shear walls both for non-nuclear and nuclear power structures. The empirical equation was obtained by modifying the above Arakawa's equation (Ref. [8]):

$$V_u(Kg) = \left\{\frac{0.0679\ p_t^{0.23}\ (F_c + 180)}{\sqrt{M/VD + 0.12}} + 2.7\sqrt{f_{yw} \cdot p_w} + 0.1\sigma_o\right\} b_e\, j \tag{3}$$

where b_e = effective thickness of wall (when a wall has flanges, 'b_e' is calculated from a uniform-thickness cross-section with equal area);

j = 0.83 D (D = total length of wall)

p_t (%) = 100 $A_t/(b_e \cdot j)$, tension axial steel ratio considering a wall as a column (A_t = total axial steel area in a flange);

M/VD = shear span ratio;

p_w = horizontal steel ratio using the effective thickness b_e;

F_c = concrete strength (kg/cm^2)

f_{yw} = steel yield stress (kg/cm^2)

σ_o = average axial stress (kg/sm^2)

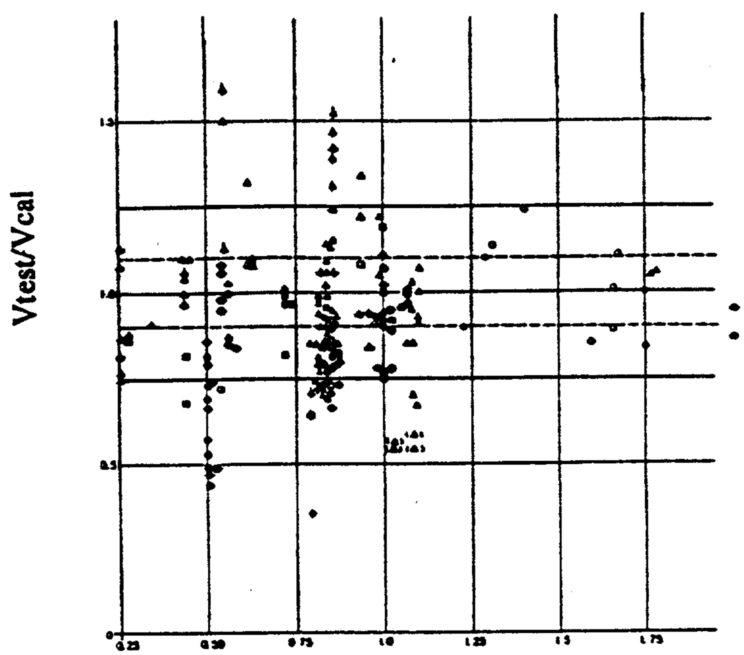
Figure 5 and Table 3 show part of the correlation studies on this equation.

JEAG Equation In the nonlinear dynamic analysis of reactor buildings, the so-called bending/shear beam model is extensively used (Ref. [1], [5]). In this model the bending and shear deformation properties are separately determined, and the restoring force characteristics of a reactor building are obtained as a series combination of two nonlinear springs as illustrated in Figure 6. The JEAG equation (or Yoshizaki's equation) was developed for this purpose. Based on a correlation study on the past cyclic loading tests of cylindrical, box-shaped, cone-shaped and I-shaped shear walls, the parameters in Figure 6 are determined as summarized in Table 4.

Table 3. Correlation of Hirosawa's Equation (Ref. [8]).

Range of Parameters	**Number of Shear Walls**	**Range Vtest/Vcal**	**Mean Vtest/Vcal**	**Standard Deviation**
0<M/VD≤0.75	52	0.44~1.59	0.90	0.23
0.75<M/VD≤1.0	79	0.35~1.52	0.92	0.21
1.0<M/VD	37	0.54~1.24	0.91	0.17
0<t/L<0.3	47	0.35~1.24	0.81	0.18
0.3≤t/L<1.0	84	0.54~1.59	0.97	0.21
t/L=1.0	37	0.64~1.19	0.95	0.13
$p_w=0$	15	0.35~1.59	0.99	0.39
$0<p_w<0.4$	46	0.44~1.32	0.89	0.23
$0.4 \le p_w<0.8$	50	0.53~1.50	0.91	0.18
$0.8 \le p_w<1.2$	26	0.72~1.08	0.94	0.11
$1.2 \le p_w$	31	0.70~1.22	0.90	0.12
Total	168	0.35~1.59	0.92	0.21

M/VD = shear span ratio
b_e/D = thickness ratio
p_w = horizontal shear reinforcement ratio



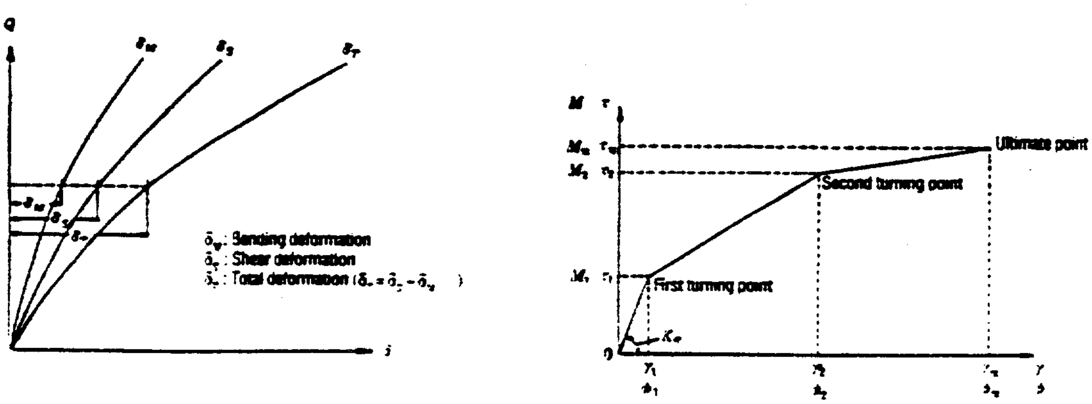
Shear Span Ratio, M/VD
Figure 5. Correlation of Hirosawa's Equation (Ref. [8]).

Table 4. JEAG Equation (Ref. [1], [5])

	Shear	Bending
Cracking	$\tau_1 = \sqrt{\sqrt{F_c}\left(\sqrt{F_c} + \sigma_v\right)}$ $\gamma_1 = \tau_1/G$	$M_1 = Z_e\,(f_t + \sigma_v)$ $\phi_1 = M_1/\,({}_cE) \cdot I_e$
Yielding	$\tau_2 = 1.35\ \tau_1$ $\gamma_2 = 3\ \gamma_1$	$M_2 = M_y$ $\phi_2 = \phi_y$
Ultimate	$\tau_3 = \left\{1-\tau_s/\left(4.5\sqrt{F_c}\right)\right\}\tau_0 + \tau_s, \tau_s \le 4.5\sqrt{F_c}$ $= 4.5\sqrt{F_c} \qquad , \tau_s > 4.5\sqrt{F_c}$ $\gamma_3 = 4.0 \times 10^{-3}$ $\tau_0 = (3 - 1.8M/VD)\sqrt{F_c}$ *When* $M/VD > 1$, *then* $M/VD = 1$ $\tau_s = \left(p_v + p_H\right)\ {}_s\sigma_y/2 + \left(\sigma_v + \sigma_H\right)/2$	$M_3 = M_u$ $\phi_3 = 0.004\,/X_{nu}$ When $\phi_3 > 20\ \phi_2$, then $\phi_3 = 20\ \phi_2$

F_c = concrete strength (kgf/cm^2)
G = concrete shear modulus (kgf/cm^2)
cE = concrete Young's Modulus (kgf/cm^2)
p_v, p_H = vertical and horizontal steel ratios
σ_v, σ_H = vertical and horizontal stress (kgf/cm^2) (positive for compression)
${}_s\sigma_y$ = steel yield stress (kgf/cm^2)
M/VD = shear span ratio

I_e = moment of inertia including steel (cm^4)
Z_e = section modulus including steel (cm^3)
f_t = $1.2\sqrt{F_c}$ = tension strength of concrete (kgf/cm^2)
ϕ_y = yield curvature
D = distance between centers of flanges
M_u = full-plastic moment (kgf • cm)
X_{nu} = distance from the neutral axis to the compressive extreme fiber



(a) Bending and Shear Deformation of Shear Wall Structures

(b) Trilinear Skelton Curves

Figure 6. Modeling of Reactor Building (Ref. [5]).

The results of the correlation studies of the above equations are summarized in Table 5 for the shear spring part only (Ref. [22]). All the empirical equations correlate well with test data except for the ultimate shear deformation (γ_3). There exists a significant uncertainty in estimating this quantity.

Table 5. Correlation of JEAG Equations for Shear (Ref. [22])

Empirical equation for	Data	Average Test/Calc.	Standard Deviation
1st turning point stress, τ_1	57	0.98	0.21
Initial stiffness, G	49	0.93	0.16
2nd turning point stress, τ_2	58	0.99	0.17
Ultimate stress, τ_3	86	1.04	0.19
Ultimate deformation, γ_3	48	1.74	0.76

DESIGN REQUIREMENTS FOR CONTAINMENT STRUCTURES

The seismic design of concrete containment structures is performed based on the MITI Notification No. 452 (Ref. [16]). This document, and in particular the background information upon which this standard is based, may be useful as the test results of large-scale containment structures are extensively utilized. The design requirements for in-plane and out-of-plane shear stresses are briefly described below.

<u>In-plane Shear</u> The design criteria of the in-plane shear stress, τ, for containment structures are given by the following simple equations:

$$\min. \begin{cases} \tau_u = \Phi 0.5\{(P_{t\phi} \cdot f_y + \sigma_{p\phi} - \sigma_{o\phi}) + (P_{t\theta} f_y + \sigma_{p\theta} - \sigma_{o\theta})\} & (4) \\ \tau_u = \Phi 3.5\sqrt{F_c} & (5) \end{cases}$$

in which,

f_y = yield stress of rebars (kg/cm^2)
F_c = concrete strength (kg/cm^2)
$P_{t\phi}$, $P_{t\theta}$ = meridional and circumferential steel ratios
$\sigma_{p\phi}$, $\sigma_{p\theta}$= meridional and circumferential membrane stress due to prestressing (kg/cm^2)
$\sigma_{o\phi}$, $\sigma_{o\theta}$= meridional and circumferential membrane stress due to external forces (considered only when the stress is in tension, and the tension stress is positive).

The reduction factor, Φ, which is the same for the out-of-plane shear, is defined in Table 6. A complete description of loading states, plant conditions and associated load combinations can be found in Ref. [1], [5] and [24].

Table 6. Reduction Factor for Containment Structures (Ref. [16]).

Loading State	I, II	III	IV
Plant Condition	Normal	S_1 Earthquake	S_2 Earthquake
Reduction factor, Φ	0.5	0.75	1.0

The factor of 0.5 in Equation 4 is to average the contributions of the meridional and circumferential reinforcement. This rule works relatively well when the steel ratios in both directions do not differ significantly. When the ratio of the two steel ratios exceeds 1.2, it is recommended to neglect the excessive part of the reinforcement.

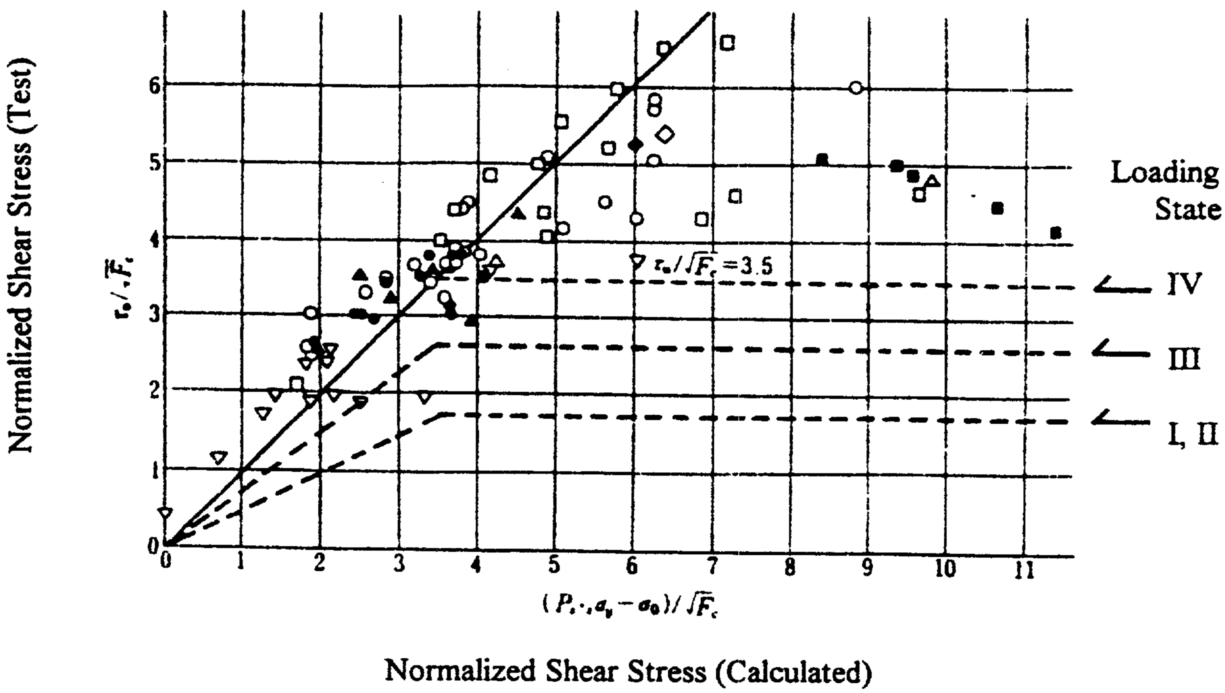


Figure 7. Shear Stress Limits for Containment Structures (Ref. [23]).

Figure 7 shows the results of a correlation study of Equations 4 and 5 using the test results of cylindrical shear wall models. The term $3.5\sqrt{F_c}$ (in kg/cm^2) or $13.2\sqrt{f_c}$ (in terms of psi), in Equation 5 does not necessarily represent the contribution by concrete. Rather, it represents a shear stress limit from which any further increase of reinforcement would not increase the strength. The combination of Equations 4 and 5 provide a conservative lower bound for the available containment model test results as shown in Figure 7.

As shown in Table 4, the above shear stress limit can be increased to $4.5\sqrt{F_c}$ (in kg/cm^2) for the median estimate of the shear strength according to the JEAG (Yoshizaki's) shear wall equation.

Out-of-Plane Shear.........Aoyagi proposed a rather complex semi-empirical equation for the out-of-plane shear strength of shear walls (Ref. [16]).

$$\min. \left\{ \begin{array}{c} \tau_R = \dfrac{(p_t f_y - \sigma_0)\cos\theta \sin^2(70^\circ - \theta) + p_w f_y \sin\theta \sin^2(\theta + 20^\circ)}{\cos^2\theta + \sin\theta\cos\theta} \\ \tau_R = 3.5\sqrt{F_c} \end{array} \right\} \qquad (6)$$

in which, θ is the angle of the shear crack plane; p_t is vertical steel ratio; p_w is the horizontal steel ratio; and σ_o is the membrance stress due to external forces (tension is positive). Assuming the crack angle, θ, to be 45°, and also by including the effect of shear span ratio, M/Vd, the design criteria for the out-of-plane shear in MITI No. 452 has been defined by the following equation,

$$\min. \left\{ \begin{array}{c} \tau_R = \left\{0.1(p_t f_y - \sigma_o) + 0.5\, p_w f_y + 0.75\sqrt{F_c}\right\} / \sqrt{M/Vd} \\ \tau_R = 3.5\sqrt{F_c} \end{array} \right\} \qquad (7)$$

in which, the shear span ratio, M/Vd, is replaced by 1.0 if lower than 1.0, and by 3.0 if higher than 3.0. The second criterion of $3.5\sqrt{F_c}$, rarely is a controlling factor.

The above equation does not include the effect of axisymmetric loads, such as those due to accident pressure. Additional equations for such loads can be found in Ref. [16].

DUCTILITY CAPACITY/REQUIREMENTS

The allowable shear deformation for both reactor buildings and containment structures are defined to be 0.2% in radians (Ref. [1], [5], [16]). Based on a statistical analysis of available box-shaped and cylindrical shear wall structures, the minimum shear deformation capacity was estimated to be 0.4% in radians (Ref. [20]). Therefore, a safety factor of 2.0 is used to account for the large scatter associated with the shear deformation capacity (see also γ_3 in Table 5).

Figure 8 shows the results of a study of available data on box-shaped and cylindrical shear walls by the authors. In the figure, the ultimate shear deformation capacity is plotted against the maximum shear stress. For high strength shear walls, the deformation capacity tends to decrease to about 0.4% in radians. However, for shear walls with less shear strength, a much higher ductility can be expected. The results of a similar study on shear deformation capacity are presented for both RCCV and PCCV structures in Ref. [16], in which the ultimate shear deformation capacity was plotted against the normalized steel ratio. According to this study, the ductility of containment structures tends to

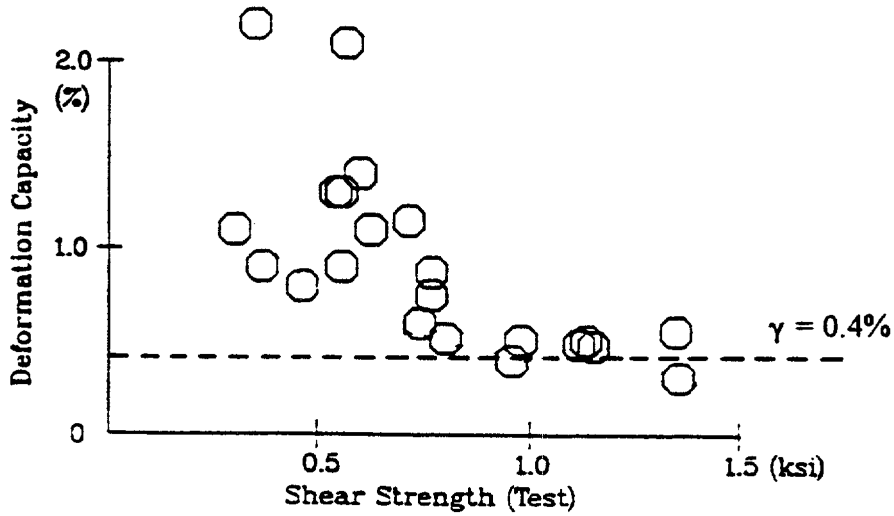


Figure 8. Shear Deformation Capacity of Box-Shaped and Cylindrical Shear Walls.

decrease as the steel ratio (therefore shear strength) increases. The minimum shear deformation capacity for heavily reinforced containment structures converges to about 0.4% in radian. A more detailed statistical study on the shear deformation capacity of ordinary RC shear walls is also described in Ref. [9].

SUMMARY AND CONCLUSIONS

The current Japanese practice for the seismic design of RC shear wall structures, particularly those related to the seismic design of reactor buildings and containment structures, and related research activities were reviewed in some detail. After three decades of extensive experimental work, it seems that all the major testing parameters have been exhausted for various structural components. However, judging from the publications, such as the transactions of the Annual Meeting of AIJ, the experimental studies on components are still increasing, rather than peaking, although the emphasis is being shifted more to newer design approaches, such as base isolation devices and pre-fabricated components.

The value of the wealth of accumulated information in Japan has been recognized by many prominent U.S. engineers. In fact, the test results of large-scale shear wall structures and pre-stressed components have been utilized in the fragility evaluation of nuclear reactor facilities, as they are the only available source of information for these types of structures. The extent to which Japanese test data have been used by U.S. engineers, however, has been seriously limited due to the language barrier. Further efforts should be made to utilize the available information.

In general, U.S. engineers are well equipped with analysis tools, e.g., computing equipment and numerous structural analysis codes. However, the critical evaluation of existing nuclear facilities and the development of new design concepts require much more than structural analysis codes. The accumulated test results, mostly performed using modern testing facilities and earthquake-like loading conditions, should also be reviewed and evaluated in detail.

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