Westinghouse

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U.S. Nuclear Regulatory Commission
ATTENTION: Document Control Desk
Washington, D.C. 20555

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Your ref: Docket Number 52-006
Our ref: DCP_NRC_003029

September 3, 2010

Subject: Supplemental Information to Resolve Shield Building Audit Questions

The purpose of this letter is to provide additional information related to Actions 11 and 12, as well as the Ductility/Benchmarking papers requested at the August NRC audit at Westinghouse Cranberry. This supplements the Shield Building Revision 2 report and the letter transmitting the WEC action items.

WEC action item 21 will follow formally on September 8th if no additional NRC comments are required to be addressed. A draft is being transmitted to Mr. Bret Tegeler. This would complete all information necessary that is associated with the Enhanced Shield Building review and approval.

Pursuant to 10 CFR 50.30(b), proprietary and non-proprietary versions of the reports and information are submitted as Enclosures 3 through 10. Enclosure 1 is one copy of the Application for Withholding, AW-10-2908 (non-proprietary). Enclosure 2 is one copy of the associated Affidavit with Proprietary Information Notice and Copyright Notice (non-proprietary).

These submittals contain proprietary information of Westinghouse Electric Company, LLC. In conformance with the requirements of 10 CFR Section 2.390, as amended, of the Commission's regulations, we are enclosing with this submittal an Application for Withholding and an Affidavit. The Affidavit sets forth the basis on which the information identified as proprietary may be withheld from public disclosure by the Commission. The information being redacted is of a type customarily held in confidence by Westinghouse and not customarily disclosed to the public.

Correspondence with respect to the Affidavit or Application for Withholding should reference AW-10-2908 and should be addressed to James A. Gresham, Manager, Regulatory Compliance and Plant Licensing, Westinghouse Electric Company, LLC, P. O. Box 355, Pittsburgh, Pennsylvania 15230-0355.

Questions or requests for additional information related to the content and preparation of this response should be directed to Westinghouse. Please send copies of such questions or requests to the prospective applicants for combined licenses referencing the AP1000 Design Certification. A representative for each applicant is included on the cc: list of this letter.

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D063
NRO

Very truly yours,

R. F. Ziesing
Director, US Licensing

/Enclosures

1. AW-10-2908 "Application for Withholding Proprietary Information from Disclosure," dated September 3, 2010 (Non-Proprietary)
2. AW-10-2908, Affidavit, Proprietary Information Notice, Copyright Notice, dated September 3, 2010 (Non-Proprietary)
3. Shield Building Benchmarking Analysis – (Proprietary)
4. Shield Building Benchmarking Analysis – (Non-Proprietary)
5. Shield Building Behavior and Design Philosophy – (Proprietary)
6. Shield Building Behavior and Design Philosophy – (Non-Proprietary)
7. Action Item 11 – (Proprietary)
8. Action Item 11 – (Non-Proprietary)
9. Action Item 12 – (Proprietary)
10. Action Item 12 – (Non-Proprietary)
11. Study on Steel Plate Reinforced Concrete Panels Subjected to Cyclic In-plane Shear

cc:	D. Jaffe	-	U.S. NRC	11E
	E. McKenna	-	U.S. NRC	11E
	P. Clark	-	U.S. NRC	11E

ENCLOSURE 1

AW-10-2908

APPLICATION FOR WITHHOLDING
PROPRIETARY INFORMATION FROM DISCLOSURE

Westinghouse

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e-mail: ziesinrf@westinghouse.com

Your ref: Docket Number 52-006
Our ref: AW-10-2908

September 3, 2010

APPLICATION FOR WITHHOLDING PROPRIETARY
INFORMATION FROM PUBLIC DISCLOSURE

Subject: Supplemental Information to Resolve Shield Building Audit Questions

The Application for Withholding is submitted by Westinghouse Electric Company, LLC (Westinghouse), pursuant to the provisions of Paragraph (b) (1) of Section 2.390 of the Commission's regulations. It contains commercial strategic information proprietary to Westinghouse and customarily held in confidence.

The proprietary material for which withholding is being requested is identified in the proprietary version of the subject briefing presentations. In conformance with 10 CFR Section 2.390, Affidavit AW-10-2908 accompanies this Application for Withholding, setting forth the basis on which the identified proprietary information may be withheld from public disclosure.

Accordingly, it is respectively requested that the subject information which is proprietary to Westinghouse be withheld from public disclosure in accordance with 10 CFR Section 2.390 of the Commission's regulations.

Correspondence with respect to this Application for Withholding or the accompanying affidavit should reference AW-10-2908 and should be addressed to James A. Gresham, Manager, Regulatory Compliance and Plant Licensing, Westinghouse Electric Company, LLC, P.O. Box 355, Pittsburgh, Pennsylvania, 15230-0355.

Very truly yours,

R. F. Ziesing
Director, US Licensing

ENCLOSURE 2

Affidavit

(with Proprietary Information Notice and Copyright Notice)

AW-10-2908
September 3, 2010

AFFIDAVIT

COMMONWEALTH OF PENNSYLVANIA:

ss

COUNTY OF BUTLER:

Before me, the undersigned authority, personally appeared R. F. Ziesing, who, being by me duly sworn according to law, deposes and says that he is authorized to execute this Affidavit on behalf of Westinghouse Electric Company LLC (Westinghouse), and that the averments of fact set forth in this Affidavit are true and correct to the best of his knowledge, information, and belief:

R. F. Ziesing

R. F. Ziesing
Director, US Licensing

Sworn to and subscribed
before me this 3rd day
of September 2010.

COMMONWEALTH OF PENNSYLVANIA
Notarial Seal
Linda J. Bugle, Notary Public
City of Pittsburgh, Allegheny County
My Commission Expires June 18, 2013
Member, Pennsylvania Association of Notaries

Linda J. Bugle
Notary Public

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(1) I am Director, US Licensing, Westinghouse Electric Company, LLC (Westinghouse), and as such, I have been specifically delegated the function of reviewing the proprietary information sought to be withheld from public disclosure in connection with nuclear power plant licensing and rule making proceedings, and am authorized to apply for its withholding on behalf of Westinghouse.

(2) I am making this Affidavit in conformance with the provisions of 10 CFR Section 2.390 of the Commission's regulations and in conjunction with the Westinghouse "Application for Withholding" accompanying this Affidavit.

(3) I have personal knowledge of the criteria and procedures utilized by Westinghouse in designating information as a trade secret, privileged or as confidential commercial or financial information.

(4) Pursuant to the provisions of paragraph (b)(4) of Section 2.390 of the Commission's regulations, the following is furnished for consideration by the Commission in determining whether the information sought to be withheld from public disclosure should be withheld.

(i) The information sought to be withheld from public disclosure is owned and has been held in confidence by Westinghouse.

(ii) The information is of a type customarily held in confidence by Westinghouse and not customarily disclosed to the public. Westinghouse has a rational basis for determining the types of information customarily held in confidence by it and, in that connection, utilizes a system to determine when and whether to hold certain types of information in confidence. The application of that system and the substance of that system constitutes Westinghouse policy and provides the rational basis required.

Under that system, information is held in confidence if it falls in one or more of several types, the release of which might result in the loss of an existing or potential competitive advantage, as follows:

(a) The information reveals the distinguishing aspects of a process (or component, structure, tool, method, etc.) where prevention of its use by any of Westinghouse's competitors without license from Westinghouse constitutes a competitive economic advantage over other companies.

(b) It consists of supporting data, including test data, relative to a process (or component, structure, tool, method, etc.), the application of which data secures a competitive economic advantage, e.g., by optimization or improved marketability.

(c) Its use by a competitor would reduce his expenditure of resources or improve his competitive position in the design, manufacture, shipment, installation, assurance of quality, or licensing a similar product.

(d) It reveals cost or price information, production capacities, budget levels, or commercial strategies of Westinghouse, its customers or suppliers.

(e) It reveals aspects of past, present, or future Westinghouse or customer funded development plans and programs of potential commercial value to Westinghouse.

(f) It contains patentable ideas, for which patent protection may be desirable.

There are sound policy reasons behind the Westinghouse system which include the following:

(a) The use of such information by Westinghouse gives Westinghouse a competitive advantage over its competitors. It is, therefore, withheld from disclosure to protect the Westinghouse competitive position.

(b) It is information that is marketable in many ways. The extent to which such information is available to competitors diminishes the Westinghouse ability to sell products and services involving the use of the information.

(c) Use by our competitor would put Westinghouse at a competitive disadvantage by reducing his expenditure of resources at our expense.

(d) Each component of proprietary information pertinent to a particular competitive advantage is potentially as valuable as the total competitive advantage. If competitors acquire components of proprietary information, any one component may be the key to the entire puzzle, thereby depriving Westinghouse of a competitive advantage.

(e) Unrestricted disclosure would jeopardize the position of prominence of Westinghouse in the world market, and thereby give a market advantage to the competition of those countries.

(f) The Westinghouse capacity to invest corporate assets in research and development depends upon the success in obtaining and maintaining a competitive advantage.

(iii) The information is being transmitted to the Commission in confidence and, under the provisions of 10 CFR Section 2.390, it is to be received in confidence by the Commission.

(iv) The information sought to be protected is not available in public sources or available information has not been previously employed in the same original manner or method to the best of our knowledge and belief.

(v) The proprietary information sought to be withheld in this submittal is that which is appropriately marked in the presentation entitled, "Supplemental Information to Resolve Shield Building Audit Questions", in support of the AP1000 Design Certification Amendment Application, being transmitted by Westinghouse letter (DCP_NRC_003029) and Application for Withholding Proprietary Information from Public Disclosure, to the Document Control Desk. The proprietary information as submitted by Westinghouse for the AP1000 Design Certification Amendment application is expected to be applicable in all licensee submittals referencing the AP1000 Design Certification and the AP1000 Design Certification Amendment Application in response to certain NRC requirements for justification of compliance of the safety system to regulations.

This information is part of that which will enable Westinghouse to:

(a) Manufacture and deliver products to utilities based on proprietary designs.

(b) Advance the AP1000 Design and reduce the licensing risk for the application of the AP1000 Design Certification

(c) Determine compliance with regulations and standards

(d) Establish design requirements and specifications for the system.

Further this information has substantial commercial value as follows:

(a) Westinghouse plans to sell the use of similar information to its customers for purposes of plant construction and operation.

(b) Westinghouse can sell support and defense of safety systems based on the technology in the reports.

(c) The information requested to be withheld reveals the distinguishing aspects of an approach and schedule which was developed by Westinghouse.

Public disclosure of this proprietary information is likely to cause substantial harm to the competitive position of Westinghouse because it would enhance the ability of competitors to provide similar digital technology safety systems and licensing defense services for commercial power reactors without commensurate expenses. Also, public disclosure of the information would enable others to use the information to meet NRC requirements for licensing documentation without purchasing the right to use the information.

The development of the technology described in part by the information is the result of applying the results of many years of experience in an intensive Westinghouse effort and the expenditure of a considerable sum of money.

In order for competitors of Westinghouse to duplicate this information, similar technical programs would have to be performed and a significant manpower effort, having the requisite talent and experience, would have to be expended.

Further the deponent sayeth not.

PROPRIETARY INFORMATION NOTICE

Transmitted herewith are proprietary and/or non-proprietary versions of documents furnished to the NRC in connection with requests for generic and/or plant-specific review and approval.

In order to conform to the requirements of 10 CFR 2.390 of the Commission's regulations concerning the protection of proprietary information so submitted to the NRC, the information which is proprietary in the proprietary versions is contained within brackets, and where the proprietary information has been deleted in the non-proprietary versions, only the brackets remain (the information that was contained within the brackets in the proprietary versions having been deleted). The justification for claiming the information so designated as proprietary is indicated in both versions by means of lower case letters (a) through (f) located as a superscript immediately following the brackets enclosing each item of information being identified as proprietary or in the margin opposite such information. These lower case letters refer to the types of information Westinghouse customarily holds in confidence identified in Sections (4)(ii)(a) through (4)(ii)(f) of the affidavit accompanying this transmittal pursuant to 10 CFR 2.390(b)(1).

COPYRIGHT NOTICE

ENCLOSURE 4

Westinghouse Non-Proprietary Class 3

Shield Building Benchmarking Analysis - (Non-Proprietary)

BENCHMARKING ANALYSIS

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ENCLOSURE 6

Westinghouse Non-Proprietary Class 3

Shield Building Behavior and Design Philosophy – (Non-Proprietary)

SHIELD BUILDING BEHAVIOR AND DESIGN PHILOSOPHY

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ENCLOSURE 8

Westinghouse Non-Proprietary Class 3

Action Item 11 – (Non-Proprietary)

11. ***Consider use of ACI 318-08 CH 21. Indicate specific type used for Shield Building and if it is type 1, Westinghouse will provide a justification. Provide qualification and production criteria.*** a,c

[ ]

ENCLOSURE 10

Westinghouse Non-Proprietary Class 3

Action Item 12 – (Non-Proprietary)

12. ***Provide a typical load case at SSE to show the cross sectional forces for both the Standard (ANSYS) and Confirmatory (LS_DYNA) analyses.***

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ENCLOSURE 11

Study on Steel Plate Reinforced Concrete Panels Subjected to Cyclic In-plane Shear

Available online at www.sciencedirect.com
SCIENCE DIRECT®
ELSEVIER
Nuclear Engineering and Design 228 (2004) 225–244
Nuclear Engineering and Design
www.elsevier.com/locate/nucengdes

Study on steel plate reinforced concrete panels subjected to cyclic in-plane shear

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Abstract

This paper describes the derivation of the equation for evaluating the strength of steel plate reinforced concrete structure (SC) and the experimental results of SC panels subjected to in-plane shear.

Two experimental research programs were carried out. One was the experimental study in which the influence of the axial force and the partitioning web were investigated, another was that in which the influence of the opening was investigated.

In the former program, nine specimens were loaded in cyclic in-plane shear. The test parameters were the thickness of the surface steel plate, the effects of the partitioning web and the axial force. The experimental results were compared with the calculated results, and good agreement between the calculated results and the experimental results was shown.

In the later programs, six specimens having an opening were loaded in cyclic in-plane shear, and were compared with the results of the specimen without opening. FEM analysis was used to supplement experimental data. Finally, we proposed the equation to calculate the reduction ratio from the opening for design.

1. Introduction

The SC structure consists of a flat composite plate made by connecting a pair of surface steel plates in which stud bolts are built with partitioning webs and filling the boxes so-formed with concrete. The stud bolts operate as shear connectors between steel plate and concrete (Fig. 1).

The SC structure has several merits from the points of view of the structural aspects and the efficiency of execution works:

(1) As the surface steel plates are the form work in the execution work, the execution work periods will be reduced.
(2) No wooden form work is used, so the destruction of the environment will be reduced.
(3) They will be able to set up the mechanic support after concrete casting and it has flexibility to set up supports.

Previously, several papers for the study of SC structure were presented (Kaneuji et al., 1989; Akiyama et al., 1989, 1991). One of the authors presented the experimental results of SC panels and the basic equations that evaluate the strengths of SC panel was derived (Takeda et al., 1995).

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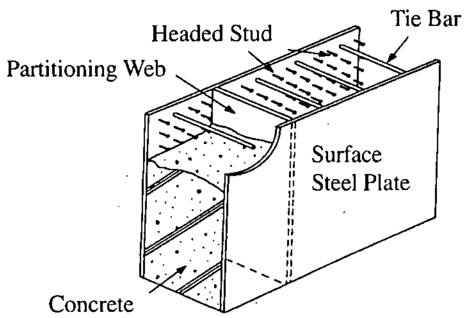

Fig. 1. Typical cross-section of SC.

However, these equations did not consider the influence of axial force. The effect of the partitioning web was not described at length, and the response of the SC panel with an opening had not been studied yet. Having an opening, the strength of the panel may be reduced, and stress concentration makes the surface plate yield earlier. The effect of the stress concentration may change the response of the panel having an opening. The investigation of the reduction ratio of an SC panel having an opening is beneficial for the design of SC structures.

Therefore, in this paper, the equation considers the influence of the axial force and the partitioning web of which the cross-sectional area was not equal in X- and Y-directions was derived.

Also two research programs were described. One was the experimental study in which the influence of the axial force and the partitioning web were investigated; another was one in which the influence of the opening was investigated. In the later program, considering the limitation of the experimental data, we tried to analyze the response of SC panels having an opening and then derived the equation of the reduction ratio for these panels.

In the conclusion, it is noted that the equations agree well with experimental results. A reduction ratio for the opening was proposed for design purposes.

2. Derivation

2.1. Assumption

The load–deformation relationship of an SC panel subjected in-plane shear is divided into three parts as follows:

(1) Elastic part (before cracking): Concrete and surface steel plates that compose the SC panel are found to be elastic. The partitioning web carries no stress. The stiffness of the SC panel in this part is given from the sum of the stiffness of concrete and surface steel plate. The perfect bond between surface steel plate and concrete is assumed.
(2) Post-cracking part (from post-cracking to surface steel plate yield): Cracks appear in concrete and the surface steel plates remain in elastic. Concrete struts carry only compressive stress and this compressive stress is in proportion to the tension stress carried by the surface steel plate and the partitioning web. These assumptions compose the truss analogy.
(3) Post-yield part: Tension failure of the surface steel plate and compressive failure of concrete determines the collapse of the panel.

2.2. Elastic shear modulus

From assumption (1), the elastic shear modulus for SC panel is derived. The relationship between shear load and shear strain,

$$q_c = G_c A_c \gamma, \quad q_s = G_s A_w \gamma \tag{1}$$

where G_c is shear modulus of concrete, A_c is the cross-sectional area of concrete, G_s is shear modulus of steel, γ is shear strain and A_w is the cross-sectional area of total surface steel plate. Then total shear load (Q_c) is,

$$\frac{Q_c}{A_o} = \frac{(q_c + q_s)}{A_o} = \left(\frac{G_c A_c}{A_o} + \frac{G_c A_w}{A_o}\right)\gamma \tag{2}$$

where A_o is the cross-sectional area of the panel. Therefore, elastic shear modulus of SC panel (G_c) is,

$$G_c = \frac{G_c A_c}{A_o} + \frac{G_s A_w}{A_o} \tag{3}$$

2.3. Post-cracking shear modulus

From assumption (2), the post-cracking shear modulus for SC panel is derived. The relationship between shear load and shear strain,

$$q_{cy} = G_{cy} A_c \gamma, \quad q_c = G_s A_w \gamma \tag{4}$$

where G_{cy} is the post-cracking shear modulus of concrete (truss analogy). Then total shear load (Q_y) is,

$$\frac{Q_y}{A_o} = \frac{(G_{cy}A_c + G_s A_w)\gamma}{A_o} = \frac{G_y \gamma}{A_o} \tag{5}$$

The post-cracking shear modulus of concrete is derived as follows.

Lateral and longitudinal bars of truss analogy are made of the surface steel plate and the partitioning web. In X- and Y-directions, the axial forces can be expressed as follows.

$$N_x = N_{wx} + N_{px}, \quad N_y = N_{wy} + N_{py} \tag{6}$$

where N_x and N_y are the axial forces of the panel for X- and Y-directions, respectively. N_{wx} and N_{wy} are the axial forces of the surface steel plate for X- and Y-directions, respectively. N_{px} and N_{py} are the axial forces of the partitioning web for X- and Y-directions, respectively.

From condition of equilibrium and compatibility, the carrying load of the surface steel plate or the partitioning web is derived. Hence, the axial force of the surface steel plate and the partitioning web are expressed as follows.

$$N_{wx} = \frac{1 + p_{dy} + v_s p_{dx}}{(1 + p_{dx})(1 + p_{dy}) - v_s^2 p_{dx} p_{dy}} N_x,$$

$$N_{wy} = \frac{1 + p_{dx} + v_s p_{dy}}{(1 + p_{dx})(1 + p_{dy}) - v_s^2 p_{dx} p_{dy}} N_y$$

$$N_{px} = \frac{(1 - v_s) p_{dx}\{1 + (v_s) p_{dy}\}}{(1 + p_{dx})(1 + p_{dy}) - v_s^2 p_{dx} p_{dy}} N_x,$$

$$N_{py} = \frac{(1 - v_s) p_{dy}\{1 + (1 + v_s) p_{dx}\}}{(1 + p_{dx})(1 + p_{dy}) - vs^2 p_{dx} p_{dy}} N_y \tag{7}$$

where $p_{dx} = A_{px}/A_w$, $p_{dy} = A_{py}/A_w$ in which A_{px} and A_{py} are the cross-sectional areas of the partitioning web in X- and Y-directions, respectively and v_s is Poisson's ratio for steel plates.

Equivalent Young's modulus of the lateral and longitudinal bars of truss mechanism are determined as follows.

$$s_x = \frac{N_x}{(A_{sx}\varepsilon_x)}, \qquad E_{sy} = \frac{N_y}{(A_{sy}\varepsilon_y)} \tag{8}$$

Eliminating strains, ε_x, ε_y,

$$E_{sx} = \frac{(1 + p_{dx})(1 + p_{dy}) - v_s^2 p_{dx} p_{dy}}{(1 + p_{dx})(1 - v_s)\{1 + (1 + v_s) p_{dy}\}} E_s,$$

$$E_{sy} = \frac{(1 + p_{dx})(1 + p_{dy}) - v_s^2 p_{dx} p_{dy}}{(1 + p_{dy})(1 - v_s)\{1 + (1 + v_s) p_{dx}\}} E_s \tag{9}$$

Finally we obtain the dimensions of the bars in Table 1. In this table, $Q_{truss} = G_{cy} A_c \gamma$.

The relationship between the lateral deformation, δ, and the dimensions of the bars is expressed as follows. The direction of the diagonal bars are assumed to be at an angle of 45°.

$$\delta = \sum_{i=1}^{5} \frac{N_i \overline{N}_i}{A_i E_i} L_i \tag{10}$$

Substituting the dimensions expressed in Table 1 in Eq. (10), gives the post-cracking shear modulus, then:

$$G_{cy} = \frac{E_s}{(4/A_c)(E_s/E_c) + (1 - v_s/A_w)(2 + (1 + v_s)(p_{dx} + p_{xy}))/((1 + p_{dx})(1 + p_{dy}) - v_s^2 p_{dx} p_{dy})} \frac{1}{A_o} \tag{11}$$

In accordance with the condition of the partitioning web, Eq. (4) is simplified as follows.

(1) Without partitioning web ($p_{dx} = p_{dy} = 0$)

$$G_{cy} = \frac{E_s}{(4/A_c)(E_s/E_c) + 2(1 - v_s/A_w)} \frac{1}{A_o} \tag{12}$$

(2) The partitioning web is arranged at the only longitudinal direction ($p_{dx} = 0$)

$$G_{cy} = \frac{E_s}{(4/A_c)(E_s/E_c) + (1 - v_s/A_w) \times (2 + (1 + v_s) p_{dy})/1 + p_{dy}} \frac{1}{A_o} \tag{13}$$

(3) The cross-sectional area of the lateral partitioning web is equal to that of the longitudinal partitioning web ($p_{dx} = p_{dy}$)

$$G_{cy} = \frac{E_s}{(4/A_c)(E_s/E_c) + (1 - v_s/A_w) \times (2/1 + (1 - v_s) p_{dx})} \frac{1}{A_o} \tag{14}$$

Table 1
The Dimensions of the bars

	Longitudinal bars	Lateral bars	Diagonal bars
Cross-sectional Area, A_i	$(A_w + A_{py})/2$	$(A_w + A_{px})/2$	$A_c/\sqrt{2}$
Young's modulus, E_i	$\mu_y E_s$	$\mu_x E_s$	E_c
Length, L_i	L	L	$\sqrt{2}L$
Nodal force, N_i	$Q_{truss}/2$	$Q_{truss}/2$	$-\sqrt{2}Q_{truss}$

where

$$\mu_y = \frac{(1+p_{dx})(1+p_{dy}) - v_s^2 p_{dx} p_{dy}}{(1+p_{dy})(1-v_s)\{1+(1+v_s)p_{dx}\}}, \quad \mu_x = \frac{(1+p_{dx})(1+p_{dy}) - v_s^2 p_{dx} p_{dy}}{(1+p_{dx})(1-v_s)\{1+(1+v_s)p_{dy}\}}.$$

2.4. Cracking strength

If the axial force is loaded at one direction, the cracking shear stress τ_c is expressed as follows.

$$\tau_c^2 = \sigma_t^2 + \sigma_t \sigma_v \tag{15}$$

where σ_t is the tension stress at cracking of concrete, σ_v is the nodal stress. The tension stress at cracking of concrete may be estimated as follows (Collins and Michell, 1991);

$$\sigma_t = 0.33\sqrt{F_c} \qquad (\text{unit}: \text{Mpa}) \tag{16}$$

where F_c is the compressive strength of concrete. Then, the cracking shear stress subjected axial force will be presented as follows;

$$\tau_c = \sqrt{0.33\sqrt{F_c}\left(0.33\sqrt{F_c} + \sigma_v\right)} \tag{17}$$

Substituting Eqs. (17) in (2) gives,

$$Q_c = \left(A_c + \frac{G_s}{G_c}A_w\right)\sqrt{0.33\sqrt{F_c}\left(0.33\sqrt{F_c} + \sigma_v\right)} \tag{18}$$

2.5. Yield strength

Yield strength is determined as the stress of the surface steel plate satisfying von-Mises' yield criterion.

$$\sigma x^2 - \sigma x \sigma y + \sigma y^2 + 3\tau x y^2 = \sigma y^* \tag{19}$$

where σy^* is the yield stress of the steel.

The effect of the axial force for yield strength is assumed to be negligible. Therefore, stresses of the surface steel plate are,

$$\sigma x = \frac{c_{qc}}{A_w}\frac{1 + p_{dy} + v_s p_{dx}}{(1+p_{dx})(1+p_{dy}) - v_s^2 p_{dx} p_{dy}} Q_y$$

$$\sigma y = \frac{c_{qc}}{A_w}\frac{1 + p_{dx} + v_s p_{dy}}{(1+p_{dx})(1+p_{dy}) - v_s^2 p_{dx} p_{dy}} Q_y$$

$$\tau_{xy} = \frac{c_{qs}}{A_w} Q_y \tag{20}$$

where

$$c_{qc} = \frac{G_{cy}A_c}{G_{cy}A_c + G_s A_w}, \quad c_{qs} = \frac{G_s A_w}{G_{cy}A_c + G_s A_w}$$

Substituting Eq. (19) in (20) gives yield strength (Q_y). Normally, the result is too complex to represent analytically. Therefore, a simplified condition, in which the cross-sectional area of the lateral partitioning web is equal to that of the longitudinal partitioning web, is considered. Hence,

$$Q_y = \frac{k(G_c + G_s)}{\sqrt{G_c^2 + 3k^2 G_s^2}}\sigma_y^* \frac{A_w}{2} \tag{21}$$

where $k = 1 + (1 - v_s)p_{dx}$.

2.6. Maximum strength

As the maximum strength, the equation for the maximum strength of RC structure is applied similarly. Then,

$$Q_u = (A_w + A_p)\sigma_y^*, \qquad \gamma_u = 6.0 \times 10^{-3} \tag{22}$$

3. Research program I

3.1. Experimental program

The experimental program I involved the testing of nine SC panels. These panels were subjected to cyclic in-plane shear. Details pertaining to the panel

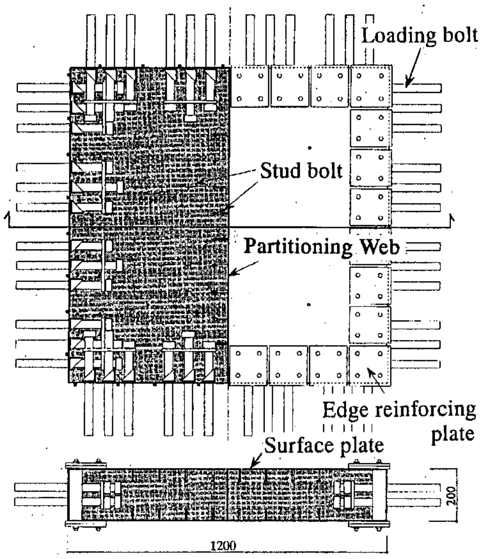

Fig. 2. Specimen properties (S3-00PS).

construction were shown in Fig. 2. The test panels were 1200 mm × 1200 mm in plan dimension, with a thickness of 200 mm. Only two specimens had a partitioning web around the center of the panel. The stud bolts were welded on the surface steel plate at intervals of a ratio in which the pitch (B) of the stud bolts was divided by the thickness (t) of the surface steel plate, $B/t = 30$. This ratio, $B/t = 30$, was decided by the experimental result that the buckling of the surface steel plate did not appear before yield (Sasaki et al., 1995).

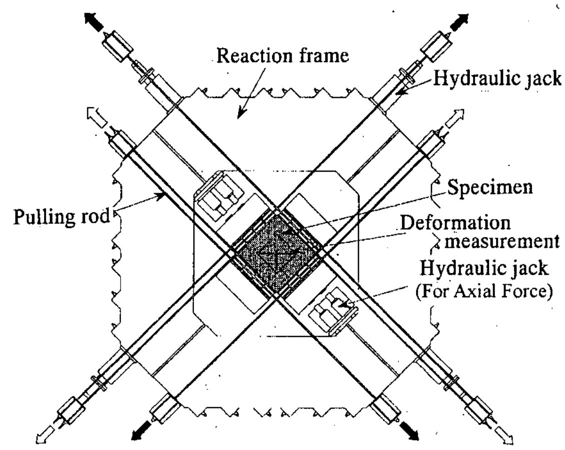

Fig. 3. Load setup.

Specimen test parameters are given in Table 2. The test parameters were the thickness of the surface plate, the partitioning web and the axial force. The surface steel plate varied three thickness (2.3, 3.2, and 4.5 mm). There were two types of partitioning web. One was the web that the stud bolts were welded, and another was the web without stud bolts. The axial force acted three levels (0.0, 1.47, and 2.94 MPa).

Uniform in-plane forces were applied to the panels using the shear bolts test facility (Fig. 3). The facility was comprised of a self-reacting frame containing eight hydraulic jacks. The loading stages are shown in Table 3.

The concrete for each panel was hand-batched using 10 mm stone. The panels and test cylinders were

Table 2
Test specimen (research program I)

Specimens	Surface steel plate (t) (mm)	Headed stud bolt			Nodal force (MPa)	Partitioning web
		Pitch in welding (B) (mm)	Diameters (mm)	B/t		
S2-00NN	2.3	70	4	30	0.0	
S2-15NN					1.47	
S2-30NN					2.94	
S3-00NN	3.2	100	5	31	0.0	
S3-15NN					1.47	
S3-30NN					2.94	–
S3-00PS					0.0	Studs were welded
S3-00PN						Without studs
S4-00NN	4.5	135	9	30		–

Table 3
Loading stages (research program I)

Loading stages	①	②③	④⑤	⑥	⑦	⑧⑨	⑩	⑪	⑫
Shear strain ($\times 10^{-3}$)	0.5	1.0	2.0	1.0	3.0	4.0	2.0	6.0	20.0

sealed and cured for 7 days and tested at approximately 80–180 days of age. The properties of concrete, determined from cylinders tested at the time of the panel tests, are listed in Table 4.

Electric strain gage measurements were made in the X-, Y-, and diagonal directions on both sides of the steel plates.

3.2. Test results

A summary of test results is given in Table 4. The response of all the specimens showed good ductility. The ultimate shear strain reached and exceeded at the level of 6.0×10^{-3}. Especially, the specimen which had a partitioning web in the center of the panel developed this tendency.

Fig. 4 shows the envelopes of the load–deformation relationships for the purpose of investigating the influence of the parameters.

Taking notice of the influence of the surface plate thickness, as the surface steel plate become thicker, the elastic shear modulus, the post-cracking shear modulus, the yield strength and the maximum strength become higher. The shear strain at the maximum strength become smaller, as the surface steel plate become thicker. If one defines ductility as ultimate to yield strain, the ductility for panels S2-00NN, S3-00NN and S4-00NN are 3.76, 2.01 and 2.82, respectively. There is the fact that increased steel plate thickness appears to reduce ductility. This phenomenon was to be explained as follows. When the steel plate thickness was increased, the shear capacity shared by the steel plates was increased but the shear capacity shared by the concrete was not increased (i.e. this experiment was done under the common panel thickness.). Therefore, if the steel plate thickness of a SC panel was increased, the concrete was to be more damaged. So, the shear strain at the maximum strength was reduced and the ductility was reduced.

The axial force had no effect upon the elastic shear modulus. However, the post-cracking shear modulus, under the higher axial force, become slightly smaller. The cracking strength was clearly affected by the axial force. However, the yield strength and the maximum strength was not so much affected by the axial force. The influence for the shear strain at the yield strength and the maximum strength was not clear.

The partitioning web made the post-cracking shear modulus rigid. With the amount of the partitioning web in this experiment, the influence for the yield strength and the maximum strength was not remarkable. The maximum strengths of these three specimens were almost the same. However, the shear strains at the maximum strength were different. The slip between concrete and partitioning web was supposed to be the reason of this result. The concrete of S3-00PN and S3-00PS were separated by the partitioning webs. Then, subjected to in-plane shear, the deformation takes place like that shown in Fig. 5. If the partitioning web had stud bolts (like panel S3-00PS), the slip would be reduced and the shear strain at the maximum strength would be smaller than that of the no stud bolt case.

The principal strains of the surface steel plate were shown in Fig. 6. Except the specimens that the axial force was loaded, the direction of the principal strain was almost 45 °C.

In this experiment, buckling of the surface steel plate before yielding was not observed. Therefore, the ratio, $B/t = 30$, between the stud bolt pitch and the thickness of the surface plate is considered as a good standardization for the control of buckling of the surface steel plate for SC panels subjected in-plane shear.

3.3. Consideration

Generally, as the axial force increases, the cracking strength becomes higher. If the axial force is loaded in one direction, the cracking shear stress τ_c is expressed as follows.

$$\tau_c^2 = \sigma_t^2 + \sigma_t \sigma_v \tag{23}$$

where σ_t is the tension stress at cracking of concrete, σ_v is the nodal stress. Then, from Eq. (16), the

Table 4
Experimental results

Specimen	Steel		Concrete	Elastic shear modulus	Post-cracking shear modulus	Cracking strength		Yield strength		Maximum strength	
	Yield stress, Young's modulus (MPa)	$A_w \times A_p$ (cm^2)	Compressive strength, tangential stiffness (MPa)	G_c ($\times 10^3$ MPa)	G_y ($\times 10^3$ MPa)	Q_c (kN)	γ_c ($\times 10^{-3}$)	Q_y (kN)	γ_y ($\times 10^{-3}$)	Q_u (kN)	γ_u ($\times 10^{-3}$)
S2-00NN	340 (1.97×10^5)	53.5 (17.1)	42.2 (2.72×10^4)	12.4	4.16	293	0.115	2290 (−2110)	2.50 (−1.99)	2960 (−2780)	9.41 (−6.12)
S2-15NN			41.6 (2.77×10^4)	13.2	4.14	433	0.133	2330 (−2290)	2.71 (−2.21)	3110 (−2930)	10.00 (−6.02)
S2-30NN			42.0 (2.79×10^4)	16.4	3.69	542	0.168	2490 (−2570)	3.01 (−2.41)	3110 (−3200)	10.48 (−6.03)
S3-00NN	351 (1.99×10^5)	75.4 (16.9)	41.9 (2.71×10^4)	12.9	4.88	311	0.134	3070 (−3070)	3.01 (−2.00)	3610 (−3430)	6.05 (−6.03)
S3-15NN			41.6 (2.67×10^4)	13.1	4.29	384	0.141	3130 (−3120)	2.99 (−3.01)	3760 (−3330)	7.99 (−6.01)
S3-30NN			40.1 (2.70×10^4)	11.9	4.67	385	0.186	3170 (−3080)	2.80 (−2.96)	3730 (−3550)	5.57 (−5.63)
S3-00PS		75.4 (25.4)	41.9 (2.71×10^4)	13.1	5.81	350	0.141	2680 (−2640)	1.93 (−1.97)	3580 (−3220)	10.87 (−5.98)
S3-00PN			39.9 (2.72×10^4)	16.4	4.92	271	0.113	2350 (−2390)	2.01 (−2.03)	3510 (−3060)	17.00 (−6.02)
S4-00NN	346 (2.07×10^5)	104.9 (16.7)	42.8 (2.76×10^4)	16.4	8.22	349	0.103	3510 (−3560)	2.01 (−2.00)	4100 (−3790)	5.67 (−4.00)

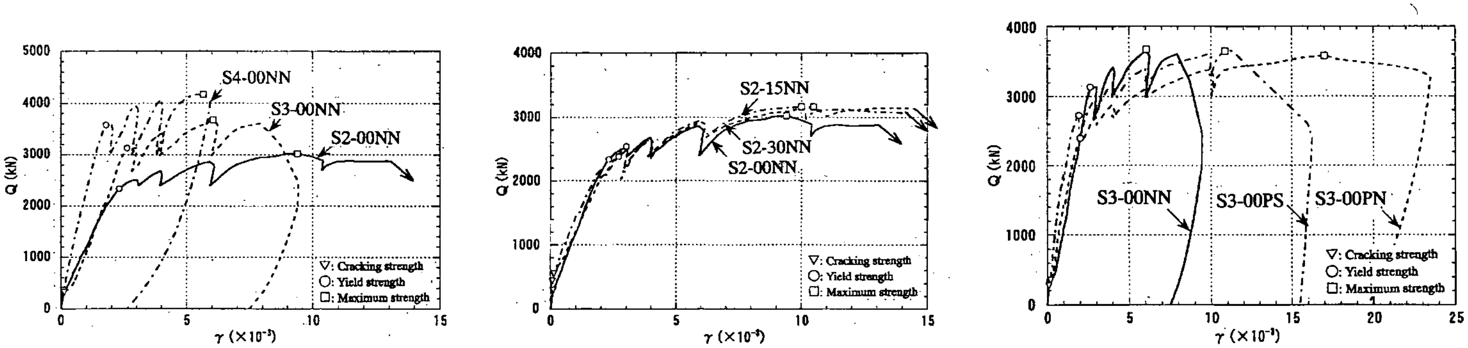

Fig. 4. Comparison about envelopes.

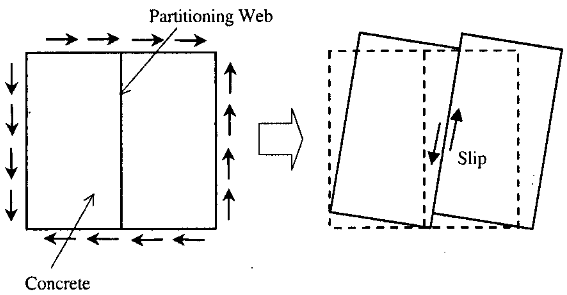

Fig. 5. Deformation of SC panel having a partitioning web.

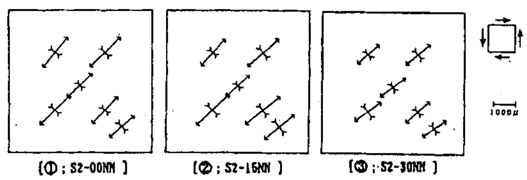

Fig. 6. Principal strains (before cracking).

direction of the principal tension stress will be presented as follows;

$$\theta = \frac{1}{2}\tan^{-1}\frac{2\tau_c}{\sigma_v} = \frac{1}{2}\tan^{-1} 2\sqrt{\left(\frac{0.33\sqrt{F_c}}{\sigma_v}\right)^2 + \frac{0.33\sqrt{F_c}}{\sigma_v}} \qquad (24)$$

The directions of cracks occur at an angle of the direction of the principal tension stress. Therefore, from Eq. (24), the calculation results of the cracking angle are shown as black points in Fig. 7. In this figure, the range of experimental results are shown as bands (for example, in S2-30NN, the range of experimental results is from 52 to 63°). This figure shows the influence of the axial force for the cracking angle, which the axial force become bigger, the cracking angle become larger. Moreover, this figure shows the good agreement between the results of Eq. (24) and the experimental results.

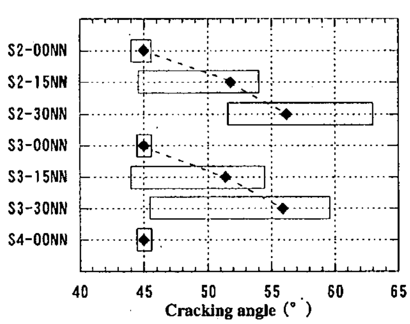

Fig. 7. Cracking angles.

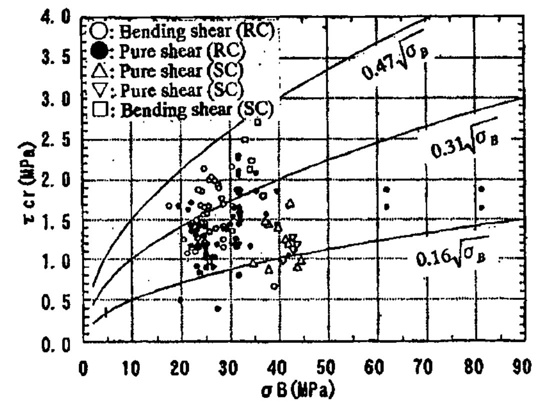

Fig. 8. Cracking stress.

Fig. 8 shows the cracking stress with other experimental results of SC structures and reinforced concrete (RC) structures. The cracking strengths are scattered widely, but the tendency of the cracking strength of SC structures and RC structures has no difference. The results of a panel subjected to in-plane shear are smaller than the results of a wall subjected to bending-shear load. The differences of these two results are considered as the influence of rigid frame. The experimental result described in this paper is the results under pure shear loading. However, real structure has flange walls, rigid floor slabs and frame girders. Cracking strength is considered to be the most sensitive strength in these elements. Therefore, the cracking strength of the wall subjected to bending-shear load is more similar to the actual situation than the cracking strength of the panel subjected to in-plane shear load. In conclusion, the cracking strength of SC structure is to be expressed Eq. (18) in the same way as RC structures.

Fig. 9 shows the relationship between the yield strength and the amount of the surface steel plate ($p_w\sigma_y^*$: $p_w = A_w/A_o$). The yield strength increase linearly as the amount of the surface steel plate increase.

3.4. *Applicability of equations*

Fig. 10 shows the comparison between the calculated results and the experimental results. Except the

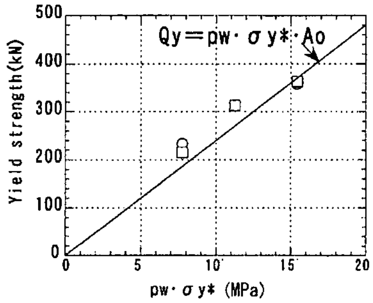

Fig. 9. $Q_y - p_w\sigma_y^*$ relationship.

cracking strength, the figure shows the good agreement between the calculated results and the experimental ones.

Fig. 11 shows the comparison in the point of view of the envelope of the load–deformation relationships. The calculated results presented good accuracy for the experimental ones.

4. Research program II

4.1. Experimental program

The experimental program II involved the testing of six SC panels having an opening. These panels were subjected to cyclic in-plane shear. The test panels were 1200 mm × 1200 mm in plan dimension, with a thickness of 200 mm. Typically, the panels had an opening at the center of the panel. There were two types of openings: a circular opening and a square opening. Whichever the types of the opening, the square root of the ratio of the opening area divided by the area of the SC panel was 0.15 in common. The corner of the square opening had a small curvature.

The results of these six specimens will be compared with the specimen without opening involved research program I, which called S3-00NN.

The reinforcement around the opening had two variations made of steel plate: sleeve reinforcement and plate reinforcement. The plate of the sleeve had 4.5 mm thickness for circular openings and 2.3 mm thickness for square openings. The plate reinforcement has a common thickness for the surface plate (3.2 mm). The reinforcement area had a half of the opening diameter width.

The reinforcing steel plate was 3.2 mm in common. Details pertaining to the panel construction were shown in Fig. 12. Specimen test parameters are given in Table 5.

The concrete for each panel was hand-batched using 10 mm stone. The panels and test cylinders were

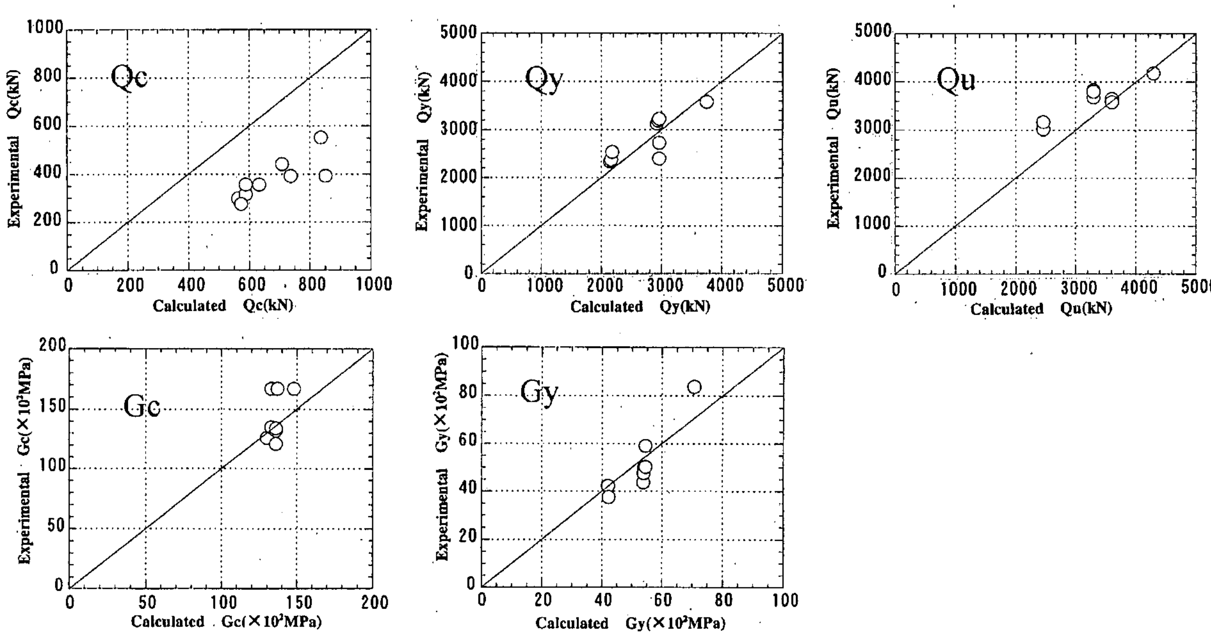

Fig. 10. Experimental results and calculated results (strength and stiffness).

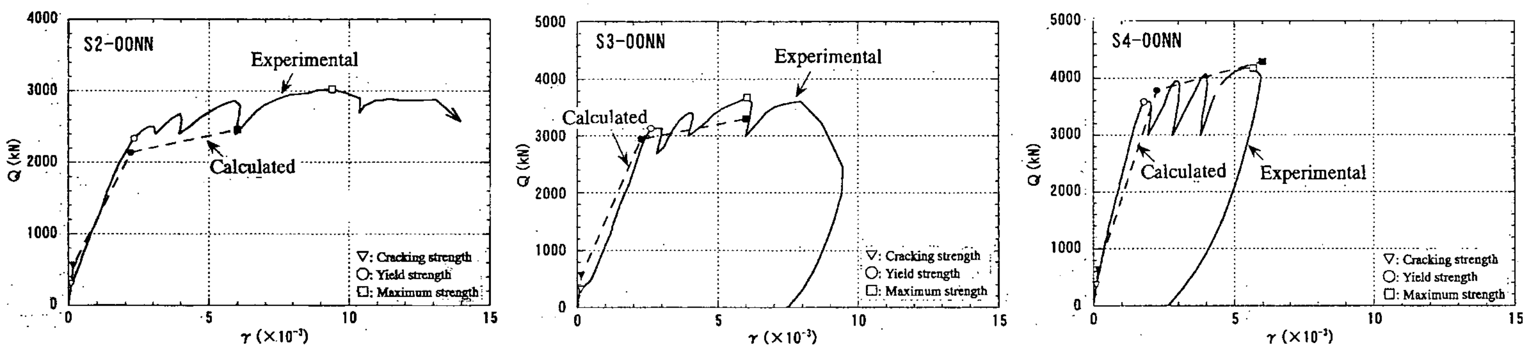

Fig. 11. Experimental results and calculated results (envelopes).

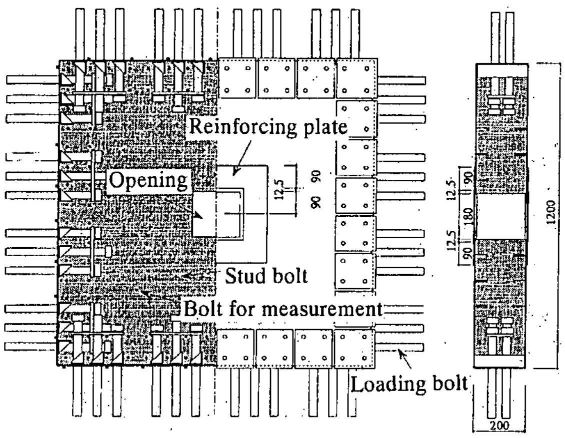

Fig. 12. Specimen properties (S3-00RP).

sealed and cured for 7 days and tested at approximately 80–180 days of age. The properties of concrete, determined from cylinders tested at the time of the panel tests, are listed in Table 7.

Uniform in-plane forces were applied to the panels using the shear bolts test facility (Fig. 3). The facility was comprised of a self-reacting frame containing four hydraulic jacks. The loading stages are shown in Table 6. To investigate the condition under large scale deformation, three loading stages were added for the loading stages of three specimens having a square opening (S3-00RN, S3-00RR, S3-00RP) and one of the specimens having a circular opening (S3-00HP).

The wire strain gage was used for the investigation of strain concentration near the opening and the inclination to decrease of strain concentration from the edge of the opening.

Electric strain gage measurements were made in the X-, Y-, and diagonal directions on both sides of the steel plates.

4.2. Test results

A summary of test results is given in Table 7. In all specimens, the cracking strength and the yield strength each appeared at almost the same shear strain.

The shear strain at the cracking strength of the standard specimen was almost equal to those of the specimens having a opening, and the shear strain at the yield strength of the standard specimen was almost equal to those of the specimens having an opening, too. However the shear strains at the maximum strength of the specimens having an opening were almost two or three times as large as those of the standard specimen. Therefore the discussion of the reduction ratio of the maximum strength, which is based on the standpoint putting only the strength in question not the shear strain, is not good for the determination of the skeleton curve of SC panels having an opening.

Therefore we will determine the nominal ultimate strength. Other experimental study which planed for H shaped specimens constructed by SC shown the shear strain of 6.0×10^{-3} at the maximum strength (Sasaki

Table 5
Test specimens (research program II)

Specimen	Thickness of surface plate (t)	Stud			Types of opening	Reinforcement
		Pitch (B)	Diameter	B/t		
S3-00NN					–	–
S3-00HN					Circular opening (diameter = 207)	–
S3-00HR						Sleeve
S3-00HP	3.2	100	5	31		Plate
S3-00RN					Square opening (180 × 180)	–
S3-00RR						Sleeve
S3-00RP						Plate

Table 6
Loading stages (research program II)

Loading stages	①	②③	④⑤	⑥	⑦	⑧⑨	⑩	⑪⑫	⑬⑭	⑮
Shear strain ($\times 10^{-3}$)	0.5	1.0	2.0	1.0	3.0	4.0	2.0	6.0	12.0	17.0

Table 7
Experimental results

Specimen	Material properties				Experimental results					
	Concrete		Steel plate		Cracking strength	Yield strength	Ultimate strength	Maximum strength	Elastic stiffness	Yield stiffness
	σ_B (σ_t)	E_c (ν_c)	σ_y	E_s (ν_s)	Q_c (γ_c)	Q_y (γ_y)	Q_u^* (γ_u^*)	Q_u (γ_u)	G_c	G_y
S3-00NN	41.9 (2.83)	2.71 (0.200)	349	1.99 (0.274)	311 (0.134)	3070 (3.01)	3610 (6.05)		12.9	4.26
S3-00HN	42.4 (3.14)	2.70 (0.200)	348	1.98 (0.282)	254 (0.127)	2230 (2.01)	2920 (6.00)	3240 (19.1)		
S3-00HR	42.4 (3.24)	2.71 (0.210)	355	2.00 (0.279)	270 (0.082)	2710 (3.01)	3060 (5.63)	3580 (17.9)	14.9	3.75
S3-00HP	31.7 (2.85)	2.72 (0.193)	380	1.93 (0.267)	251 (0.129)	2510 (2.38)	3470 (5.94)		9.53	4.39
S3-00RN	32.9 (2.78)	2.66 (0.194)	386	2.00 (0.269)	146 (0.075)	2160 (2.45)	2800 (6.10)	3020 (11.9)	10.8	3.67
S3-00RR	32.1 (2.76)	2.49 (0.186)	385	1.99 (0.272)	250 (0.082)	2170 (2.43)	2930 (6.05)	3350 (11.4)	12.7	3.72
S3-00RP	32.7 (2.76)	2.66 (0.193)	369	2.01 (0.281)	246 (0.115)	2500 (2.86)	3060 (6.15)	3210 (11.5)	11.8	3.65

Note. Material properties unit $\sigma_B \sigma_t \sigma_y$ (MPa), E_c ($\times 10^4$ MPa), E_s ($\times 10^4$ MPa).
Ultimate strength: strength at shear strain of 6.0×10^{-3}.
Experimental results: unit $Q_c Q_y Q_u$ (kN), $\gamma_c \gamma_y \gamma_u$ ($\times 10^{-3}$), $G_c G_y$ ($\times 10^3$ MPa).

et al., 1995). Considering the results of this experimental study and the result of the standard specimen from this study, we determine the ultimate strength at the shear strain of 6.0×10^{-3}.

By the reinforcement around the opening, the strength of the specimens having an opening tends to increase in comparison with the strength of those without reinforcement.

4.2.1. Load–deformation relationship

Shear load–deformation response are given in Fig. 13. The load–deformation response were very ductile. Except for the standard specimen, ultimate response of deformation for six specimens having an opening were determined by the limitations of the deformational capacity of the used facility (around 17.0×10^{-3}). The specimens which reinforced with steel plate observed small cracking in the corner of the square opening at the loading stage of the shear strain of 12.0×10^{-3}. But the cracking had not immediately grown at the end of the all load stages.

The comparisons of the responses from the point of view of the load–deformation envelope are shown in Fig. 14. In the range of the reinforcement of this experiment, the plate reinforcement was more effective than the sleeve reinforcement. Then a discussion of the relationship between the amount of the reinforcement and its effectiveness will be made.

4.2.2. Ultimate condition of concrete

The ultimate cracking pattern of concrete is shown in Fig. 15. After all loading stages, the surface plates were driven from the specimen to observe the ultimate cracking pattern of concrete. Fig. 15 shows that the cracking pattern of SC panels is very sparse.

4.2.3. Strain near opening

The strain distribution near the opening (ε_θ) is shown in Fig. 16. In these figures, the dotted lines show the distributions of the strains calculated elastically, and the black dots show the experimentally measured results for various stages of loading. The figures show that, after yielding of the steel plate, the effect of the strain concentration close to the opening is significant, but that it is negligible far from the opening (530 mm from the center) at all loading levels. The strain concentration near the opening and shear strain relationship is shown in Fig. 17.

The strain near opening of the specimen without reinforcement increased almost linearly by the shear strain increase. But the strain of the specimen with reinforcement checked the increase of the shear strain.

4.3. Consideration of the experimental results

4.3.1. Reduction ratio due to opening for stiffness or strength

We determined the reduction ratio due to the opening for tangential stiffness, yield strength and ultimate strength (K_1, K_2, and K_3). Fig. 18 shows the determination of each of them. In the experiments, the concrete properties were not uniform. Therefore we tried to standardize the reduction ratio by way of the multiplication of the square root of the concrete strength

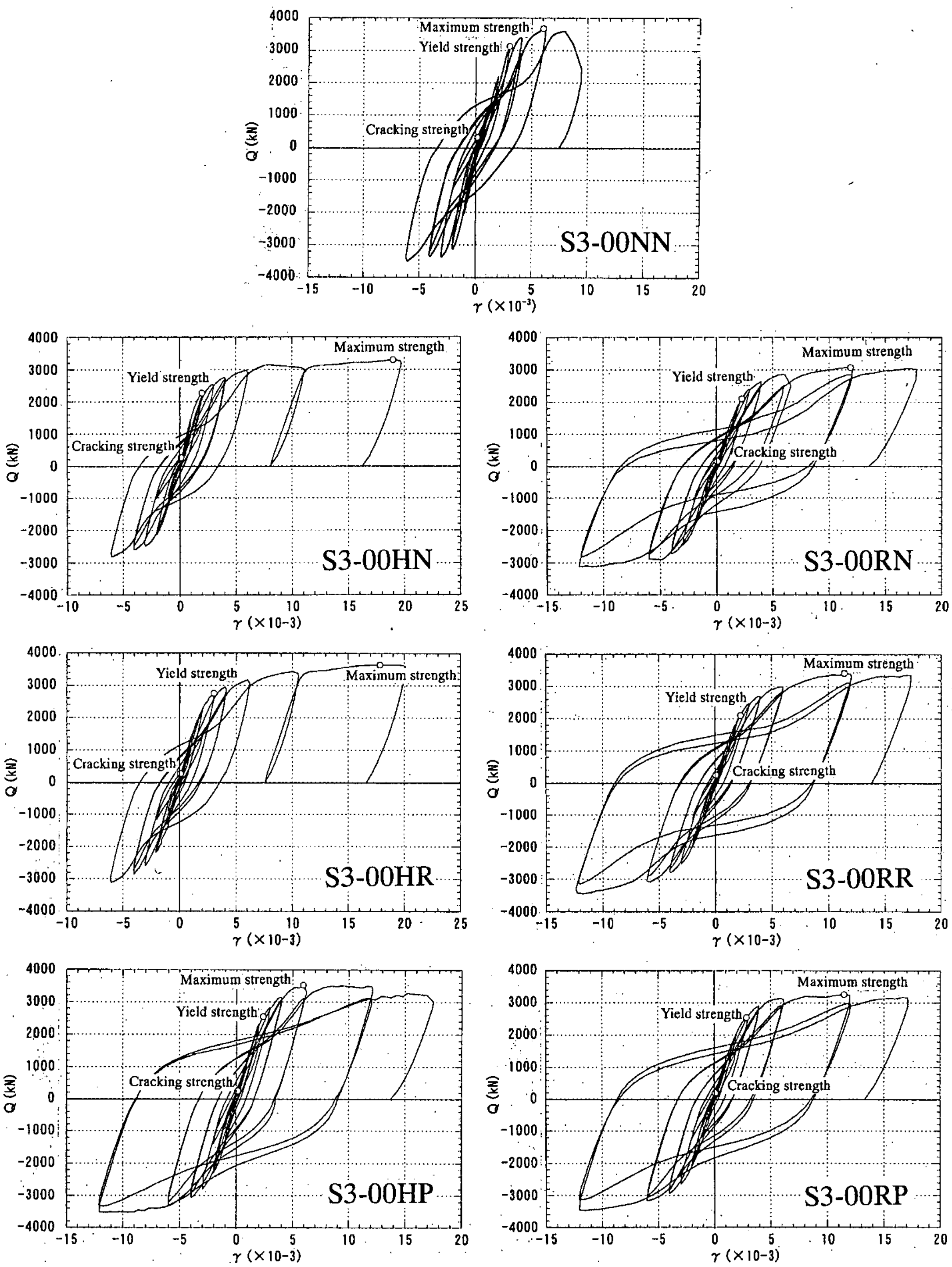

Fig. 13. Load–deformation relationship.

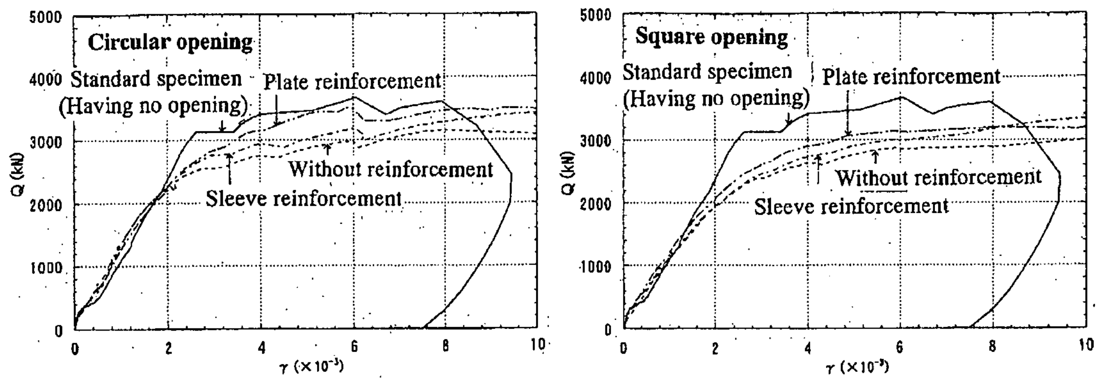

Fig. 14. Comparison about envelopes.

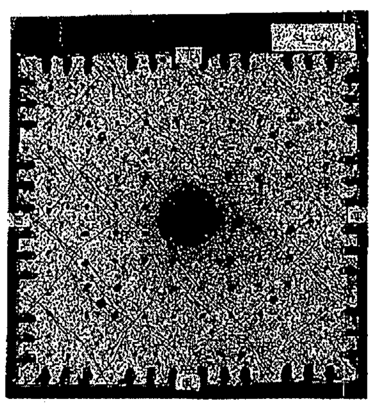

Fig. 15. Ultimate cracking patterns of concrete.

of the standard specimen divided by that of the specimen having an opening. Fig. 19 shows the results of the calculation for the reduction ratios. The results of the calculation were based on the reduction ratio for reinforced concrete wall. That is,

(1) Design recommendation in Japan for the wall having opening (Architectural Institute of Japan, 1999) is,

$$\gamma_{rc} = \min\left(1 - \frac{l_o}{l}, 1 - \sqrt{\frac{h_o l_o}{hl}}\right) \tag{25}$$

in except that the term $\sqrt{h_o l_o / hl}$ shall not exceed 0.4.

(2) Ono and Tokuhiro (1992) proposed as follows,

$$\gamma_{ot} = \sqrt{\frac{\sum A_c}{hl}} \tag{26}$$

where h is the height of wall and l is the length of wall, and h_o is the height of opening and l_o is the length of opening. A_e is the compression field area of the wall.

The results of Eqs. (25) and (26) show good agreement with the experimental results. So, for SC panels, these equations for RC walls represent good applicability.

4.3.2. Effectiveness of the amount of reinforcement

The effectiveness of reinforcement in terms of the total amount is shown in Table 8. The effectiveness for yield strength and ultimate strength is focussed herein. We consider the effectiveness as the amount of steel reinforcement. Hence,

$$\text{Strength per total amount of reinforcement} = \frac{\Delta q}{q_s} \tag{27}$$

where Δq is the difference in strength between the reinforcing specimen having an opening and the specimen having an opening without reinforcement. q_s is the total amount of the reinforcement.

Table 8 shows the effectiveness of the reinforcement of the steel plate is more effective than that of the sleeve reinforcement. So, finite element analysis is discussed in the next chapter for the reinforcement using the steel plate.

4.4. Finite element analysis

4.4.1. Introduction

The basic response of SC panels having an opening and subjected to in-plane shear was investigated

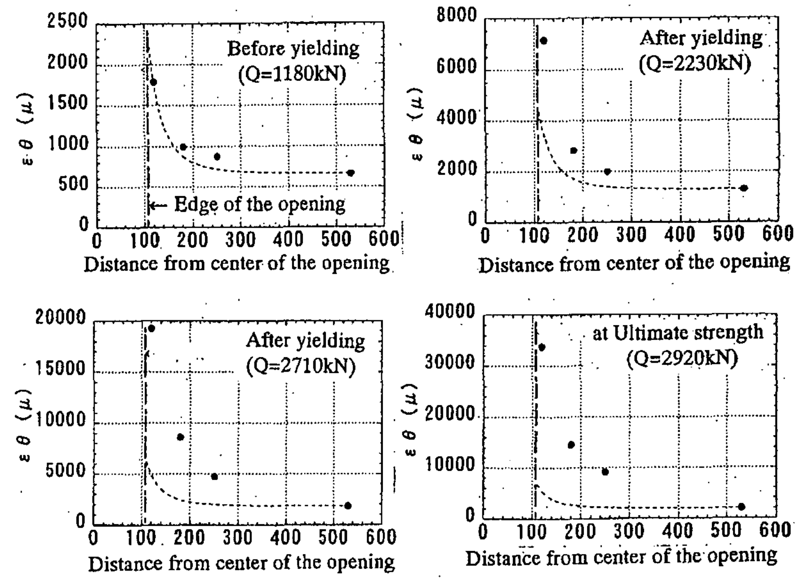

Fig. 16. Strain near opening.

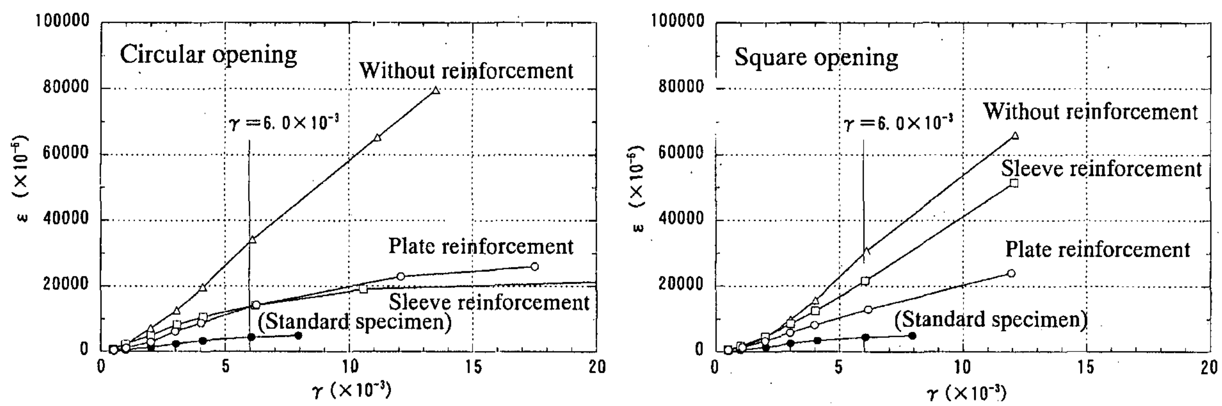

Fig. 17. Strain near opening and shear strain.

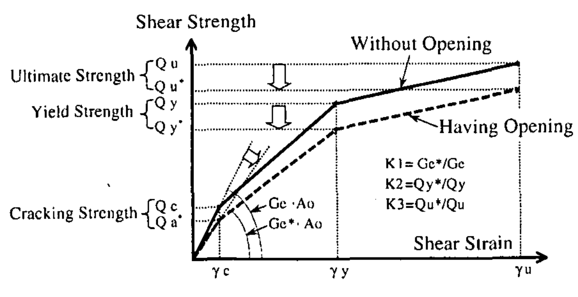

Fig. 18. Determination of reduction ratios.

using the approach described above. But experimental data was insufficient to describe the behavior at the reinforcement. Then the finite element analysis was used for the investigation of the relationship between the reinforcing area and reinforcing thickness. Loads were monotonically increased in constant ratio until failure occurred.

4.4.2. Analytical model

The analytical models were 1200 mm × 1200 mm in plan dimension with a thickness of 200 mm, the same

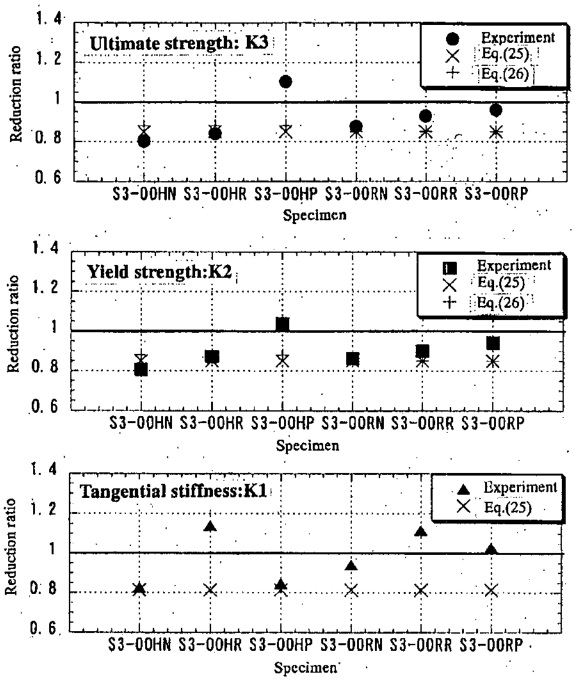

Fig. 19. Experimental results and calculative results.

as the test panels. But in this analysis, using symmetry, one-quarter the test panel was modeled (Fig. 20). The thickness of the one side surface plate was modeled as 2.00 mm. The scale of the opening was the same as the test panels. The partitioning web was modeled as a rod element because the partitioning web carried mainly axial force. The stud bolts were not considered for the analytical model.

Analytical model parameters are opening types (circular opening and square opening), thickness of the reinforcement steel plate (a half, equal, twice the surface steel plate thickness) and the width of the reinforcement (0.25, 0.50, 0.75, 1.00 times of the

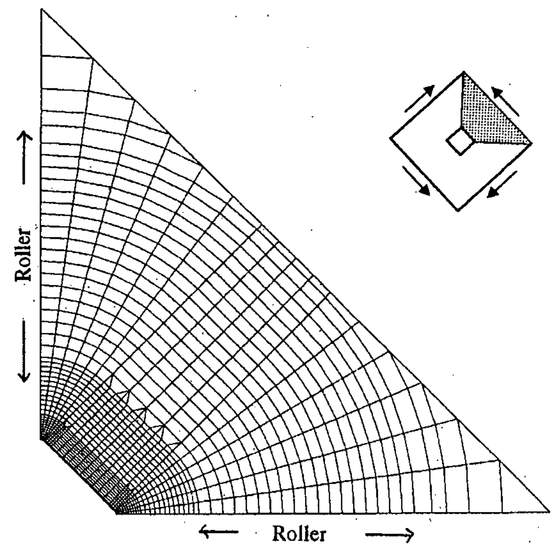

Fig. 20. Analytical model (square opening) (circular opening).

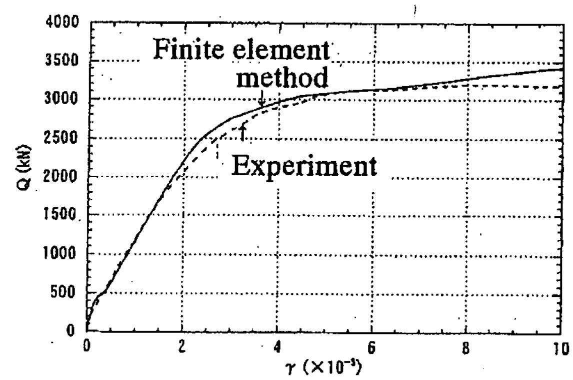

Fig. 21. Experimental results and analytical results.

diameter of circular opening or the side of square opening).

Before the analysis described, the test analysis was done to evaluate the accuracy of this method.

Table 8
Effective strength

Specimen	Concrete strength (MPa)	Yield strength		Ultimate strength		Amount of steel, q_s (cm^3)	Effective strength	
		Q_y (kN)	Δq	Q_u (kN)	Δq		Q_y	Q_u
S3-00HR	42.4	2710	74	3060	22	599	124	37
S3-00HP	31.7	2510	104	3470	168	701	148	240
S3-00RR	32.1	2170	6	2930	29	173	35	168
S3-00RP	32.7	2500	60	3060	47	680	88	69

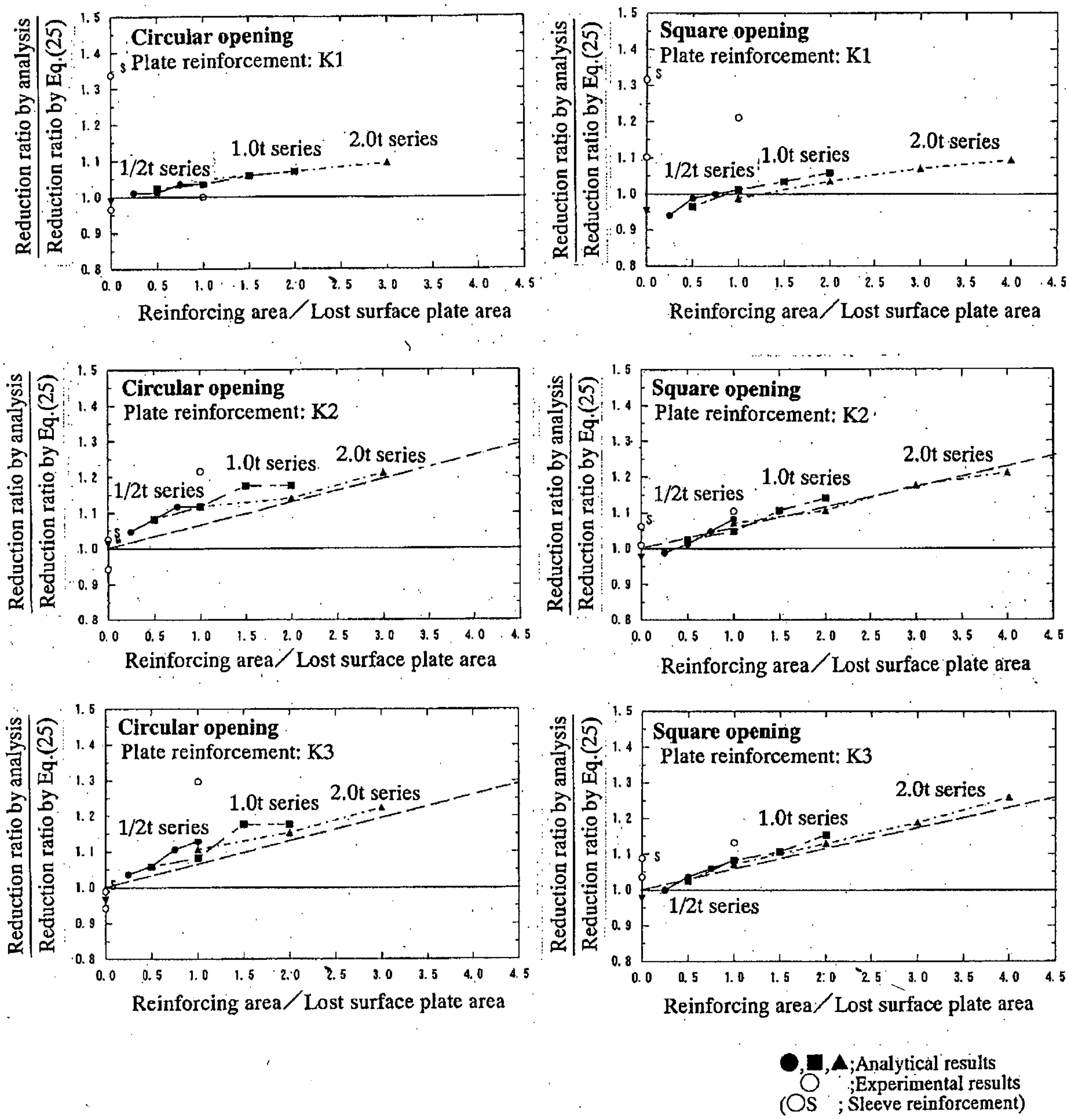

Fig. 22. Reduction ratios and reinforcement.

4.4.3. Constitutive relations

Constitutive relations generally used for RC structures were applied in this analysis. The stress–strain relationship in compression was modeled after Fafitis and Shah (1985) and Darwin and Pecknold (1977). In tension, tension stiffening was considered after Izumo et al. (1987) (parameter $c = 0.8$). Shear transfer on crack surface was considered after Al-Mahaidi (1979). Stress–strain relationship for steel plate was modeled in bi-linear.

4.5. Analytical results

The load–deformation relationship of the test analysis was shown in Fig. 21 in comparison with the experimental results. They were in good agreement. Therefore, the constitutive relationships described before have good accuracy for SC panels not only for RC structures.

The reduction ratios due to openings derived by the analysis are shown in Fig. 22. This figure shows the reduction ratio can be represented only as the amount of the reinforcing steel plate. The vertical axes show the quotient of strength or stiffness, which is the reduction ratio, given from the finite element analysis, divided by the reduction ratio calculated from Eq. (25). The horizontal axes show the quotient of reinforcing area, which is the reinforcing area in the section divided by the lost area of the surface steel plate for the opening. The figure shows that the quotient of strength or stiffness increases linearly as the quotient of reinforcing area increases. Especially, the quotient of the yield strength and the ultimate strength show the same results. These analytical results show the quotient of strength or stiffness depends only on the quotient of the reinforcing area, not on the width of the reinforcing area.

4.6. Consideration of the analytical results

4.6.1. Reduction ratio considering reinforcement

If we were able to evaluate the strength or stiffness by the calculation method based upon rational theory, the effect of reinforcement could be considered for the reduction ratio for the opening.

Practically, the calculation method for SC structure was based upon rational theory (truss mechanism) and the results of the calculation method are in good agreement with the experimental results (Takeda et al., 1995).

Therefore, the reduction ratio for SC panels could be considered to be caused by the effect of the reinforcement, except for the reduction ratio for tangential stiffness, K_1. K_1 is not included because it is insufficiently affected by reinforcement (Fig. 22). And the reduction ratios for yield strength and ultimate strength considering the effect of reinforcement were almost equal. Then we will determine these two reduction ratios as the same, K'. As the results, the reduction ratio considering reinforcement is given by the recursion formula of the results of the finite element analysis.

Hence,

$$\text{In case of circular opening;}\quad K' = 0.065 r_s + K$$
$$\text{In case of square opening;}\quad K' = 0.057 r_s + K \qquad (28)$$

where r_s is the quotient of reinforcing area, which is the reinforcing area in the section divided by the lost area of the surface steel plate for the opening. K is the reduction ratio for the panel without reinforcement. The results of these equations shall not exceed 1.00. These equations will be applied only for the reduction ratios for yield strength and ultimate strength.

The dotted line in Fig. 22 show the calculated results of Eq. (20).

5. Conclusions

The experimental and analytical study of the steel plate reinforced concrete structures (SC) subjected to in-plane shear was carried out.

The conclusions of the research program I are as follows:

(1) The yield strength increased linearly as the thickness of the surface steel plate increased.
(2) The cracking strength was influenced by axial force. However, the influence of the other strength was not remarkable.
(3) The partitioning web made the stiffness almost rigid. The influence of the strength was not remarkable.
(4) The equations based on truss analogy at post-cracking behavior were derived.
(5) The proposed equations were in good agreement for the experimental results.

The conclusions of the research program II are mainly given in the following:

(1) Although specimens have an opening, the responses under loading were very ductile.
(2) Strain near an opening increase linearly as the applied load increase. But the reinforcement kept the strain in lower value.
(3) The reduction ratio for the panel having an opening can be estimated by the calculation method for RC structure.

(4) A calculation formula of the reduction ratio by opening considering the effect of reinforcement was proposed.

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