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REPAIR AND RETROFIT OF STRUCTURES FOR EARTHQUAKE RESISTANCE (SHIZUOKA PREFECTURE)

US/JAPAN COOPERATIVE EARTHQUAKE ENGINEERING RESEARCH PROGRAM SPONSORED BY THE NATIONAL SCIENCE FOUNDATION THROUGH GRANT NUMBER CEE-7816730

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16. Abstract (Limit: 200 words)

Reports evaluating the earthquake resistance of existing buildings are organized into four categories: (1) earthquake control guidelines for building installations; (2) seismic diagnosis of reinforced concrete buildings; (3) methods for determining earthquake resistance and earthquake proof designs of pre-existing steel-skeleton construction; and (4) determination of earthquake resistance of buildings such as a gymnasium. Guidelines for applying seismic diagnostic standards are provided. Appendices contain computations of resistance and other data for columns, beams, and connections, and an example of the computation of index V for a gymnasium.

17. Document Analysis a. Descriptors

Earthquakes	Steel structures	Beams
Earthquake resistant structures	Reinforced concrete	Joints (junctions)
Dynamic structural analysis	Gymnasiums	Columns
Buildings		

b. Identifiers/Open-Ended Terms

Ground motion	R.D. Hanson, /PI
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PREFACE

Research, design and construction activities in the repair and retrofit of structures for earthquake resistance both in Japan and the United States have been increasing rapidly over the last decade. One way to maximize the benefits of research and experiences of others is to share them at an early stage of development and discuss alternative approaches and techniques. This was the purpose of the US/Japan Co-operative Research Program in Earthquake Engineering on Repair and Retrofit of Structures sponsored by the National Science Foundation through grant number CEE–7816730 to The University of Michigan.

A series of three seminars (May, 1980 in Los Angeles; May, 1981 in Sendai and Tsukuba, Japan; and May, 1982 in San Francisco) were held to share and discuss research results and field experiences. The Proceedings of these three seminars have been published in three volumes. A fourth volume contains an English translation of several Japanese reports on evaluation of earthquake resistance of existing buildings prepared for Shizuoka Prefecture as part of their Earthquake Hazard Reduction Program.

The financial support of the National Science Foundation, and the personal efforts by Dr. John B. Scalzi, NSF Program Manager, in establishing this program; the contributions of Mihran S. Agbabian and James Warner in organizing the Los Angeles meeting and field trip; and the contributions of Loring A. Wyllie, Jr. and Oris H. Degenkolb in organizing the San Francisco meeting and field trip are sincerely appreciated. The meeting and field trip in Japan was organized by Dr. Makoto Watabe and by Dr. Masaya Hirosawa who receive the sincere thanks and appreciation of all US participants.

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US/Japan Cooperative Research Program in Earthquake Engineering
on Repair and Retrofit of Structures

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(Translations of Japanese Documents)

INTRODUCTION

In 1978 Japan formulated the Large-Scale Earthquake Countermeasures Act which outlines the basic steps required of the national, prefectural and local governments once an area has been designated an "intensified area" (that is, an area that can expect significant damage. Reinforcing buildings, lifelines and other structures which are likely to receive great damage are part of these required steps.

Shizuoka Prefecture is in the Tokai intensified area and has initiated many efforts to prepare for the impending earthquake. The documents included in this volume are the direct result of these efforts. Shizuoka Prefecture commissioned the preparation of reports and manuals necessary to assist in the identification and evaluation of hazardous buildings and recommendations for their retrofit and strengthening. The original documents (in Japanese) were made available to the US Government. The Department of Housing and Urban Development had these documents translated into English. From the documents provided those included in this volume were selected as being of the most interest to the US structural engineering professionals.

The opinions, findings, conclusions and recommendations expressed herein are those of the individual contributors and do not necessarily reflect the views of the NSF or other private or governmental organizations. The accuracy of the translations have not been verified and may not necessarily reflect the opinions, findings, conclusions and recommendations of the individual contributions.

Our sincere appreciation is extended to the original contributors, and to officials of Shizuoka Prefecture and HUD for providing the documents included in this volume.

EARTHQUAKE CONTROL GUIDELINES FOR BUILDING INSTALLATIONS

Shizuoka Prefecture

April 1980

EARTHQUAKE CONTROL GUIDELINES FOR BUILDING INSTALLATIONS

In Shizuoka prefecture, potential occurrences of Tokai earthquakes originating in the vicinity of Suruga Bay are being discussed. Attempts are being made by this prefecture to promote systematic measures to control earthquakes from various angles as the most important program to be enforced.

Even earthquakes which originated in the sea near Oshima in Izu Islands and in the sea off the shore of Miyagi prefecture caused numerous damages to building installations, such as, collapsing of boilers, destruction of water tanks on the roof of buildings and breakdown of elevators.

Damage to building installations such as elevators, coolers, water tanks, boilers, etc. can cause a cut in the water supply, blackouts and suspension of elevator services if they are slightly pushed out of perfect alignment or their accesaries or parts are damaged. Buildings loose functions and daily lives are impeded after earthquakes. Furthermore, long time and large expenses are spent for restoration.

We have asked Assistant Professor Sakamoto of the University of Tokyo to research seismic designs, conduct a diagnostic examination of existing installations for earthquake resistance and study reinforcement methods, in order to design building installations more resistant to earthquakes. The guidlines we present at this time are prepared based upon the results of his research efforts.

I hope you will utilize this publication and make an earnest effort to make building installations more earthquake resistant.

April 1980
Shizuoka Prefecture

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I. POLICY FOR PREPARATION OF THE PRESENT GUIDELINES

The guidelines describe earthquake control for building installations, and are prepared in line with the following points.

i) Systematic and coherent earthquake control of installations is emphasized, which can be achieved by giving thought to both the earthquake resistance of individual machinery, equipment and piping installations, and comprehensive earthquake resistance which puts the former in perspective in respect to the entire building.

ii) Views expressed and results obtained by related programs implemented in the past or being in effect at present through public, academic and private sectors were referred to as necessary in order to avoid conflicts as much as possible.

On the other hand, it is recommended to pay heed to the following details when interpreting or making use of the contents of the guidelines.

i) It is desirable that related programs currently in progress in various fields be referred to for detail at your convenience whenever the results of the programs are made available.

ii) In implementing the control measures presented in these guidelines, it is necessary to examine fully the adequacy from the special conditions under which each building concerned is placed, the economical feasibility and the technical adoptability of the equipment itself.

II. SEISMIC DESIGNS AND IMPROVEMENTS FOR BUIDLING INSTALLATIONS

1. CONCEPT AND PROCEDURE

The guidelines interpret seismic designs and improvements to imply widely applicable earthquake countermeasures which include not only hard matters limited to, for inctance, the dynamic strength of machinery, equipment and piping installations but also what is so called soft control.

Seismic designs such as this, in a broad sense, are configured in accordance with the systematic procedures shown in Figure 2.1.

Likewise, when improving the earthquake resistance of building installations in a broad sense a problem (weakpoing) will be detected through the systematic procedures in the same table as described above, and the most efficient idea will be first implemented in consideration of the efficiency (expense required for improvements and the magnitude of the results) to be achieved after improvements.

2. ASSUMPTION OF SEISMIC MOTIONS AND DESIGN EARTHQUAKE LOAD

As the first step of the systematic approach to earthquake resistant designs and improvements for building installations, seismic motions shall be assumed. The seismic motions will be determined theoretically by the magnitude of the earthquakes, the hypocentral distance and the ground conditions. The trend of recent years pertaining to the earthquake resistant design methods that govern main structures generally takes into considerations the two or more levels of seismic motions assumable at the construction site of the building concerned.

a. Strong Seismic Motions: Strong seismic motions that rarely occur.

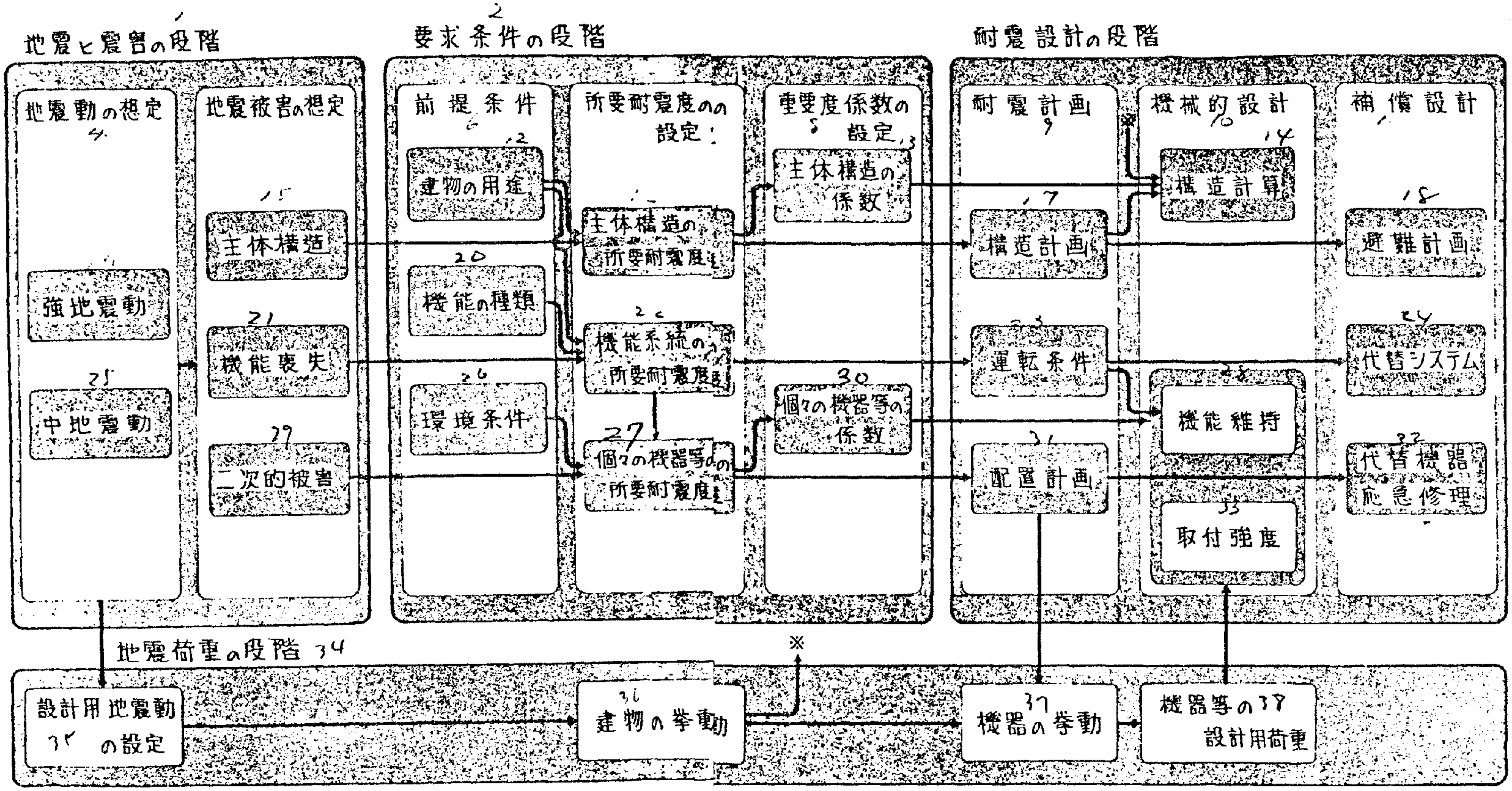
b. Moderate Seismic Motions: Moderate seismic motions that occur once in a while.

As will be described in section II-5, seismic designs of main structures aim to achieve a resistance level high enough to incur hardly any damages by moderate

Figure 2.1--key

1. stage of earthquake and its damage
2. stage of requirements
3. stage of seismic designs
4. assumption of seismic motions
5. assumption of earthquake damages
6. preconditions
7. establishment of required earthquake resistance level
8. establishment of importance factor
9. seismic plan
10. mechanical designs
11. compensating designs
12. use of buildings
13. factor for main structure
14. structure calculation
15. main structure
16. required earthquake resistance level for main structures
17. structure plan
18. escape plan
19. strong seismic motions
20. type of functions
21. loss of functions
22. required earthquake resistance level for functional systems
23. operational conditions
24. substitution system
25. moderate seismic motions
26. surrounding conditions
27. required earthquake resistance level for individual machinery and equipment
28. maintenance of functions
29. secondary damages
30. factor for individual machinery and equipment
31. arrangement plan
32. substitution machinery and equipment
33. installation strength

Figure 2.1 SEISMIC DESIGN SYSTEM FOR BUILDING INSTALLATIONS



34. stage of earthquake load
35. establishment of design seismic motions
36. behaviors of buildings
37. behaviors of machinery and equipment
38. design load of machinery and equipment

seismic motions of "b" and to withstand destruction by strong seismic motions of "a". The New Seismic Design Method (regulation) requires a 2 step calculation check for certain buildings (structures).

Since it is difficult to render a 2 step check for seismic designs for building installations in the same manner as will be rendered for main structures, ideally 2 levels of seismic motions are assumed as described above. However, it is the recent general trend to think of single level seismic motions in designing. Concrete values will be discussed in Chapter V, but if you insist, the level considered in designing can be said closer to the level of strong seismic motions of "a".

After the level of the seismic motions is set as above, it is necessary to present these seismic motions as the design seismic motions in order to create seismic designs in a narrow sense. The latest trend in seismic designs for main structures generally sets up this as the "design (seismic motion) spectra". This "design spectra" has already taken into consideration the response (behavior) of buildings, and the earthquake input into the machinery, equipment and piping installations can be determined based upon this spectrum.

Nevertheless, ground properties and the oscillation characteristics (proper period, proper oscillation form, damping constant) of the main structures must be assessed in regard to individual buildings for the determination of the earthquake input into machinery and equipment installations. Since assessment of this sort will be encountered with various difficulties, we would like to express uniformally the input (floor response, floor seismic intensity) values by showing the seismic intensity as is often practiced in the seismic designs for installations, according to the ratio of the installation height (story) of the machinery and equipment concerned to the total height of the building. Values in detail will be described in Chapter V.

3. ASSUMPTION OF DAMAGES

In assuming earthquake damages, we will think of potential damages extending over a range as wide as possible based upon the past cases of earthquake damages and the predictions by the engineers in charge, regardless of the degree of the earthquake resistivity of the installations about to be designed or reinforced. This work is necessary to determine whether or not the occurrences of such damages are permissable when later undertaking the task of "setting a required earthquake resistance level" (II-5).

An item "main structure" is included in the system shown in Figure 2.1, so that seismic designs and improvements for building installations dealt with in these guidelines can be worked out collectively from the viewpoint of the entire building. Damages to main structures can be adequately assumed by using common grading such as, minor damage, moderate damage, major damage and destruction.

In the case of functional losses of installations, these guidelines include not only the functional losses of individual machinery, equipment and piping installations but also broadly the loss of functions (for instance, blackout and cutting of water supply) to be serviced by the integral system of the installations such as the electric system and water supply and drain system. The installation system meant by the latter will be called the "functional system" (Classification of the functional systems will be related in section II-4). Consequently, the above described "functional losses of installations" can be described as "destruction of the functional system", and this will be the grounds to decide upon the "setting of the required earthquake resistance level for the functional system" later in section II-5. As reference materials for making such a decision, conditions of damages to each machine and equipment classified in each functional system are complied in Chapter III.

"Secondary damages pertaining to installations" are the inconvenie
incurred in the vicinity of destroyed individual machinery, equipment and pi
installations, and they are not related to the presence of the previously de
"destruction of the installation system". Secondary damage is exemplified b
water spills and flooding due to broken water tanks (especially flooding of
elevator machine room and suspension of elevator service due to the broken w
tanks in penthouses) and leaks and explosions due to the damaged gas pipes.

4. SETTING OF PRECONDITIONS (FUNCTIONAL SYSTEMS AND SURROUNDING CONDITIONS)

For carrying out the "setting of required earthquake resistance le
to be described in the next section (II-6), it is necessary to establish the
of the building concerned, the type of the functional systems of installatio
and the conditions surrounding the machinery, equipment and piping as precon

In seismic designs for main structures, the idea to differentiate
the required earthquake resistance levels in correspondence to the use of th
building concerned is likely to be adopted by means of creating an importanc
in compliance with the use. The use will be usually dividied into groups ac
to the roles to be played by the building concerned during and immediately a
an emergency, the earthquake. Here, the following classification is employe

(1) Buildings required (especially) to maintain functions during a
buildings containing large amounts of hazardous materials and
which contain a large number of people.
(2) Buildings other than described in (1) and (3).
(3) Buildings hardly likely to cause human casualties during a dis

In line with the above described main structures, it is rational t
seismic designs and improvements also for functional systems of installation
correspondence to the importance of the functions to be fulfilled by the sys
concerned. Therefore, here, we have decided to classify functional systems
installations as shown in Table 2.1.

Concerning the secondary damages incurred in conjunction with the destruction of individual machinery, equipment and piping installations, the impact of the effect of secondary damages can be one of the conditions to determine the required earthquake resistance level for the individual machinery and equipment (as will be described later, the required earthquake resistance level will be put under restraint by another angle in that individual machinery and equipment, needless to say, are a component which constitutes a part of a functional system). The impact of the effect of this secondary damage can be influenced by the conditions around the machinery and equipment. Accordingly, the impact can be judged not only by the details such as flooding, fire damage, poisoning and suspension of other machinery and equipment but also by prospects such as the range of space that can be affected, the use of the room and the functional system to which the machinery and equipment that cause the damage belong.

5. ESTABLISHMENT OF REQUIRED EARTHQUAKE RESISTANCE LEVEL

Assumption of seismic motions, assumption of damages and setting of preconditions have been completed in the prior sections. Here, the required earthquake resistance level will be set based upon the results obtained in preceeding sections. This required earthquake resistance level is set in terms of an allowable level for each damage incurred under each precondition (functional system, surrounding conditions) relative to each strong and moderate seismic motions.

In reference to the main structure, a comparatively general consensus is obtained regarding this setting of the required earthquake resistance level. Contents of the requirement set for buildings of average importance are more or less as follows, and it is presupposed that this approach will be also followed in these guidelines.

Strong Seismic Motions: Main structures shall not collapse, and human lives shall be protected.

Moderate Seismic Motions: Main structures shall not be extensively destroyed, (and of course, human lives shall be protected).

Regarding the setting of the required earthquake resistance level for for functional systems, we hope to make decisions in these guidelines based upon references to some past cases where functional systems were reviewed and mutually compared in respect to the required earthquake resistance level. For instance, naturally a high level of earthquake resistivity shall be required for an emergency power source installation which is expected to demonstrate its function particularly during and immediately after an earthquake, one of the emergencies. On the other hand, the required earthquake resistance level can be set relatively low for general office use air conditioners. Also, the required earthquake resistance level for individual machinery, equipment and piping installations are established primarily from the three viewpoints: the required earthquake resistance level for the functional systems that contain the said machinery and equipment, importance of the said machinery and equipment to the said systems and the impact of the effect of the secondary damage. (Additionally, it is necessary to consider the degree of importance according to the use of the building.)

One example of the degree of importance of each functional system and each piece of machinery, equipment and piping are presented in Table 2.1.

Table 2.1 Degree of Importance of Functional Systems and Major Machinery and Equipment Installations

機能系統 1		重要度 3	2 主要機器	重要度 3
4 電気設備	受変電設備 5	B	6 変圧器	B
			7 配電盤	B
			8 その他（継電器，幹線）	B
	非常用電源設備 9	A	10 蓄電池	A
			11 自家用発電機	A
	照明・弱電設備 12	B	13 照明	B
			14 弱電（電話，放送）	A
給排水設備 15		A	16（高架）水槽	A
			17 ポンプ	A
			18 配管	A

冷暖房空調・換気設備 19	B	20 ボイラー 21 冷凍機 22 冷却塔 23 空調機 24 パッケージ空調機 25 放熱器 26 ダクト	B B A B B B B
ガス設備 31	B	27 配管 28 その他（器具、ボンベ）	A B
昇降機設備 32	A	29 非常用エレベーター 30 一般用エレベーター	A B

key-1. functional system

2. major machinery and equipment
3. importance factor
4. electric system
5. power receiving and substation system
6. transformer
7. power board
8. others (relay, main line)
9. emergency power source system
10. battery
11. home generator
12. lighting and light duty system
13. lighting
14. light duty (telephone and broadcasting)
15. water supply and drain system
16. (elevated) water tank
17. pump
18. piping
19. cooling, heating, air conditioning / ventilating systems
20. boiler
21. refrigerator
22. cooling tower
23. air conditioning unit
24. package air conditioning unit
25. radiator
26. duct
27. piping
28. others (instruments, bottles)
29. emergency elevator
30. general elevator
31. gas system
32. elevator system

6. ESTABLISHMENT OF IMPORTANCE FACTOR

As one of the specific measures to secure the required earthquake resistivity for individual machinery, equipment and piping installations, an importance factor will be established. When incorporating the importance factor in seismic designs, there are, in a broad sense, the following two lines of thought concerning the nature of the factor to be incorporated in the designs.

(i) Factor that gives extra (discounted) load values

(ii) Factor that expresses margin (safety ratio) of structures (materials)

The difference between the two relate to the basic approaches to seismic designs. No matter which thought is adopted, both can eventually bring the same level of earthquake resistivity for the designed structures. Therefore, we will not go into the details of this subject, but the explanation below will be carried assuming that the designs will be made in line with the approach in (i).

Values shown in Table 2.2 are conceivable as an importance factor for main structures in correspondence to their use listed in section II-4.

Table 2.2 Importance Factor of Main Structures

use of structures	importance factor
(1)	1.5
(2)	1.0
(3)	0.7

Values shown in Table 2.3 are conceivable as an importance factor of individual machinery, equipment and piping installations which are classified into A, B and C from the viewpoint described in section II-4.

Table 2.3 Importance Factor of Machinery, Equipment and Piping

Machinery, Equipment and Piping	importance factor
A	1.5
B	1.0
C	0.7

7. SEISMIC PLAN

The first step of seismic designs is to formulate a seismic plan that will measure up to the prescribed required earthquake resistance level.

In case of the main structure, so called a structural plan is equivalent to this, but an evacuation plan will be, in a slightly broad sense, included in this discussion.

The seismic plan pertaining to functional systems determines operational conditions regarding the maintenance of functions during and immediately after an earthquake. Another task is to determine which functional systems need a substitution system. The following are the feasible types of operational conditions.

i) Totally earthquake resistant type
ii) Earthquake control automatic reinstating type
iii) Earthquake control manual reinstating type
iv) others

The seismic plan for individual machinery, equipment and piping installations primarily concerns the arrangement plan which decides the location (especially the story) for installation, selection of machinery and equipment and choosing of machinery and equipment support methods from the seismic point of view.

8. MECHANICAL DESIGNS

In this section, designs, in a narrow sense, based upon calculations and experiments, will be described. This can be considered a control before the fact in contrast to compensating designs that serve as a control after the fact to be described in the next section.

The mechanical design of a main structure entails so called structure calculations.

This section of "mechanical designs" is not necessary for functional systems. Mechanical designs required of the functional systems for the purpose of earthquake resistance will be provided as mechanical designs for individual machinery, equipment and piping to be described in this section. The mechanical designs for individual machinery, equipment and piping are divided into the following two main categories.

i) Function Maintenance Designs for the Machinery Proper: This concerns mechanical designs, and manufacturers are responsible for them. Consequently, construction companies are required to make special specifications relating to the earthquake resistivity of individual machinery and equipment based upon the design load, importance factor, operational conditions, etc. Since the function maintenance designs for the machinery and equipment proper are, as described above, produced by the manufacturers' side, it is desirable that a special specification shall be prepared and machinery and equipment that meet the specification shall be used as far as new machinery and equipment installations are concerned.

Even now, makers display guarantee limit against external forces such as vibrations for some products. But the values presented include a certain safety factor, and they are in some cases lower than the value of an external force that can be actually borne. Evidentally, it is necessary to receive confirmation from manufacturers that functions can be maintained at a considerably higher probability than the special specification values, although it is not necessarily required that this guarantee limit values should exceed the special specification values.

The following are, for instance, conceivable as special specifications.

"Functions shall not fail under an oscillation of sine waves with a dg (refer to section V-3, d is the design seismic intensity on the floor where the machinery and equipment are installed) acceleration amplitude and a 1-5 Hz frequency".

Incidentally, it is preferable to make special mention jointly with matters effective for the improvement of earthquake resistivity after simple modifications, such as, "In case of equipment with a door like a cubicle type, specify those which come with a lock on the door."

ii) Dynamic Designs for Parts and Pedestals of Machinery, Equipment and Piping: This determines materials and types of casings, support materials, frame materials, pedestals and foundations, and also determines the required sections. Some may overlap those in "i)". Constructors and manufacturers must get together for adjustment, but what is more important is that both sides should cooperate and investigate lest something may be carelessly overlooked.

Refer to Chapter V for concrete methods.

9. COMPENSATING DESIGNS

It is essential to formulate in advance a control after the fact in case the control before the fact ends up in failure, from the standpoint of fail safe thinking. Of course, this will not apply to functional systems and individual machinery and equipment with a low required earthquake resistance level.

In respect to the main structures, deciding on evacuation methods is equivalent to compensating designs.

Compensating designs for functional systems propose substitution systems that can render services in lieu of the destructed functional systems. For example, emergency power source installations can be considered as a substitution system to an electric receiving and substation installation.

The following two different methods can be cited as compensating designs for individual machinery, equipment and piping.

i) Spare machinery and equipment which can render the same services as the damaged machinery and equipment, shall be ready for use.
ii) An emergency repair system shall be established.

III. ACTUAL CONDITIONS OF DAMAGES AND PROBLEMS

1. ELECTRICAL SYSTEMS

(1)Electric Receiving and Substation System (Including Main Line Installations)

i) Transformers

Concerning the electric receiving and substation system, transformer damage is outstanding. Damages characteristic in that the transformer proper shifts and breaks its wiring contacts, occur frequently.

Cubicle power switchboards and power boards were damaged relatively less. A few cases of damages to relays, disconnecting switches and circuit breakers are reported to have occurred during the Kanto Earthquake, the Niigata Earthquake and the San Fernando Earthquake but not during the Miyagi Prefecture Offshore Earthquake.

The major cause of the damage to transformers can be the use of improper installation (bearing) methods. Those which had not been fixed on a floor largely shifted, and in some cases transformers also shifted by slipping or breaking of bolts even if they had been fastened, due to a deficiency in the strength of the bolts.

Transformers come with various efficiencies and in various forms, but they are all relatively heavy and shaped with a high center of gravity. Transformers can be easily shifted or turned over by strong seismic motions unless they are firmly fastened.

Generally, transformers are fixed to a floor by means of channel materials but many of them are unstable and their upper section can readily vibrate.

ii) Others

In the San Fernando Earthquake, there was a case where a generator did not start during the blackout due to an operational error of a relay. There was also a case associated with the Miyagi Offshore Earthquake where a generator stopped because it was unable to change over due to a breakdown of a transformer in the cubicle. These incidents indicate that the possible damage to the electric receiving and substation system will affect emergency power source installations.

(note) It is desirable that the electric receiving and substation system associated with the installations in the buildings of electric companies be in compliance with Seismic Control Guidelines JEAG 5003-1980 for electric systems such as those in substations".

(2) Emergency Power Source System

i) Batteries

Only a few cases of damages to batteries were reported in connection with the Miyagi Prefecture Offshore Earthquake. However, with the Tokachi Offshore Earthquake and the Niigata Earthquake, many such cases were reported.

The damage description of the majority of the cases refer to broken contacts of electrolytic cells and wiring, caused by the shifted or turned over batteries which had not been fastened. Other reported damages are simply shifting of pedestals and electrolytic cells in the cubicles and damaged pedestals and cubicles. These incidents were attributable to the pedestals and electrolytic cells which had not been fastened or imporperly fastened to the floor. Cubicle type batteries were also damaged when cubicle containers were not stabilized or stabilizers were not provided for the internal electrolytic cells.

Batteries which serve as an emergency power source are for supplying electricity to emergency lighting and surveillance machinery and equipment, when commercial power is not usable. They should exhibit prescribed functions particularly during an emergency. In some cases, emergency home generators did not start because the batteries failed to perform their normal function.

The Tokachi Offshore Earthquake damaged many electrolytic cells which had been simply installed in line on the floor. Recently, however, many of them are set on pedestals or in cubicle containers. Even so, those not fastened adequately to the pedestals or in the cubicles were often damaged.

Damages were almost exclusively related to shifting and falling, which indicates that the most important point of earthquake control is to fasten an object very securely.

ii) Home Power Generation Installations

Among power generation installations, generators primarily suffered damages. Damages to the generator proper (including internal combustion engines) were rare, but in many cases, connecting pipes were damaged due to the shifting of the generator itself, resulting in operational failure of the generator. In one of the said cases, an exhaust pipe was severed and continuous operation was withheld. Usually, an exaust pipe is connected to the upper part of a generator, and suspended from a ceiling slab together with a muffler. It appears as if the damage to this pipe is not essential to the power generating function, but the broken pipe cannot fulfill the exhausting function, and the functions of the power generating installation as a whole will actually be lost. Likewise, in some cases, cooling water pipes cracked, and operators coped with this situation by securing the cooling water using backets. Various types are available for the generator cooling system, but any of them cannot operate without a supply of cooling water (excluding gas turbine type). In order to enhance the earthquake resistivity of the entire system, it is necessary to pay heed even to peripheral parts and minute parts.

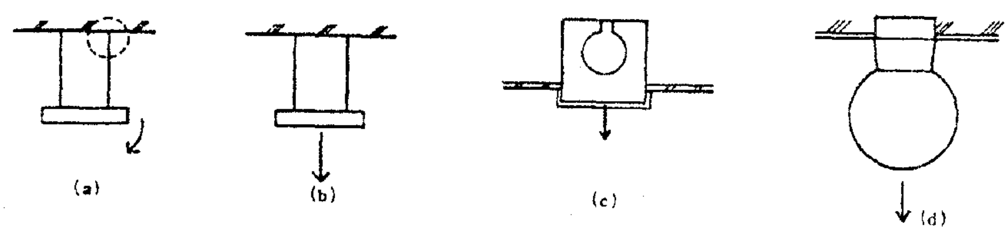
Incidentally, generators are usually supported with rubber vibration insulators so that the noise and vibrations generated will not be conveyed to the chasis. Large si æ generators are considerably heavy, and the bearing with rubber vibration insulators is smaller in strength (sheaing strength, tensile strength) than fixation by bolts. Furthermore, it is possible that machinery and equipment may resonate with an earthquake. It is probably necessary to investigate the bearing of heavy machinery and equipment with rubber vibration insulators. Those projected to shift shall be treated with some kind of measures such as installation of stoppers. Connecting pipes shall be connected by flexible joints os that vibration and displacement of the generator proper can be absorbed to an extent.

Diesel generators are generally designed to receive the fuel supply to the diesel engines from a fuel service tank which receives the fuel first from the fuel tank. This fuel service tank sends fuel to the engines by means of gravity and is usually installed high on a pedestal. Although there have been no reports on the damages to this tank, it is highly probable that earthquake damages occur if the rigidity of the pedestal is low or the fixation of the pedestal is inappropriate. Damage to the fuel service tank can incapacitate the generator or cause great trouble. Furthermore, not only the loss of the system function but also secondary damages such as outbreaks of fire may occur. It is therefore desirable that the control measures be in detail.

(3) Lighting and Light Duty System

The following are the details of damages to lighting fixtures classified by the sites where damages occurred.

a. Cracked bearings of lighting fixtures hanging from a ceiling.
b. Cracked or dropped lighting fixtures hanging from a ceiling.
c. Shifted, slipped or dropped covers of a lighting fixtures directly attached (embeded) to the ceiling.
d. Dropped lighting fixtures directly attached (embeded to the ceiling).



Also, damages classified by causes are,

i) Damages to hanging materials by large oscillations (particularly chain hanging and pipe hanging types)
ii) Inappropriately fixed light bulbs and lighting bubes
iii) Shifting and dropping of ceilings

Among the damage cases, some are damaged by the shifting of system ceilings. This is one example of damages attributable to the deficient strength of non-structural materials in buildings and inappropriate fixation. System ceilings integrate lighting fixtures and duct blow-off openings, and can be easily built. The earthquake resistivity of these system ceilings must be reinvestigated including the building methods.

Many lighting fixtures are hung from ceilings. This may be because the general public is only vaguely aware of the danger of these lighting fixtures, and they accept hanging lighting fixtures if installed secure enough not tp drop during normal time. It is impossible to provide sufficient earthquake control measures to all of them from the points of cost and design, but it is necessary to give some earthquake control consideration to lighting fixtures. Of course, lighting fixtures installed in important places of buildings (for example, near evacuation routes, places where a large number of people gather like halls) absolutely require seismic considerations. Especially, chandeliers shall be installed with caution, since they are heavy and dangerous enought to cause casualties if they drop.

All and only control measures to be rendered for them is nothing but the enforcement of fixation.

(note) For electrical installations, mundane daily maintenance is just as important as the implementation of earthquake control measures.

2. WATER SUPPLY AND DRAIN SYSTEM

i) Elevated Water Tank

With the Miyagi Prefecture Offshore Earthquake, damages to the elevated water tanks on the roof of buildings were very much noticed. The description of damages are classified into the following 4 categories.

a. Cracking and breaking of water tank itself.
b. Breaking of joints between piping and water tank.
c. Breaking of connecting pipes.
d. Damage to bearings (water tank fixation site, pedestal and foundation).

These damages do not necessarily occur independently. Damages may occur simultaneously at different parts, or damage to one area may invite damage to other areas in some cases. However, the above classification may become useful to assess which part of the elevated water tanks is most vulnerable. The causes for the damages suffered were investigated and can be roughly divided into the following three groups.

i. Deficient strength of the water tank itself.
ii. Deficient strength of the water tank fixation site.
iii. Absent or deficient flexibility of piping.

"i" often manifests as the side plates break off the bottom plate or crack due to a great load added to the side plates by the largely waving water in the water tank. In the light that the majority of damaged water tanks were made of FRP and steel water tanks were rarely damaged, much remains to be investigated. In recent years, FRP water tanks are very popular because of the merits of inexpensive and simple construction and cleanliness. We hope that manufacturers will promptly work to solve this problem. (Refer to section IV-2).

Incidentally, spilling of a large body of water from the broken water tanks and piping means an incapacitated total water supply system. Usually, the water supply system provides water to other functional systems (for instance, fire extinguishing system), which translates that damages to this system affect all the related systems. Especially this system heavily affects the fire extinguishing system since the indoor fire hydrants and sprinklers will be gravely impaired to be able to function on the upper floors of buildings if elevated water tanks break down.

In one case, a lot of water flowed into an elevator machine room due to a broken water tank and elevator service was withheld. This can be regarded as typical secondary damage by water. Thirty water related elevator accidents were reported in association with the Miyagi Prefecture Offshore Earthquake due to the broken water tanks and cooling towers installed on the top of buildings. It is necessary to have some control measures against the secondary damages such as these in case they should happen by any chance. As specific water damage control measures, we can recommend, for instance, not to install elevated water tanks above or near the machine rooms, or provide adequate emergency water drain routes.

In some cases, the water pipe from a water pump was directly connected to the water supply pipe (bypassing the water tank to shorten the supply line) to cope temporarily with the situation when the water supply was cut by the broken elevated water tank. Providing substitution such as this or having an emergemcy repair system at hand will enhance the earthquake resistivity of the machinery and equipment per se as well as will be commendable as compensating designs.

ii) Others

Damages to the water supply system other than those associated with elevated water tanks are the damages related to hot water tanks installed on upper floors. These damages were caused by inadequately fixed pedestals and foundations and insufficient strength of foundation concrete. It is desirable that tanks and the like be installed upon firmly immobilized pedestals and foundations, and that the foundation concrete be of an adequate size and made integral with the slab by the reinforcing steel.

In one case, a water pump was operated without water and broke down because the power was on in spite of the fact that the water supply was cut off. This is an example which indicates the necessity of investigating the emergency power machine and equipment control system.

Piping plays an essential role for transportation of water from ground floors to elevated water tanks and for the supply of water to appointed machinery and equipment installations. In reality, a large number of damaged piping cases are expected to occur including minor damages. Minor damages may not have been included in the results of the survey, since the damage inside a pipe shaft or to the piping above the ceiling are hardly visible. It seems we must correct our notion that the piping which connects machines to machines are secondary to major machinery and equipment, and must assess piping as a part of machinery and equipment in administering earthquake control.

Among drain related damages, there was a case where a drain pipe broke and water was not usable in spite of the fact that the supply side was intact. This example indicates that even a section that belongs to the peripherals can inflict functional loss of the total system depending upon the size of the damage.

Damage to the water supply and drain system not only makes daily life difficult but also affects the cooling of geerators and refrigerators, and may even incapacitete fire extinguishing systems.

The suspension of operation of air conditioners can result in no more than a reduction in pleasantness, but an incapacitated fire extinguishing system may lead to the loss of human lives. In view of these points, water supply and drain installations should be given a high earthquake resistivity (although the level of the resistance differ in correspondence to the use of buildings and the system of the installations).

3. COOLING, HEATING AIR CONDITIONING AND VENTILATING INSTALLATIONS

i) Boilers

In almost all cases, damages were characteristically related to broken connecting pipes due to the shifting of the boiler itself. Excluding some sectional boilers, cases where a boiler itself was damaged were rare. Large size boilers have a steaming weight exceeding 20 tons.

It can be said that the main cause of the damages was generally unfastened or inadequately fastened large heavy boilers. It is natural that the connecting pipes break when the boiler itself shifts or turns over. Consequently, earthquake damage can be controlled by:

i. Securely fixing the boiler itself.
ii. Providing flexibility to connecting pipes.

Inoperable boilers imply loss of heating or hot water supply functions in air conditioning. Furthermore, the breaking of oil pipes and fuel tanks may invite secondary damage such as oil leaks and breaking out of fires. Incapacitated heating systems are not so important, but secondary damages (accidents) associated with fuels may involve human lives, and it is desirable that some care be taken to prevent these accidents.

ii) Refrigerators

Several cases of damages to refrigerators were reported in relation to the Miyagi Prefecture Offshore Earthquake, The contents of the damages were shifting of the refrigerator itself and the imparing of connecting pipes. Damages to the refrigerator itself were not reported.

There are various types of refrigerators, turbo refrigerators, absorption refrigerators, gas water chilling and heating units and reciprocating refrigerators.

Except for the absorption type, generally a vibration proof bearing is provided by using metal springs against the vibration and noise that are generated. (Only rubber vibration insulating pads are placed under an absorption type refrigerator since it hardly vibrates).

The cases of damage in the Miyagi Prefecture Offshore Earthquake were dominated by impaired or disconnected vibration proof springs of the bearing. It seems that a refrigerator proper supported by vibration proof springs resonated with the seismic motions of the earthquake vigorously resulting in a shifting of the refrigerator itself and the disconnecting of the springs (Damaged absorption refrigerators were merely placed on a rubber pad).

An unoperable refrigerator naturally disables the total cooling system. Suspension of the cooling system during summer is anticipated to reduce drastically the efficiency of the activities conducted within the buildings.

Large size refrigerators, absorption refrigerators in particular, exceed 40 tons. These damages occurred probably due to the lack of considerations for earthquakes in connection with such heavy machinery and equipment.

To deal with this, some kind of fixation must be provided to the refrigerator proper and the foundation for the prevention of the disconnection of vibration proof, springs. Stoppers for example must be installed to prevent the refrigerator proper from shifting. Needless to say, those unsecured absorption refrigerators must be fastened.

In general, the importance and necessity of the cooling system which is for pleasantness can be said to be lower than those of the emergency power installation, but control measures should be administered by all means since the strengthening of fixation largely increases the eqrthquake resistance (however, long suspension of the cooling system is critical to computers and communications machinery and equipment).

iii) Air Conditioners (Air Conditioning Units)

Air conditioning units were damaged by the Miyagi Prefecture Offshore Earthquake, for example, broken bearings with rubber vibration insulators and vibration proof springs resulted in shifting of the refrigerator proper.

An air conditioning unit is an integral unit which combines an air blower, cooling and heating coils, a humidifier and filters. In the past, respective parts, iron plates and steel frames were assembled at the local site, but recently, in many cases, air conditioning units are being manufactured as a unit in factories and only installed at the local sites. Air conditioning units come in considerably large sizes and cause noise and vibration.

Evidentally, they are provided with a vibration proof bearing and installed in an air conditioning room or a machine room. Cases of damage by the Miyagi Prefecture Offshore Earthquake revealed sheared rubber vibration insulators and separated rubber materials and metal parts. The strength of rubber vibration insulators and the method for their use must be reinvestigated. Installation of stoppers can be cited as a countermeasure (especially in cases where the units are installed on upper floors).

v) Cooling Towers

Cooling tower damages reported in relation to the Miyagi Prefecture Offshore Earthquake were shifting of the tower due to the bending of its legs and the tilting of foundations.

Cooling towers are classified into four groups by the cooling system, natural draft cooling towers, orthogonal current cooling towers, closed cooling towers and countercurrent cooling towers (natural draft cooling towers are very rarely used today). Also, various shapes and various efficiencies are provided depending upon their use. Circular shaped cooling towers are usually supported by three or four legs and weigh, if large, nearly 5 tons. Some square shaped ones exceed 12 tons. In the common fixation method, the legs are fixed on a concrete foundation by boxed anchors. In view of the fact that they may be installed on the top of buildings with a large floor seismic intensity, it is necessary to determine whether or not the strength of the legs and fixation sites can fully withstand the seismic motions. In one of the damage cases, a cooling tower fell over and spilled a large amount of water which damaged the elevator installation. This is an example of secondary damages by water spills similar to the previously described elevated water tanks. Due to the nature of being installed on the top of a building, cooling tower damages are likely to connect to water related accidents.

Some square cooling tower foundations tilted. In these cases, concrete foundations were merely lined on a slab and not connected to the slab. There are plenty of examples of construction which do not have a concrete foundation and the slab made into an integral body. If foundations are not fixed to the slab, they may be shifted by the horizontal force of earthquakes or show different behaviors at different locations causing bending of the legs.

It is necessary to think of connecting the foundation with the slab by means of reinforcing steel in stead of making an independent clustered foundation.

The suspension of the cooling tower function by its breakdown makes refrigerators unoperable and eventually the function of the total cooling system is lost. Therefore, we should administer control measures that not only maintain the function of the system but also give thoughts to the prevention of the previously described secondary damages. Specifically, cooling towers should not be installed sbove or near a machine unit with important machinery and equipment. Another example is to secure drain routes with some margin.

4. ELEVATOR SYSTEM

Damages revealed in association with the Miyagi Prefecture Offshore Earthquake were derailing of balancing weights and shifting of lifters and motor generators.

Derailing of balancing weights claimed 40% of the damages to elevators. The San Fernando Earthquake that hit the Los Angeles area also reported an overwhelming number of balancing weight related damage cases. Derailing of balancing weights does not merely end up with suspended operation, but the weights which pick up force may crash into the elevator box and endanger human lives. Althogh no human lives were lost, 110 cases of damage to the box by the balancing weights were accounted for in relation to the San Fernando Earthquake (if ascending box collides with descending balancing weights, an extremely large impact force will be added).

Suspension of elevator service will be translated only as the loss of a convenience for low and medium height buildings, but will be a tremendous reduction of efficiency in a building such as a high rise where the elevators are essential. In addition, damages to elevators are highly likely to endanger the lives of passengers. The earthquake control measures for elevators must first consider the securing of the safety for passengers and then the maintaining of the function.

The point of the countermeasures for the former is the control system during earthquakes. It is absolutely necessary to let passengers off the elevator promptly by stopping it at the nearest floor when an earthquake above a certain level of seismic intensity occurs. For this purpose, an earthquake detector must be installed. Nevertheless, merely less than 10% of the total ±6000 elevators and escalators installed in six prefectures in the northeast region came with a detector at the time when the Miyagi Prefecture Offshore Earthquake occurred. It is, furthermore, revealed that only less than 1% of 146,867 elevators (as of

March 31, 1978) installed throughout Japan are equipped with detectors. It presents a problem that only such a nominal number of detectors are installed in spite of the fact that the installation of earthquake detectors are indispensible for the controlled operation of elevators to secure the safety for passengers.

Next, speaking of the maintenance of functions, especially high earthquake resistivity should be given to emergency elevators in high rise buildings for evacuation and fire extinguishing activities. As described before, the Miyagi Prefecture Offshore Earthquake reported lots of elevator damage due to water spills by broken elevated water tanks and cooling towers. It will be necessary to give some sort of consieration to prevent these types of secondary damages.

5. PIPING

When classifying damage to piping by cause, the following groups are made.

a. Damage probably due to vibration of the piping itself (especially piping with a long hanging length).
b. Broken pipes due to the shifted or fallen machinery and equipment.
c. Damage due to the displacement of piping relative to nonstructural materials such as ceilings.
d. Damage due to the inability to follow the deformation of the constructed structures (especially expansion).
e. Damage due to ground fluctuations and the displacement of the ground relative to the structures.

Piping renders the function of blood vessels connecting machinery and equipment to other machinery and equipment and terminals, and the functions of systems in various places of buildings can be given life only after this function can be normally rendered. In systems which depend upon pressurized water such as indoor fire hydrants and sprinklers, damages to the piping are fatal.

Damage inflicted by a large deformation or destruction of a constructed structure cannot be helped, but cases of "a" and "b" are preventable.

Incidentally, broken piping can lead to the functional loss of systems as well as secondary damages just like other machinery and equipment. For example, broken water supply pipes, as described previously can cause water related damages and broken oil supply pipes can invite fires due to leaking of oil. Control measures with consideration to these points are required.

IV. EARTHQUAKE CONTROL FOR FUNCTIONAL SYSTEMS AND MAJOR MACHINERY AND EQUIPMENT

1. ELECTRICAL SYSTEMS (ELECTRIC RECEIVING AND SUBSTATION SYSTEM, EMERGENCY POWER SOURCE SYSTEM, LIGHTING SYSTEM)

(1) Outline

Electrical installation systems are generally required to have a high earthquake resistance level. Therefore, the function of electrical systems are often required to recover immediately after an earthquake even if power may blackout temporarily during the earthquake.

Immediately after an earthquake, it is anticipated that electricity that is normally obtained from outside will be out of service, and it is desirable to secure an emergency power source (generators and batteries) as a compensating design. However, if this emergency power source is destroyed, the compensating effect can not be demonstrated. Apparantly, seismic designs and improvements for emergency power sources are a priority matter. For the matters related to this, it is desirable to comply with "Guidelines (Proposition) (Part 1) for Home Power Generation Installation Seismic Designs" issued in August 1978 by Japan Internal Combustion Power Generation Installation Association (incorporation), Investigation and Research Committee for Earthquake Control Measures.

Generators which are the center of an emergency power source system generate vibtations from their internal combustion engines, and are normally provided with vibration proof bearings. For this point, please refer to the section, "Stoppers" in VI-5. Also, in order to secure the function of the internal combustion engines, cooling water is indespensible. In some of the past earthquake damage cases, this cooling water supply was interrupted by the destruction of cooling water tanks and breaking of piping. It is necessary to secure the earthquake resistivity from this point by using the explanations relating to water tanks and piping in the following section (IV-2) as a reference.

In numerous cases, batteries which are expected to pick up the function during an emergency just as generators, were vulnerable to earthquakes. It is necessary to increase earthquake resistivity by adopting support frames, spacers and flexible conductors.

As described above, if an emergency power source can display its function as expected when electric supply from outside is cut during an earthquake, the next seismic problem to be solved is the earthquake resistivity of various machinery and equipment for transmitting electricity of a rated voltage in the building. Among them, transformers of a substation system are especially important. The present state indicates that there are many cheap installation parts. It is necessary to attempt to absorb the displacement in transformers using flexible conductors for wiring as well as to prevent falling and shifting during earthquakes by referring to sections VI-1 to VI-3. Besides the ones mentioned above, it is also necessary to provide a measure first to prevent falling and shifting of machinery and equipment such as circuit breakers and power switchboards. For this purpose, installation of stay supports shown in section VI-6 is very effective.

Seismic problems are relatively few as far as wiring in buildings is concerned. Bath ducts and the like are also likely to withstand earthquakes if installed in accordance with general installation methods.

As a typical item that uses electricity, lighting fixtures can be cited. They frequently fall off during earthquakes. Unless an electrical blackout, not all of the light fixtures simultaneously cease to light. Secondary damages due to dropped light fixtures are more problematic in respect to the lighting system per se. Earthquake resistivity primarily depends upon the supporting forms and supporting details (including constructional precision).

Telephones are essential as a means to transmit information during emergencies, and it is especially necessary to provide earthquake resistivity to switchboards in separate buildings. It is assumed that telephone switchboards themselves have good earthquake resistance. The earthquake control therefore shall be focussed on installation methods which produce structures that can well resist shifting and falling of the switchbaords. Particularly, it is better to enhance the degree of fixation by adding stays on the upper part in stead of using a self-supporting type.

(2) Check Points

i) <u>Transformers</u>

a. The size of the transformer installation anchor bolts shall be large enough to endure shearing forces and reaction loads created by earthquakes.

b. Anchor bolts for a large transformer shall be desirably welded to reinforcing steel of the slab and the foundation.

c. When using post-driven anchor bolts for installation of transformers, an adequate size shall be chosen.

d. Anchor bolts shall be installed with a proper pitch, and an adequate concrete covering depth (edge distance) shall be provided when anchoring the bolts on the edges of the foundation concrete.

e. Fixation bolt pitch shall be not too large (A reaction load by the moment shall be kept small).

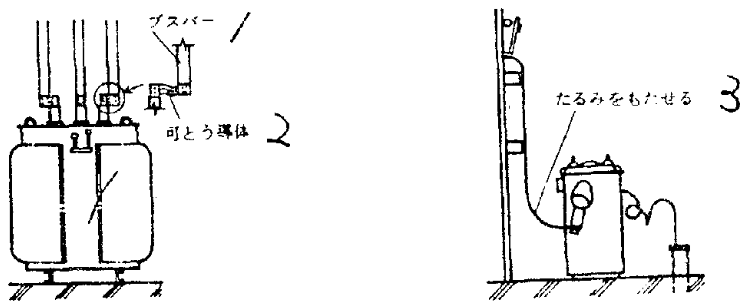
f. When the strength of transformer installation bolts is inappropriate, supplementary measures such as stays and stoppers shall beprovided.

g. Channel bases will be effective if they are inlayed in the lower part of the concrete.

h. Fixation and welding of the channel base to the transformer itself shall be done firmly.

i. Joints at the ends of machinery and equipment shall be provided with flexibility (figure).

j. Wiring shall be provided with some slack (figure).



Key-1. bus bar
2. flexible conductor
3. given slack

ii) <u>Cubicle Power Switchboards</u>

a. Large boards (special high voltage power switchboards) shall be installed closer to the basement.

b. It is desirable that the natural frequency of the board be made above 10 Hz. (to enhance regidity)

c. The top of the board may be supported by stays.

d. Relays may be installed at the lower section of power switchboards.

e. Locks on double doors be firmly placed to minimize the space.

f. Support space shall not be too large. With a large support space, internal wires vibrate extensively and may contact internal equipment and devices or break terminal connections.

g. Refer to control measures for transformers for other hints.

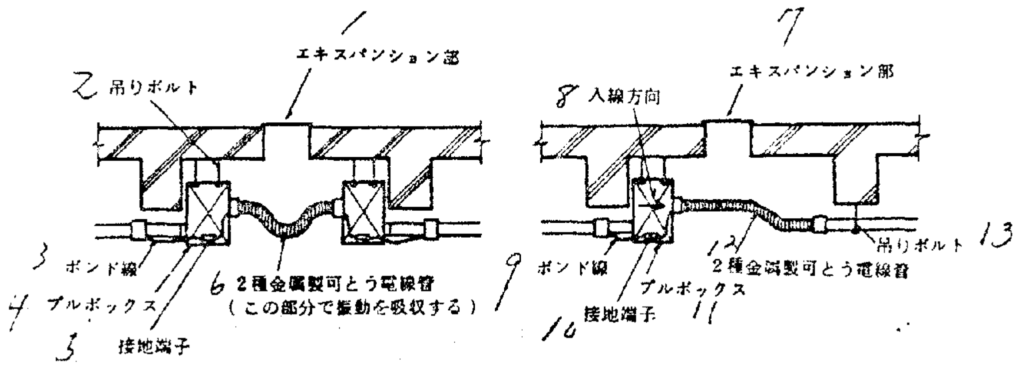
iii) <u>Others</u>

a. Static type relays and non-contact relays shall be preferably used (Countermeasures may be provided to conventional mechanical or electromagnetic contacts).

b. Equipment and devices with vibrating parts such as relays should be installed at the lower section of a board.

c. Flexible wires shall be used for wiring in the area of expansion in buildings. (Figure)

d. A frame made of angle materials shall be installed so the load will not concentrate on the hanging metals of horizontal main lines (Also for the purpose of preventing vibration).



Key-1. expansion
2. hanging bolt
3. bond wire
4. pull box
5. earth terminal
6. bimetal flexible conduit tube (vibrations are absorbed here)
7. expansion
8. inlet direction
9. bond wire
10. earth terminal
11. pull box
12. bimetal flexible conduit tube
13. hanging bolt

e. Cables above the cable rack shall be pulled together and fastened to the rack.

f. Hanging bolts for the main line shall not be longer than necessary and required.

g. Supports for bends and joints of piping shall be firmly installed.

h. Bath ducts and transformer connections shall be flexible.

i. Stabilizers and rubber covers shall be used to safeguard rising main lines and others which might contact building materials by oscillations, as a protection against impact.

j. Mounting of heavy items on walls shall be avoided. If necessary for good reasons, those which are wall mounted shall besupported by a pedestal which stands on the follor.

iv) <u>Batteries</u>

a. Batteries shall not be simply placed on a floor, but shall be framed in and fastened to the floor.

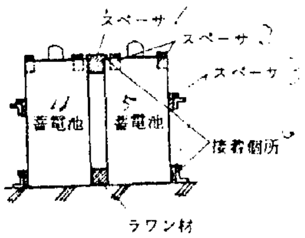
b. Batteries which can beplaced on a pedestal shall be set on a pedestal which will be fastened to the floor and the wall.

c. Elastic spacers shall beinstalled between electrolytic cells for the prevention of vibration and collison (Figure)

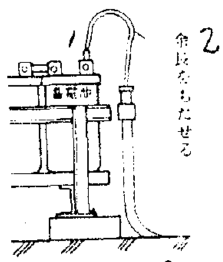
d. Angle stabilizers shall be provided for batteries contained in a cubicle.

e. Wiring joints shall be flexible (use flexible conductors).

f. Some slack shall be given in wiring (Figure).



8 スペーサそう入図



蓄電池への接続 3

Key-1. spacer
2. spacer
3. spacer
4. battery
5. binding site
7. lauan material
8. installation of spacers

key-1. battery
2. give some slack
3. connection to batteries

v) <u>Home Power Generation Installations</u>

a. Installation bolts chosen for prime mover and generator itself shall be sufficiently strong.

b. Stoppers shall be installed on the lower section of the main body. (Figure)

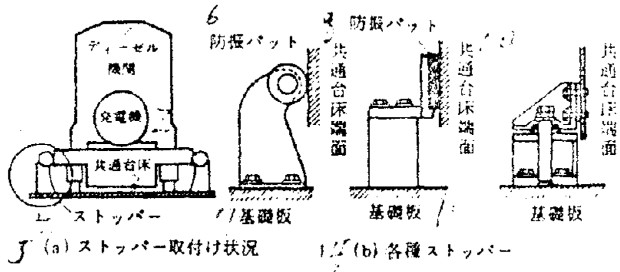
c. Rubber vibration insulators to be applied shall have high unit strength.

d. The joints of the main body to exhaust pipes, cooling water pipes, oil pipes and wiring shall be flexible.

e. An oil service tank on a pedestal shall be fixed, if possible, all around to the pedestal, and sections of the elements shall be selected so that the elements of the framework for the pedestal shall not buckle.

f. Starting air tanks shall be supported, for instance, by stays from a wall to prevent them from falling.

Examples of Stoppers



key-1. diesel engine
2. generator
3. common bench floor
4. stopper
5. (a) Stopper fixing condition
6. vibration proof pad
7. base plate
8. common bench floor end surface
9. vibration proof pad
10. common bench floor end surface
11. base plate
12. common bench floor end surface
13. base plate
14. (b) various stoppers

vi) <u>Lighting Installations</u>

a. Lighting fixtures to be used shall be desirably an embeded type, semi-embeded type or directly attached type with less inertia.
b. Pipes and chains shall not be longer than necessary and required when hanging fixtures directly from a ceiling.
c. Strength of hanging metal fittings for pipe hanging light fixtures shall be appropriate.
d. Stabilizers shall be installed for chain hanging light fixtures to prevent them from bouncing by the force of seismic motions.
e. Light fixtures with acryl or other louvers shall be so designed to protect the louvers from falling off by the force of seismic motions.
f. Sockets of fluorscent lamps shall be so designed to protect lamps from falling off by the oscillations of seismic motions.
g. Hanging strength shall be fully adquate for heavy lighting fixtures (chandeliers).
h. Hanging bolts and metal fittings of lighting fixtures shall be strong enough not to be deformed or detached by the force of seismic motions.

vii) Telephone Installations

a. Switchboards shall be firmly installed lest they will turn over.

b. Wiring for terminals shall be given some slack.

viii) Broadcasting Installations

a. Amplifiers shall be fitted securely.

b. Wall speakers shall be desirably fixed with mounting aids such as metal fittings rather than be hooked up.

2. WATER SUPPLY AND DRAIN SYSTEM

(1) Outline

The water supply and drain services are indispensible functions when buildings will be used uninterrupted after earthquakes. In some past earthquake damage cases, the water supply was cut due to the destruction of water tanks and broken piping. Simultaneously this breakdown caused secondary damage that manifested as sporadic floodings in the buildings.

Pumps are either directly supported on the floor or provided with vibration proof bearings. In the latter case, it is necessary to install stoppers and flexible joints between pipings and joints.

Many water tanks have a questionable main body from the aspect of earthquake resistivity, and also not a few of them are placed on a pedestal which lacks in strength. "FRP Water Tank Seismic Design Standards" was prepared by Reinforced Plastic Technology Association (Incorporation). Please refer to this publication for control measures specifically for FRP water tanks which were markedly the victem of the recent earthquake casualties among all types of water tanks. (It is justifiable to consider that guidelines in this book and the standards above mentioned basically aim to achieve a similar level of earthquake resistance.)

Concerning pedestals (a base for items to be elevated and installed on top of an building), it is necessary to design then with full rigidity by referring to section VI-4. (Generally speaking, new ones shall be given much more strength compared to existing pedestals.)

Independent elevated water tanks (independent outside a building) can be itself called a structure, and so treated differently from other installations starting with the design earthquake loads. The design seismic intensity K shall be given based upon the oscillation characteristics of the tanks concerned just as for a building itself. However, in this book, we leave no doubt about the value of design seismic intensity and designate it as the 0.5 level presuming an elastic design. In this case, it is assumed that the structure of the pedestals are constructed of steel with proper ductility (for instance, a static truss that will collapse by the buckling of one diagonal element cannot be applicable).

Generally, septic tanks are embedded in the ground, and therefore it is necessary to secure and harden the ground fully to prevent it from sinking and shifting. Also, since water is lighter than soil, water tanks may float up when sands are fluidized during earthquakes. When the bottom and circumference of FRP septic tanks are to be enclosed with a concrete box, this box shall be constructed with reinforced concrete in consideration of earth pressure during earthquakes.

Water supply and drain pipes in buildings shall be provided with flexible joints at essential points so that they can cope with the relative displacements occurring in joints with water tanks and machinery and equipment as well as to cope with the interlayer displacement of buildings. Embedded pipes that connect to water pipes and sewage pipes inside and outside buildings shall be, if possible, given considerations not to be directly affected (not to receive forced deformation) by the destruction of the ground itself (sinking and shifting) (For instance, double piping is suggested).

The installations that supply cooling water to internal combustion engines that drive home generators described in previous section IV-1 shall be designed in accordance with the guidelines for water supply system described in this section. The importance of the installations shall be duely reminded.

As a water supply related substitution system, it is desirable to secure emergency water, particularly, drinking water. One way to do this is to keep a vessel filled with drinking water. Also, it will be necessary to think in advance of proper directions not to waste water remaining in water tanks in the building immediately after earthquakes.

(2) Check Points

i) Elevated Water Tanks (Including Water Tanks in Machine Rooms on Intermediate Floors)

a. Foundations shall be large enough for pedestals, and the concrete cover for the anchor bolts shall be sufficiently provided.

b. Fixation of pedestals and foundations shall be firmly secured.

c. Foundations shall be reinforced with steel and welded to the steel in the slab to become integral with the tanks.

d. Steel reinforced pedestals shall be given extra rigidity by braces and corner gusset plates as well as shall be free of manufacturing defects.

e. Water tanks shall be securely fastened to the pedestals, using clasps as broad as possible.

f. If circumstances allow, stays shall be installed on water tanks in the direction vulnerable to displacement.

g. Joints of piping on a water tank shall be given flexibility by use of flexible joints.

h. When heavy and large valves are attached to piping, valves shall be supported by pedestals.

i. Water tank connecting pipes shall be made as long as possible in the distance from where pipes penetrate through the building structure to the water tank.

j. Foundations for piping pedestals shall be directly installed on the floor surface to enhance the bond to the floor surface, or shall be made larger than required to prevent them from slipping, shifting and turning over.

k. In order to prevent secondary damages due to the spilling of water, some measures shall be taken to prevent the water from entering electric rooms, elevator shafts and interiors of the buildings.

ii) Pumps

a. Fixing bolts shall be large enough to withstand the force of seismic motions.
b. In case pumps are provided with earthquake proof bearings, stoppers shall be installed.
c. For joints, flexible joints shall be installed.
d. Piping and wiring shall be given extra slack or shall use flexible conduit tubes.

3. COOLING, HEATING, AIR CONDITIONING AND VENTILATING SYSTEMS

(1) Outline

Including ventilation installations, cooling, heating and air conditioning systems are different from electrical and water supply and drain systems, and are not of vital importance to buildings. However, in recent years, these facilities have spread remarkably, and the contents have been sophisticated. Seismic designs for cooling, heating and air conditioning facilities shall be dealt with stressing economical efficiency.

Nevertheless, air conditioning installations which primarily render cooling service are necessary particularly for the maintenance of the functions of electronic machinery and equipment headed by computers. In such a case, loss of cooling functions long after earthquakes must be avoided.

Pumps used for these systems shall be treated in accordance with the section relating to water supply and drain system.

The inherent design of some refrigerators cause vibrations, and vibration proof bearings must be provided. In such a case, stoppers are necessary. Even when they are directly supported on the floor, it is necessary to provide some countermeasures in connection with installation as will be shown in Chapter VI lest they shift and turn over.

Cooling towers which function in conjunction with refrigerators are usually supported by pedestals on the top of buildings. Tectonic installation methods mainly dealt with pedestals are important.

Regarding boilers, tectonic installation methods are the essential seismic point.

Air conditioning units and air blowing and exhausting units both vibrate and must be given vibtation proof supports. Tectonic installation methods including stoppers are also very important.

Cooling and heating peripherals (fan coil units) are often independetly erected from the floor or mounted on walls. In past earthquake damage cases, there were numerous reports relating to fallen or disconnected peripherals due to simple detail and cheap work of fittings. It is desirable that peripherals shall be supported both from the floor and the wall.

Passages (ducts) for routing air itself, and piping for distribution of hot water and steam are indispensible components of cooling, heating and air conditioning system.

Ducts which are light and easy to deflect may appear advantageous from the point of earthquake loads and relative displacements, but they vibrate widely and are vulnerable to damages such as leaking of air depending upon how they are hung. The basic policy for seismic designs, therefore, shall be preferably the adoption of a suspended pedestal structure that constricts vibtation. Please refer to Chapter VII for concrete types of this structure.

Piping shall be handled in accordance with the guidelines for piping in the section relating to water supply and drain system.

Ventilation installations can be viewed by functional system as a component in cooling heating and air conditioning system. In general, ventilation installations are expected to have a comparatively low required earthquake resistance level as a member of functional systems. Also, the secondary damages inflicted by damaged air blowing and exhausting units and damaged ducts are not expected to amount to a great problem. Necessity for the seismic designs is slightly lower in this system compared to other installation systems, and likewise the priority for improvements can be also considered low.

However, it is reminded that fresh air is necessary both for machinery and equipment and people in a nearly closed space such as a basement room.

(2) Check Points

i) Boilers

a. The main body shall be fastened firmly with bolts large enough to endure the force of earthquakes.
b. The main body and the channel base shall be firmly fixed.
c. Connecting pipes such as water supply pipes and oil pipes shall use flexible joints.
d. Flue duct bends and joints shall be strong enough to endure the force of earthquakes.

ii) Refrigerators

a. Foundation concrete shall be sufficiently large.
b. Foundations shall be made integral with the slab by use of reinforcing steel.
c. For refrigerators supported by vibration proof springs,
   * Vibration proof springs shall be fixed to the main body and the foundation.
   * Stoppers shall be installed
d. Connecting pipes shall be flexible.
   (Same treatment shall be given even for absorption refrigerators with minimal vibtations)

iii) Air Conditioning Units

a. Those with vibration proof supports,

* High strength and endurance vibration proof materials (devices) shall be used.

* Joints of ducts andpiping shall be flexible. (Use flexible joints and canvas ducts)

iv) Air Blowers

a. The type of air blowers which is hung from the ceiling shall be avoided. If this type must be used, vibration shall beprevented by installing angle materials fixed to the ceiling slab.

b. Stoppers shall be installed.

c. Canvas ducts shall be used for duct joints. It is desirable that high strength canvas ducts be used.

v) Package Air Conditioners

a. For those not fixed but merely set upon a vibration proof pad, stays and stoppers shall be installed to prevent them from vibrating and shifting.

b. Flexible joints shall beinstalled for connecting pipes to add flexibility.

vi) Radiators

a. Cast iron radiators are heavy and large, and must be fastened to floors or walls.

b. Sufficient strength shall be given to piping joints.

vvii) Cooling Towers

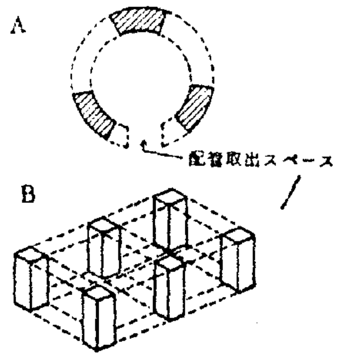
a. Foundation concrete shall have a large area for stable shape or shall not be segmented independently but shall be continuous for stability (Figure)

b. Foundation concrete shall be integral with the salb by use of reinforcing steel.

c. Anchor bolts to be used for fixation of legs shall be of sufficient strength.

d. When bending of legs is anticipated, reinforcing material such as stays shall be installed from the floor.

e. Flexible joints shall be installed for piping joints.

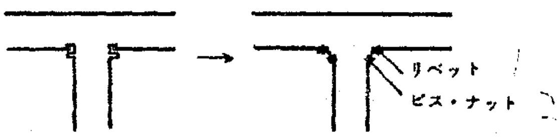


Key-1. piping outlet space

viii) Ducts

a. Ducts hung by hanging bolts from a ceiling, shall be stabilized with angle materials.

b. Sufficient strength shall be given to branch sections. (Figure)



Key-1. rivet
2. vis nut

4. GAS SYSTEM

(1) Outline

For urban gas service, the targets of the seismic designs are piping and terminal appliances which use gas. Gas installations as a functiona system cannot be deemed absolutely essential for general buildings. Since gas itself is toxic, the seismic designs must be primarily conceived from the point of secondary damages.

Piping shall be given a high earthquake resistivity compared to piping for other purposes so that it can endure interlayer displacement and relative displacement. Particularly, attention must be paid to avoid resonance with buildings.

incidentally, efficiency of couplings shall be considered in calculating stress.

Individual terminal appliances which use gas shall be given serious attention not because of their functions but because of the dangers of the possible fire hazards as secondary damages. For this purpose, prevention of shifting and falling are demanded. Different from oil stoves, those with an automatic fire extinguisher are not available. Rather than depending on the earthquake resistivity of the appliances themselves, it is necessary to formulate a system where people close plugs of appliances and gas main plugs as a part of a compensating design.

It is difficult to make a decision as to whether or not to shut the gas supply for the entire building, but the measures to be complied with during earthquakes can be formulated beforehand and make known to the public.

Also, it is desirable that the feasibility of adopting an automatic gas leak preventive device be investigated.

(2) Check Points

a. Piping in sections that enter buildings and connect to appliances shall be given flexibility.
b. Piping inside buildings shall be fastened to structures.
c. Terminal appliances shall be fastened to floors and walls.
d. Gas leak automatic preventive devices shall be installed.
e. Gas bottles shall be connected to structures or strong independent frames for the prevention of falling shall beprovided.

5. ELEVATOR SYSTEM

(1) Outline

Elevators are either of the cable type or oil pressure type, but the former dominates in nunber. Also, during recent earthquakes, cable elevator damages are notable. Most of the damages were attributable to derailing of balancing weights and shifting of lifters and motor generators.

(2) Check Points

a. Derailing of balancing weight shall be prevented. Specifically, the balancing weight shall interlock 10mm more than the anticipated deflection of rails.

b. Lifters and motor generators shall be installed in such a manner as to be able to endure horizontal seismic instensity of 0.1 and vertical seismic intensity of 0.5. If shifting is anticipated, stoppers shall be installed.

c. Boexes and balancing weights shall be so designed to withstand a horizontal seismic intensity of 0.6.

d. When machinery and equipment such as control boards are anticipated to turn over, they shall be fastened with stays.

e. With elevators above 50m lift (from ground surface), vibration stoppers and guards are provided to prevent cables and cords from oscillating.

f. Elevators shall be equipped with an earthquake control system and shall have an earthquake detector.

g. An emergency lamp shall be installed in the box.

k. Considerations must be given to prevent water damages due to broken water tanks and water cooling towers.

V. DYNAMIC CALCULATION METHOD

1. DESIGN FLOOR SEISMIC INTENSITY

As described in section II-2, there are some proposals already made in regard to input seismic intensity K_1 (floor response; seismic intensity on the floor of each story; floor seismic intensity) for seismic designs of building designs. Many of them fall into the range of Figure 5.1. According to this chart, once the floor on which machinery and equipment installations are to be installed is given, an input value can be obtained correspondingly to the floor location. Using this as a basis, an earthquake design load for the machinery and equipmet installations can be determined.

The above described range is comparatively narrow, and the values within the range are not significantly different. If it is our intention to determine the one and only value, it will be best to adopt a distribution which designates 0.3 for the first floor and 1.0 for the roof top. This K_1 value can be expressed in an equation as follows.

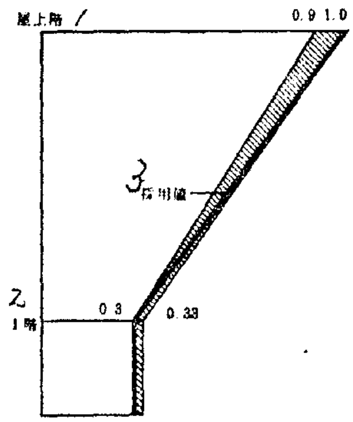
$$K_1 = \begin{cases} 0.3 + 0.7\,\dfrac{n-1}{N} & : \text{first floor} \\ 0.3 & : \text{basement} \end{cases}$$

whereas, N:Number of stories above the ground of the building.

n:The story where machinery and equipment are to be installed.

Incidentally, vertical seismic intensity Kv is designated as 1/3 to 1/2 of K_1.

Figure 5.1 Design Floor Seismic Intensity



Key-1. top floor
2. ground floor
3. adopted value

2. OSCILLATION CHARACTERISTICS OF MACHINERY AND EQUIPMENT AND RESPONSE MAGNIFICATION

Behavior of machinery and equipment relative to floor during earthquakes is the same kind as the beharior of the main structure relative to the ground. Evidentally, the size of the response of the machinery and equipment to movemets of the floor can be greatly affected by the oscillation characteristics (proper osciklation, proper oscillation form and damping constant) of the machinery and equipment and bearings.

Seismic designs for machinery and equipment are made by simplifying these conditions to a great extent. Therefore, the concept of response magnification is adopted, which expresses the ratio of a seismic intensity that affects the machinery and equipment to a seismic intensity of the floor on which the machinery and equipment are placed.

It is adequate to adopt the values as in Table 5.1 for the design value K_2 of this response magnification in consideration of the current trends.

Table 5.1 Design Response Magnification

conditions of machinery and equipment and bearings	response magnification k_2
The main body of machinery or equipment is rigid enough and suppo rted directly on the floor (those with total system vibration above 20 Hz)	1.0
The main body of machinery or equipment is set on vibration proof bearings, supported by high pedestals or stands independently.	2.0

Also, when it is necessary to know the relative displacement of various parts of buildings in designing piping, the following values can be used as standard values.

interlayer displacement Do 1/50 radian

expansion relative displacement Eo 1/100 x story with the expansion

whereas, if the main structure is of reinforced pure rahmen, it is better to give values about double of these values.

3. DESIGN LOAD AND STRUCTURE CALCULATION FOR MACHINERY AND EQUIPMENT

Design Seismic Instensity K relative to machinery and equipment is the value obtained from multiplying the standard design seismic intensity (Input Seismic Intensity K_1 x Response Magnification K_2) given in preceeding section 2 by Imortance Factor I established in section II-6. Specifically,

$$K = K_1 \cdot K_2 \cdot I$$

However, as a real case, it is better to give the following ceiling.

$$K_1 \cdot K_2 \leqq 1.5$$

Also, Piping Design Interlayer Displacement D and Expansion Relative Displacement E are the products of the standard values Do and Eo established in the preceeding section multiplied by the importance factor. Specifically,

$$D = D_0 I$$
$$E = E_0 I$$

However, similarly as in the case of K, a ceiling is set.

$$D \leqq 1/120$$
$$E \leqq 1/75 \times 3$$

The above dynamic calculations for load values are made on the premise of elastic designs.

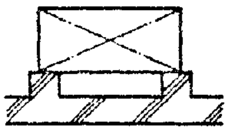
VI. SEISMIC DESIGN FOR BEARINGS

1. FOUNDATIONS

Much of the damage to machinery and equipment installations is attributable to improperly equipped foundations. Consequently, seismic designs for foundations are extremely vital.

Foundations are classified by shape and shown in Table 6.1.

Table 6.1 : Shapes of Foundations

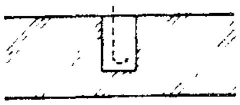
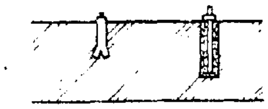
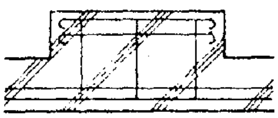
shapes	outlines
(i) 	Floor slab itself or the same surface as the floor slab
(ii) 	Installation surface made on a level above the floor slab
(iii) 	Foundation is partially raised from the floor slab, or raised surfaces are constructed. Some are merely blocks placed on the slab.

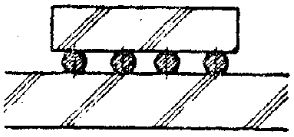
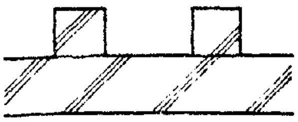
Among them, (ii) and (iii) are sometimes installed on the roof top, and it is necessary to attend to the waterproof layer treatment.

Also, Table 6.2 comments on the shapes by internal structure of the foundations.

Table 6.2: Shapes of Foundations and Comments On Earthquake Resistance

Slab itself or same suface as slab		
Drawing	Machinery & equipment installed	outline
	receiving board, power switchboard, surveillance board	After setting the position of anchor bolts a slab is installed (embedded anchor). This is not suitable for heavy machinery and equipment since the embedding depth anchors is not deep enough, but suitable for boards and light machinery and equipment.

	receiving boards, power switchboards, surveillance boards	Foundation with boxed anchors. The part which was installed later is sometimes called grout (secondary concrete). As in the above column, this is not suitable for heavy machinery and equipment or for those that vibrate.
	same as above	Post-driven anchors (expansion anchors, chemical anchors) are installed. Similarly to the above, great grength cannot be expected.
Installation surface on a level above floor slab		
	transformers, oil tanks, pumps, blowers, generators	Reinforcing steel is preliminary raised from the steel placed in the slab, and the concrete is poured in afterwards. The foundation and the structure are made integral by using reinforcing steel. It is generally used for heavy machinery and equipment required to be vibration proof. The foundation is stable and good in earthquake resistance.
	generators	Steel material (reinforcing steel, H-shaped steel, channel steel) base is assembled before the foundation is installed. It is used for heavy machinery and equipment and large machinery and equipment units. It is desirable that steel anchors stick out of the body

	transformers small pumps, blowers	The foundation is installed after the slab surface is made rough, simply on to of the slab. Sometimes, reinforcing steel materials are not embedded. At any rate, it may shift during an earthquake since it is not connected to the body, and evidentally it is poor in earthquake resistance.
	pumps, blowers, refrigerators	Vibration proof materials are placed on the slab, and the foundation is installed on top of these materials (vibration proof foundation). Although the vibration proof effect is, large, the foundation may shift or fall off the vibration proof materials by the horizontal earthquake load applied to the foundation, machinery and equipment.
Partially raised from the floor slab		
	cooling towers, elevated water tanks, tanks (on pedestals)	Concrete benches or blocks are merely placed on top of the slab. There are many actual examples, but it is extremely inadequate as a foundation for pedestals. Shifting and falling during earthquakes are anticipated.
	cooling towers, elevated water tanks, tanks (on pedestals)	Reinforcing steel was welded to the reinforcing steel of the slab, and subsequently the concrete was poured in. This foundation is more stable than the above example. Suitable for heavy machinery.

2. ANCHOR BOLTS

(1) Variety of Anchor Bolts

• Anchoring of machinery and equipment to the structure with anchor bolts and building of foundations are often accomplished after the structure is virtually completed. This will not result in full strength anchoring and often is connected to the damage during earthquakes. It is important to deal with these works with a clear plan from the initial stage of designing.

The following are the variety of anchor bolts.

a. embedded anchors

b. boxed anchors

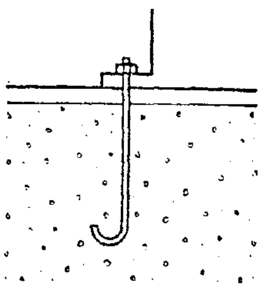
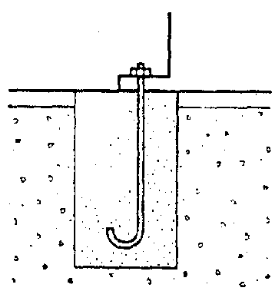
c. post-driven anchors { mechanical anchors
  resin anchors

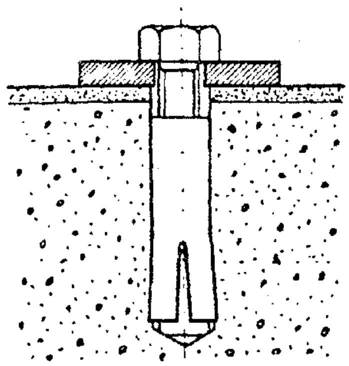
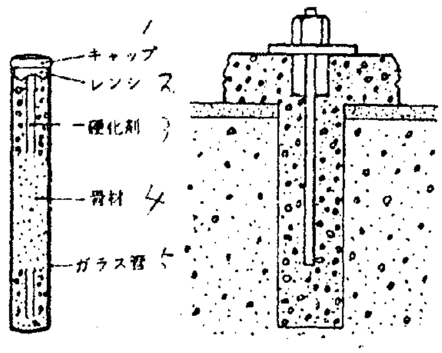
Essentials of these anchors are shown in Table 6.3.

Incidentally, as a shape of the tip of anchor bolts, the J-type and those with anchor heads (stud, nut and end plate) are desirable.

(Guidelines in paragraph (2) and below of this section are based upon "Guidelines for Home Power Generation Installation Seismic Designs (proposal)".

Table 6.3 Variety of Anchor Bolts

	embedded anchors	boxed anchors
drawing		

characteristics	Anchor bolts are embeded in the foundation concrete itself, and have a large reaction load than boxed anchors. Consequently, this anchor is desirable when a heavy load is installed on it. Incidentally, since the setting will be completed with the installation of the foundation concrete, high work precision is required.	When installing concrete, a hollo. boxed-in space is set aside for setting of anchor bolts. When installing a machine or equipment, anchor bolts are set in position and embedded with mortar. Since the anchor is indirectly placed on the foundation through the mortar which is filled in at a later time, reaction strength is not so large, and its relaibility is poor.
building process	Bolts are set accurately in position ↓ Installation of doundation concrete	Installation of foundation ↓ Temporary placement of machines and bolts ↓ filling of mortar (grout)
	mechanical anchors	resin anchors
drawing		
	A prescribed hole is drilled in the concrete and an anchor is set in position. The lower part of the anchor is mechanically expanded to	Key-1. cap, 2. retainer, 3 hardener 4. aggregate, 5. glass tube A glass tubular capsule (figure to the left) filled with resin, hardener and

characteristics	bind the anchor to the concrete. It indicates significant strength in a laboratory. At an actual work site, the reaction strength is anticipated to fluctuate largely depending upon the condition of the concrete and the appropriateness of the work. A large safety factor must be taken into account. Some makers put out various structure types on the market. When using these, carefully work with them following respective instructions.	aggregate is inserted into a vertical hole punched in theconcrete, and an anchor bolt is driven in on it by the rotational impact of an impact drill. The resin, hardener, aggregate and the crushed glass tube are mixed, harden and are bonded by the bonding strength. Comparatively high strength can be obtained. Incidentally, threaded bolts or special reinforcing steel shall be used in stead of a round bar.
building process	Make a hole in concrete embed the bolt tighten the bolt	Puncture a hole in concrete and insert the capsule Attach a bolt to a hammer drill or a machine drill and drive it by ratational impact.

(2) Reaction Load and Shearing Force Applied to Anchor Bolts

i) Reaction Load

Assuming a rigid machine, let usthink of the direction to which the machine is likely to fall.

Reaction load per one bolt Rb applied by the force of an earthquake can be given by the following equation in accordance with the balance with the moment. (see figure)

$$R_b = \frac{F \cdot h - (W - F/2) \cdot \ell/2}{n/2 \cdot \ell}$$

...... (1 - 1)

where, F: horizontal load by the earthquake

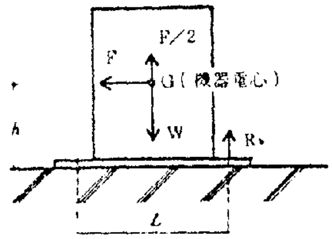
h: height to G from the installation floor surface

ℓ: bolt span viewed from the direction to which the machine is likely to fall

F/2: vertical load

n: total number of bolts

w: weight of the machine



Key-1. center of gravity of the machine

ii) Shearing Stress

It is assumed that the horizontal force applied to the machine is equally received by all of the anchor bolts. Shearing stress τ is given by the following equation.

$$\tau = \frac{F}{n \cdot A} \qquad \cdots\cdots (1-2)$$

where, A is the nominal sectional area per one bolt.

iii) Selection of Anchor Bolts

1) When $Rb < 0$

Reaction load by the earthquake will not affect bolts.

Bolts shall be selected in such a way as to make the shearing stress below the allowable limit calculated by equation (1-2)

2) When $Rb \geqq 0$

Reaction load affects bolts. Obtain τ by equation (1-2), then select bolts according to the following procedures.

a. $Rb \geqq 0$ and $\tau \leqq 4.5 kg/mm^2$

Bolts shall be selected in such a way as to make Rb below the allowable reaction load limit to be discussed in paragraph (3).

b. $Rb \geqq 0$ and $\tau > 4.5$ kg/mm^2

Bolts shall be selected in such a way as to make Rb below the allowable limit. Additionally, the tensile stress of the bolts shall be obtained, and the strength of the bolts that simultaniously receive tensile and shearing forces shall be verified based upon thetable below.

iv) Values from Bolt Allowable Stress Table 6.4 shall be used.

Table 6.4 Allowable Stress of Bolts

1 ボルト径 d mm	2 降伏点 (Sy)	3 引張強さ(Su)	4 短期許容応力	
			5 引 張 (ft)	6 せん断(fs)
d ≦ 16	25 以上	41 ~ 52	18.0	13.5
16<d ≦40	24 以上 7	同上 8	18.0	13.5
d >40	22 以上	同上	16.5	12.3

9 単位 kg／㎟

Key-1. volt diameter, 2. yield point, 3. tensle strength, 4. short term allowable stress, 5. tensile, 6. shearing, 7. above, 8. same as above 9. unit

Strength of bolts which simultaneously receive tensile and shearing forces shall be verified by the following.

$$f_{ts} = 14\, f_t - 1.6\, \tau \quad \text{and} \quad f_{ts} \leqq f_t\ ,\ \tau \leqq f_s$$

When tensile stress of bolts is below ft, the strength of bolts is adequate.

fts: allowable tensile force on bolts that also receive shearing force

ft : Allowable tensile force on bolts that only receive tensile force (values in Table 6.4)

τ : Shearing force that affects bolts.

fs : Allowable shearing force on bolts that only receive shearing force (values in Table 6.4)

(3) Allowable Reaction Load on Anchor Bolts

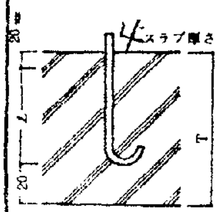
Table 6.5 Allowable Reaction Load on Anchor Bolts

setting on a general floor or ceiling slab

boxed anchors

1 ボルト径	2 スラブ厚さ(mm) 120	150	180	200
M10	270kg	400kg	550kg	660kg
M12	320	450	630	750
M16	430	590	760	910
M20	540	740	950	1050
M24	650	890	1140	1200

3(この表の太線の下側はすべて1200kg)



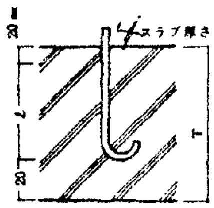
If the shape and the embedding depth of bolts are different from what is shown in the drawing, follow "*" below.

key-1. bolt diameter
2. slab thickness
3. (all below the fat line in the table are 1200 kg).
4. thickness of slab

embedded anchors

1 ボルト径	2 スラブ厚さ(mm) 120	150	180	200
M10	400kg	610kg	830kg	990kg
M12	480	670	950	1130
M16	650	890	1140	1200
M20	810	1110	1200	1200
M24	970	1200	1200	1200

3(この表の太線の下側はすべて1200kg)



If the shape and the embedding depth of bolts are different from what is shown in the drawing, follow "*" below.

key-1. bolt diameter
2. slab thickness
3. (all below the fat line in the table are 1200 kg).
4. thickness of slab

(Same both for boxed and embedded anchors)

Use the larger result of the 2 equations described below.

$$\begin{cases} F_1 = \pi d \ell_1 f c \\ F_2 = 1.5 \pi d \ell_2 f c \end{cases}$$

setting on a hard concrete foundation

Where, F_1: Allowable Reaction Load (K_g) obtained by using ℓ_1 in the drawing to the right.

F_2: Allowable Reaction Load (K_g) obtained using ℓ_2 in the drawing to the right.

(It is assumed that the hook shares 1/3 and the shaft shares 2/3 of the reaction load.)

d: nominal diameter of bolt (cm)

ℓ_1: bolt embedding depth (cm)

ℓ_2: bolt embedding depth (cm)

} refer to the drawing to the right

fc: Allowable adhearing stress level to reinforced concrete

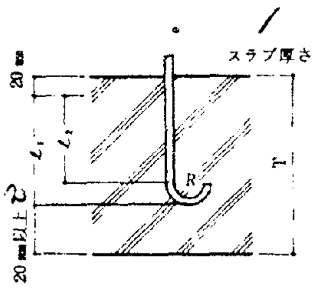
$$fc = \frac{9}{100} Fc \text{ and } fc \leqq 20.25 \text{ kg}$$

Fc: concrete design standard strength

boxed anchors........ Fc ≒ 120 kg/cm^2

embedded anchors..... Fc ≒ 180 kg/cm^2

※



Key-1. thickness of slab

2. above 20mm

setting on a general floor or ceiling slab

mechanical anchors	resin anchors

mechanical anchors

Exactly as shown in the table.

¹ボルト径	²引抜き許容荷重
M 6	80 kg
M 8	100
M 10	140
M 12	220
M 16	360
M 20	500
M 22	550

(note) It is assumed that anchor bolts installed on the slab are unable to bear more than 1200 kg per bolt.

Key-1. bolt diameter

2. allowable reaction load

resin anchors

Assuming that the anchor bolts installed in the slab are unable to bear more than 1200 kg of reaction load per bolt, the allowable reaction load for bolts above M 10 is designated as 1200 kg.

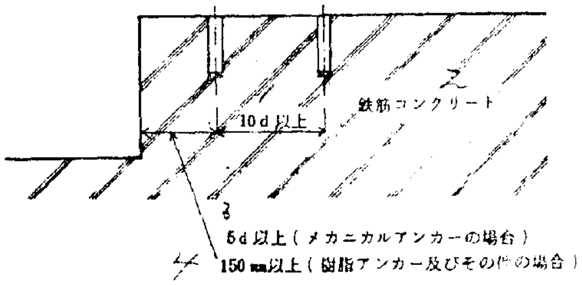
<table>
<tr>
<td>setting on a hard concrete foundation</td>
<td>Same as above.</td>
<td>Exactly as shown in the table.
<table>
<tr><th>ボルト径</th><th>引抜き許容荷重</th></tr>
<tr><td>M 10</td><td>1300 kg</td></tr>
<tr><td>M 12</td><td>2000</td></tr>
<tr><td>M 16</td><td>3600</td></tr>
<tr><td>M 18</td><td>4500</td></tr>
<tr><td>M 22</td><td>6800</td></tr>
<tr><td>M 24</td><td>8100</td></tr>
<tr><td>M 30</td><td>12700</td></tr>
<tr><td>M 36</td><td>18300</td></tr>
</table>
Key-1. bolt diameter
2. allowable reaction load</td>
</tr>
</table>

(4) Anchor Bolt Arrangement

For arrangement of anchor bolts, machine makers often designate the number and the location of the anchor bolts. For those specified, installation standards and points suggested by the makers shall be followed.

For installing other machinery and equipment, the following points shall be considered.

a. For those which will be fixed by means of steel material such as transformers, the length of the steel material shall be sufficient and the bolt span shall be ample if possible. (Reaction load per bolt will be reduced.)
b. Considerations as shown in the figure below shall be given for the anchor driving locations.
c. Embedded anchors, if possible, shall be welded to the reinforcing steel of slabs and foundations before the concrete is poured.



(note) In case of concrete without reinforcement, it is a standard practice to adopt measurements two times more than the above. However, if the actual anchor supporting strength is less than 1/2 of the allowable reaction load described in paragraph (3), the measurements in the drawing can be adopted.

Key-1. above 10d, 2. reinforced concrete, 3. above 5d (for mechanical anchors), 4. above 150mm (for resin anchors and others)

3. DIRECT BEARINGS

Types of bearings for machinery and equipment can be classified as follows according to the location relative to the supporting body (floor, ceiling).

a. direct bearing
b. pedestal bearing
c. swing bearing

Adirect bearing among these can be divided into the following two classes according to whether or not the machine itself vibrates.

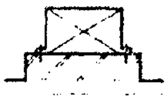
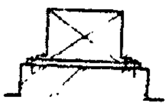
a. fixed bearing
b. elastic bearing (vibration proof bearing)

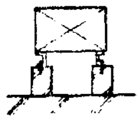
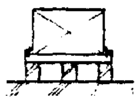
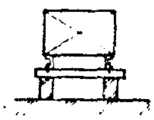
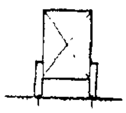
Vibration proof bearings encounter lots of seismic problems, and installation of stoppers is necessary as shown in Section VII-5.

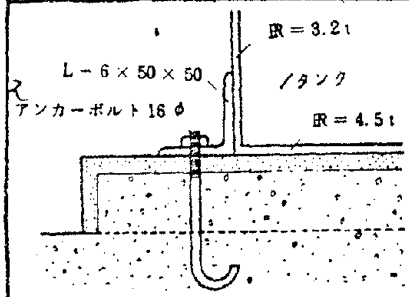
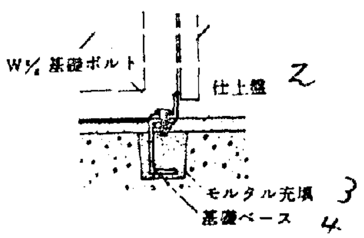
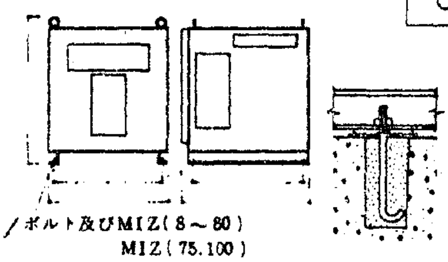
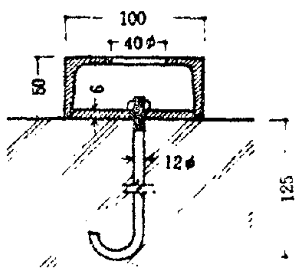
Types and concrete examples of fixed bearings are shown in Tables 6.6 and 6.7, and types and concrete examples of elastic bearings are shown in Tables 6.8 and 6.9.

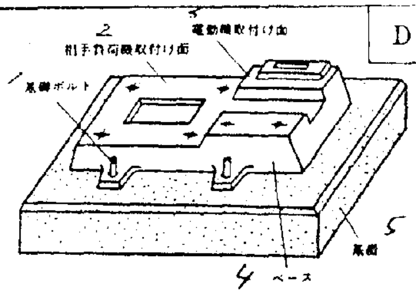
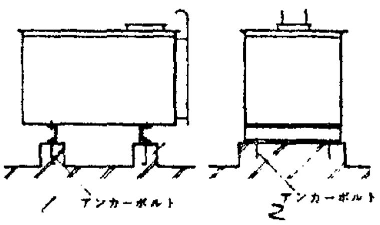
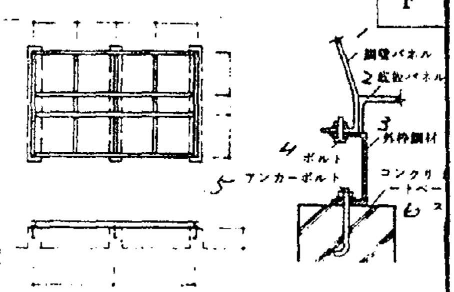
Incidentally, please refer to section VII-4 for pedestal bearings and Section VIII-2 for swing bearings.

Table 6.6 Types of Fixed Bearings

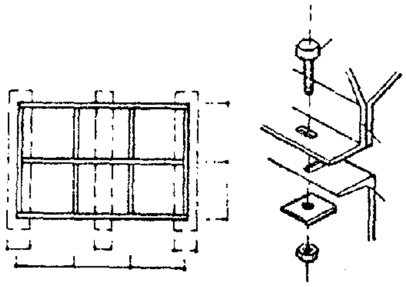
code	drawing	machinery and equipment concerned	outline and comments
A		oil tanks	machinery and equipment---anchor bolts---foundation * Be mindful of the strength of the bolts and the anchoring sites of the machine.
B		power switchboards surveillance board	machine---anchor bolts---embedded steeel materials---foundation * Same as A, but a slightly higher strength can be expected because of the embedded steel materials.
C		transformers power switchboards	machine---anchor bolts---steel materials---anchor bolts---foundation * Be mindful of the strength of the fixation of the machine and the steel materials.
D		pumps	machine---anchor bolts---stell base---foundation---machine---bolts---base---anchor bolts---bolts---foundation * Point is the bolt strength.
		cooling towers water tanks	machine---bolts---steel materials---independent foundation---slab

E			* This is not sufficient for heavy machinery and equipment. Point is in the bolt fixation site. Incidentally, the shape of the foundation shall be stable and the foundation shall be integral with the slab.
F		water tanks cooling towers	machine---bolts---steel pedestal---independent foundation---anchor bolts--slab *The poing is in the fixation of the pedestal with the machine and the foundation. As for the foundation, the same can be said as in E.
G		water tanks cooling towers	machine---welded bolts---steel materials---(welding)---steel bench---(anchor bolts)---slab * Same as in E. Be careful of those with unfixed benches and foundations.
E		pressure water tanks cooling towers crude fire foam tanks	machine---legs of the main body---anchor bolts---foundation * Be careful of the fixation sites of the base, legs and the foundation.

EXAMPLE	type code	explanations and comments
 Key-1. tank, 2. anchor bolt	A	Oil tank for home power generation The bottom of a 2800ℓ (1800 x 1000 x 1700^H) tank is fixed with angle materials and four anchor bolts. When an earthquake hits, a large stress is expected to develop in the welds of the tank and the angle materials. Fixation merely at the bottom is not secure enough.
 key-1. foundation bolt 2. finish board 3. mortar packings 4. foundation base	B	Special High Voltage Power Switchboard An example of installing a 2000 kg, 1400 x 2800 x 4000^H, board. Bolts are short, but embedded channels serve as anchors.
 Key-1. bolt and MIZ	C	Cubicle Three-Phase Transformers Channel materials are premounted on the main body. It is very common that transformers are supported through channel materials. Also, boxed anchor bolts are dominant among anchor bolts.
	C	High Voltage Power Switchboard An example of installing a 2100 x 1960 x 2300^H cubicle high voltage power switchboard. The main body is fixed to this channel material.

D  Key-1. foundation bolt 2. cedar hand pump installation surface 3. motor installation surface 4. base 5. foundation	Pump Common Base A motor and a pump are placed on a rigid commonbase lest they get out of alignment. Sufficient strength shall be given to bolts.
E  key-1. anchor bolt 2. anchor bolt	Water Tank There are only two pieces of steel materials (channels) on which to rest the water tank. It is desirable to have a pedestal which can support the water tank on the entire surface, since large loads will be shouldered on the fixed sites between the water tank and the channels and the fixed sites between the channels and the foundation.
F  key-1. steel wall panel 2. bottom panel 3. outer frame steel material 4. bolt, 5. anchor bolt 6. concrete base	Water Tank (Pedestal, Foundation) An example of a panel water tank by T company. It is desirable that the foundation shall be made of a considerably high strength concrete, and be made integral with the structure using reinforcing steel. Be careful of the strength of the fixation between channel materials and the main body of the tank.

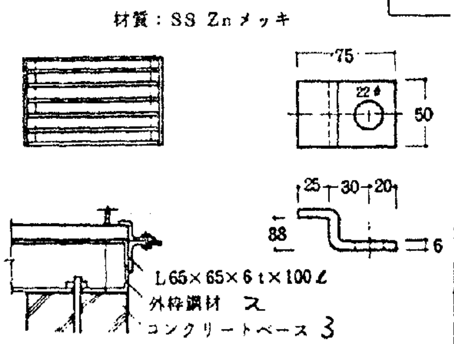
F



Water Tank (Pedestal, foundation)

An example of an FRP panel water tank by B company. It is said that a great deal of consideration has been given to earthquake resistivity (Various re-inforcing materials are installed inside the tank). The foundation shall be desirably integral with the slab.

F



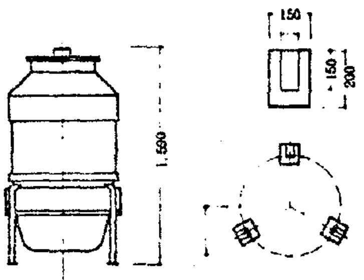
key-1. material quality: SS Zn plating
2. steel outer frome
3. concrete base

Water Tank (Pedestal, Foundation)

An example of an integral water tank by T company. The main body is fastened by clasps. The maker must have completed its investigation, but there was a case where these clasps were broken. Sufficient strength must be given.

It is desirable that the foundation be integral with the slab (Pedestal steel material dimension: ⊏ shape 150 x 75 x 9).

E · H



Cooling Tower

A cooling tower is small and weighs only 122kg (during operation) (an example of H company product). The strength of the fixation on the joints of legs, legs and the foundation is questionable.

The foundation must be integral with the slab.

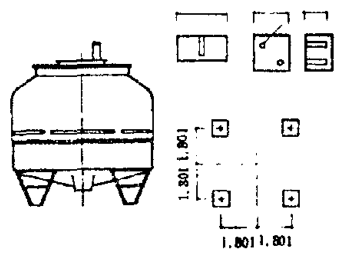
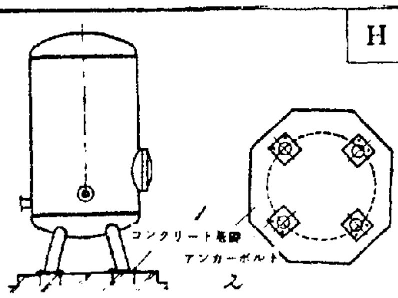
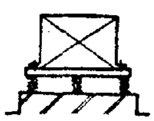
E · H 	Cooling Tower This is large and weighs 3870 kg (during operation) (Also, an example of the towers made by H company). It is necessary to investigate if the strength of the fixation by the anchor bolts is adequate. The foundation must be integral with the slab.
H  Key-1. concrete foundation 2. anchor bolt	Pressure Water Tank Compared to the height of the vessel, the cente of gravity is not so high, but the tank is unstable. It is doubtful if the strength of the joints of legs is sufficient.

Table 6.8 Types of Elastic Bearings

Code	Drawing	machinery & equipment concerned	outline and comments
a		refrigerators package air conditioners	machine---vibration proof material---foundation * Be mindful of the strength of vibration proof materials. Stoppers are required.
b		generators blowers	machine---steel base---vibration proof material---foundation * same as "a"

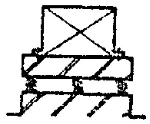
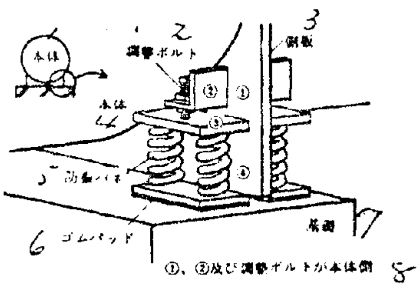
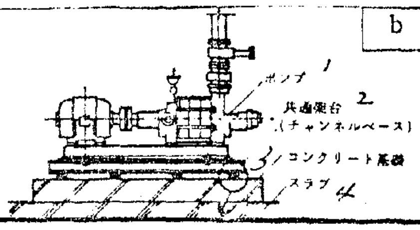
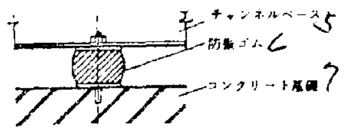
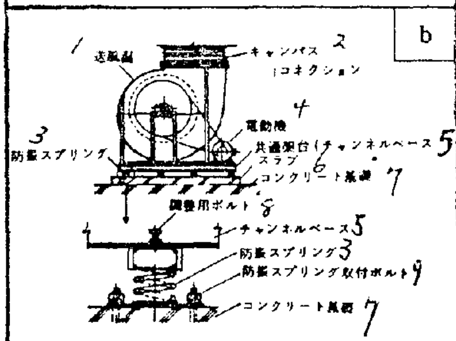
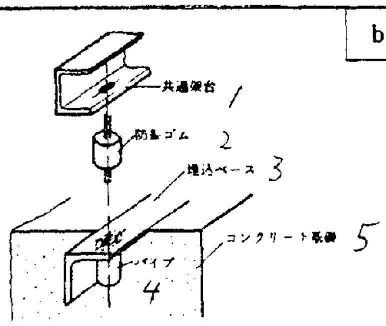
c		pumps	machine---bolts---vibration proof foundation---vibration proof material---foundation * A large load will be added to the vibration proof materials. Be mindful of the vibration proof materials. Stoppers are required.

Table 6.9 : Concrete Examples of Elastic Bearings

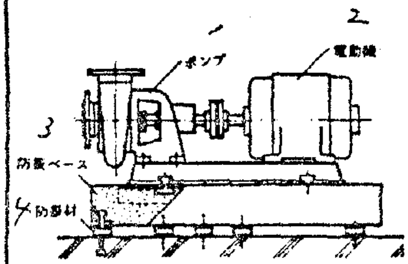
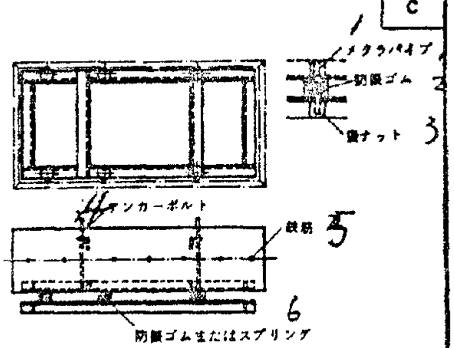
EXAMPLE	type code	explanations and comments
 Key-1. main body 2. adjusting bolt 3. side board 4. the main body 5. vibration proof spring 6. rubber pad 7. fundation 8. ①,② and the adjusting bolt are on the side of the main body	a	Closed Single Stage Turbo Refrigerator The main part of the body measures 1100ϕ x 6000L (rough dimension), and weighs 9300 kg during operation. It is supported by vibration proof springs at a total of 6 sites, on both ends and the mid section, which involves 24 springs. The body is merely sitting on the vibration proof springs and is anticipated to fall of by strong oscillations. Stoppers are required.
	b	Pump An example of the vibration proof bearings for machinery and equipment, using rubber vibration insulators inserted under the common pedestal.

<table>
<tr>
<td>


Key-1. pump
2. common pedestal
(channel base)
3. concrete foundation
4. slab
5. channel base
6. rubber vibration insulator
7. concrete foundation
</td>
<td>
Stoppers are required to restrain large vibration and displacement.
The shearing yield strength of the rubber vibration insulators is questionable.
</td>
</tr>
<tr>
<td>
b


key-1. hot air blown
2. canvas connection
3. vibration proof spring
4. motor
5. common pedestal
(channel base)
6. slab
7. concrete foundation
8. adjusting bolt
</td>
<td>
Air Blower
An example of the spring vibration proof bearings.
It is desirable to install stoppers just as in the above examples.
</td>
</tr>
<tr>
<td>
b


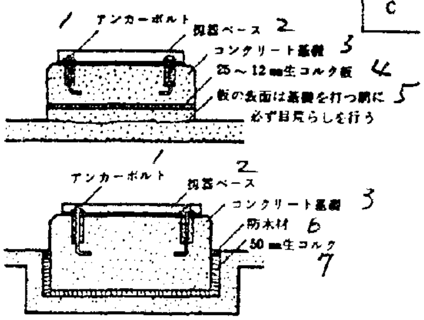
</td>
<td>
Rubber Vibration Insulator Fixation Example
Bolts are attached to the top and the bottom of the rubber vibration insulators, and it is designed to be fastened by nuts to machinery and equipment and steel materials.
</td>
</tr>
</table>

Key-1. common pedestal 2. rubber vibration insulator 3. embedded base 4. pipe 5. concrete foundation	
c  key-1. pump 2. motor 3. vibration proof base 4. vibration proof material	Pump A bearing example with a vibration proof base for enhancement of the vibration proof effect. The weight above the vibration proof material will be heavy, and it is necessary to be careful about the strength of the vibration proof material itself and the fixation to the foundation. Installation of stoppers is desirable.
c  key-1. blank pipe 2. rubber vibration insulator 3. cap nut 4. anchor bolt 5. reinforcing steel 6. vibration proof spring or rubber vibration insulator	Pump The rubber vibration insulator is fastened to the angle material below but not fastened to the vibra tion proof foundation. The elastic bearing may pop out by vertical seismic motions. Vertical and horizontal stoppers are desired.

C

Floating Foundation for Vibration Proof

There are problems related to the durability of cork against water and the bonding of concrete. In the above case, the part above the cork may shift.



key-1. anchor bolt
2. plate base
3. concrete foundation
4. 25-12mm raw cork board
5. The surface of the slab must be always roughened before the foundation is installed.
6. wood material trap
7. 50mm raw cork

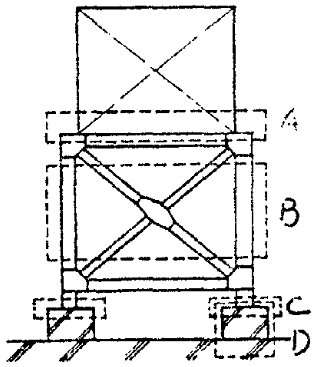
4. PEDESTALS

(1) Structure of Pedestals

Supporting by a pedestal may not only increase the response magnification but also will be disadvantageous from the point of strength. If it is necessary to provide pedestal bearings from the structural aspect of the machinery and equipment installations, the bearings shall be rigid and strong enough to resist the design seismic force. In this section, in view of the contents of the damages, points to be noted when designing pedestals will be indicated.

In the following, pedestals are viewed from the four grouped parts.

A : Pedestal and machine fixed junction
B : Pedestal frame
C : Pedestal and Foundation fixed junction
D : Foundation

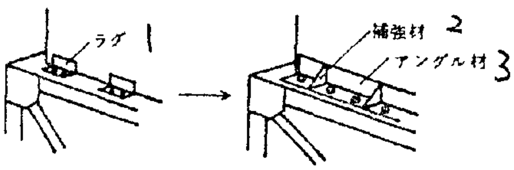


(2) Pedestal and Machine Fixation Junction

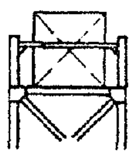
a. Bolts and metal fittings shall have sufficient strength.

b. Excessive concentrations of stress shall be avoided (Figure).

c. Legs of the pedestal shall be extended to support the machine at an upper level. (Figure)



Key–1. rug
2. reinforcing material
3. angle material



(3) Pedestal Frame

a. Corners shall be propped and bonded securely with gasset plates.

b. The size of legs shall be large enough not to buckle.

c. Braces shall be used as much as possible (to enhance the total regidity).

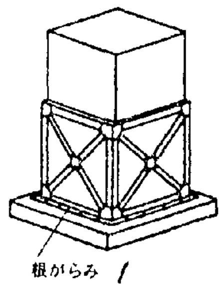
(4) Pedestal and Foundation Fixation Junction

a. Anchor bolts to be used shall have a sufficient strength, and shall be welded to the arranged reinforcing steel.

b. A large enough concrete covering depth shall be given lest the anchor bolts slip through or the surrounding concrete chips off.

c. The base plate shall be sufficiently large and strong, and the base legs shall be reinforced with ribs.

d. Batten bridging shall beconstructed the base of legs. (Figure)



Key-1. batten bridging

(5) Foundation

a. A sufficient contact surface shall be given.

b. When an equipment(pedestal) is not so large, it shall be in sinngle continuous shape (Figure).

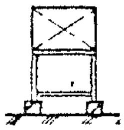
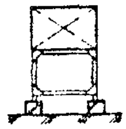
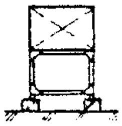
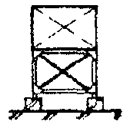
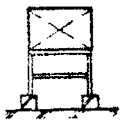
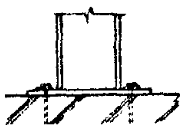
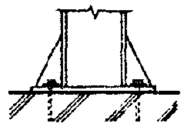
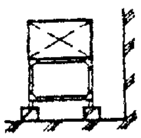
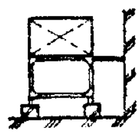
c. Reinforcing steel shall be installed and welded to the steel arranged in the slab.

d. A high strength concrete shall be used.

(6) Reinforcement and Improvement of Existing Pedestals

A great number of existing pedestals are soft in earthquake resistance. In Table 6.10, examples of methods to reinforce these weakpoints are Presented.

Table 6.10: Existing Pedestal Reinforcing Examples

before reinforcement		after reinforcement	
	weak corners		reinforced with gasset plates
	no braces		braces installed
			chord element supplemented
	base of legs		reinforced with ribs
			reinforced with wall mounted stays

5. STOPPERS

Internal combustion engines of home power generators, recipro type refrigerators, blowers, etc. generate vibrations when operating, and they are usually provided with vibration proof bearings. However, these vibration proof bearings do not demonstrate a vibration proof effect under earthquakes, but rather give the effect of raising the response magnification of the machinery and equipment.

Thus, sometimes earthquake resistive stoppers are installed to deter excessive displacement of these vibration proof bearings during earthquakes. There are different kinds of stoppers to be used depending upon the type of the vibration proof bearings and the shape of the machinery and equipment, for instance, stoppers that inhibit horizontal directional shifts and stoppers that inhibit vertical directional shifts.

Examples of stoppers are shown in Table 6.11 and Table 6.12.

Incidentally, the following considerations are required in respect to the stopper types and setting locations.

a. Direction of the movement to be restrained shall be accurately checked, and the stoppers which render proper service shall be adopted.
b. Stoppers shall be installed on a solid foundation which can give a large reaction force to them. If the foundation size is not large enough, an additional portion of the foundation shall be installed.
c. Stoppers shall be arranged in such a way as to divide equally the load by the machinery and equipment as much as possible.
d. Stoppers shall be fitted to the strong and hard part of the machine, or fitted so as to restrain the strong and hard part.
e. When dealing with heavy machinery a nd equipment, stoppers shall be placed at two points on one side.
f. Contact surface shall be lined with a buffer.
g. Stoppers shall be arranged in such a way as to prevent a large displacement of connecting pipes.
h. Stoppers shall be fitted properly so that they will not interfere with the operation and repair of the machinery and equipment.

Table 6.11: Examples of Stoppers for Horizontal Constraint

	Slip guard angle, the simplest type Unless the material is thick, strength will be insufficient. (one way constraint)

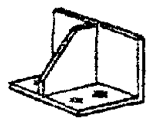
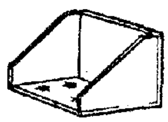
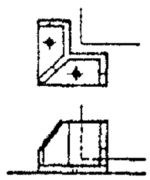
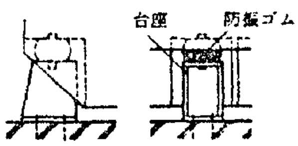
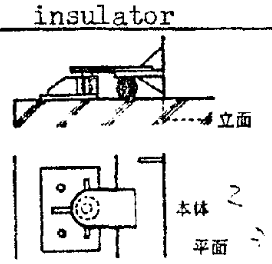
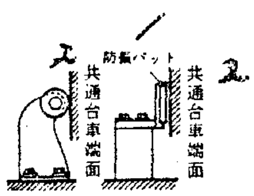
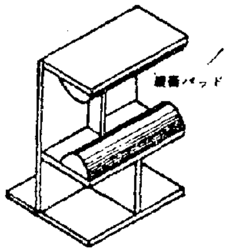
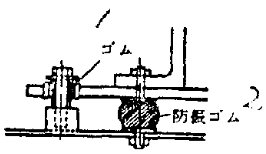
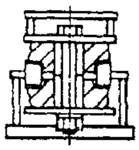
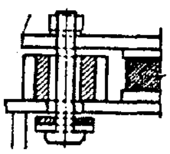
	Reinforcement added to the above example This type is desirable when constraining heavy machinery and equipment or constraining them at a high position. (one way)
	Further reinforced second example. This type is suitable for heavy machinery and equipment such as generators. (one way)
	Those installed on the corners of machinery and equipment. (two way)
 key-1. seating base 2. rubber vibration insulator	Seating base of a rubber vibration insulator functioning as a stopper since it is surrounded by the (base of) a machine.
 key-1. elevation 2. main body 3. plane	Example of multi-directional constraint stoppers with a complicated structure and moderate strength. It seems a simpler design may be of greater benefit.
 key-1. vibration proof pad 2. common base wheel end	A vibration proof pad placed at contact with the machine.

Table 6.12: Examples of Stoppers for Vertical Constraint

	Simplest type of crank Those with sufficient thickness shall be used.
 key-1. buffer pad	Type which holds the pedestal from top and bottom through buffer pads (rubber material)
 key-1.rubber material 2. rubber vibration insulator	Type which constrains displacement with holes and bolts. The bolts are attached with rubber vibration insulators and must be firmly tightened.
	Bolts which penetrate through rubber vibration insulators serve as stoppers. Loosening of nuts shall be prevented.
	Type which constrains displacement with holes and bolts. Be careful of loosening nuts.

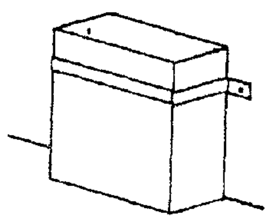
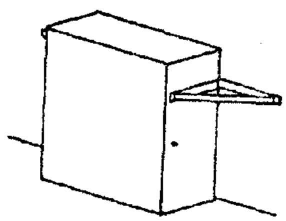
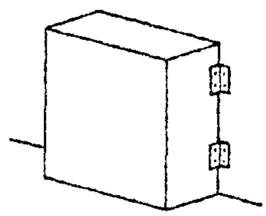
6. STAYS

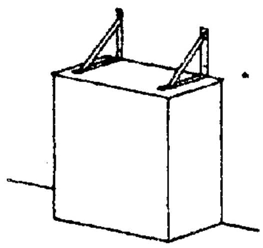
Stays are very effective prevent machinery and equipment installations from turning over. Furthermore, they cannot only be incorporated when designing but also can be applied when improving the earthquake resistance. The installation of stays will be restricted by the conditions of the environment such as availability of walls and ceilings nearby, but they are classified as follows according to the objects to which they are fastened.

a. Stays fixed on walls.
b. Stays fixed on ceiling slabs and beams.
c. Stays fixed on floors.
d. Stays fixed in cubicle containers (iron plates).
e. Stays fixed on other secured machinery and equipment.

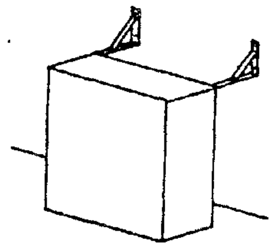
Examples of stays for earthquake control are shown in Table 6.13.

Table 6.13: Examples of Stays

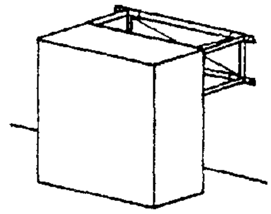
Fixed on Walls		
 wrapped with a band	 fixed with horizontal bracket	 fixed with clasps and angles



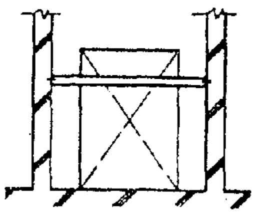
fixed with brackets on the top



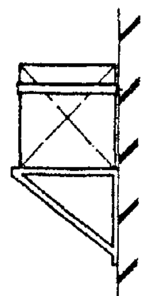
fixed with angles on the back



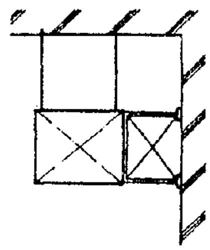
fixed with a frame on the back



fixed on both walls

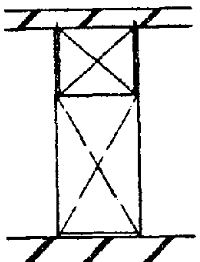


wrapped with a band

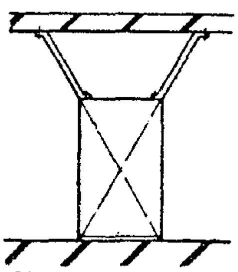


vibration proof with angles

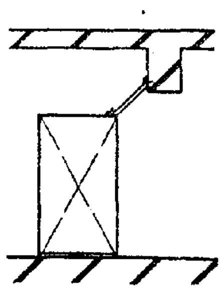
Fixed on ceilings and beams



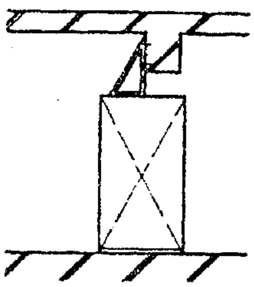
fixed with a frame



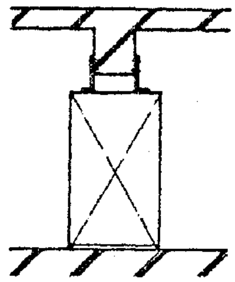
fixed with angles



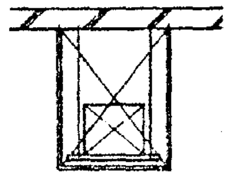
fixed on a beam with angles



fixed on a girder with brackets

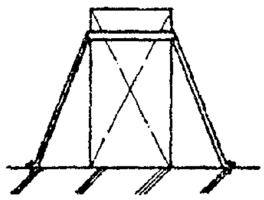


fixed on a girder with angles

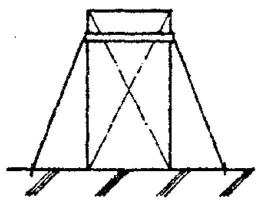


A machine hanging from ceiling enclosed with a frame (or possibly with brakets)

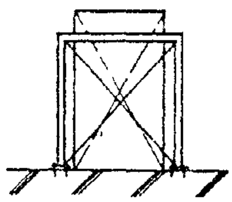
Fixed on floors



Fixed on a floor with angles

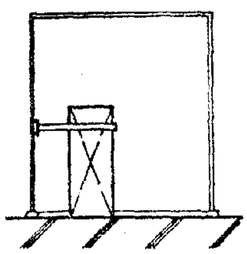


Fixed with wires

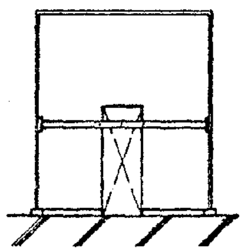


Enclosed in a frame

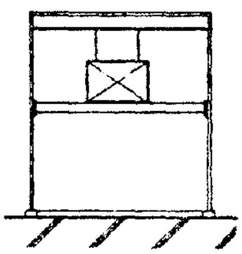
Fixed in cubicle containers



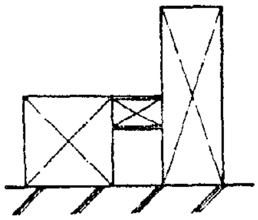
Fixed on a closer side plate with angles



Fixed on both side plates with angles



Fixation of hanging machinery and equipment

Fixed on other machinery and equipment		
 integral with secured machinery and equipment nearby		

VII PIPING AND BEARING DESIGNS

1. EARTHQUAKE RESISTIVITY OF PIPING

Piping is subjected to force of inertia (force of an earthquake) corresponding to its own weight and the forced deformation due to the relative displacement between fulcrums.

Regarding the force of inertia, the former, the earthquake force can be established according to the machinery and equipment. It is however wise to avoid destruction by implementing countermeasures such as increasing the rigidity of bearings and shortening the distance between bearings in order to prevent machines from resonating with the vibrations of buildings.

As for the latter, the forced deformation, it is necessary to provide some measures to prevent piping damages inflicted by interlayer displacement and the relative displacement of the expanded area as explained in section V-2. However, in the present state, it is very difficult to verify this by calculation.

Destructio n of piping of a high required earthquake resistance level shall be avoided by prearranging of flexible joints.

2. STRUCTURE OF BEARINGS FOR PIPING

Following types of piping bearings are available.

a. Pedestal bearings from the floor
b. Swing bearings from the ceiling
c. Bracket bearings from the wall

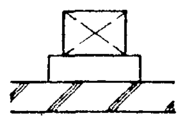
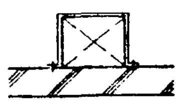
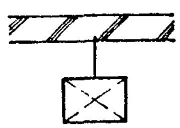
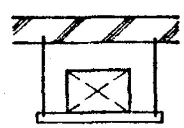
Also, solid vertical pipes penetrate through other objects and are supported at those breakthroughs.

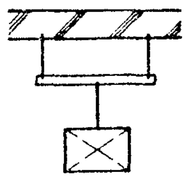
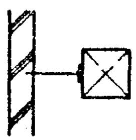
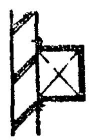
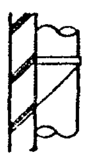
Concerning the seismic considerations for the piping bearings, swaying of piping in excessive amplitudes shall be prevented. Also, the piping shall be designed to withstand interlayer displacement, relative displacement and expansion of the piping itself due to the temperature changes.

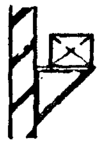
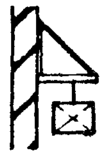
Especially with swing bearings, braces applied to hanging materials are very effective in preventing horizontal vibration. When the hanging length is long, stabilisers are particularly important. Likewise, the distance between bearings shall be on the short side.

Types of piping bearings and related earthquake control examples are shown in Table 7.1 and Table 7.2, and types of the floor penetration of piping and related earthquake control examples are shown in Tables 7.3 and 7.4.

Table 7.1 : Types of Piping Bearings

support partner	symbol	conceptural drawing	outline	general seismic evaluation
FLOOR	A		Support base is constructed on top of the slab, and the piping shall be fixed on top of it. (Sometimes piping is merely seated on it.)	When the diameter and the weight of pipes are large, or multiple pipes are involved, the support base must be reinforced, and fixed securely to the slab.
	B		Piping is directly set on the floor slab with a saddle band. Sometimes, insulators are placed inbetween.	This rigid support is preferable from the seismic aspect, but not suitable for the expansion of the piping. Where pipe bends and rises, measures to absorb displacement are necessary
CEILING	C		Piping is suspended by round steel and flat steel.	This type is likely to cause wide transverse oscillations and is not suitable as is. Stabilizers are necessary.
	D		Angles and channels are hung from the ceiling, and the piping rests on them. There are two ways, one which fastens the piping to the steel and the other which does not fasten it to the steel.	Transverse oscillation occurs. So This type is not suitable. The piping must be fastened to the steel Stabilizers are needed.

CEILING	E 	This is perhaps a special example of C. Piping is hung from hanging angles. This type is used for piping where surface condensation is a factor.	Complex oscillation is anticipated. At any rate, stabilizers are desirable. Above C,D and E patterns' strength in the inserted area is also important.
WALL	F 	Vertical Pipe is supported by metal fittings coming out of the wall.	When distance between the wall and the pipe is large, sufficient strength shall be given to the inserted area.
	G 	A pipe is fixed on the wall with U bolts and bands.	There are no significant problems, but consideration must be given to counter the interlayer displacement.
	H 	The pipes and the flanges of the duct are placed on the angle brackets mounted on the wall. Sometimes, vibration proof bearings are provided.	The point is the strength of the flange of pipes and whether or not the pipes are fixed. It is better to fix them loosely.

WALL	I 	Pipes are placed or fixed on brackets mounted on walls.	It is desirable to fix the pipes (However, allow some expansion). Brackets shall be arranged to have horizontal rigidity.
	J 	Pipes are hung from brackets mounted on walls. This type is used for pipes with due condensation.	When transverse oscillation is anticipated to cause the pipes to bump the wall, stabilizers shall be utilized.
	K 	Pipes are tightened by flat steel bands and fixed to walls.	Significant problems are not anticipated. Flexibility is desirable in the vicinity of bends.

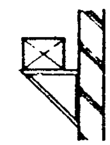
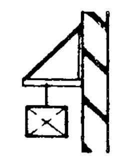
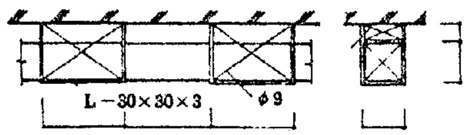
WALL			
	I 	Pipes are placed or fixed on brackets mounted on walls.	It is desirable to fix the pipes (However, allow some expansion). Brackets shall be arranged to have horizontal rigidity.
	J 	Pipes are hung from brackets mounted on walls. This type is used for pipes with due condensation.	When transverse oscillation is anticipated to cause the pipes to bump the wall, stabilizers shall be utilized.
	K 	Pipes are tightened by flat steel bands and fixed to walls.	Significant problems are not anticipated. Flexibility is desirable in the vicinity of bends.

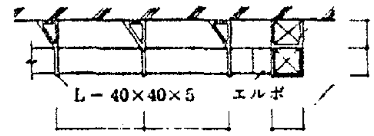
Table 7.2 Examples of Earthquake Control for Piping Bearings

a



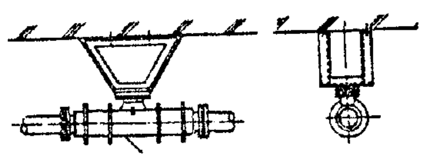
Support of ducts by frames

a



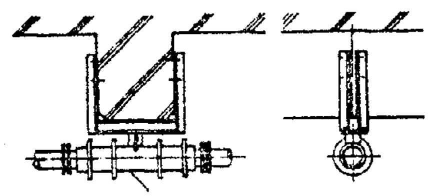
Support of ducts by reinforcements
key-1. elbow

a



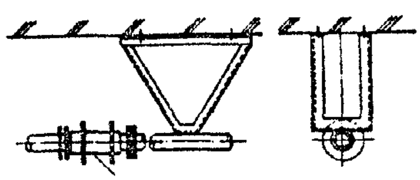
Solid and hard support from a frame

a



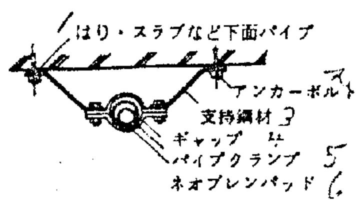
Solid and hard support from a beam

a



Solid support by a frame

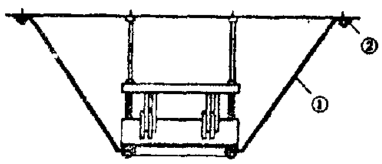
a



Vibration proof fixation of pipes running across

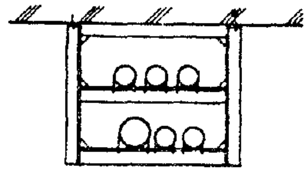
key-1. pipes below beams and ceiling slabs
2. anchor bolt, 3. supporting steel
4. gap, 5. pipe clamp, 6. neoprene pad

a



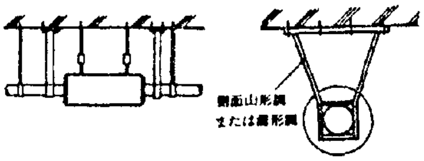
Stabilizers for bath ducts

a



Fixation of multiple piping by a rigid frame pedestal

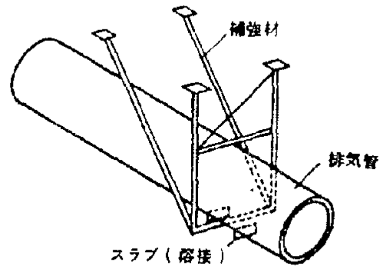
a



Fixation of mufflers

Key-1. adjusters angled or channeled on the side

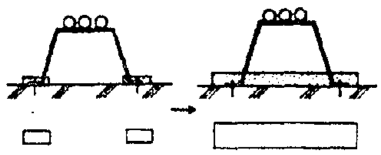
a



Bearing for exhuast pipes

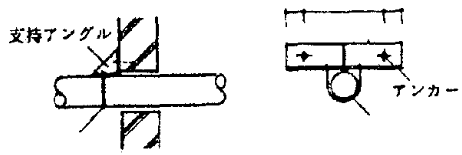
key-1. reinforcement, 2. exhaust pipe, 3. slab (welded)

c



Reinforced pedestal foundation

d



Fixation of wall penetration

Key-1. support angle, 2. anchor

Table 7.3 Types of Floor Penetration of Pipes

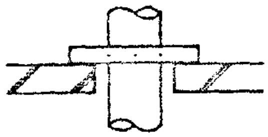
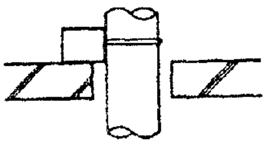
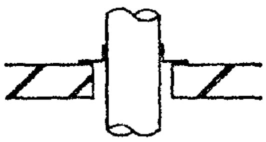
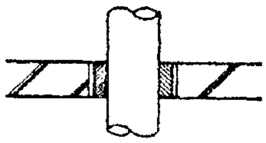
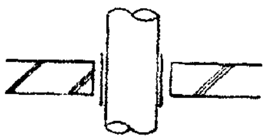
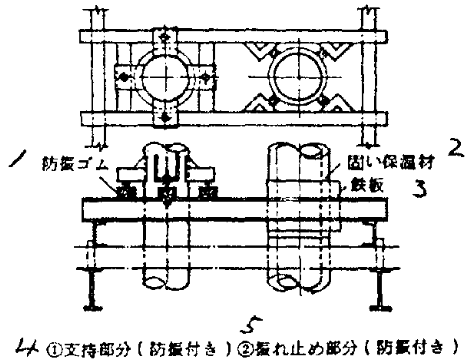
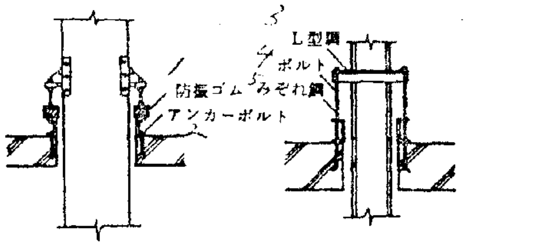
support or no support	symbol	conceptual drawing	outline (comments)
supported	L		Rising pipes (rising ducts) are fixed on the floor angles and channels. (Fixed to steel with rivets and welding). No particular problems are anticipated if designed not to concentrate the stress on the fixed part of pipes.
	M		Shape steel is fixed to floor slabs. Pipes are fixed to U bolts that connect to the steel. It is desirable to see to it that the stress will not concentrate on the bolt tightened part.
penetration only	N		This type is used where water proofing is required, such as on a roof top. The penetrated area is covered by iron plates, or the slab is made to rise.
	O		Sleeve is installed to re-embed the pipe with mortar or lockwall.
	P		Only sleeve is installed. Sometimes lead caulking is applied.

Table 7.4 Examples of Earthquake Control for Floor Penetration of Pipes

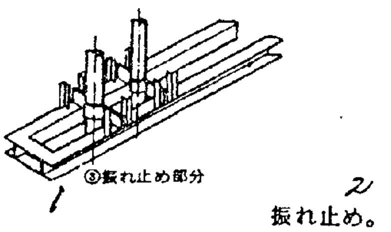


Key-1. rubber vibration insulator
2. hard heat insulator
3. iron plate
4. supported area (with vibration proof)
5. stabilized area (with vibration proof)

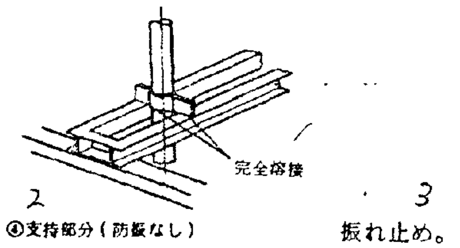


Elastic Support for Penetration of A Bath Duct Through Floor

Key-1. rubber vibration insulator
2. anchor bolt
3. L shape adjuster
4. bolt, 5. grooved steel



Key-1. stabilized area, 2. stabilization



Key-1. complete welding
2. supported area (without vibration proof)
3. stabilization

3. JOINTS OF MACHINERY/EQUIPMENT AND PIPING

Relative displacement develops in the joints between machinery/equipment and piping due to the responses of the two. The maximum value is estimated to be almost the same as the simple addition of the maximum value of the respective responses. However, it is very difficult to obtain the response displacement of machinery/equipment and the response displacement of piping by calculation, and such data is presently virtually non-existant. Therefore, it is desirable to arrange flexible joints as required for these joints in consideration of the required earthquake resistance level of the functional systems to which these joints are included and the rigidity of the machinery/equipment and pedestals. Especially, with vibration proof bearings, the joints must be flexible against the displacement until the stoppers work.

PRIMING WATER TANK

Functional System:	Fire Extinguishing Installation
Floor Installed:	Basement
Boundary Dimensions:	tank 1000 x 600 x $^{H}755$
Weight:	
Capacity:	360 ℓ
Evaluation:	B

****

INSTALLATION CONDITION AND COMMENTS

The tank is placed on two brackets mounted on the wall.

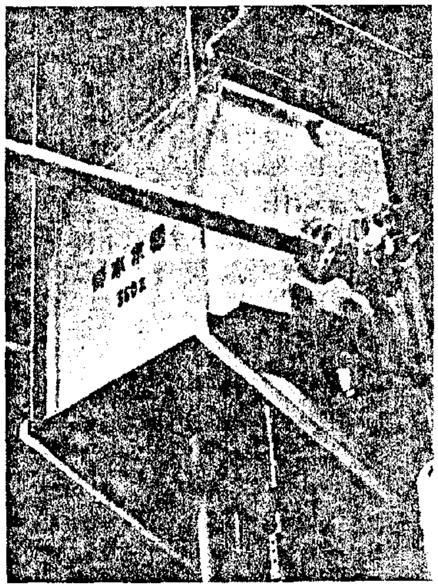
Tips of brackets are pulled by round steel. However, round steel is low in rigidity, and it is not expected to be effective against horizontal oscillations.

Brackets also seem vulnerable to transverse oscillations. Also, the joints of the tank and the pipe look unstalbe.

It is recommended that the upper portion of the water tank be fixed to the wall.

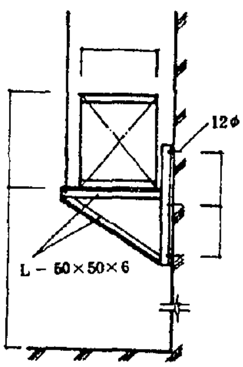
bracket: L- 50 x 50 x 6^t

swing bolt: 12ϕ



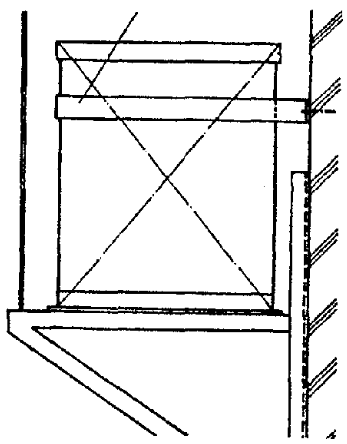
SKETCH DRAWING AND DETAIL DRAWING

* Side View



PROPOSAL FOR REINFORCEMENT & IMPROVEMENT

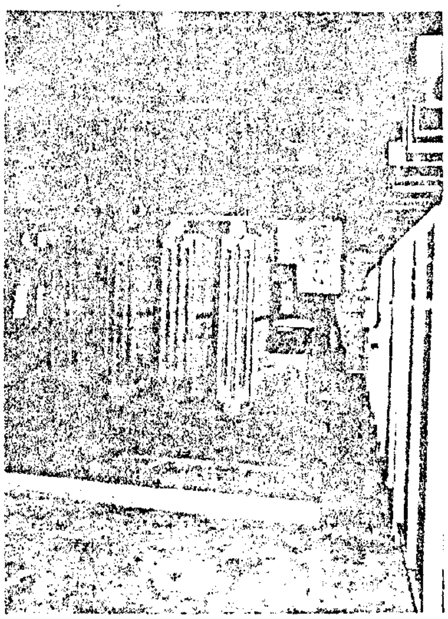
* Fix on the wall with a band.



TRANSFORMER (SPECIAL HIGH VOLTAGE 2000KVA OIL COOLING TYPE)

Functional System:	Electrical Installation
Floor Installed:	Basement
Boundary Dimensions:	Essentials of Main Body 657 x 1920 x $^{H}2940$
Weight:	
Evaluation:	B

****



INSTALLATION CONDITION AND COMMENTS

It is fixed on the foundation approximately 160 mm high with 4 bolts (24ϕ) through channel materials (⊏ - 100 x 50 x 5).

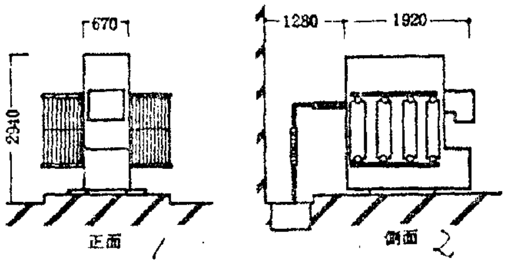
It is connected to the wiring in the cable pit at the back.

Channel materials were welded to the main body in the factory, and the details of the welds are unknown.

Fixation span appears to be narrow relative to the proportion of the transformer.

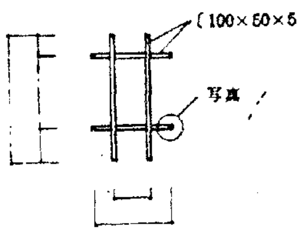
SKETCH DRAWING AND DETAIL DRAWING

* Outline of the transformer



key-1. front view, 2. side view

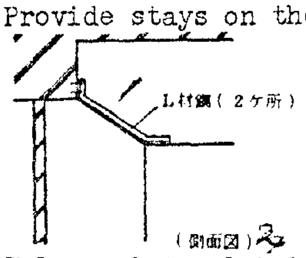
* Channels and bolts fixation location



key-1. photo

PROPOSAL FOR REINFORCEMENT & IMPROVEMENT

* Provide stays on the upper part



key-1. L steel (2 sites)
2. side view

* Enlarge lateral bolt span.
  (Under the present condition, it is small)

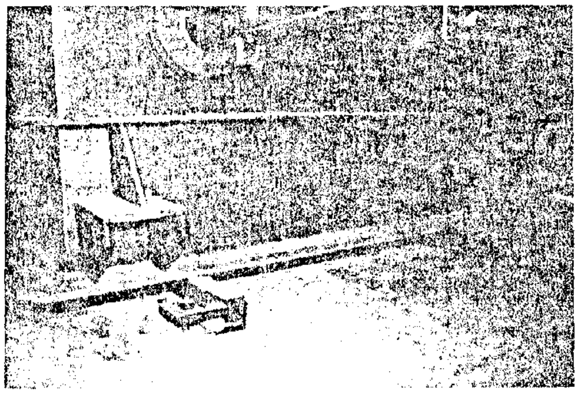
* Bolt Fixation Site

Even if bolts will not slip off, channel-plate welding may break up.

Channel-main body welding is, in this case, unknown, but the strength is questionable.

Some clearance is present in the concrete around the bolts. Is the work done properly?

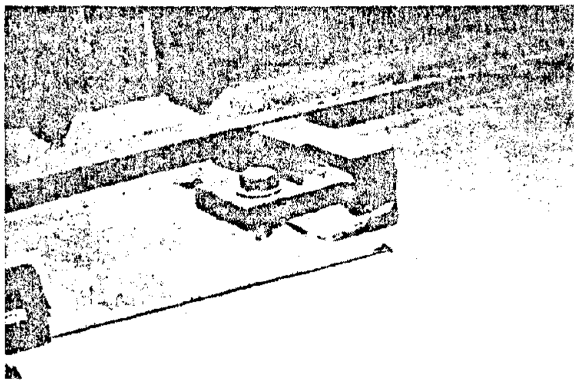
It is felt that the details of the channel is too complicated.



* Bolt Fixation Site (Boxed)

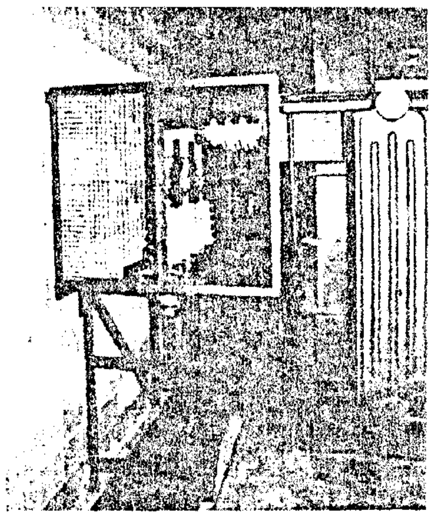
It is requested that the adequacy of the bolt diameter be investigated.

The ancor bolt is 24ϕ, thick enough in calculation, but the slab side is anticipated to break.



* Wiring Joints

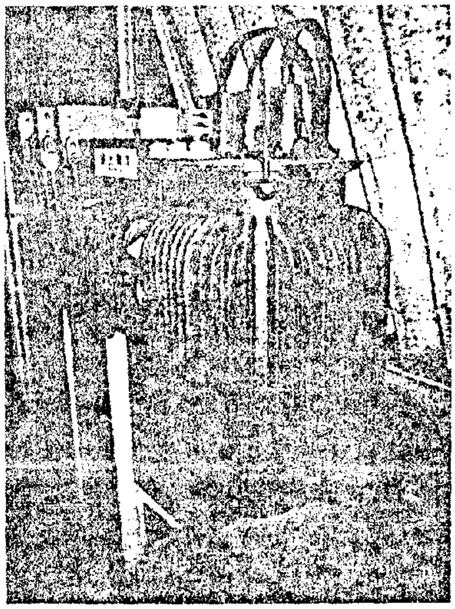
It is anticipated that the insulators break due to the oscillations of the main body during earthquakes.



Functional System:	Electrical Installation
Floor Installed:	Upper Floor
Boundary Dimensions:	810 x 1330 x ^{H}1295
Weight:	1550 kg
Evaluation:	C

****

INSTALLATION CONDITION AND COMMENTS



The transformer is supported with rubber vibration insulators on an H shape steel semi-embedded in the foundation.

There are 4 fixation sites.

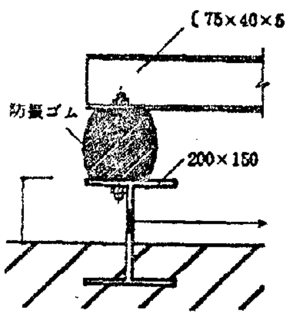
It is expected to receive a considerable amount of earthquake force since the transformer is installed on an upper floor. Fixation on the top is likely to improve earthquake resistance.

This bearing is insecure, since sometimes, vibration proof bearings resonate with earthquakes creating large acceleration.

H shape steel size: 200 x 150
(estimation; confirmed by drawing)

SKETCH AND DETAIL DRAWING

* Detail of Vibration Proof Bearing



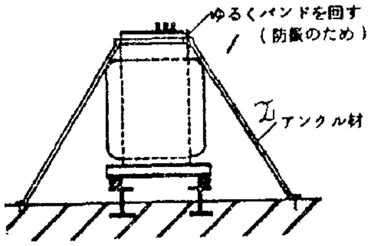
Key-1. rubber vibration insulator

(Probably the bolt appearing on the top and the bottom of the insulator is not connected inside the rubber vibration insulator, which means that the support is weak against the up and down earthquake motions.

If connected, the bolt can function with the nut as a verticaldirectional stopper....)

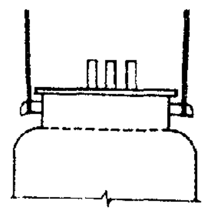
PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

* Given enough strength to the welding of the channel and the main body.
* Use high strength rubber vibration insulator.
* Apply stays on the top extending from the floor.



Key-1. wrap with a band loosely (for vibration proofing)

2. angle

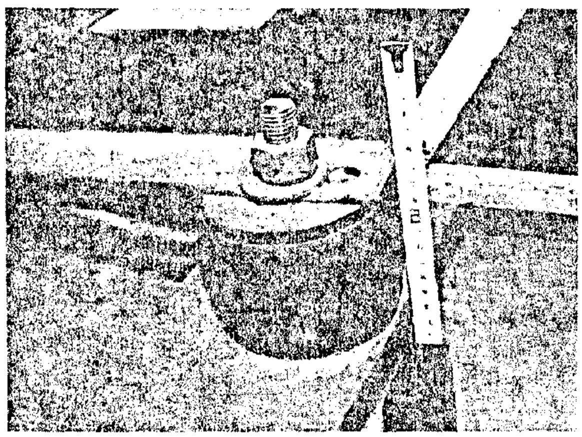


*Sling with wires (if it can be anchored on the top).

Vibration Proof Bearing

Bolts are 16 mm in diameter.

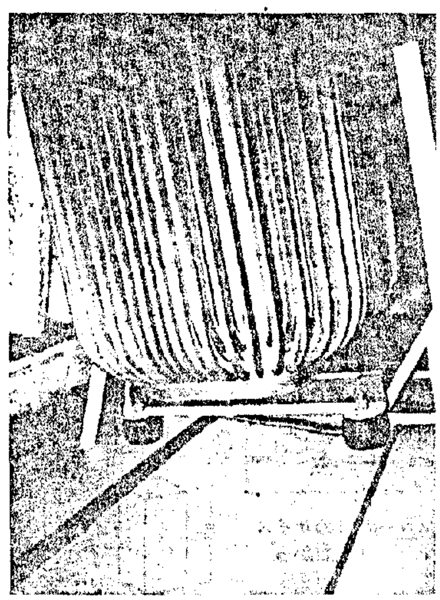
A large load may be applied to the bearing due to the resonance of the rubber vibration insulator.



Channels in Bearing

The welding of channels and the main body is unknown.

H shape steel was probably selected to support the transformer for vibration proofing purposes but the rigidity will be low compared to direct fixation to the floor.



DIESEL GENERATOR

Functional System:	Home Power Generation Installation
Floor Installed:	Basement
Boundary Dimensions:	1820 x 4288 x ^{H}2015
Weight:	8850 kg (Engine 5450 kg, Generator 3400kg)
	* (10 100kg including the common base)
Evaluation:	C

****

INSTALLATION CONDITION AND COMMENTS

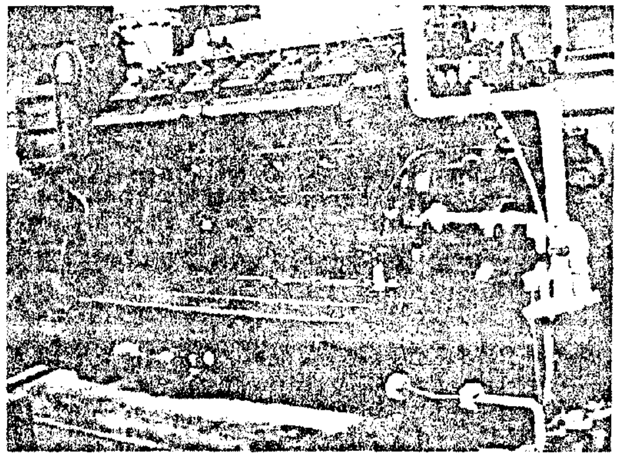
It is supported on a foundation of approximately 100 mm high with rubber vibration insulators (see drawings below).

There are 10 rubber vibration insulators. The machine is heavy and contains many connecting pipes. Solid stoppers must be installed.

Flexible pipes are used for fuel inlet pipe, cooling water outlet pipe, cooling water inlet pipe and starting air inlet pipe.

Incidentally, foundation bolts and nuts are accessories.

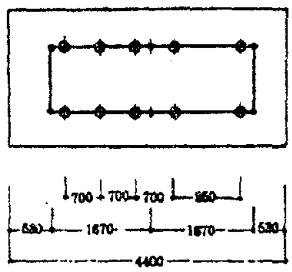
(bolt diameter: 24 mm)



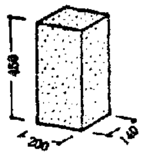
SKETCH AND DETAIL DRAWINGS

*Location of anchor bolts and *rubber vibration insulators
(*fixation of embedded steel material)

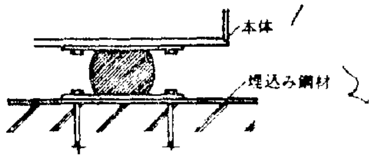
•bolts
o rubber vibration insulators



* Size of Boxed Anchors (from drawing)



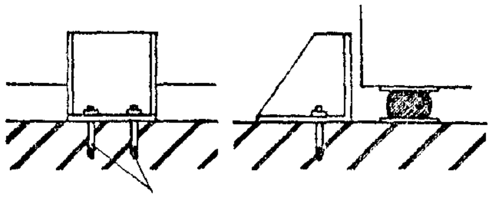
* Main body and rubber vibration insulator



Key-1. main body
2. embedded steel material

PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

* It is absolutely necessary to install stoppers (water table) and constrain the sides of the common base. At least 2 stoppers are desired on each side.



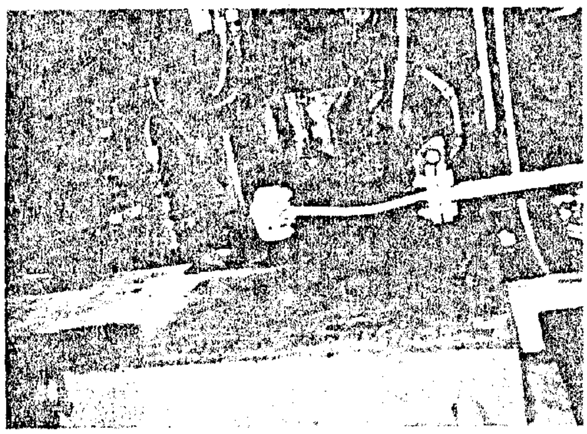
* Lower Diesel Engine

It is supported on steel materials embedded in the foundation with rubber vibration insulators.

The bolts used for fixing the body to the steel materials are 22 mm.

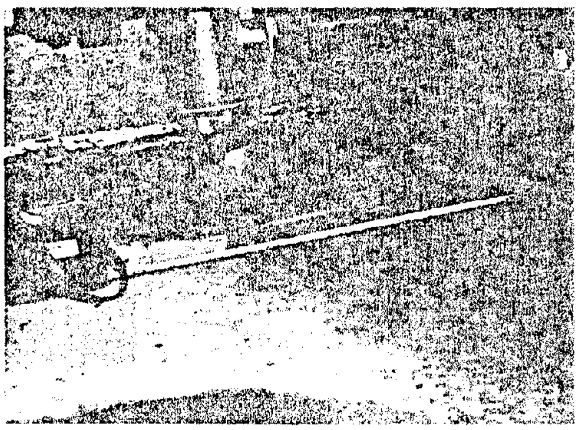
Stoppers that stop the lower side plates are necessary.

Incidentally, the pipe to the right from the center is the starting air inlet pipe. Some slack is given to allow for tensile.



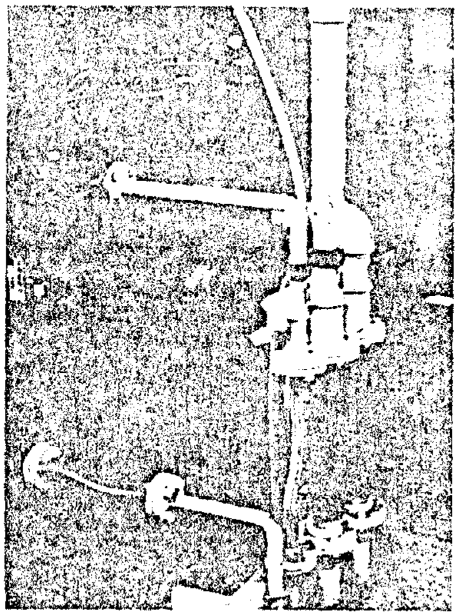
* Rubber Vibration Insulator

Stoppers are necessary.



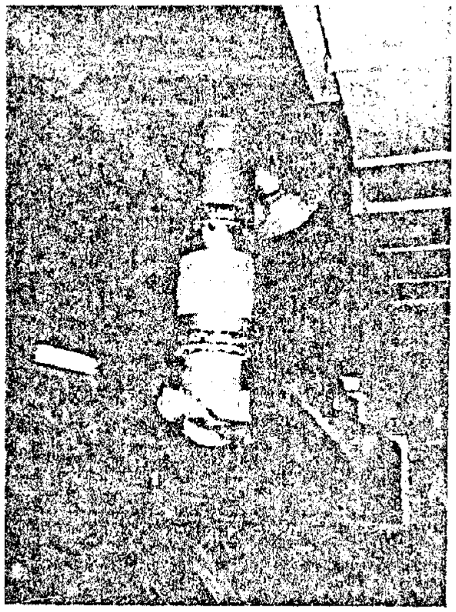
* <u>Cooling Water Inlet and Outlet Pipes</u>

Rubber flexible pipes are used to absorb the vibration of the generator.



* <u>Upper Muffler</u>

A canvas duct is used. The muffler is slung from the ceiling by vibration proof hanging bars. Transverse osciilation preventive measures are desirable.



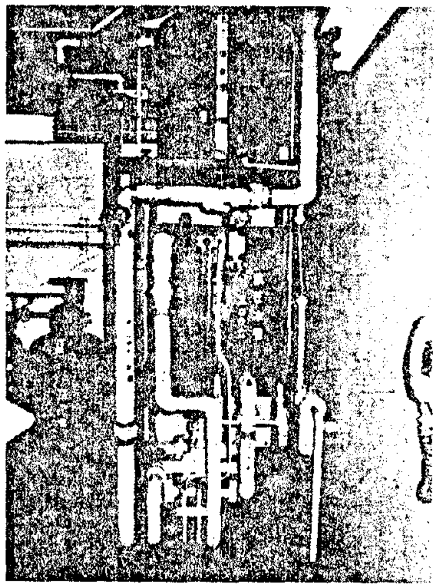
OIL SERVICE TANK AND DEPRESSURIZED WATER TANK

Functional System:	Home Power Generation Installation
Floor Installed:	Basement
Boundary Dimensions:	Upper Tank ø830 x ^{H}867
	Lower Tank 900 x 780 x ^{H}914
Weight:	unloaded Upper Tank 45 kg, Lower Tank 100kg
Evaluation:	C

****

INSTALLATION CONDITION AND COMMENTS

An upper and lower two stage pedestal is built with 50 x 50 x 6 angles. A fuel service tank and a depressurized water tank are installed on the upper shelf and the lower shelf respectively.



Capacity of the tanks are 390 liters and 500 liters respectively.

The oil service tank supplies fuel to diesel engines by a gravity system and is placed on a high plane. The pedestal is made of pure rahmen and seems dangeous without modification. If this tank breaks down, not only is the emergency home power generation jeopadized but also this may cause secondary damages.

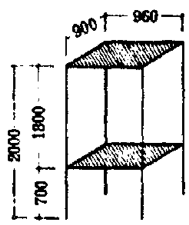
Since there is a wall nearby, it is perhaps necessary to fasten the pedestal by stays.

Pedestal Angles: L- 50 x 50 x 5

Anchor Bolt: 10ø

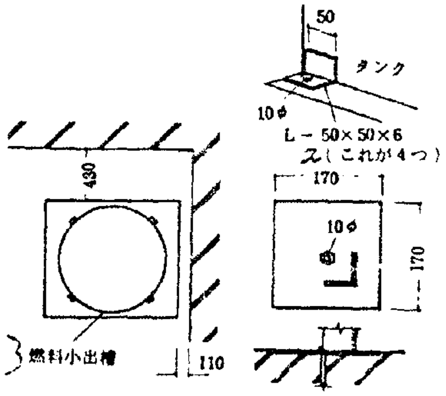
SKETCH AND DETAIL DRAWINGS

* Pedestal Dimensions



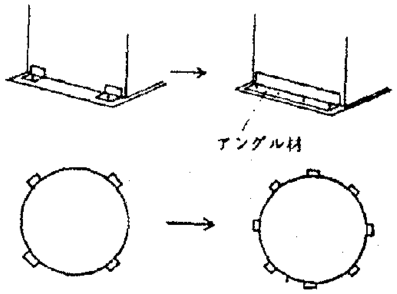
* Fixation of Two Tanks (Drawing 1)
Relation to Wall (Drawing 2)
Fixation of Base of legs (Drawing 3)

Key-1. tank
2. (4 of these)
3. fuel service tank



PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

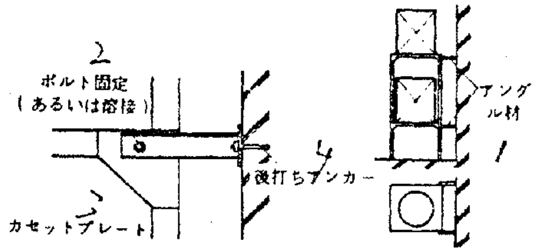
* Reinforce the fixation of the pedestal to the tanks.



key-1. angle

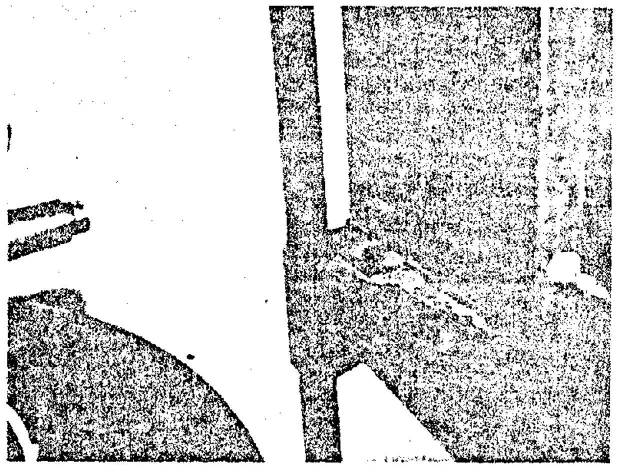
* Give stays from the wall.

key-1. angle
2. fixation by bolts (or welding)
3. gusset plate
4. post-driven anchor



* Fixation of Depressurized Water Tank on Lower Shelf

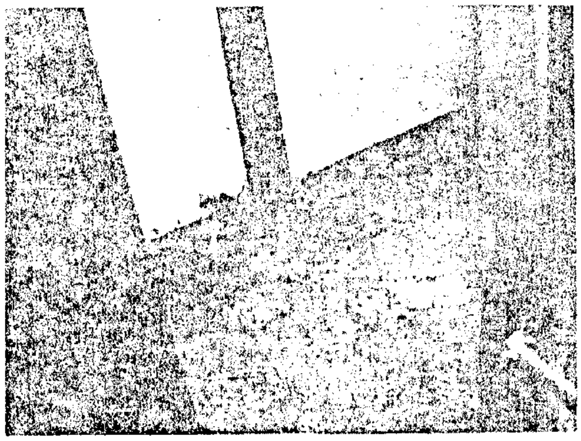
Clasps and bolts appear inadequate.
There are 4 of thse fixation sites.
(clasp: L-50 x 50 x 6^t) (bolt diameter 10mm)



* Base of Pedestal Leg

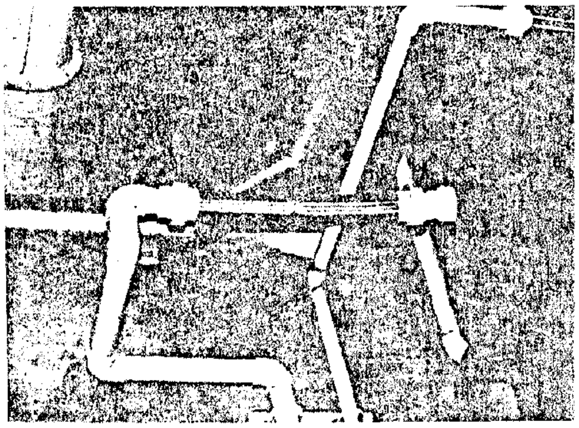
Angles of legs are welded to a plate which is fixed by an ancor bolt of 10mm diameter.

Diameter of bolts and welding must be investigated.



* Flexible Pipe on the Upper Part of the Fuel Service Tank

It is desirable to use flexible pipes for the joints of machinery and equipment and piping as seen here.



OIL STORAGE TANK

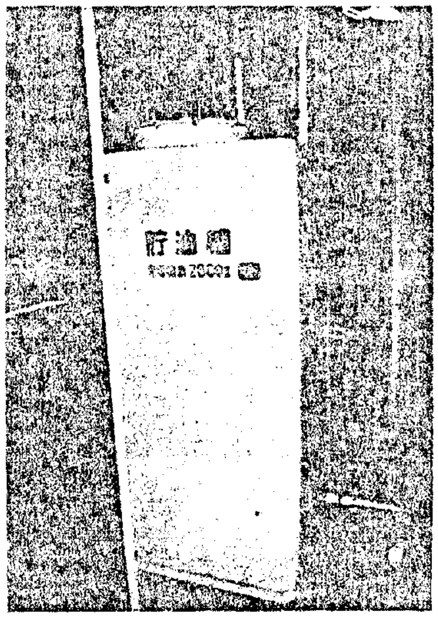
Functional System:	Home Power Generation Installation
Floor Installed:	Basement
Boundary Dimensions:	1800 x 1000 x ^{H}1700
Weight:	Approximately 270 kg
	(Unloaded condition, calculated from the plate thickness)
Evaluation:	C

****

INSTALLATION CONDITION AND COMMENTS

It is merely fastened by 4 anchor bolts of 10mm diameter.

When it is full, the weight increases. When sloshing occus, a large load will be added to the bottom.



When this tank breaks down, not only is the function of power generation lost but also secondary damage may occur.

Reinforcement of the bottom, and support on the upper section are desirable.

Incidentally, out of the 4 bolts, one of them had a loose nut.

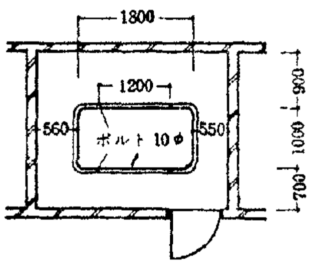
Bolt strength is adquate in calculation, but the strength around the legs on the main body side was questionable. Stays are desirable.

steel plate thickness: side 3.2mm
bottom 4.5mm
lid 2.3mm

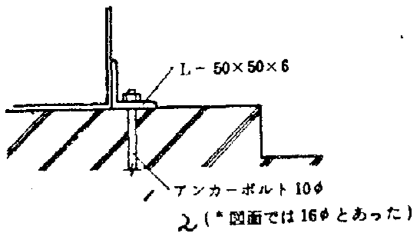
SKETCH AND DETAIL DRAWINGS

* Fixation of Oil Storage Tank

key-1. bolt



* Detail of Fixed Site



key-1. anchor bolt ϕ 10

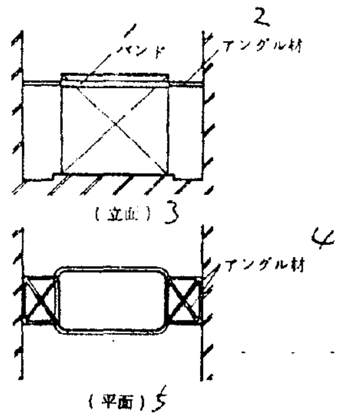
2. (* in the drawing, it was indicated as 16ϕ)

PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

* Increase the number of bolts (for each side).
* Use reinforcement with larger sectional surface for the bottom.
* Provide stays (Make sure that the anchor side is not the block wall).

key-1. band

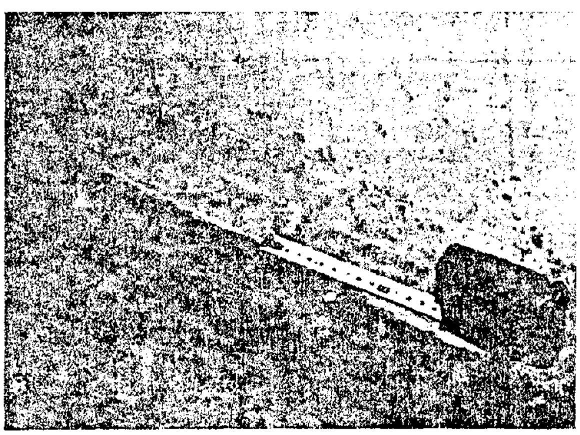
2. angles
3. (standing view)
4. angles
5. plane view



* <u>Bolt Fixation Site</u>

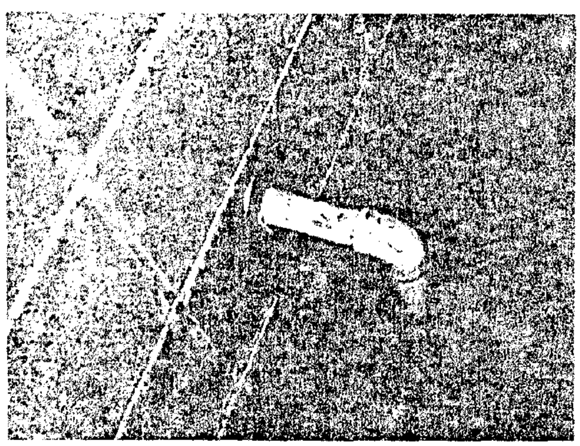
Bolt Diameter : 10mm

Bottom Reinforcement: L-50 x 50 x 6



* <u>Block Penetration of Oil Supply Pipe</u>

There is a possibility that the block walls will collapse and the pipe mat break.



ALKALI BATTERIES (100V 200 AH)

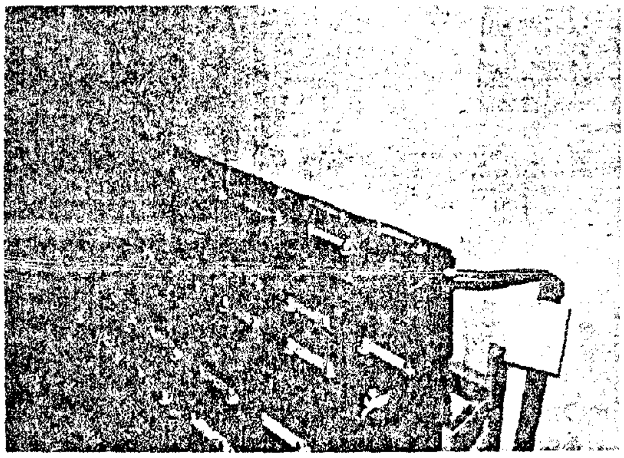
Functional System: Electrical Installation
Floor Installed: Basement
Boundary Dimensions: Batteries 115 x 170 x ^{H} 400
(Refer to the drawings below for the pedestal)
Weight: 935 kg in toal
Evaluation: C

****

INSTALLATION CONDITION AND COMMENTS

A total of 80 batteries are lined on the 3 stage pedestal, 27,27 and 26 batteries from the bottom to the top shelf.

The batteris and the pedestal are not fastened. This arrangement has the risk of having the batteries fall down by vigorous oscillations.



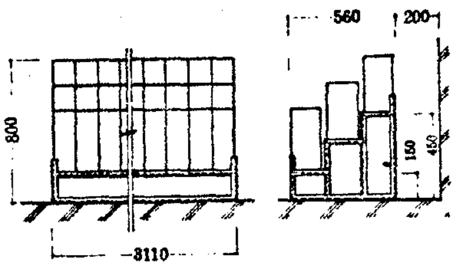
Some measures are necessary, for instance, securing of battries by fixation, since these batteris supply electricity to the central surveillance board during an emergency.

The wall in the back is earthquake proof and solid. One idea is to fasten the pedestal to this wall.

Incidentally, pedestal is composed of L - 40 x 40 x 5.

SKETCH AND DETAIL DRAWING

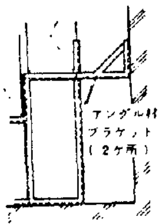
* Total View

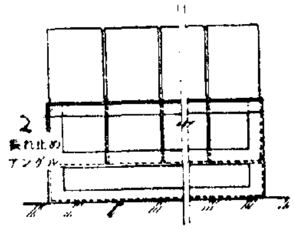


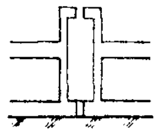
PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

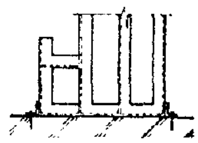
* Fix the upper part of the pedestal with stays from the wall (Drawing 1)
* Lay a stabilizer across the lowest shelf (Drawing 2)
* Bind two pedestals (fixing) (Drawing 3)
* Fix legs of the pedestal to the floor.

(Drawing 1) (Drawing 2) (Drawing 3) (Drawing 4)









Weld with plate(or simply tie with

Side view

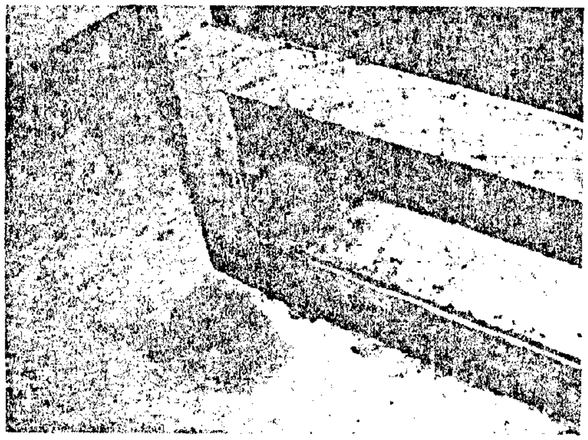
key-1. angle bracket (at 2 sites)wires).

2. stabilizing angle

*Pedestal Legs

Not fixed.

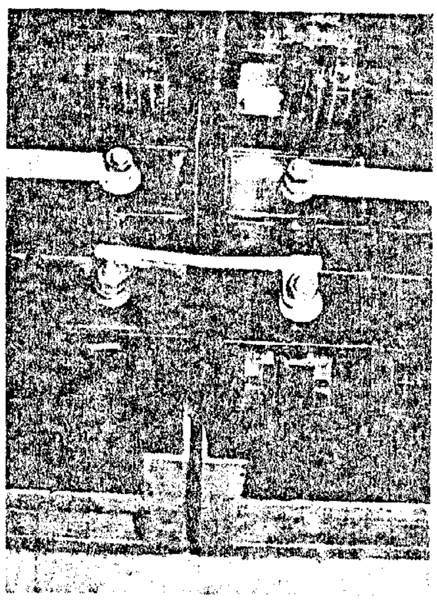
The pedestal may fall in some osccasions, and fall proof measures are necessary.



* Battery Connections

Since the pedestal is divided into two sections, the wiring over the boundary shall be given some play.
(The case shown here indicates no slack.)

Also, it is desirable to have the two sections of the pedestal integral.



BOILER OIL SERVICE TANK (FUEL SERVICE TANK)

Functional System:	Air Conditioning Unit
Floor Installed:	Basement
Boundary Dimensions:	ø 700 x L1400
Weight:	capacity 500 liter
Evaluation:	C

****

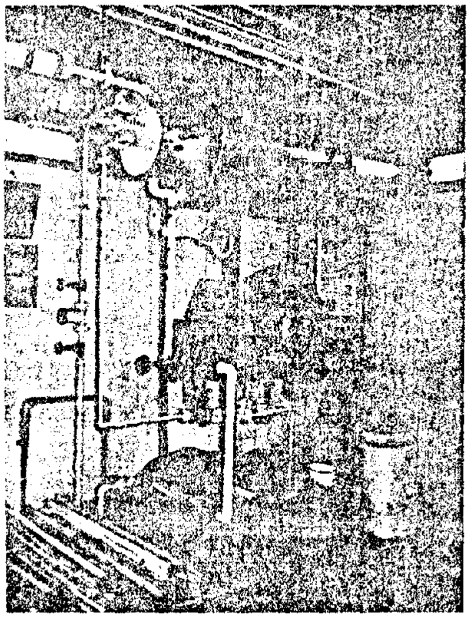
INSTALLATION CONDITION AND COMMENTS

This is just as dangerous as the tank previously shown.

At least, braces should be installed for the pedestal. Also, it is desirable that the top of the tank or pedestal be fixed with stays from the wall.

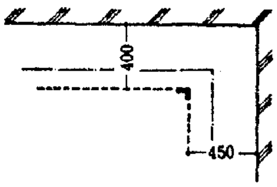
Also, flexible joints must be used for connecting pipes.

Pedestal: L-50 x 50 x 6

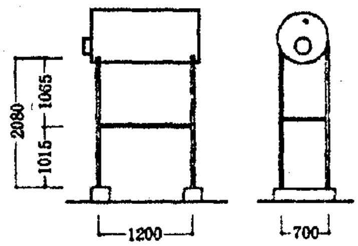


SKETCH AND DETAIL DRAWINGS

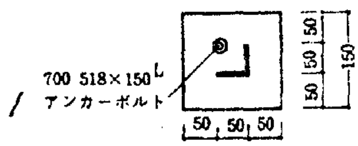
* Relation to Wall



* Pedestal Dimensions



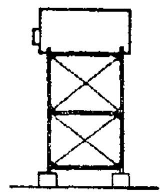
*Fixation on the Base of Legs



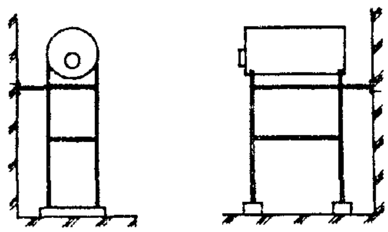
key-1. anchor bolt

PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

* Reinforcement of Pedestal.....Install braces



* Fix the top of the pedestal with stays from wall (at 4 sites).



AIR CONDITIONAER OUTSIDE AIR CIRCULATION DUCT

Functional system: Air Conditioning Unit
Floor Installed: Upper Floor
Boundary Dimensions: ∅ 1000-1100
Weight:
Evaluation: B

****

INSTALLATION CONDITION

It is hung by round steel rods of approximately 12 mm in diameter.

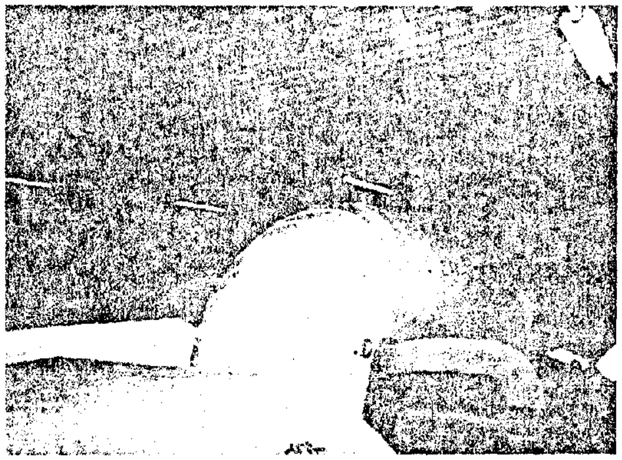
The strength of the round steel fixing sites at both ends is questionable.

Bending of stoppers were seen in several sites on the side of the duct.

Transverse oscillation control measures are necessary.

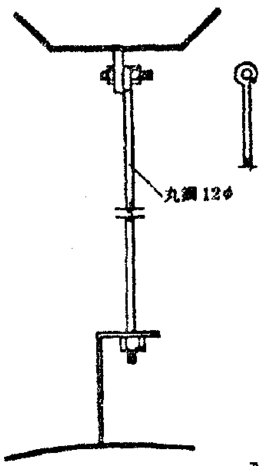
Also, fluorscent lamps hunb by chains, might break by bumping into the side walls due to resonance.

The outline of the fixation of the tope of the round steel rods is as shown below.



SKETCH AND DETAIL DRAWINGS

* Fixation of the Round Steel Upper End

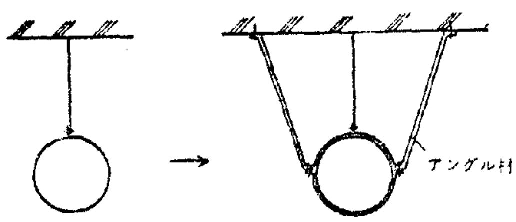


* Round Steel Lower End

Key-1. round steel
12∅

PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

* Prevent transverse oscillations



key-1. angle

TURBO REFRIGERATOR

Functional System:
Floor Installed: Upper Floor
Boundary Dimensions: ϕ 1116 x ^{L}5995 (Cyrinder of the main body)
Weight: 9300 kg
Evaluation: C

****

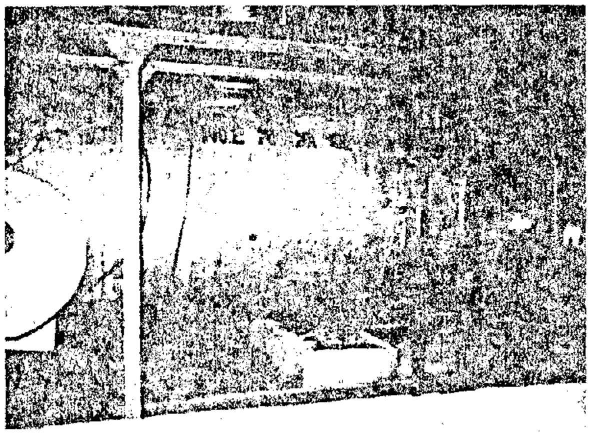
INSTALLATION CONDITION AND COMMENTS

A total of 6 sites are supported by a vibration proof spring. In appearance this machine is merely sitting on the springs.

If this assumption is correct, the main body will easily shift during earthquakes.

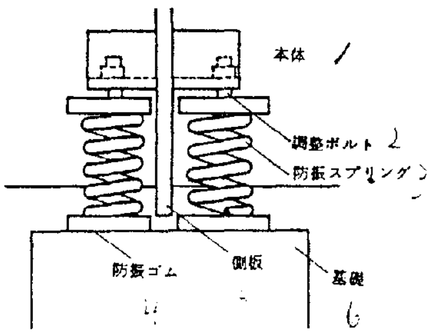
Stoppers are absolutely necessary (Springs are also not fastened to the foundation).

The structure of the bearing is shown below.



SKETCH AND DRAWINGS

*Vibration Proof Bearing

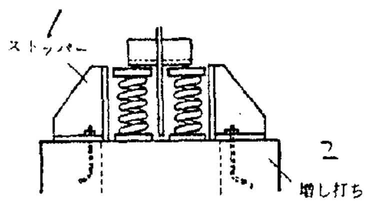


The main body is merely seated on springs with adjusting bolts.

key-1. main bode, 2. adjusting bolt, 3. vibration proof spring, 4. rubber vibration insulator, 5. side plate, 6. foundation

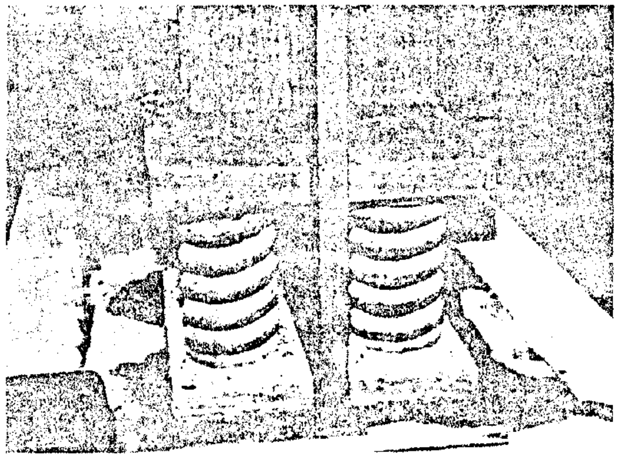
PROPOSAL FOR REINFORCEMENT & IMPROVEMENT

* Install stoppers.
* Install additional foundation.



Key-1. stopper
2. additional foundation

* Vibration Proof Springs



This is the picture of the central installation among the 3 sites on one side.

There are 4 springs in one site, a total of 24 springs.

The plate in the center is the flange from the main body.

The width and the height of the foundation are 490 and 205mm respectively.

* Vibration Proof Springs (Magnification)

This bolt appears to be the only means to support the main body on the springs.

Also, springs appear not to be fixed to the foundation at all.

If so, it is very likely that springs fall off and the main body shifts by earthquakes.

Control measures are necessary.

EXPANSION WATER TANK

Functional System: Air Conditioning/Ventilation
Floor Installed: Upper Floor
Boundary Dimensions: 1200ϕ x 1200^H
Weight:
Evaluation: C

****

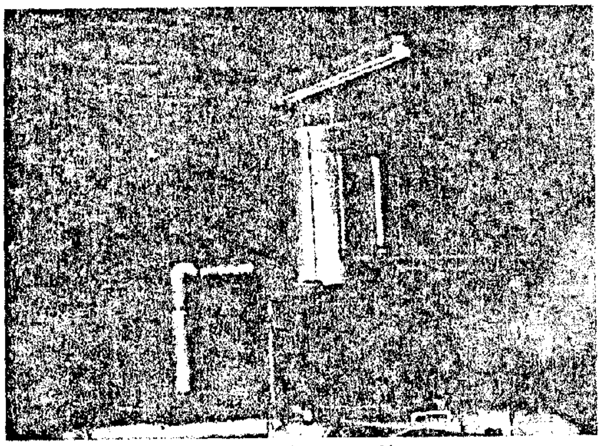
INSTALLATION CONDITION AND COMMENTS

It is hung by 4 round steel bars mounted on the girders of the ceiling.

Without modification, transverse oscillations occur, and round bars break off at the base causing the tank to drop, or in a less critical case, the connecting pipes may break off at the joints with the tank.

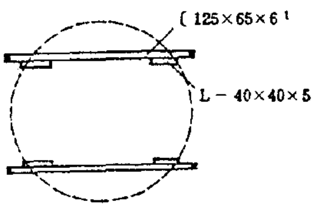
Transverse oscillation control measures are indispensible, for instance, construct a frame with angles.

Bottom Channel: [- 125 x 65 x 6^t

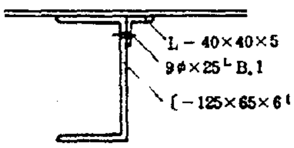


SKETCH AND DETAIL DRAWING

*Bottom of the Water Tank (from drawing)

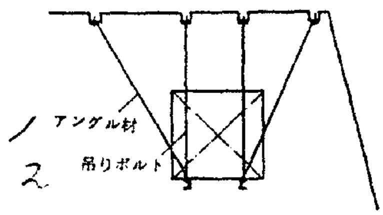


*Water Tank Installation Pedestal Detail (from drawing)



PROPOSAL FOR REINFORCEMENT & IMPROVEMENT

* Install stabilizers



key-1. angle
2. swing bolt

PACKAGE AIR CONDITIONER

Functional System: Air conditioning/Ventilation
Floor Installed: Upper Floor
Boundary Dimensions: 1100 × 565 x H2000
Weight:
Evaluation: A

****

INSTALLATION CONDITION AND COMMENTS

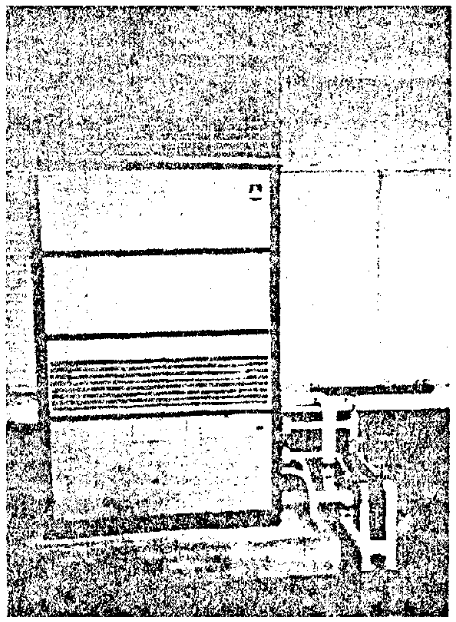
The unit is fixed on the upper part band from the girder behind the unit.

The bottom is not fixed but the unit merely sits on a vibration proof pad.

Therefore, theexistance of this band makes a distinctive difference.

This unit is worth referring to as an earthquake control example.

The earthquake resistivity will be further increased if metal fittings for the prevention of transverse shifts are installed at the bottom.

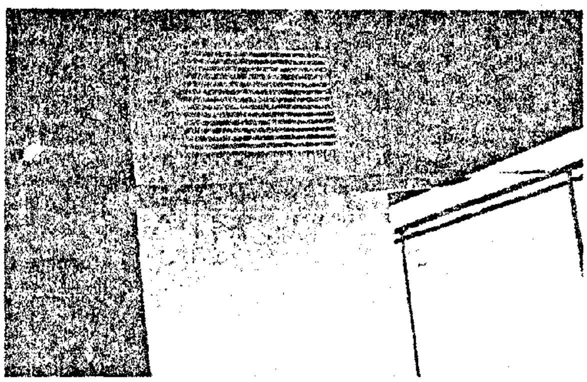


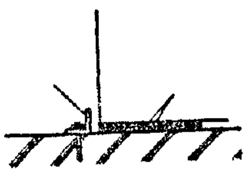
SKETCH AND DETAIL DRAWINGS

*Band on the Upper Portion

Section of the flat steel is 30 x 4. It prevents the unit from falling forward.

Vertically long machinery and equipment can be effectively protected by fixing the upper portion as shown in this case. The band is fixed on the back of the girder.





PROPOSAL FOR REINFORCEMENT & IMPROVEMENT

* Prevent transverse shifts.

ELEVATED (PALCED HIGH) FEED WATER TANK

Functional System:	Water Supply andDrain Installation
Floor Installed:	Upper Floor
Boundary Dimensions:	4000 x 2500 x $^{H}3600$
Weight:	(capacity 25,000 liters)
Evaluation:	?

****

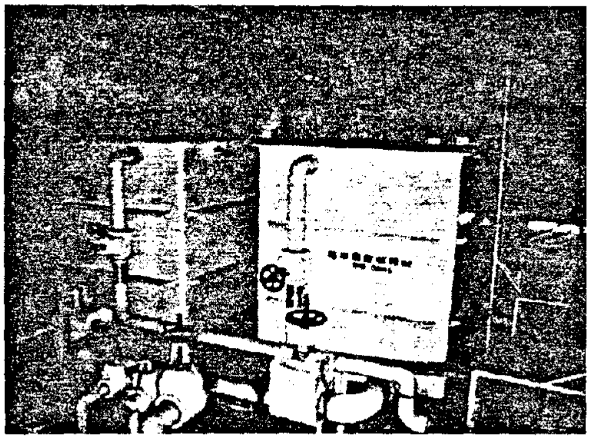
INSTALLATION CONDITION AND COMMENTS

Two foot lock foundations made on the slab receive the tank.

Presence of the fixation to the foundations is not known, but it appears the tank is merely seated on the foundations.

If not fixed, the tank may shift earthquakes, and as a result, piping may fail.

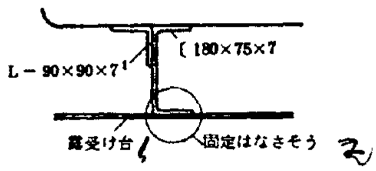
Although not related to the water tank, some transverse oscillation control measures are probably necessary for the ducts and fluorscent lamps which are merely hung from the ceiling.



The tank bottom detail is as shown below.

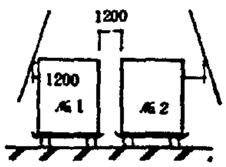
SKETCH AND DETAIL DRAWINGS

* Detail of Tank Bottom

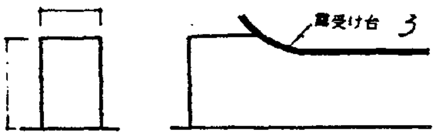


key-1. condensation receiver
2. no apparent fixation
3. condensation receiv

* Relation to Surroundings

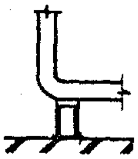


* Foundation and Water Tank



PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

*The suction water pipe (vertical pipe in the photo) and water tank joints may fail. The valves are heavy. Preferably, the pipe shall be supported at the part where it rises, so that a large load shall not be added to the joints with the water tank. Also, flexible joints shall be desirably used as appropriate.



* Since it is tremendously heavy, an easy treatment is insignificant. It is also feasible to fix the two tanks and then fix them to the steel columns on both sides or to the girders.

ELEVATOR CONTROL BOARD

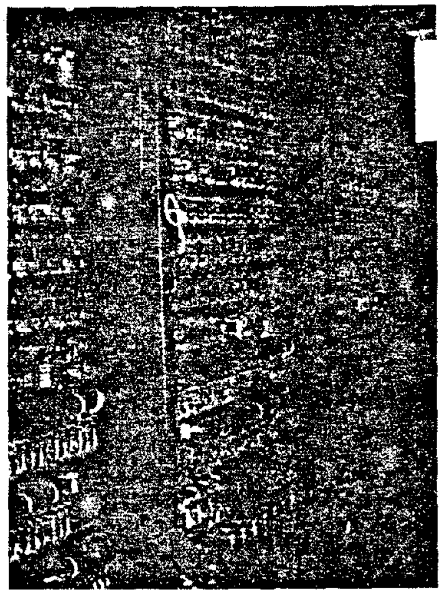
Functional System: Elevator Installation
Floor Installed: Upper Floor
Boundary Dimensions: 609 x 1372 x ^{H}1969 (one unit) 2905 (3 units)
Weight:
Evaluation: B

****

INSTALLATION CONDITION AND COMMENTS

Fixation to the floor is unknown. At any rate, the installation work is not secure enough.

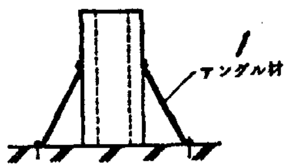
The center of gravity is high, and it is an important equipment. Some fall preventive measures are absolutely necessary.



SKETCH AND DETAIL DRAWING

PROPOSAL FOR REINFORCEMENT & IMPROVEMENT

* Fix the control boards to the floor or to the ceiling (when ther is a frame) with stays.



key-1. angle

CO_2 BOTTLE

Functional System: Fire Extinguishing Installation
Floor Installed: Basement
Boundary Dimensions: (tank) $\phi 270 \times {}^{H}1515$
Weight:
Evaluation: C

****

INSTALLATION CONDITION AND COMMENTS

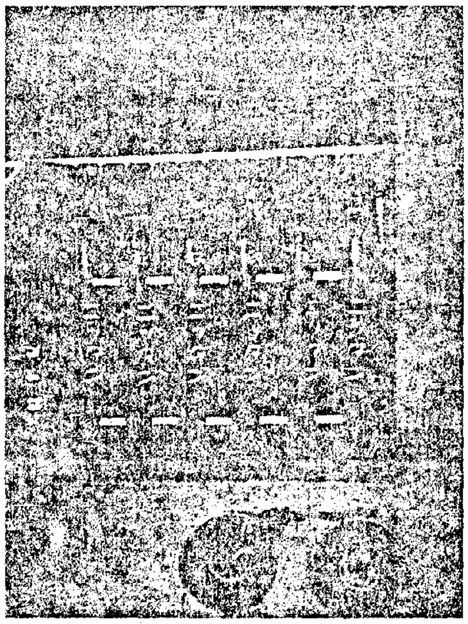
These are bottles for CO_2 fire fighting.

Apparently, 12 bottles are connected to the frame with metal fittings, but the frame is flimsy and may fall.

Either the frame must be reinforced, or stays must be provided from the floor or the wall as a desirable control measure.

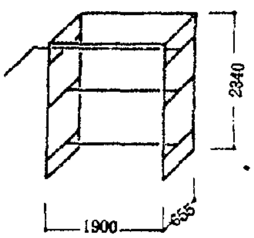
The upper horizontal pipe is fixed to the frame, but may not be able to meet large oscillations of the frame and breake.

It is requested that the strength of the frame and the fixed bolts be investigated.

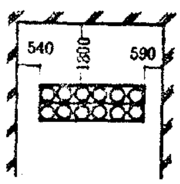


SKETCH AND DETAIL DRAWING

*Frame Dimensions
L-50 x 50 x 4

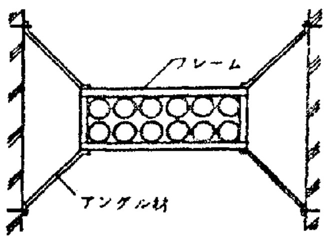


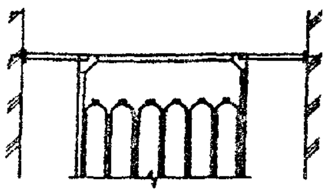
* Relation to Wall



PROPOSAL FOR REINFORCEMENT & IMPROVEMENT

* Fix the upper portion with stays.
(Stays from the floor are not appropriate since they will be in the way when exchanging tanks)





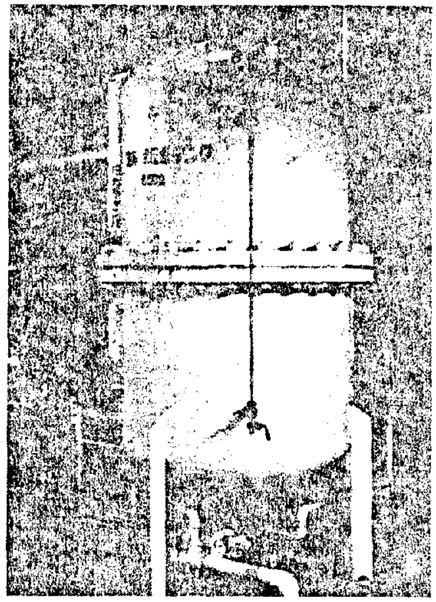
key-1. frame
2. angle

FIRE FOAM PRESSURE FEED TANK

Functional System:	Fire Extinguishing Installation
Floor Installed:	Basement
Boundary Dimensions:	ø850 x ^{H}1700
Weight:	510 kg (tank weight)
Evaluation:	B

****

INSTALLATION CONDITION AND COMMENTS



The tank is supported with three angle iron legs, but the strength is questionable.

It is necessary to investigate the problematic welding of the main body and the fixed sites to the floor.

As a control measure, the main body may be wrapped by a band or supported by stays.

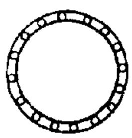
Incidentally, legs: L- 75 x 75 x 60

tank capacity: 800 ℓ

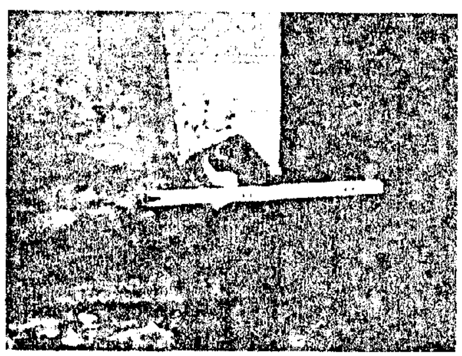
Refer to the photo for the base of legs.

SKETCH AND DETAIL DRAWING

* Relation to the Wall

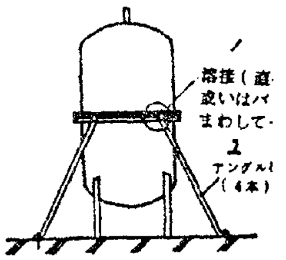


*Fixation of the base of legs



PROPOSAL FOR REINFORCEMENT AND IMPROVEMENT

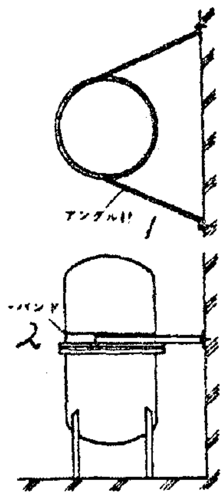
* Provide stays from the floor.



Key-1. welding(directly or on the band
that wraps around the body)
2. angles (four)

* Provide stays from the wall.

key-1. angles
2. band



SEISMIC DIAGNOSIS OF REINFORCED CONCRETE BUILDINGS

SHIZUOKA PREFECTURE URBAN HOUSING DIVISION

Bi

FORWARD

In Shizuoka prefecture, potential occurrences of Tokai Earthquake (Magnitude 8) originating in the vicinity of Suruga Bay are being discussed. Attempts are being made by this prefecture to promote systematic measures to control earthquakes from various angles as the most important program to be enforced.

Shizuoka prefecture will be shortly designated as an earthquake disaster prevention reinforcement area based upon the Law for Large Scale Earthquake Special Control Measures. Once designated, emergency disaster control measures and control measures essential for the prevention of disasters such as providing evacuation sites are expected to be stipulated. Currently, the task of formulating "Tokai Earthquake Control Measures" under the Shizuoka Prefecture Area Disaster Prevention Plan is being undertaken.

As you know, in respect to the earthquake control measures for buildings, we developed seismic diagnostic methods for existing buildings, and compiled guidelines for seismic designs for new buildings to be constructed in 1978, primarily for wooden housing, and they are already in use.

On the other hand, research on the earthquake resistivity of reinforced concrete buildings is actively encouraged. In 1977, the national government developed a technique to evaluate the earthquake resistivity of existing buildings. In addition, New Seismic Design Method (proposal) has been announced.

Consequently, technical standards of the Building Standard Law will be furnished one after another. In Shizuoka prefecture, as previously mentioned, in view of the announcement of the Tokai Earthquake Theory, we live under a special condition and the earthquake resistivity of reinforced concrete buildings must be promptly bolstered. Among reincorced concrete buildings, special buildings and buildings which serve as a base for activities involving disaster prevention, evacuation and rescue during earthquakes should be given special considerations.

Under these circumstances, it is thought necessary to raise the criteria for the evaluation of the strength of existing buildings and the standards for the design target strength of buildings to be constructed in the jurisdiction of this prefecture. Professor Kai Umemura, former Dean of Engineering Department, University of Tokyo (currently, professor at Shibaura Institute of Technology); Professor Hiroyuki Aoyama, University of Tokyo; Assistant Professor Tsuneo Okada, University of Tokyo and Professor Masaya Murakami, University of Chiba, are assigned to conduct research on possible earthquake inpur forces that can affect buildings.

This publication is a compilation of the results of the commissioned research and the research results achieved in the past by the professors mentioned above, and it is meant to be used as a text in the future service training to be administered. I trust you will use this document effectively and hope for the further improvement in earthquake resistivity of the reinforced concrete buildings.

February, 1979

In this publication, Part 1, Part 2 and Part 3 were borrowed from the Fiscal 1978 Public School Technical Employee Service Training Text, Journal Kenchikukai vol 26 No 12, 1977, and Journal Kenchiku Gijitsu No 317, 1978 respectively.

Table of Contents

PART 1 SEISMIC STRUCTURE PLAN FOR REINFORCED CONCRETE SCHOOL BUILDINGS

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1. PREFACE

You will remember for a long time 1978 as a year with a lot of earthquakes. On January 4, when the general public was still euphorious with the celebration of the New Year, the Izu Oshima Nearsea Earthquake occurred. On February 20th, the Miyagi Prefecture Offshore Earthquake followed it. School buildings suffered considerable damages. On top of it, the Miyagi Prefecture Offshore Earthquake recurred causing sizable damage primarily to the city of Sendai.

It is therefore quite appropriate that the public school technical employee service training class of this fiscal year chose the seismic plan for school buildings as its main topic. The author of this paper will be very happy and honored if this small contribution can be of any service during a group discussion. For this purpose, school buildings which suffered some degree of damage during the above three eqrthquakes are introduced in the text, if possible, with drawings and analytical research results, in order to discuss the relationship between the seismic structure plan and earthquake damages.

However, I would like to mention before entering into discussion that it is actually just a little too soon at this point of time to write something like this, since the research relating to the earthquake damages is in its initial stage and has not yet produced any significant quantitative conclusions as I am taking a pen to write this paper. Accordingly, the content of this text is not only inconclusive but also may be subjected to future partial correction by necessity.

Incidentally, the author gave a lecture titled "Seismic Structure Plan for School Buildings" at the Public School Facility Service Technical Staff Liaison Conference of this fiscal year, and the essentials of the speech was printed on the Conference Data[1].

The conference was held on 22-23 June, immediately after the second Miyagi Prefecture Offshore Earthquake, and the contents of the data is limited to the earthquakes which occurred in January and February. This time, in writing the text by putting the three earthquakes together, I would like to mention that some parts of the previously written compositions are reused as indicated below.

The part relating to the Izu Oshima Nearsea Earthquake which occurred in January, is almost the same in content as in the previous case, and the drawings and photos of the school buildings are also largely the same. However, the content of the part related to the analytical research was made more substantial.

The part relating to the Miyagi Prefecture Offshore Earthquake which occured in February, contains the general explanation and the photos virtually the same as the previous report, but the drawings are complete and the analytical research results have been newly added.

The part concerning the Miyagi Prefecture Offshore Earthquake of June has been written for the first time in this article, and naturally analytical research has hardly been conducted. This is the least accomplished part.

The writings regarding the above three earthquakes are expected to be incorporated in "Construction of School Buildings—Plans and Designs" scheduled to be published in 1978 by the Architechtural Institute of Japan.

2. ESTABLISHMENT OF SEISMIC DIAGNOSTIC STANDARDS

When promoting a seismic structure plan, the future direction will be the utilization of so called "seismic diagnosis"[2]. The conference data[1] rather elaborately describes the history and the essential content of the established seismic diagnostic standards. In the following, the data will be summarized.

The Tokachi Offshore Earthquake which occurred on 16 May, 1968, exactly 10 years ago, was the greatest test to the history of the development of the Japan's seismic designs since the Meiji Era[3][4]. The earthquake demonstrated very clearly that regular medium and low rise buildings designed in accordance with the Building Standard Law and Architectural Institute Calculation Standards were not necessarily safe against earthquakes.

The seismic structure research workers who took this as a lesson given by mother nature have been involved in various activities since that time, which are shown in Table 1.

Summing up these activities, the following is the common knowledge of today shared by those related to the architechtural tectonics. Specifically, the force of the earthquake motions that affect medium and low rise buildings during major earthquakes can be tremendously greater, 3 times or 5 times more than the design seismic intensity stipulated in the current Building Standard Law. In order to withstand this force, the strength that is about to be able to meet the stipulated level in the Building Standard Law is not at all sufficient. Larger strength (superflorous strength)and ductility are required. On the other hand, the actual strength of buildings varies significantly, although the design seismic intensity is set at 0.2. The strength of some buildings only bearly meets the design seismic intensity (Anthing lower than this is not expected to exist.) while the strength of others can be more than 5 times the design seismic intensity. Therefore, all of the real buildings will not be flattened like dominos during major earthquakes. Indeed, most of them can manage to stay intact. (In this respect, Japanese seismic designs have spread and reached the world's top level, owing to the long standing efforts of the forerunners. Japanese buildings are probably the strongest in the world against earthquakes), but those with insufficient strength and ductility are unfortunately mixed in and suffer damages during earthquakes.

Spreading of the awareness as described above entails various pronouncements regarding seismic designs shown in Table 1, and also embodies items regarding the seismic diagnosis of existing buildings. Among them, a lively exchange of opinions concerning the earthquake resistive designs for new buildings are repeated presently at the Ministry of Construction and the Architectural Institute of Japan. However, it appears to take much more time until a final conclusion can be reached in view of the fact that the issue of the design seismic intensity has a long tradition since the Urban Building Law.

In contrast to that, the seismic diagnosis of existing buildings promtly materialized since it was a new issue. The Japan's capital of the high economic growth period has accumulated as facilities such as buildings. Today, outstanding street districts spread out no matter which city is may be throughout Japan. When we think of earthquake control measures now in Japan, it is of course important to improve the seismic designs for new buildings, but it is much more important and urgent to evaluate whether or not existing urban buildings are safe. Incidentally, checking of the seismic design calculation does not help at all when trying to evaluate the adequacy of the earthquake resistivity of existing buildings. They are certainly designed in accordance with the Building Standard Law and the Architectural Institute Standards, and there is no difference in earthquake resistivity as far as that is concerned.

The real difference in earthquake resistivity is dominated by the size of the reserve yield strength of a building and its destruction mode. The reserve yield strength of medium and low rise buildings is always expected to be above o.2 in terms of seismic intensity. If the reserve yield is high and the destruction mode is rich in ductility such as bending destruction, buildings will be safe. Inversely, if the reserve yield strength is low and the destruction mode lacks ductility such as shearing destruction, the buildings are critically vulnerable to earthquakes. Depending upon the level of the danger, necessary measures such as reinforcement may have to be implemented.

What is generically called seismic diagnosis entails a process of judgement such as above, and works to cull out buildings with questionable resistivity from the existing buildings.

It may be considered natural that the reserve yield strength of actual buildings exceed the design seismic intensity since it is after all a design. As far as reinforced concrete structures are concerned, the difference between the two varies depending upon buildings and the designers, almost to a hopeless extent. Table 2 indicates the examples of these conditions [3)5)6)]. Reserve yield strength is expressed by yield layer shearing force coefficient in this table. As it is revealed, there are buildings which exceed the design seismic intensity only by 10% whereas some buildings exceed it by 400-500%.

I have described in the previous report[1)] why yield strength increases differently in various structures. An especially large factor is the presence of of external walls such as a lower sectional wall which is considered not to contribute to the structural yield strength and is ignored in calculation. In genuine pure rahmen structures which do not include these external walls, generally, the reserve yield strength is low. On the other hand, actual buildings that have these external walls, for instance, Nemuro Elememtary School and Hachinohe Specialized High School, indicated a very high reserve yield strength. If shearing destruction of columns are not incurred, external walls made of reinforced concrete and installed on the spot are very helpful. Unfortuantely, shearing destruction often occurs at a location where the effective length of columns are shortened by these lower sectional walls, and thus the lower sectional walls are blamed as the cause of hazards.

In the seismic diagnosis, a seismic index is computated, taking into consideration both the reserve yield strength and the destruction mode, as the product of strength index relative to the former and the ductility index relative to the latter. This is designed to give high scored to both the buildings with a high strength but a low ductility and the buildings with a not so high strength but a very high ductility.

Originally, the seismic diagnostic standards were for existing buildings. It is however a great step forward even for new building structural plans since the diagnosis has made it possible to express the seismic efficiency by a countinuous quantity. Conventionally, qualitative guidance singly dominated "notabilia" of structure plans such as "Place walls as evenly as possible". Utilization of seismic diagnosis in the stage of structure plan will allow more quantitative planning.

Table 1: Progress After Tokachi Offshore Earthquake

Date	Particulars
1968. 5.	Tokachi Offshore Earthquake (M 7.9)
	ADMINISTRATIVE AND TECHNICAL GUIDANCE
1969. 1.	"Earthquake Control Measures for RC Structure" (Journal Kenchiku Zasshi)
1970. 12.	"Building Standard Law Enforcement Ordinance" revision

1971. 5.	"RC Calculation Standards" revision
1975. 11.	"SRC Calculation Standards" revision
	INVESTIGATION AND RESEARCH
1968- 70.	Casualty and Damage Investigation, Experiments, Analyses
1968. 12.	"Tokachi Offshore Earthquake Casualty Investigation Report" (Architechtural Institute)
1971.	"Tokachi Offshore Earthquake Composite Report" (Keigaku Publishing Co.)
1971. 9.	Symposium based upon the Science Paper Reports (Architectural Institute)
	TRIAL SEISMIC DESIGNS
1971. 8.	"Special Reinforced Concrete Design Method" (Edited by the committee for the same)
1971. 11.	"School Building Plan" (Architectural Institute)
1973. 8.	"Dynamic Seismic Design Method for RC Buildings" (Edited by Kai Umemura
1976. 8.	"Seismic Safety of RC Structures" (Edited by Minoru Yamada)
1978. 4.	"Seismic Design Method for Medium and Low Rise Reinforced Buildings" (Edited by Steel Material Club)
	EXISTING BUILDINGS
1973. 11.	"Seismic Judgement Standards for Existing RC Buildings" (Architectural Institute)
1975. 6.	"Seismic Diagnostic Methods and Reinforcement Methods for RC School Buildings" (Architectural Institute)
1977. 4.	"Seismic Diagnostic Standards for Existing RC Buildings" (Special Building Safety Center)
1978. 6.	"Seismic Diagnostic Standards for Existing Reinforced Structures" (Special Building Safety Center)
	INTERNATIONAL COOPERATION
1970. 9.	"Japan/US Seminar (Tokachi Offshore Earthquake)" Sendai
1973. 9.	"Japan/US Seminar (San Fernando Earthquake)" Berkley
1973- 75.	Japan/US Cooperative Research (RC School Buildings)
1975. 7.	"Japan/US Cooperative Research and Investigation Meeting" Honolulu

1977-	"Japan/US Joint Research and Project with Large Scale Experimental Facilities"
	SEISMIC DESIGNS
1974.	Earthquake Load Joint Committee (Architectural Institute)
1977. 1.	"Earthquake Load and Earthquake Resistivity of Building Structures" (Architectural Institute)
1972- 77.	Comprehensive Project (Ministry of Construction)
1977. 3.	"New Seismic Design Method (Proposal)" Ministry of Construction

Table 2: Design Seismic Intensity and Reserve Yield Strength

	建 物	階 数	設計震度	保 有 耐 力			文 献
				1 階	2 階	3 階	
9	標 準 設 計 A 型	3	0.30	0.49	0.51	0.56	
	〃 A′ 型	3	0.20	0.30	0.31	0.34	
	〃 B 型	3	0.30	0.56	0.64	0.71	
10	〃 C 型	3	0.30	0.33	0.32	0.33	
	〃 C′ 型	3	0.20	0.25	0.26	0.28	
11	根 城 小 学 校 (C型)	3	0.18	0.61	0.56	0.86	3)
12	八 戸 高 専 南 棟	3	0.18	0.51 (0.73)*	0.82 (0.89)*	1.37	
13	同 西 寮	4	0.18	0.63	0.54	0.69	
14	試 設 計 A − WG	3	0.30	0.45	0.36	0.92	
	〃 B − WG	3	0.30	0.44	0.47	0.86	
15	〃 B−純ラーメン	3	0.30	0.37	0.35	0.80	
16	〃 B − 腰壁	3	0.30	0.46	0.52	0.87	
17	〃 建物全体	3	0.30	0.49	0.52	1.03	
18	設 計 例 1 (x方向)	3	0.30	0.51	0.52	1.07	
19	〃 (y方向)	3	0.30	0.47	0.47	0.95	
20	Olive View 病 院	5.B1	0.086	0.39	(P − δ曲線) 21		5)
				0.52	(単純和) 22		
23	適 用 例 A	3	0.20	0.27	0.35	0.44	
24	適 用 例 B	8	0.20	0.32	−	−	
25	適 用 例 C (x方向)	3	0.20	0.93	1.20	1.49	6)
26	(y方向)	3	0.20	0.42	0.55	0.68	

27 *せん断破壊しないとした時の値

Key-1. buildings
2. number of stories
3. design seismic intensity
4. reserve yield strength
5. first floor
6. second floor
7. third floor
8. literature
9. standard design type A
10. Type B
11. Nejiro Elementary School (Type C)
12. Hachinohe Specialized High School, South Wing
13. Hachinohe Specialized High School, West Wing
14. trial design
15. B- pure rahmen
16. lower sectional wall
17. entire building
18. design example 1 (x direction)
19. (y direction)
20. Olive View Hospital
21. (P-δ curve)
22. (simple sum)
23. applicational example A
24. Applicational example B
25. Applicational example C (x direction)
26. Applicational example C (y direction)
27. * values when shearing destruction is not assumed to occur

3. IZU OSHIMA NEARSEA EARTHQUAKE DAMAGE TO SCHOOL BUILDINGS

A moderate earthquak of magnitude 7 originating on the sea bed of approximately 17 km in depth between Izu Oshima and Izu peninsula occurred at 12:24, 14 Jan., 1978. The seismic intensities reported at various localities were V in Izu Oshima and Yokohama, and IV in Tokyo, Shizuoka and Ajiro. Actually, the damage suffered in Izu Oshima was minor. The major damage, namely, buildings roads, rail-roads and water system damages, was seen in Higashi-Izu-machi (Inatori, Atagawa) and Kozu-machi located in the east coast of the south Izu peninsula.

According to the investigation by the Shizuoka prefecture Board of Education, 59 schools throughout the Izu peninsula suffered damages, if damages to finished materials and broken glasses were accounted for. However, only four of the reinforced concrete school buildings reported damages to the structure of the buildings —Inatori Elementary School, Itatori Junior High School, Atagawa Elementary School and Atagawa Junior High School in Higashi-Izu-machi. The level of the damage was also minor, and the school buildings were reusable after partial repairs except for the Inatori Junior High School. It can be stated that the damage to the reinforced concrete school buildings was generally lighter than the ground damage such as landslides, or damages to wooden buildings and steel structure buildings.

The heaviest damage was noted in the Inatori Junior High School in Photo 1. It is a three story building built in 1955, and has a plane which offsets at the east side of the center entrance hall as seen in the plane view indicated in Figure 1. There were no expansion joints. Damages found were bending and shearing cracks on the columns and wing walls on all floors in the building due to the oscillations in the direction of the cross beams. As seen Phto 2, the slab of the staircase rooms on each floor were cracked. Some cracks were as much as 2 cm wide. An inspection of the building revealed 4-5 cm axial expansion displacement in the east and the west wings. The major cause of the damage was jolting of the joints and the forced displacement which worked as if to tear the entire top apart from the foundation due to the complex oscillations occurring in both wings. Judging from these conditions, it is possible that the footing beams in the entrance were broken off.

Figure 1. Inatori Junior High School Second Floor Plane View

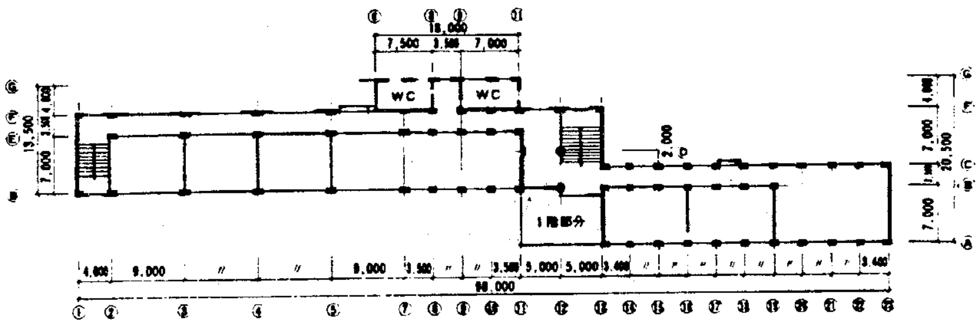
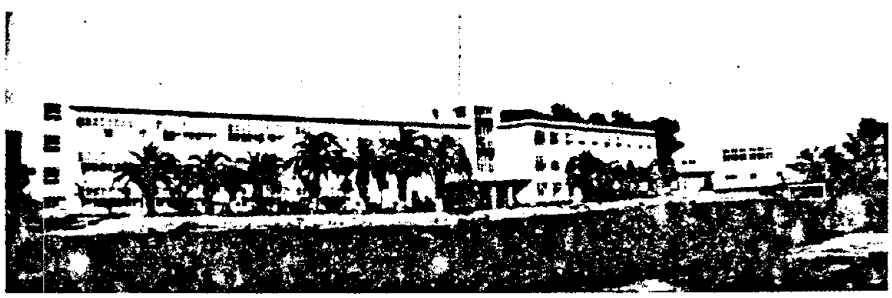


Photo 1. Inatori Junior High School South Front



In Table 3, cross-beam direction seismic diagnostic results are shown[7)8)]. Incidentally, beam direction earthquake resistance was also diagnosed, but both first and second diagnoses often inciated a high seismic index value of around 1.0 because of the presence of numerous reinforced concrete seismic walls in the beam direction used as room dividers. Therefore, there is absolutely no problem in respect to the beam direction earthquake resistance.

Photo 2. Inatori Junior High School
Cracked Center Staircase Room

The east wing of the Inatori Junior High School is structured with an uniformal rahmen consisting of an array of columns with a 3-4m span, without lower sectional walls, and it is dominated by curved columns high in firmness. In contrast, the west wing is basically a 9m span Type A building. Many shearing destruction sites were present in the columns. Columns with wing walls on the north side have a not-so-high yielding strength. Because of these, the seismic index of the west wing was much lower than the east wing. Furthermore, what must be noted is the fact that the seismic index obtained for the entire building combining the east and west wings, was lower than either of the west or east wing. This happened because it was disadvantageous for both wings to be joined only by partial contact. It is obvious that the earthquake resistivity of this building will be improved by installing expansion joints.

Table 3. Seismic Index of School Buildings (Cross-beam Direction)

	1 次数	2 第1次診断					3 第2次診断				
	4 階	1	2	3	4	5	1	2	3	4	5
5	稲取中・東棟	0.48	0.53	0.84	-	-	0.54	0.58	1.25	-	-
6	同・西棟	0.54	0.66	1.02	-	-	0.40	0.49	0.66	-	-
7	同・全体	0.44	0.51	0.82	-	-	0.33	0.40	0.64	-	-
8	熱川中	0.36	0.34	0.55	-	-	0.32	0.34	0.68	-	-
9	稲取小	0.52	0.44	0.56	0.97	-	0.57	0.63	0.74	1.09	-
10	熱川小・北棟	0.55	0.50	0.80	1.46	-	0.62	0.58	0.80	1.39	-
11	同・南棟	0.43	0.54	0.90	-	-	0.67	0.62	1.23	-	-
12	稲取高・教室棟	0.24	0.25	0.32	0.53	-	0.35	0.42	0.38	0.75	-
13	伊東高	0.25	0.25	0.23	0.30	0.45	0.53	0.59	0.69	0.83	1.74

Key-1. order,
2. first diagnosis
3. second diagnosis
4. floor
5. Inatori Junior High School, east wing
6. Inatori Junior High School, west wing
7. same, entire building
8. Atagawa Junior High School
9. Inatori Elementary School
10. Atagawa Elementary School, north wing
11. same, south wing
12. Inatori High School, classroom wing
13. Ito High School

Atagawa Junior High School is a three story B type buildings (span 4.5m) built in 1962, and has a plane which elbows in a mild angle as shown in Photo 3 and Figure 2. The cross-beam direction structure is practically made of pure rahmen. Columns on each floor indicated bending cracks due to cross-beam direction oscillations. The surrounding ground was considerably jolted, and piping on the north side of the school building was broken. A wooden room divider in the science room at the west end of the first floor was buckled as shown in Photo 4 due to the swelling of footing beams and the concrete floor by the sinking of the school buildings. The foundation of this building has independent footings without piles. Incidentally, a steel brace broke in the gymnasium.

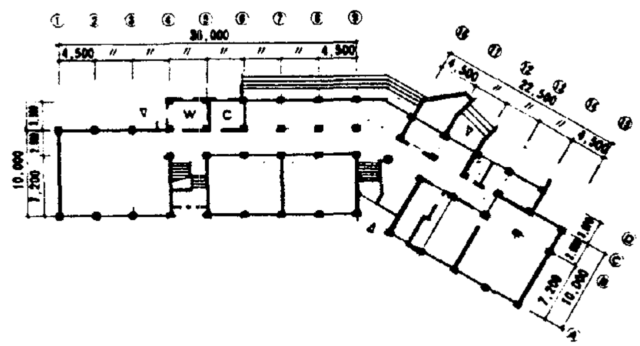


Figure 2. Atagawa Junior High School
First floor Plane View

Photo 4. Atagawa Junior High School
Buckled Wooden Room Divider

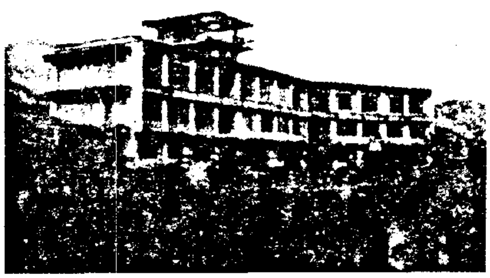


Photo 3. Atagawa Junior High School South Front

From the seismic diagnostic results in Table 3, seismic index values both in the first and second diagnoses were low, especially the earthquake resistivity of the first and second floors was insufficient. When buildings are of pure of near pure rahmen structures at a design seismic intensity of 0.2, the earthquake resistivity will be deficient unless the firmness of buildings is very well attended to. This point has been often reminded in the past[3)5)] but it is reconfirmed by the seismic diagnosis.

Inatori Elementary School is a four story A type building (span 9m) built in 1970, and it is in straight line as shown in Photo 5, a picture of the south front, and in Figure 3, a plane view of the first floor. Stairways, bathrooms and other rooms are all arranged to the north of the middle corridor. The arrangement gives a structural plan with some walls in the cross-beam direction. Main damages were seen in shearing cracks of cross-beam direction walls in the corridor and bending destruction of the entrance hall eaves at two sites. Others noticed were broken fixed sash windows of the entrance hall, fallen bookcases on the second floor, a flown grand-piano on the fourth floor and a dropped ceiling of the gymnasium in a separate building.

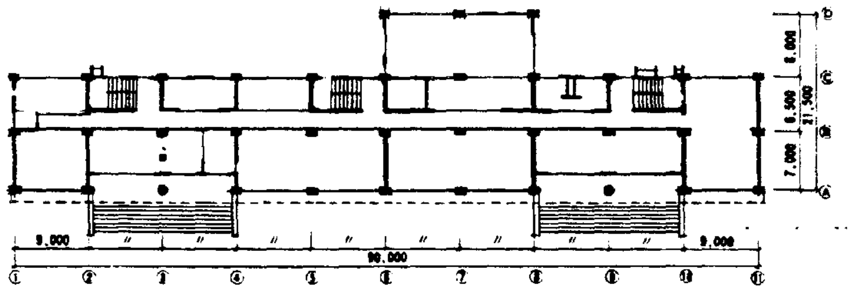


Figure 3. Inatori Elementary School First Floor Plane View

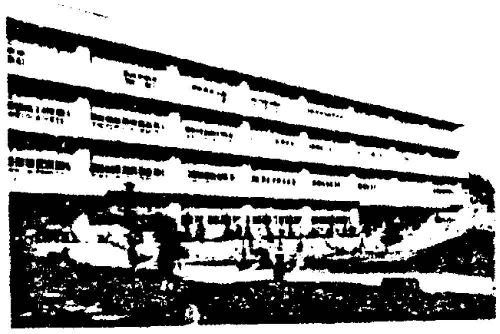


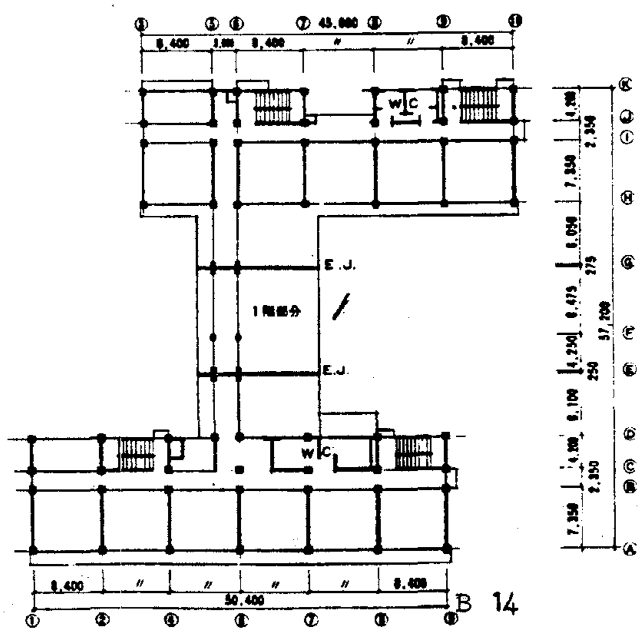
Photo 5. Inatori Elementary School South Front

The seismic diagnostic results in Table 3 also show values over 0.5–0.6 both in the first and second diagnoses owing to the well demonstrated supportive effect of the cross-beam direction walls.

However, in the case of this school, it is quite disturbing to find that the damages concentrated around the vicinity of the doorways, for instance, bending destruction of the cantilevers of the entrance eaves and broken fixed sash windows, since it is a school building. Fortunately, the warthquake occurred on a Saturday afternoon, and human casualties in the vicinity of the doorways were not reported. But under different circumstances, it could have been very dangerous. "Designs" for the doorways and their immediate area should not be elaborate but desirably stress safety.

Atagawa Elementary School was build in 1975, and has a H shaped block plan as shown in Figure 4. The four story north wing and the three story south wing are connected by the two story connecting corridor with expansion joints. Photos 6 and 7 show south front and north front of the north wing. The building has some walls in the cross-beam direction as in Inatori Elementary School. It is also characterized with a middle corridor stype with four rahmens installed in the cross-beam directio. Damages were primarily shearing cracks of seismic walls and some external walls. Expansion joints were designed and constructed very carefully and demonstrated their full function. Incidentally, a ceiling in the gymnasium in a separate building dropped just as in the Inatori Elementary School.

Figure 4. Atagawa Elementary School Second Floor Plane View



key-1. part of first floor

Photo 6. Fouth Front

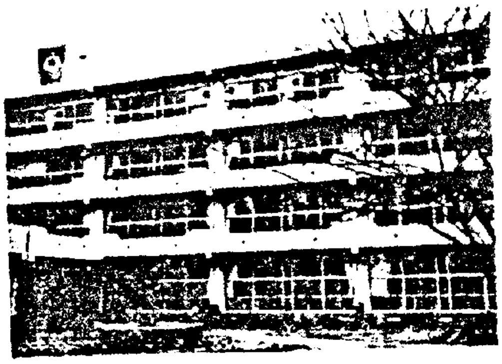
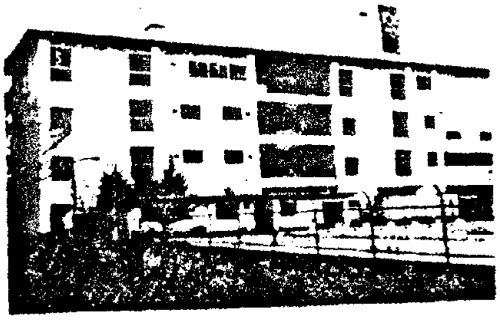


Photo 7. North Front



Atagawa Elementary School North Wing

The seismic results are also similar to those of the Inatori Elementary School, and considerably high index values were obtained both in the first and the second diagnoses. It may be possible to increase further the seismic index by tidying the external wall of the north front in Photo 7.

The above discussion of some Izu Oshima Nearsea Earthquake damages to the four elementary and junior high schools mainly focussed on the structure plan comparing the state of the damage and the results of the seismic analysis. The studies on the absolute seismic index values and the seismic judgement have not been completed in detail, and it is better to think of this value, in this paper, as a yardstick that indicates relative size of the earthquake resistivity of each school. In this perspective, Table 3 apparently indicates that the earthquake resistivity of Inatori and Atagawa elementary schools was better than that of Inatori and Atagawa junior high schools. This trend also coincides with the trend in the size of the damages incurred to a certain degree. There is a meaning in that this difference was produced not because of the appropriateness of the structure calculation but because of the difference in structure plan tied to the plane plan.

The seismic diagnostic results of two high schools in the same area are shown in Table 3, although they did not suffer damages. Inatori High School was build in 1965 and has an A type four story administration/general class room wing and a B type (cross-beam direction 1 span) four story special class wing connected by connecting corridors with expansion joints. In Table 3, the administration/general classroom wing was diagnosed, but the seismic index in the first and second diagnoses was very low. The school did not suffer damages because of the low earthquake input force and probably because of the virtual absence of the cross-beam direction secondary elements such as non-structural walls that give an impression associated with damages. Nevertheless, these buildings can be classified as problematic buildings with respect to earthquake resistivity judging from the seismic index values.

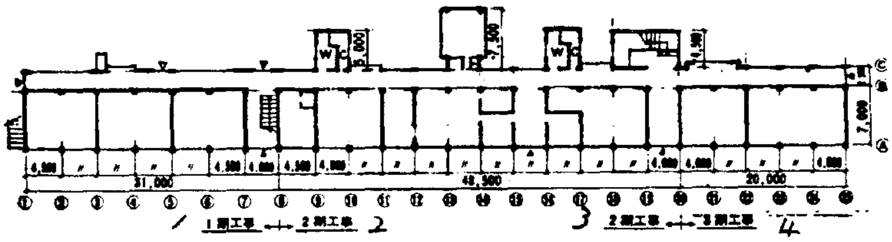
In comparison, Ito High School was built in fiscal 1976 and 1977, and is a B type (beam direction 1 span, partially middle corridor style with 2 spans) five story building with a cross-beam direction design seismic intensity of 0.24. Also, the seismic index value is considerably improved by installing walls such as bathrooms to the north of the corridor. If rigid frames are installed in the boundary of classrooms and the corridor to raise the beam direction span to 2 spans, more superior earthquake resistivity can be achieved.

4. MIYAGI PREFECTURE OFFSHORE EARTHQUAKE (FEBRUARY) DAMAGE TO SCHOOL BUILDINGS

This moderate earthquake of a 6.8 magnitude occurred at 13:37, 20 February, 1978, and originated in the sea bed of 56 km in depth, approximately 110 km northeast of Sendai and approximately 40km southeast of Ofunato[9]. The seismic intensities reported at various localities were V in Ofunato and IV in Morioka, Miyako and Sendai, but the oscillations in the city of Sendai were rather severe and many cases of broken window glasses in medium and high rise buildings were reported, for instance, some strong motion seismograph recorded maximum acceleration of 170 and 144 gals. Damages to wooden houses, roads and railways were generally light, but reinforced concrete school building and other public building damages occurred in Minamikata-machi, Ishikoshi-machi, Tsukidate-machi, Shiwahime-machi and Ichihasama-machi in Miyagi prefecture, and Ichinoseki-shi in Iwate prefecture, in the inland 60-80 km from the hypocenter. The majority of the buildings which incurred damage had flimsy "non-structural walls" over the cross-beam direction low rigidity pure rahmen, and only a few cases incurred damage to the main structure. However, it is desirable that a deliberate investigation be conducted regarding whether or not these damages by moderate seismic motions as seen at that time can be tolarated, and whether or not it is better to add regidity to the main structures to provide for the future.

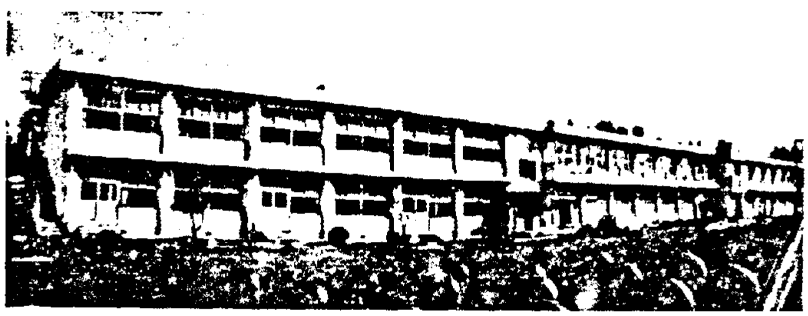
The largest school building damage was seen in Nishigo Elementary School in Nishigo, Minamikata-machi (Photo 8, Figure 5). This school incurred damage to the wooden school building of that time during the North Miyagi Prefecture Earthquake of April 1962. The school was reconstructed as the first 7 span structure and completed with reinforced concrete as a disaster restoraration in March 1963. The second and third term construction works were done over in 1973 and 1974, and it became a very long school building as seen in Photo 8. The first term construction work was designed in accordance with the old RC standards of the Architectural Institute of Japan, and the second and third term construction work was designated in accordance with new standards revised in 1971 which incorporated lessons learned from the Tokachi Offshore Earthquake.

Figure 5. Nishigo Elementary School First Floor Plane View



key-1. first term construction work
2. second term construction work
3. second term construction work
4. third term construction work

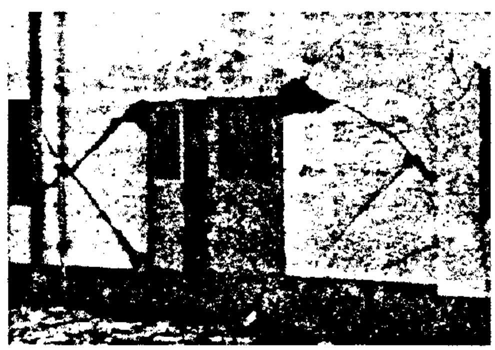
Photo 8. Nishigo Elementary School South Front



The structure of the Nishigo elementary school is of the B type, 4.5 span in the cross beam direction. The corridor is cantilever stype and has only two lines of cross-beam rahmens. Damages suffered indicated good contrast—shearing cracks and/or shearing destruction of columns (Photo 9) built during the first construction work completed in 1963 and bending cracks and/or bending destruction of columns built during the second and the third construction works completed in 1974. The north side 12 cm thick reinforced concrete external wall is considered as non-structural element and suffered damages as shown in Photo 10. The structural body is relatively thin (column dimensions 50 cm x 55 cm), and it deformed largely due to the absence of cross-beam direction seismic walls. The external wall probably could not follow this deformation. It should have been designed with the external wall as a part of structural body if this wall was made of concrete installed right on the spot, or it should have been designed to give much more rigidity to rahmens so that the deformation that destroyed "non-structural wall" could have been prevented. Likewise, inversely, if it is preferred that the structural body should remain thin as in the current building, deformation during earthquake is unavoidable, and the external wall should have been a curtain wall that can better cope with deformation.

Photo 9. Shearing destruction of columns

Photo 10. Destruction of North Front External Wall

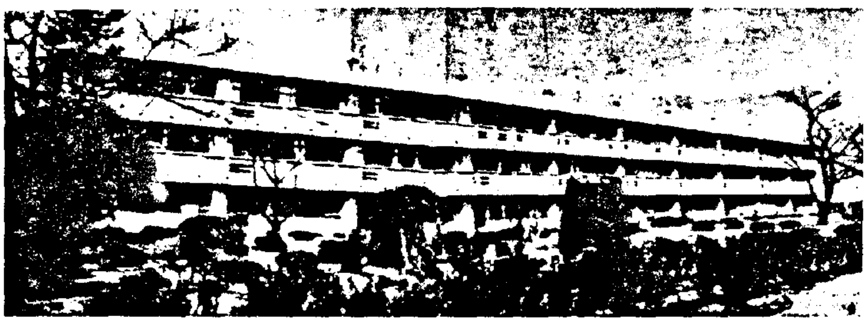


Nishigo Elementary School

The Ichihasama elementary school in Ichihasama-machi and the Shiwahime junior high school in Shiwahime-machi(Figure 6) are both A type 9m span three story buildings. There are balconies on the south side and corridors on the north side extended by cantilevers. There is a bathroom wing built separately on the north side connected by staris to the corridors. Accordingly, the floor of the bathrooms are offset by a half floor level. In the Ichihasama elementary school, the two wings built side by side are connected by a two story structure with expansion joints (Photo 11) whereas the Shiwahime junior high school building stretches straight in line (Photo 12). However, the structural seismic efficiency of the two are considered practically the same, to begin with the dimensions of the columns (southside 45 x 90, north side 50 x 90). The former was built in 1963 and the latter was built in 1967 both in accordance with the old RC standards of the Architectural Institute.

Photo 11. Ichihasame Elementary School South Front

Photo 12. Shiwahime Junior High School South Front



Most of the damage concerned the shearing destruction of the non-structural walls installed on the edge of the corridor held by cantilevers. In this point, these cases resemble the case of the Nishigo Elementary School. Some columns received shearing cracks which are not as bas as shearing desctruction. Considering that these buildings were designed in accordance with old standards, if the seismic motions had been just a little more fierce, damages such as shearing destruction of columns and further the collapsing of the entire building as seen during Tokachi Offshore Earthquake might probably have occurred.

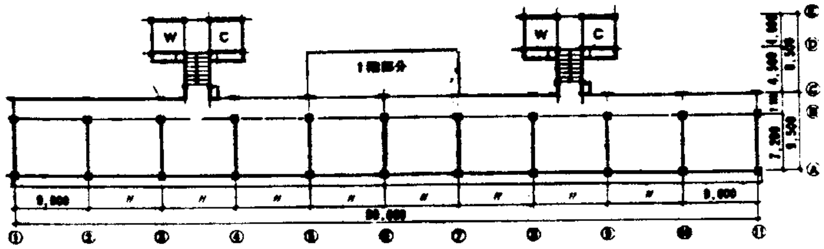


Figure 6. Shiwahime Junior High School Second Floor Plane View

The Shiwahime elementary school was designed and constructed in 1973, six years later than the junior high school in accordance with new standards (Photo 13, Figure 7). The entire building is divided into several blocks some with middle corridor style and others with side corridor style. Each block has cross-beam direction walls and columns are comparatively fat. The damages found were hair cracks surrounding the cross-beam windows, shearing cracks of the cross-beam direction walls in the joints and peeling of the cover mortar of the expansion joints, which were considered light damages to the structure.

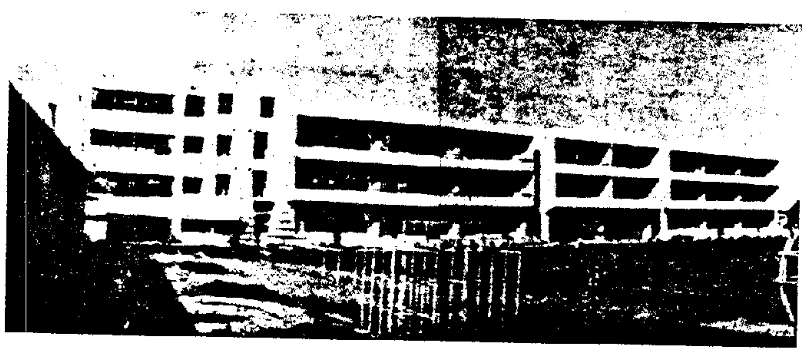


Photo 13. Shiwahime Elementary School South Front

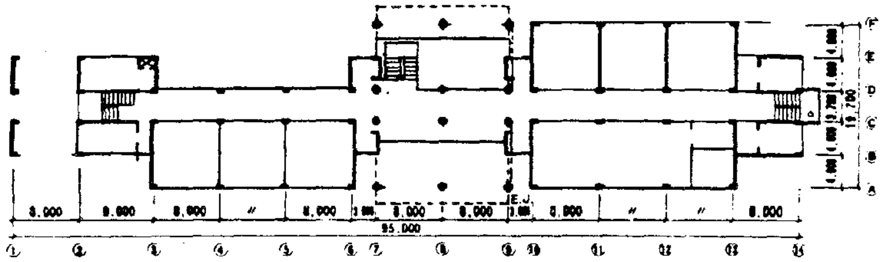
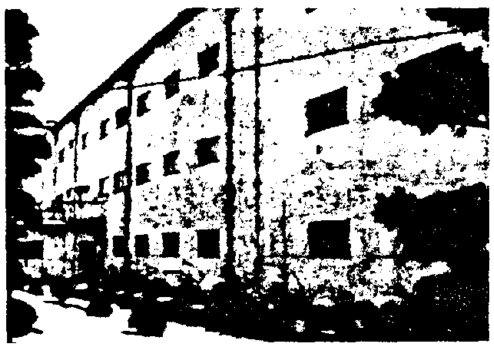


Figure 7. Shiwahime Elementary School First Floor Plane View

The Kurihara Agriculture High School (Photo 14) in Wakayanagi-machi is a B type cross-beam 4m span three story building with a span of 2 spans in the side corridor stype structured in 1961. The rahmen to the north of the corridor has lower sectional walls, drop walls and wing walls, and the entire rahmen is constructed like one sheet of wall. Damages were light—shearing cracks in one of the columns on the first floor on the sourth side, cracks around the openings of walls on the north side and peeling of the finishing coat.

Photo 14. Kurihara Agriculture High School
North Front



The Yasakae Junior High School (Photo 15) locates in the east end of Ichinoseki-shi, Iwate prefecture, and it is a three story building completed in 1965. The plane is of the battery type as shown in Figure 8. Structurally, it is virtually a pure rahmen in the cross-beam direction. As seen in the photo, there was a 3-layer pentohouse, but this penthouse suffered the heaviest damage. The three story pure

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arranged for the prevention of shearing destruction.

Table 4 Seismic Index of School Buildings (Cross-Beam Direction)

1 次数	2 第1次診断				3 第2次診断			
4 階	1	2	3	4	1	2	3	4
5 志波姫中	0.18	0.23	0.38	−	0.30	0.30	0.95	−
6 志波姫小・東棟	0.38	0.58	0.80	4.87	0.67	0.68	0.84	3.88
7 同・西棟	0.29	0.30	0.44	1.10	0.65	0.80	1.14	1.22

key-1. order, 2. first diagnosis, 3. second diagnosis, 4. floor
5. Shiwahime Junior High School, 6. Shiwahime Elementary School, east wing, 7. the same, west wing

5. MIYAGI PREFECTURE OFFSHORE EARTHQUAKE (JUNE) DAMAGE TO SCHOOL BUILDINGS

On 12 June, 1978 at 17:14 when the momory of the February earthquake was still fresh in everyone's mind, a 7.4 magnitude major earthquake occurred in the sea bed approximately 40 km in depth and approximately 130 km to the east of Sendai[10]. This location is approximately 80 km to the sourth east of the hypocenter of the February earthquake. Seismic intensities in various localities were V in Sendai, Fukushima, Yamagata, Ishinomaki and Ofunato, and IV in almost all areas of Tohoku region and Kanto region. Especially, the seismic motions were violent in the city of Sendai and the adjacent city of Izumi. According to the recording of the strong motion seismograph in Sendai, the south north directional seismic motions were fierce, and the maximum acceleration was 250-300 gals on hills and soft grounds and 150-180 gals on hard ground[10].

Disasters caused by this earthquake presented various problems pertaining to urban type lifeline damage, housing ground damage, ways to make up a loss of shared property such as condominiums and deficiency in earthquake resistivity of outside structures such as block fences. Damages to building structures extended over to RC, SRC, S and wooden structures, and among RC, the collapse of the five pilotis

3-4 story retail store/office buildings in Wakatake and Oroshi-machi districts in Sendai, were the focuss of attention.

A joint investigation team from the University of Tokyo and the University o f Chiba investigated, especially with emphasis on school buildings, almost all cases of damages to them[11]. Table 5 is a list of school buildings which incurred damages to the structural body, and the damage ratio of school buildings calculated for Sendai is presented in Table 6.

The classification basis for major, moderate and minor damages was in compliance with that used for the Tokachi Offshore Earthquake[12]. Roughly speaking, major damage implies buildings with loss of horizontal resistance, moderate damage implies partial destruction of the structural body and minor damage means cracks in the structural body and destruction of non-structural materials.

When plotting damaged schools on a map, a good number of schools were concentrated in a zone 3km in the north to south direction, approximately 1 km to the east of Sendai station, which coincided generally with the damage distribution of general buildings. It is assumed that the seismic motions of this zone was larger than any other areas in Sendai.

Table 5. List of Schools Damaged by Miyagi Prefecture Offshore Earthquake

場所	大破	中破	小破
仙台	市立図南高 (3)	東仙台小 (4)	南材木町小 (4)
	県立総合衛生学院 (3)	愛宕中 (4)	八本松小 (4)
	私立東北工大・3号館 (5＋3)	長町中 (4)	若林小 (4)
	同，5号館 (3)	私立聖和学園高 (5)	台原中 (3)
		私立仙台育英高 (5)	県立仙台三高 (4)
		私立常盤木学園 (4)	市立仙台商高 (4)
			私立電子工業高 (3)
			私立宮城学院 (3～6)
泉	県立泉高 (3)	将監西小 (4)	南光台小 (4)
		私立三島学園短大 (3)	私立三島学園高 (3)
一関	弥栄中 (3)		

(註) （　）内の数字は階数

表中の小中学校はすべて公立校

key-1. place, 2. major damage, 3. moderate damage, 4. minor damage, 5. Sendai, 6. Tonan High School (City Operated), 7. Higashi Sendai High School 8. Minami-Zaimokucho Elementary School, 9. Comprehensive Health Academy (Prefecture operated), 10. Atago Junior High school, 11. Happonmatsu Elementary School, 12. Tohoku Institute of Technology (private), No. 3 Building, 13. Nagamachi Junior High School, 14. Wakabayashi Elementary School, 15. Seiwa Gakuen High School (private), 16. Dainohara Junior High School, 17. Yohoku Institute of Technology (private), No. 5 Building, 18. Sendai Ikuei High School (private), 19. Sendai Daisan High School (prefecture operated), 20. Tokiwagi Gakuen (private), 21. Sendai Business High School (city operated), 22. Electronic Technical High School (private), 23. Miyagi Gakuin (private), 24. Izumi, 25. Izumi High School (prefecture operated), 26. Shokan Nishi Elementary School, 27. Minami Kodai Elementary School, 28. Mishima Gakuen Junior College (private), 29. Mishima Gakuen High School (private), 30.Ichinoseki, 31. Yasakae Junior High School, 32. (note) numerals in ( ) are the number of stories, 33. Elementary and junior high schools in the table are all public.

In Table 6, ten city elementary and junior high schools not investigated are all in the mountains to the west of the city. These schoolscan be treated separately without any problems from the schools in the streets of Sendai. The damage ratio of urban Sendai rather should include thecity of Izumi adjacent to the north. However, the investigation did not cover the total area of the city of Izumi, and the damages in Izumi were omitted from Table 6.

In Column K of Table 6, a damage ratio above moderate damage was indicated. The similar damage ratio calculated pertaining to the school building damages by the Tokachi Offshore Earthquake was approximately 25%[3). Compared to that, the damage ratio of this earthquake is low.

The damage ratio for private schools is higher than for the public schools. One of the reasons for this is accountable for the uniformal and monotonous design of the public schools in Sendai. The elementary and junior high schools of Sendai are almost all designed by the Construction Division, Building and Repairs Section of the city, and there are many school buildings with a good structure and design.

Table 6. School Damage Ratio in Sendai

	1 市立小学校	2 市立中学校	3 県立高校 市立高校	4 県立各種学校 国立小中校 私立学校
A 総数	63	27	15	
B 調査せず	8	2	0	
C 調査校	55	25	15	25
D 木造・S造	8	4	1	1
E RC校	47	21	14	24
F 大破	0	0	1	2 *
G 中破	1	2	0	3
H 小破	3	1	2	2
I 軽微	16	14	8	12
J 無被害	27	4	3	6
K 被害率 $(\frac{F+G}{E})$	.021	.095	.071	.208
		.049		
L 被害率 $(\frac{F+G+H}{E})$	.085	.143	.214	.292
		.122		

＊東北工大は一校と数える。

Key-1. city elementary schools, 2. city junior high schools, 3. prefecture and city high schools, 4. various prefecture schools, national elementary and junior high schools, private schools
A. total, B. not investigated, C. schools investigated, D. wooden buildings and special buildings, E. RC schools, F. major damage G. moderate damage, H. minor damage, I. slight damage, J. no damage K. damage ratio, L. damage ratio, * note: Tohoku Institute of Technology was counted as one school.

Also, there are city standard designs. It is a future subject to evaluate the seismic efficiency of these schools, but probably they are considerably high in earthquake resistivity. In comparison, the designs of the private schools are unique and individual. Schools with high and low earthquake resistivities are mixed, and thus approximately 20% of the schools incurred moderate or greater damages.

Among the schools which suffered major damages, the Comprehensive Health Academy (prefecture operated) uses as a school building a renovated agricultural experimental station built in 1952. Theshearing destruction of the walls penetrated the columns. The quality of the concrete also appeared to be questionable. Other severely damaged schools except for Izumi High School incurred shearing destruction of cross-beam direction pure rahmen columns which were designed in accordance with old RC standards. As one of the examples, a panoramic view of Tonan High School and the shearing destruction of columns are shown in Photos 16 and 17.

Photo 16. Tonan High School East Front

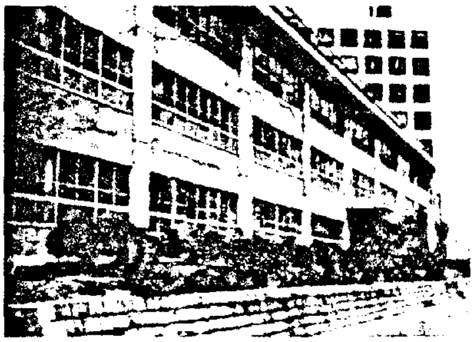


Photo 17. Tonan High School Shearing Destruction of Columns

The plane of Izumi High School is as shown in Figure 9. Three 3 story buildings are connected by connecting corridors with expansion joints. The structure was designed in accordance with the new standards revised in 1971. Since the columns were designed without paying attention to the cross-beam direction lower sectional walls, shearing destruction of columns, as a result, occurred just as in other buildings. (Photos 18, 19). The new standards do not require total and complete prevention of shearing destruction. Therefore, destruction such as this, in a way, cannot be helped. It is appreciated that the shearing destruction which occurred in this school was not as severe as the destruction in Tonan High School, and the buildings tentatively resisted vertical loads. However, this case once again indicated that utilization of seismic walls is the most practical solution as it is very difficult to design earthquake resistant pure rahmen structures.

Figure 9. Izumi High School First Floor Plane View

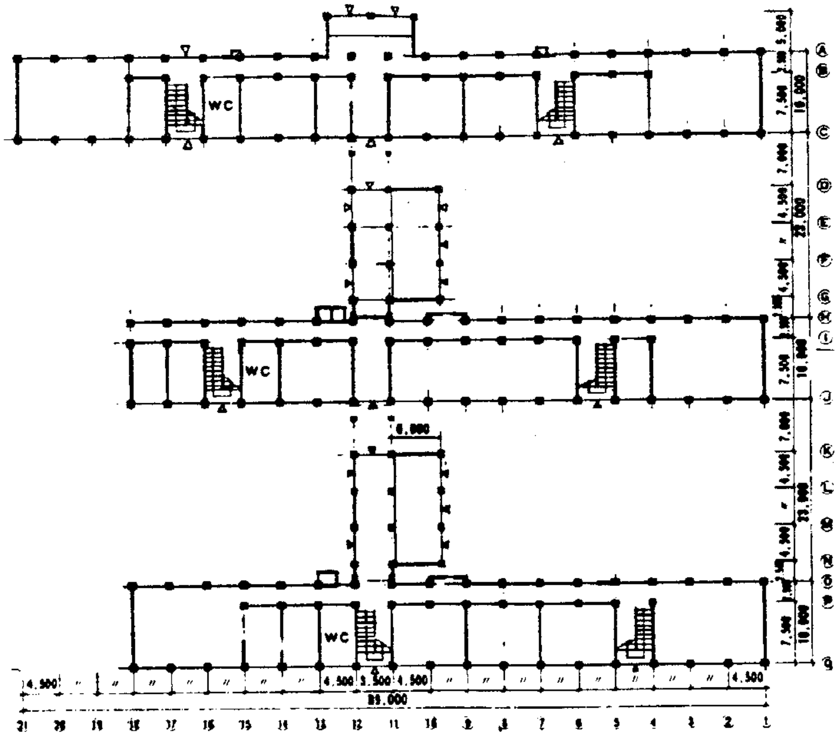


Photo 18. Izumi High School C wing
North Front

Photo 19. Izumi High School
Shearing Destruction of Columns

Shokan Nishi Elementary School in the city of Izumi located approximately 400m to the northeast of Izumi High School is a B type four story building, and incurred sizable damage--shearing destruction of some columns andcracks on the joints of lower sectional walls and floor slabs (Photo 20), which distinctively suggests that the earthquake input in the general area of Izumi High School was larger than other areas. Also, the Shokan Junior High School located approximately 400m to the east north east of the elementary school suffered only slight damage. Figure 10 indicates a plane view of Shokan Nishi Elementary School. Shokan Junior High School also has practically the same plan except that it does not have the sourth side balcony. The span is basically 1 span and haunches are installed on the beam. Cross-beam class rooms and corridors are divided by block walls. Tow staircases, one for outdoor footwear and the other for indoor footwear, are installed side by side on the north side. Each floor has footwear shelves. The building is characterized by installation of a considerable volume of cross-beam direction walls which utilize staircases and bathrooms.

Figure 10. Shokan Nishi Elementary School Plane View

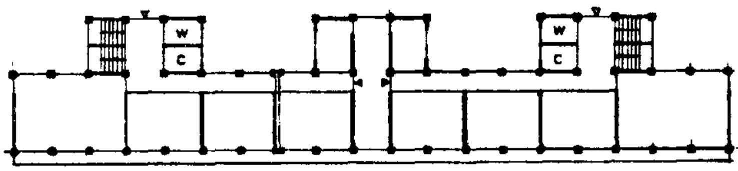
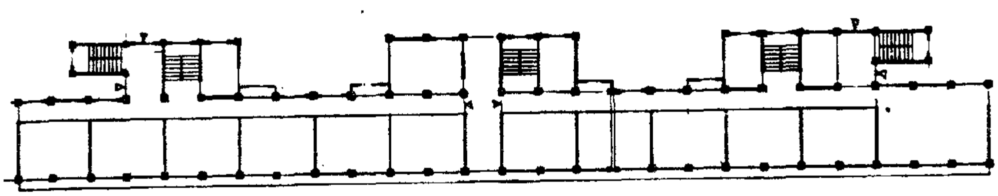


Photo 20. Shokan Nishi Elementary School Damage

In Izumi, Nankodai Elementary School suffered minor damage. Characteristics of the structure plan of this school also indicates, as seen in Figure 11, an installation of cross-beam direction walls by dividing staircases for indoor and outdoor footwears and additionally an installation of wing walls for the north side rahmen. From the point of earthquake resistance, column shaped seismic walls are much more advantageous than wing walls, but the intent of the design can be appreciated. The damage to this school appeared primarily as cracked external walls of upper structures due to the unequal settlement.

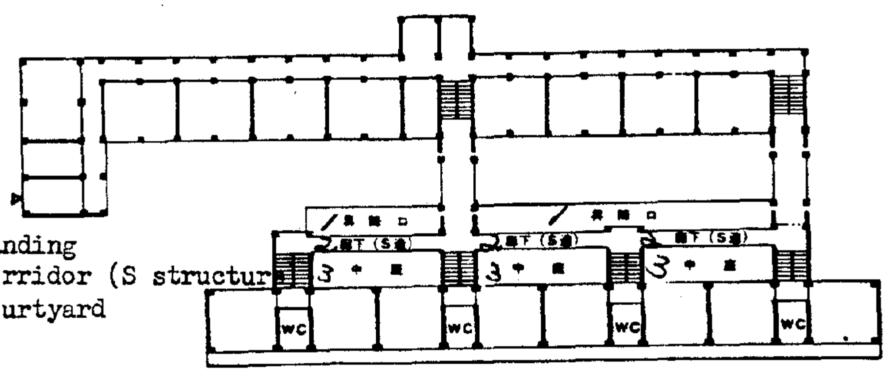
Figure 11. Nankodai Elementary School plane View



Many plane plans similar to those described above are adopted in public elementary and junior high schools in the city of Sendai. First, explaining the special ones, Kimachi Dori Elementary School and the Second Junior High School have a 3m cross-beam span, C type building constructed long time ago, probably around 1955. Both are a three story buildings and characterized with a look of multiple windows placing window sashes outside the columns on the second and third floors. Among school buildings of the A type, 1 classroom and 1 span in the cross-beam direction, there are the five story Higashi Nibancho Elementary School and the three story Nakada Junior High School. Both are somewhat specially designed, but did not suffer any damage.

The outstanding buildings among those built comparatively earlier are what is generically called a battery type school building without side corridors. There are three styles in this type. The first one is seen in Dainohara Junior High School which incurred minor damage, and Miyagino Junior High School which drew attention because it did not incur damage although it is adjacent to the south of Tonan High School. Figure 12 shows a plane view of Miyagino Junior High School.

Figure 12. Miyagino Junior High SchoolPlane View



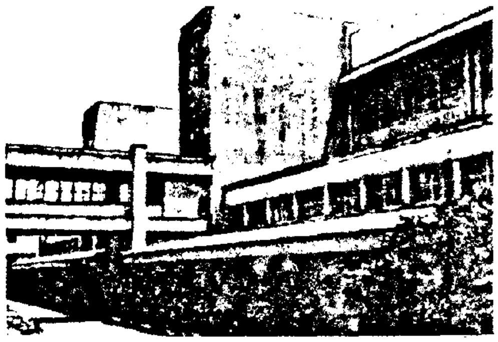
Key-1. landing
2. corridor (S structur
3. courtyard

The south wing (constructed in 1951) is a battery type three story building whereas the north wing (cOnstructed in 1966) is a regular B type four story building. The south wing is shown in Photos 21 and 22. Dainohara High School is almost the same as this, and received shearing cracks on the columns of the south front of the south wing.

Miyagino Junior High School South Wing

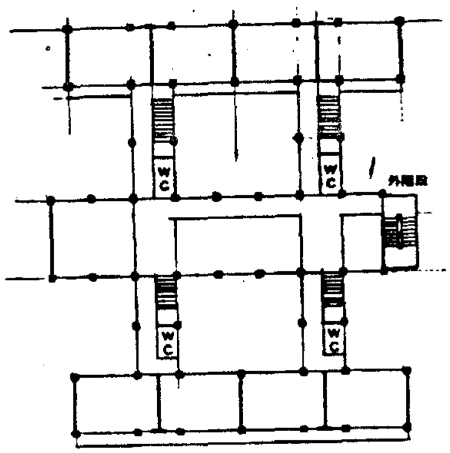
Photo 21. South Front

Photo 22. North Front



The second one is the style adopted only by elementary schools, for instance, Toricho Elementary Schho, Shichinosato Elementary School, Takasago Elementary School, Kuroshiro Elementary School, etc. Representing them all, a plane view of Takasago Elementary School is shown in Figure 13. The central building has corridors from which connecting corridors extend to the north and the south to two sets of classrooms. This plan is seen im many of buildings built after 1963. Photo 23 shows the south front of the south wing of Takasago Elementary School, and Photo 24 shows the east front of Shichinosato Elementary School. These buildings are commonly characterized with landings installed on each floor and outdoor staircases with elaborate design decorated with school emblem in the center wing.

Figure 13. Takasago Elementary School Plane View



key-1. outside staircase

Photo 23. Takasago Elementary School South Wing South Front

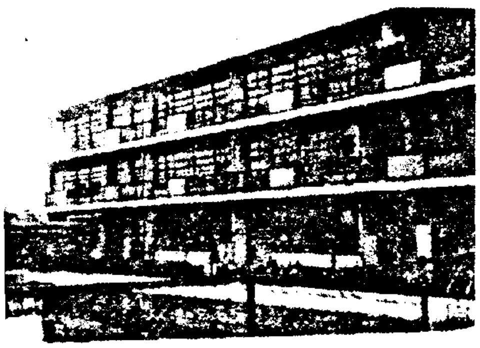
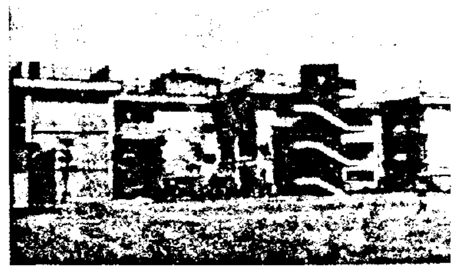


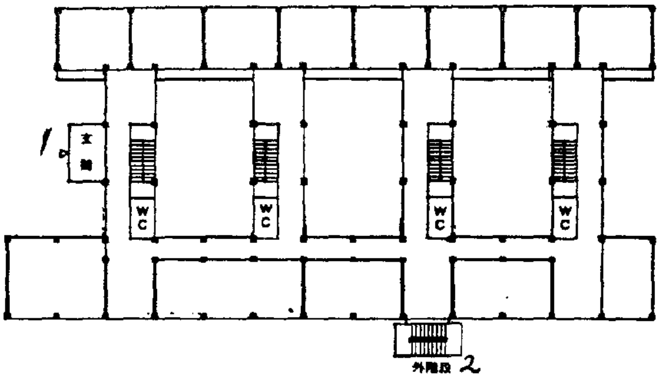
Photo 24. Shichinosato Elementary School East Front



The third style is also adopted only by elementary schools, and can be seen in Higashi Rokubancho Elementary School, Uesugi Yamadori Elementary School, Haramachi Elementary School, Nagamachi Elementary Schoo, Shinden Elementary School, Asahigaoka Elementary School, Tomizuka Elementary School and Nakayama Elementary School. They were designed in 1967 and built from that year to 1976. Representing them, Nakayama Elememtary School is shown in Figure 14 and Photos 25 and 26. The south wing has corridors, and outside staircases with an elaborate design. Three or four connecting corridors that extend from these staircases lead to the classrooms in the north wing. The above three styles of battery type buildings suffered no damage or minor damage by the earthquake of this time except for Daihara Junior High School.

The cross-beam direction was virtually pure rahmen, and it remains to be investigated in the future as to the level of the earthquake resistivity.

Figure 14. Nakayama Elementary School Plane View



key-1. entrance hall
2. outdoor stairs

Nakayama Elementary School

Photo 25. South Wing South Front

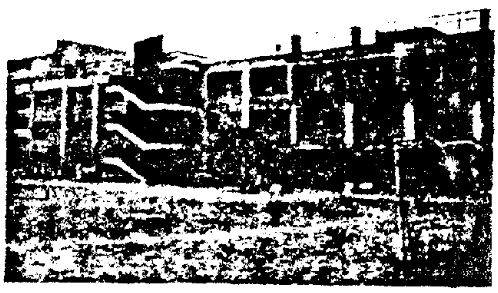
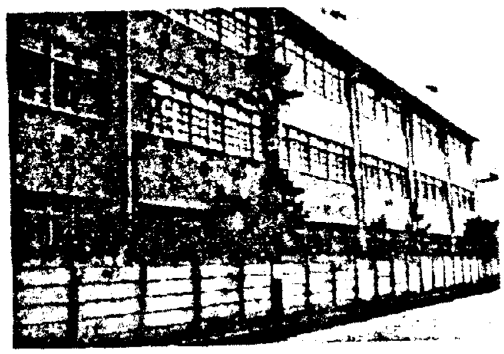


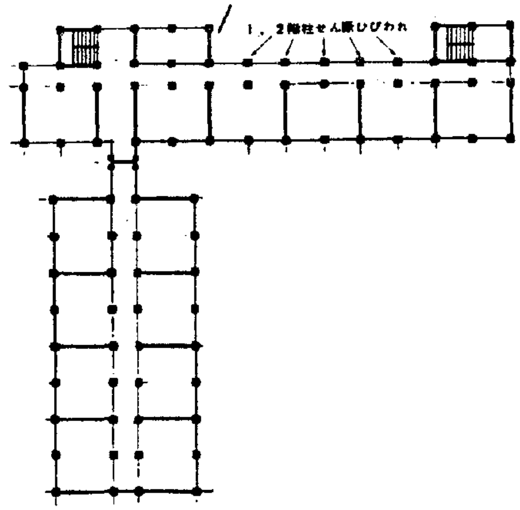
Photo 26. North Wing North Front



After 1966, "B" type school buildings have become the main stream. Elementary schools built during the initial "B" period, for instance, the Minami Zimokucho Elementary School and Aramachi Elementary School, have outside staircases decorated with the school emblem just as seen in battery type buildings. Junior high schools of the same initial period, for instance, Hakken Junior High School, Nakamachi Junior High School, Takasago Junior High School and Kitasendai Junior High School, were concurrently built using the middle corridor type. The common characteristic is the absence of the south front veranda which is seen in the standard type to be discussed next. As one example, Nagamachi Junior High School which incurred shearing destruction to some columns will be shown in Figure 15 and Photos 27 and 28.

Figure 15. Nagamachi Junior High School Plane View

key-1. column shearing cracks on first and second floors



Nagamachi Junior High School

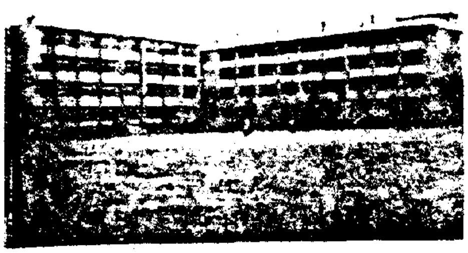


Photo 27. Southeast Front

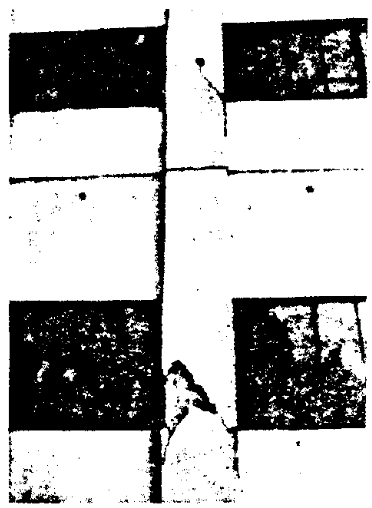
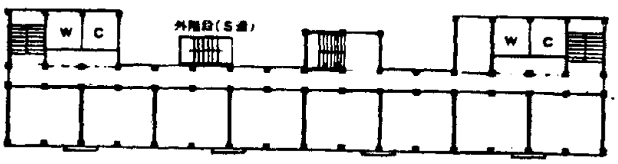


Photo 28. Shearing destruction of North Side Column

After 1969, school building types have been settled in accordance with the standard designs of the Sendai city government, which are currently in effect. At present, 60% of the RC buildings of elementary schools, equivalent to approximately 30 schools, are of this type, starting with Tachimachi Elementary School, Katahira

Elementary School and Renbokoji Elementary School. Also, 50% of existing RC junior high school buildings, equivalent to 10 schools, are of this type, for instance, Uesugiyama Junior High School and Gojo High School. As a typical example, Miyagino Elementary School is shown in Figure 16 and Photos 29 and 30. From the plane view, it is B type and made up of three rahmen structures in the cross-beam direction with staircase rooms and bathrooms to the north of corridors. However, some schools added classrooms on the north side and the corridors appear more or less like middle corridors. A special intent to build more walls in the cross-beam direction is not detectible. They are usually four stories but sometimes three stories. As a design, balconies are not provided on the south front, but, in stead, small verandas (size large enough to place flower pots) are installed at a location that connects the adjacent two classrooms. As can be seen in the photos, these verandas were installed because of the need of the design, and cannot be used as an escape route to the adjacent classroom during fires. One more common characteristics is the installation of wing walls for columns at the landing on the first floor.

Figure 16. Miyagino Elementary School Plane View



key-1. outside staircases (special structure)

Photo 29. South Front

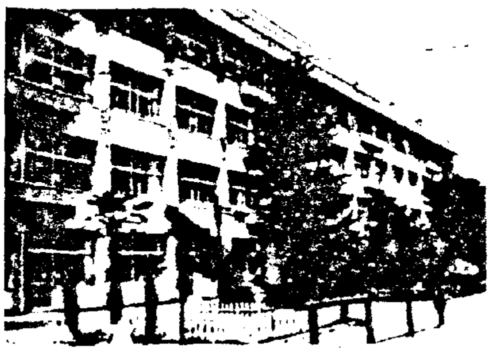
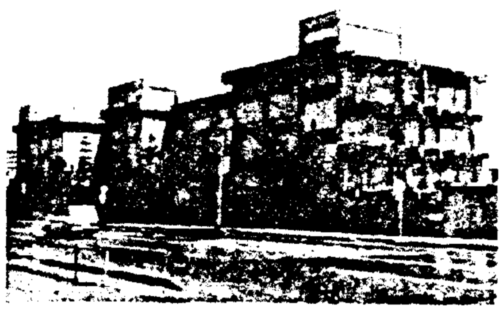


Photo 30. North Front



Miyagino Elementary School

The majority of schools of this type suffered none or only slight damage. Higashi Sendai Elementary School, however, suffered shearing destruction to some short columns in the corridors on the north side as seen in Photo 31. Also, Atago Junior High School suffered destruction of columns and walls on the north front as shown in Photo 32. It seems an urgent task to investigate promptly the level of the earthquake resistivity of the standard designs of Sendai in the view of the fact that there are so many schools deal with.

Photo 31. Higashi Sendai High School
Shearing Destruction of Column

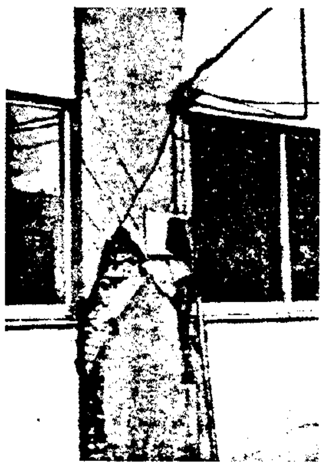


Photo 32. Atago Junior High School
Shearing Destruction of Wall

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PART 2 SEISMIC DIAGNOSIS OF REINFORCED CONCRETE BUILDINGS

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1. FORWARD

In recent years, a growing tendency has been felt in that seismic diagnosis of existing buildings is performed and seismic reinforcement is implemented as necessary and required. The fact that this publication is issued as a special edition with this subkect as a theme is also one of the manifestations of the trend in sight.

The main object of this paper is medium and low rise reinforced concrete buildings. The former half will deal with the necessity of seismic diagnosis and the relationship between seismic diagnosis and seismic design, and the latter half explains the concrete procedures of seismic diagnosis.

2. NECESSITY OF SEISMIC DIAGNOSIS

The terminology "seismic diagnosis" itself is not especially new in Japan. It is our old practice to diagnose aged buildings with questionable yield strength, and in some cases, to repair or reinforce them. However, the concept of "Seismic Diagnosis" which has been recently brought up or "Seismic Diagnosis" which is the subject of this paper is slightly different from the old practice.

"Seismic Diagnosis" of the new concept is designed to diagnose the seismic efficiency of buildings regardless of the age of the buildings and to make preparations for future earthquakes. Speaking in terms of medical science, while conventional seismic diagnosis examines only the patients with symptoms, the new seismic diagnosis is equivalent to the physical examination which also checks healthy people without any symptoms.

The following can be cited as reasons that call for "seismic diagnosis" which renders services similar to preventive medicine.

The progress of earthquake proof engineering and earthquake engineering yearly imporved seismic design methods. Also, seismic safety of newly constructed buildings tends to increase gradually, although with some differences in the level of safety. On the other hand, existing buildings designed and built in accordance with outdated seismic design technology, ie, the buildings left behind in the progress of seismic designs, may be naturally questioned concerning the integrity of their safety, when they are reviewed by the yardstick of new knowledge and information. This is one of the reasons that the importance of so called seismic diagnosis has lately come to be recognized.

In addition, although this is what is pointed as an introspection for the currently commonly used seismic designs, the conventional seismic designs seem to tend, in many cases, to complete the work by mechanically checking the stress of each part of the buildings against the design earthquake force prescribed in Building Standard Law Enforcement Ordinance, using variousstructure calculation standards of the Architectural Institute of Japan. This procedure lacks the ability to give proper evaluation to the seismic efficiency which si reserved in the buildings. The shortage of the conventional design gave motivation for the recent outcome of various proposals relating to seismic diagnosis, and is the reason for the presentation of new seismic design methods which contain evaluation of seismic efficiency.

the necessity of this seismic diagnosis which can be considered equivalent to a physical examination, probably began to be earnestly recognized after the 1968 Tokachi Offshore Earthquake which brought more than anticipated damages to reinforced concrete buildings in Hokkaido and Aomori prefecture. As seen in a number of investigation reports, the damage by this earthquakewas minor. For instance, in terms of the damage ratio itself, it was some 10% even in the city of Hachinohe which had a comparatively high damage ratio. The majority of the reinforced concrete buildings were undamaged. Society and the architectural world made a great point of publicizing the damge which happened to occur to reinforced concrete structures which had been once believed to be the safest against earthquakes.

Although the after-the-fact activities were not limited to this earthquake, studies on the cause of damage and investigation and research related to the earthquake resistivity of reinforced concrete buildings were promoted, and many results were presented (Literatures 1 and 2). The direct cause of damage to individual buildings varied, but many lessons were learned from the 1968 Tokachi Offshore Earthquake including the results of investigation of unharmed buildings. Among them, the most important lesson regarding the ideology of seismic design was, according to the opinion of the author of this paper, the following as pointed out in Literature 3.

"Buildings, although equally designed in accordance with the rules in the Building Standard Law, the Engorcement Ordinance of this law, and the "Reinforced Concrete Structure Calculation Standard" of Architectural Institute of Japan, have various seismic efficiencies, and some of the buildings suffer damage from an earthquake of an intensity which can be normally anticipated."

All the actions taken after this earthquake are attributable to the lessen learned from this earthquake, for instance, the amendamant of the Building Standard Law Enforcement Ordinance (1970), revision of the "Reinforced Concrete Structure Calculation Standard" of the Architectural Institute of Japan, proposals of new seismic design methods (Literature 4-10) and proposals of seismic diagnostic methods for existing buildings which are the main theme of this paper. Also, the common objective of the various proposals relating to the seismic design methods and seismic diagnostic methods can be said to be "accurate evaluation of the seismic efficiency of buildings". The above is a movement to advocate accurate evaluation of the seismic efficiency of a building to be constructed when it is being designed in order to clarify probable behaviors of a building under an earthquake. It is also a movement to advocate reinforcement of existing buildings before earthquakes hit them if found necessary after the evaluation of seismic efficiency.

The necessity of new conceptual "seismic diagnosis" was created from these backgrounds, and is naturally unseparable from "seismic design".

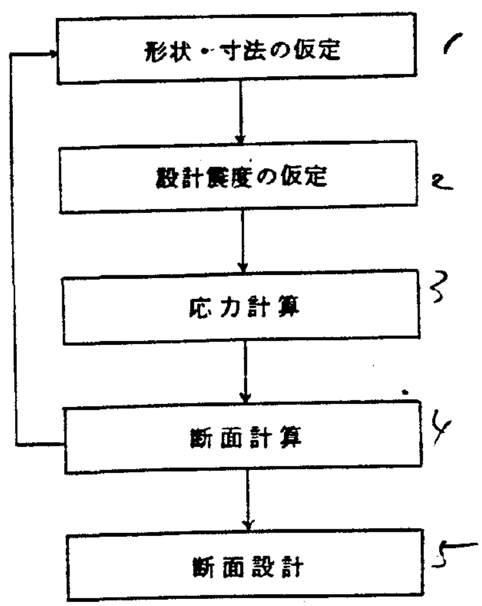
3. SEISMIC EFFICIENCY OF EXISTING BUILDINGS AND SEISMIC DESIGN METHOD

In this section, I would like to discuss a little more about the diversity of seismic efficiencies of existing reinforced concrete buildings and how they relate to seismic design methods.

First of all, let us review the seismic designs at the time of the Tokachi Offshore Earthquake. Figure 1 is the flow sheet of the seismic design method of that time. As indicated in the figure, the procedure observed for designing consisted of stress calculations for each part when a horizontal force equivalent to the design seismic intensity worked upon a building of an assumed shape and dimensions; determination of the quality of reincorcing steel arrangement in accordance with the stress; and investigation whether the unit stress of each part is within the allowable unit stress limit. Standard design seismic intensity was 0.2, but this can be lowered according to regional factors which take into consideration the seismicity. Later, improvement were made by revision of the Building Standard Law and "Reinforced Concrete Calculation Standard" of the Architectural Institute of Japan. However, wise readers will notice that the flow of the outline indicated in Figure 1 is the same as the current commonly used design method. In this paper, this flow of the design will be termed as the common design method, but the seismic design is actually made from so called static seismic intensity method. The static seismic intensity method was adopted in Japan after the 1923 Kanto Earthquake, and basically design methods in compliance with this method are also used throughout the world.

Figure 1 Flow of Common Seismic Design

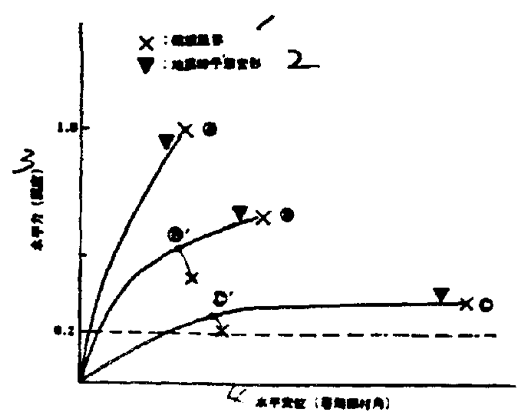
Key- 1. assumption of shape and dimensions
2. assumption of design seismic intensity
3. stress calculation
4. section calculation
5. section design



Now, the static seismic intensity method is an excellent engineering method which simply substitutes the effect of earthquakes on buildings by horizontal force and expresses it in the form of design seismic intensity. If we depend only to this method, buildings will be constructed without knowing the seismic efficiency of the designed buildings, and erroneous use of this method will allow an earthquake to affect the blind points of the buildings.

Figure 2 is the schematic display of the relationship between the horizontal force when a building designed according to the common design method was destroyed by a lateral force and its horizontal displacement.

Figure 2. Properties of RC Buildings



Key-1. destruction limit
2. anticipated deformation during earthquake
3. horizontal force (seismic intensity)
4. horizontal displacement (interlayer element angle)

The axis of the ordinates is the horizontal intensity obtained by dividing the horizontal force by the weight of a building, and the axis of the abscissa is an interlayer element angle expressed, for instance, by dividing horizontal relative displacement of the floor on the first story and the floor on the second story by the height of the story. This figure indicates properties of medium and low rise reinforced concrete buildings, and is plotted using a graph contained in Literature 3. Now, the seismic efficiency of existing reinforced concrete buildings will be explained using this figure.

First, paying attention to the axis of theordinates, the horizontal strength varies even if buildings are designed similarly using a design seismic intensity of 0.2. Some buildings with many walls (A in the figure) have a strength exceeding 1.0 in terms of seismic intensity, while generally pure rahmen buildings (C in the figure) are low in strength, only slightly exceeding the design seismic instensity. This phenomenon has a close relation to a part of the design flow of the common design method indicated in Figure 1 below the stress calculation. In the common design method, the safety factor relative to buildings with many walls tend to come out comparatively large, while the safety factor relative to the pure rahmen structure does not come out very high. Many reasons can be assumed for this, but the following can be plainly said.

(1) Buildings with Many Walls: Sometimes a designer intended to install them from the point of the structure plan, but in many cases, walls are provided due to the actual needs from the aspect of the plane plan in spite of the fact that a structural calculation is not required as long as the plan is in compliance with the common design method, that is, the building can meet the condition of 0.2 design seismic invensity even when the number of the walls may be reduced. Walls between classrooms in school buildings are good example of these. Also, when attempting to maintain the same numberof walls but to reduce the thickness of walls, it is often found that thicker walls cannot be scaled down too much due to the problems related to the construction workability and noise insulating efficiency.

(2) Pure Rahmen Structure: It is possible to design a building to meet the allowable minimum design seismic intensity by using the common design method. In this case, there are few other structures that give superfluous strength to the building, and consequently the final strength will not be higher than the design seismic intensity.

Next, letus talk about the axis of the abscissas. Destruction of buildings will be dominated by the destruction of the walls in buildings with many walls. Generally, destruction of walls manifest in a brittle destruction mode called a shearing destruction except for special walls, and the displacement at this time will not be large. Also, the destruction of a rahmen structure may take a form of destruction with very potent deformation ability usually called the bending destruction mode as indicated by the symbol C in the figure, or of shearing destruction with less deformation ability just as with walls as indicated by the symbol C' in the figure.

According to the common design method, the lower limit of the strength is numerically established to an extent by the design seismic intensity while the deformation ability cannot be examined numerically but is given indirect considerations by means of so called structure regulations, for instance, setting of the minimum quantity of hoops of columns, setting of the upper limit value of the axial directional force of the columns or providing means prevent shearing destruction. The revision of the related regulations after the 1968 Tokachi Offshore Earthquake especially emphasizes structure regulations that ensure the deformation ability. Compared to the past, buildings inferior in deformation ability are decreasing, but we have not yet reached a stage where we can treat this quantitatively.

Well then, what kind of properties do these reinforced concrete buildings with various strengths and deformation abilities demonstrate when they are exposed to earthquakes? Mark ▼ indicates anticipated displacement of buildings during earthquakes. Looking at this simply from the point of the previously described strength of buildings, the pure rahmen buildings symboled by C seem inferior in seismic efficiency compared to the other buildings. However, as shown in the figure, if the destruction limit deformation (x) of buildings is larger than the anticipated deformation during earthquakes, the building will be stable. Rather, buildings symboled by B' higher in strength and lower in destruction limit displacement than C will be low in seismic efficiency.

The anticipated deformation of buildings during earthquakes are complex and diversified by the properties of earthquakes, properties of buildings and properties of the ground on which the buildings stand. Following are the simplified descriptions of the characteristics.

(1)Buildings with many seismic walls have small anticipated deformation during earthquakes but encounter considerably large input force due to the high rigidity. The size of this force can be possibly 2-3 times more than the seismic intensity of the ground surface. Since 0.3 level of seismic intensity is fully predictable, the final strength of these types of buildings shall be preferably 0.6-0.9.

(2) The anticipated displacement of buildings the strength of which cannot be raised, such as pure rahmen structures, shall be expected to be much larger than the buildings with many walls. Therefore, they must be designed with appreciably large destruction limit displacement.

As described above, there is a possibility of producing buildings very rich in variety of strength and deformation ability when reinforced concrete buildings are designed in accordance with the common design method. Also, according to this design method, a certain degree of seismic efficiency is guaranteed to be secured, but the level of the efficiency is not clarified.

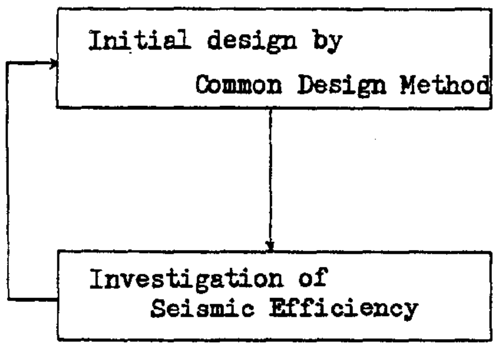
The seismic diagnosis accesses the point marked x, ie, the level of strength and the deformation ability, and the point marked ▼, ie, the level of anticipated deformation during earthquakes, in Figure 2, and finds out the seismic efficiency of buildings. As will be described below, if a seismic efficiency evaluation process is incorporated into the stage of the seismic design, the author of this paper believes that the task of seismic diagnosis for existing buildings will no longer be necessary in addition to the seismic diagnosis necessitated by the aging of the buildings and the re-checking of the seismic efficiency due to the progress of sciences (The author terms this as the seismic diagnosis due to the aging of seismic design method). Also, the author of this paper wishes a speedy establishment of a seismic design method that will not call for an seismic diagnosis.

4. SEISMIC DESIGN AND SEISMIC DIAGNOSIS

As has been described so far, numerous seismic designs and seismic diagnostic methods proposed after the 1968 Tokachi Offshore Earthquake are based upon the same idea in the view that the seismic efficiency of buildings is evaluated.

For example, the first proposal (Literature 8) for the earthquake load submitted by the Earthquak Load Sub-Committee of the Architectural Institute of Japan is built on the thought to provide buildings with a deformation ability larger than the roughly calculated anticipated displacment (earthquake response displacement) relative to the maximum seismic motions (maximum acceleration almost 300 gal class seismic motions) of buildings estimated from the initial design prepared in accordance with the currently used design method. As shown in Figure 3, it is an idea to add a procedure "Investigation of seismic efficiency" to the common design method shown in Figure 1. This method uses a similar procedure to the design method used for designing super high rise buildings, that is, the procedure where the initial design is created in accordance with, for instance, "High Rise Architectural Technology Guidelines" of the Architectural Institute of Japan, analyzed for earthquake response and modified if necessary and required, although there is a difference in technique. If the buildings produced by the initial design are imagined as the existing buildings on the drawings, the part for the seismic efficiency investigation can be called seismic diagnosis.

Figure 3. Flow of New Seismic Design Method (example)



Also, the method in Literature 5, co-proposed by the author of this paper, is for low rise reinforced concrete buildings. Paying attention to the possibility of requiring frequent modification to the seismic efficiency investigation results if the initial design is prepared in accordance with the currently enforced design seismic intensity as shown in Figure 3, this method changes the design seismic intensity in correspondence to the seismic efficiency of the buildings to be designed in the initial design stage, and also proposes to investigate the seismic efficiency of the buildings obtained as the result of the former step, as shown in Table 1. The "New Seismic Design Method (proposal)" introduced recently by theBuilding Research Institute of the Ministry of Construction can be regarded to have the same idea as the former two.

Table 1. Design Objective and Design Principle for 3 story RC Buildings (from Literature 3)

	design objective	design principle (for ground floor)	
		concerning shearing strength	bending strength
pure rahmen structures *	2 times the yield displacement relative to input seismic intensity 0.3 4 times the yield displacement relative to input seismic intensity 0.45	column average shearing stress 12 kg/cm^2 relative to design seismic intensity 1.0 (ku = 0.85 secured)	Calculated by the common design method relative to design seismic intensity 0.3 (ky = 0.45 secured)
structures with many walls *	Shearing cracks of walls relative to input seismic intensity 0.3 Walls protected from destruction relative to input seismic intensity 0.45	Average shearing stress of walls 10 kg/cm^2 relative to design seismic intensity 1.0 (ku ≃ 0.9 secured)	Calculated by the common design method relative to design seismic intensity 0.6 (ky ≃ 0.9 secured)

(note) ku: shearing final strength, ky: bending yield strength

As above, proposals that recommend an inclusion of the procedure to investigate seismic efficiency in seismic design methods have started to come forward. It is therefore possible to perform seismic diagnosis of existing buildings using the part for the seismic efficiency investigation from these proposals. However,

(1) It is actually difficult to investigate the seismic efficiency of all of the existing buildings exerting labor equal in level to that extent for designing new buildings. Therefore, it is necessary to have simplified techniques which employ simpler calculations than the design method.

(2) Many of the proposed seismic design methods are for comparatively simple structures while many of the existing buildings are generally complicated. In many cases, it is not practical to use the proposals submitted as seismic design methods in their original form for seismic diagnoses.

Table 2. Difference Between Seismic Design and Diagnosis

	Seismic Design	Seismic Diagnosis
object	new buildings	existing buildings
design books	newly prepared	non existant in many cases on site investigation required
materials	can be appointed	estimation or on ssite investigation possible quality deterioration by aging
position of those in charge	Designers can aim to create their ideal buildings	Often those in charge of diagnosis must handle buildings designed quite different from their design idea.
Measures to be taken when earthquake resistivity is not sufficient	change designs affect the structure expenses	Require new budget for reinforcement or demolition

5. EXAMPLES OF SEISMIC DIAGNOSIS

Literatures 11-16 are methods recently proposed for medium and low rise existing buildings of reinforced concrete. Literature 16 will be commented upon in detail in the following section. Here, Literatures 11, 12 and 13 will be briefly introduced.

Literature 11 was developed by the Building Research Institute of the Ministry of Construction. Literature 12 is the method which improved it. The outline of the two methods combined in Table 3 primarily focus on the numerical value called S_E' which expresses the axis of the ordinates in Figure 2, that is, the final strength of buildings, and attempts to judge the earthquake resistivity of buildings.

Table 3 Diagnositc Method in Literature 11

(a) Items for Judgement of Earthquake Resistivity

A Judgement Concerning Grounds

Landsliding, Fluidization of Ground

B Judgement Concerning Upper Structures
(Comparison of S_B and qBO described below)

1. Building Strength Coefficient

$$S_B = \text{correction coefficient (a)} \times \frac{\text{building horizontal strength(Q)}}{\text{building weight (W)}}$$

2. Required Building Strength Coefficient

$$qBO = 1.0 \times \underset{(1.0-0.5)}{\text{importance factor}} \times \underset{(1.0-0.8)}{\text{locality factor}} \times \underset{(1.0-0.5)}{\text{interaction factor}}$$

whereas, $1.0 \geqq qBO \geqq 0.5$

3. Correction Coefficient of Building Strength

$$d = \frac{Ri}{Rc \times Re} \quad \text{whereas} \quad 1.0 \geqq a \geqq 0.5$$

Ri: Presence of basement. yes: 1.2, no: 1.0

Rc: Degree of Construction Work. good: 1.0, inferior: 1.5

Re: Adequacy of rigidity distribution. good: 1.0, inferior: 1.5

(b) Contents of Judgement

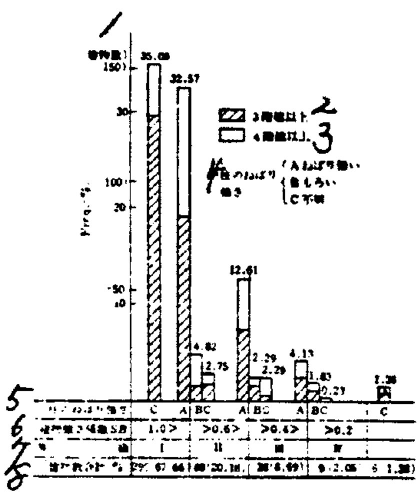
	contents	conditions to be met	conclusion
first judgement	from volume of walls: Rw	$Rw \geqq 2N$ and $5 cm/m^2$ (N 6) $Rw \geqq N + 6$ (N 7)	fully strong
second judgement	from summary calculation of column and wall strength ($qBO = 1$)	$S_B = qBO$ whereas, $Q = \sum Qc + \sum Qw$ $= (5-10) \times \sum Ac + (10-30) \sum Aw$	fully strong

third judgement	From precision calculation of column and wall strength ($qBO \fallingdotseq 1$)	a) $S_B' \geqq qBO$ b) $S_B' \geqq 0.6\ qBO$ and column shearing strength larger than column bending strength c) $S_B' \geqq 0.45qBO$ and reinforcement ratio larger than required value 1 d) $S_B' \geqq 0.3\ qBO$ and reinforcement ratio larger than required value 2	fully strong considerably strong slightly ductile strong and ductile slightly strong considerably ductile
fourth judgement	Repetition of 2nd and 3rd judgements from concrete strength and proper period of buildings and grounds	Repetition of 2nd and 3rd judgements	Same as 2nd and 3rd judgements. If buildings fail the tests, scrutinize buildings wing by wing, and determine countermeasures.

If it is assumed that the deformation ability of buildings is large (In the combined method, a building with the bending strength lower than the shearing strength), the standard value of judgement is lowered so that the earthquake resisting ability of buildings with low strength but high deformation ability shown in Figure 2 can be evaluated. Examples of seismic diagnosis of existing buildings performed in accordance with this method are shown in Figures 4 and 5 (Literatures 17, 18). As described previously, these figures also reveal that there are considerable error of accuracy in the final strength of existing buildings.

The method in Literature 13 was jointly developed by the author of this paper and Professor Bresler of Berkeley campus, University of California as a link of a chain in "Earthquake Engineering Stressing Safety of School Buildings" which is part of the Japan/US Science Cooperation Work cosponsored by Japan Society for the Promotion of Science and National Science Foundation (NSF) of America from 1973-1975. This is a modified version of the method in Literature 5 which is developed as dynamic seismic design method for medium and low rise reinforced concrete buildings, and it is made to be used for seismic diagnosis. In the United States, seismic diagnosis of existing buildings is also considered important after the (Garrison) Law was established in the state concgress of California in 1968 for the re-examination of the seismic safety of aged school buildings and the 1971 San Fernando Earthquake that damaged buildings with seismic designs.

Figure 4 Seismic Safety Investigation Results of Public Buildings (Literature 16)



key-1. number of buildings
2. above 3 story
3. above 4 story
4. column ductility { A. ductile, B. brittle, C. unknown }
5. column ductility
6. strength coefficient of buildings
7. classification
8. total number of buildings (%)

In this method, earthquake response is roughly calculated according to the final strength and destruction mode of buildings, and calculation results were used to judge whether or not the buildings pass the standards expressed by matrixes in Table 4. It is equivalent to the 3rd judgement level of the method expressed in Literature 11 indicated in Table 3. Figure 6 shows examples of this method applied to damaged and undamaged buildings during the 1968 Tokachi Offshore Earthquake.

Figure 5. Seismic Diagnostic Examples of Private Buildings (from Literature 18)

key-1. seismic diagnosis of RC buildings
2. number of cases by class
3. number of cases by yield strength coefficient
4. class

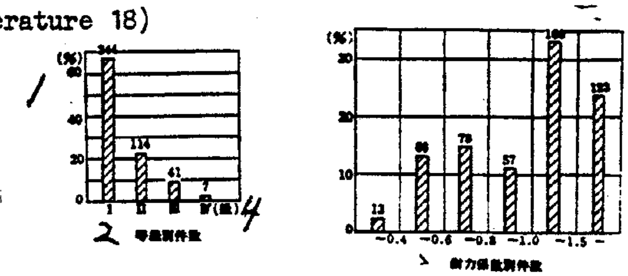


Table 4. Seismic Safety Judgement Standard Matrixes
(From Literature 13)

(a) General Standards

magnitude of seismic motions	strong earthquake	ruinous earthquake
level of damage	repairable	not collapsed

(2) First Screening Standards

destruction mode	ground motion 0.3g	ground motion 0.45g
bending mode (ductile)	below 2 plasticity ratio (μ)[1]	below 4 plasticity ratio (μ)
shearing mode (brittle)	shearing crack level	no shearing destruction [2)]
shearing/bending mode	shearing crack level	near yield point[3)]

1) plasticity ratio= macimum displacement /yield displacement

2) shearing deformation $\gtrsim$ 1/2 of final shearing deformation ($\gamma_{uit} = 4 \times 10^{-3}$ radian)

3) deformation equivalent to plasticity ratio of 2 of the bending type buildings

Figure 6. Seismic Diagnostic Examples (from Literature 13)

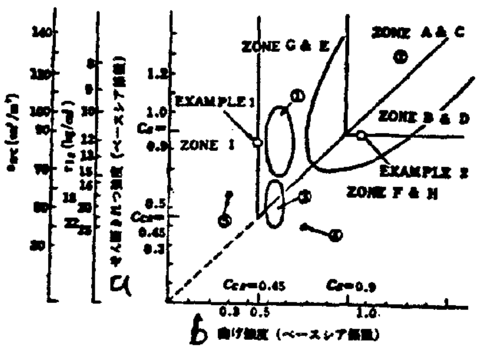
key-a. shearing crack strength
(base shear coefficient)

b. bending strength
(base shear coefficient)

1. C shaped school building, unharmed
2. buildings with many walls, unharmed
3. moderate damage
4. Hachinohe Specialized High School major damage
5. Hachinohe Library, major damage

Examples 1 and 2 are cases from Literature 5

Properties of buildings affected by the 1968 Takachi Offshore Earthquake



Safety Ranking

zone	type	ground motion 0.3g		ground motion 0.45g		ranking
		strength	deformation	strength	deformation	
A	bending	safe	safe	safe	safe	
B	shearing or shearing/bending	safe	safe	safe	safe	I
C	bending	safe	safe	unknown	safe	
D	shearing or shearing/bending	safe	safe	unknown	safe	
E	bending	unknown	safe	——	safe	II
F	shearing or shearing/bending	unknown	unknown	——	safe	III
G	bending	——	safe	——	unknown	
H	shearing or shearing/bending	——	——	——	unknown	IV
I	bending	——	——	——	unknown	

6. SEISMIC DIAGNOSTIC STANDARDS
(Proposal by Japan Special Building Safety Center)

In this section, the outline of "Seismic Diagnostic Standards for Existing Reinforced Concrete Buildings" will be introduced, which is a proposal submitted in March, 1977 by the Existing Building Seismic Diagnostic Standard Planning Committee (Chairman, Professor Kai Umemura, University of Tokyo) organized in 1976 within Japan Special Building Safety Center (Foundation) as a project of the Construction

Guidance Section, Housing Bureau of the Ministry of Construction. Incidentally, details of the standards are available at the center (Yamazaki Building, 1-2-1, Akasaka Minato-ku, Tokyo, TEL 03-586-2881), please refer to them. Also, the original plan of the standards was the results of efforts by the members of the Sub-Committee for Draft Preparation Work with the author of this paper as the chief investigator.

6.1 COMPOSITION

The standards were developed aiming to diagnose the earthquake resistance of 5-6 story and lower existing reinforced concrete buildings using a simple method which is still right on target, referring to various recent proposals relating to seismic diagnosis (Literatures 11-15). The standards were developed also for possible use in design stage of new buildings.

According to this diagnostic method, the structural seismic index I_S which indicates dynamic earthquake resistivity of framework, and seismic index of non-structural elements I_N which indicates safety primarily against damages to be incurred by the falling of finishing materials on the exterior of buildings, are cited as factors that express the seismic efficiency of buildings. Also, the diagnostic method is classified by the level of precision into first, second and thrid diagnoses. For instance, the strength of the framework is calculated from the concrete sectional area of vertical elements (columns and walls) in the first diagnostic method. In the second diagnostic method, the same strength is obtained from horizontal yield strength of vertical elements and the destruction mode. In the third diagnostic method, the destruction mode of the framework such as yield of beams, is also taken into account. In the following, mainly a procedure to obtain structural seismic index I_S in the first and second diagnoses will be introduced.

6.2 BASIC POLICY

The seismic efficiency of structure on each story and each direction of a building is expressed by Structural Seismic Index I_s indicated in Equation (1).

$$I_S = E_0 \times G \times S_D \times T \quad \cdots\cdots (1)$$

Eo is a basic index that expresses reserve seismic efficiency as will be explained in the next section, and G, S_D and T are indexes that modify Eo index using 1.0 as a standard. That is,

G: Ground motion index that expresses the scale of ground motion which affects building foundations. The standard value is designated as 1.0. This value is characteristically lowered when it is possible that the ground motion is locally amplified. This value is fixed at 1.0 at the present stage.

S_D: Shape index or structure plan index . It is used to take into consideration, by the check list system, the effect on seismic efficiency of factors difficult to be incorporated in a summary calculation such as plane and elevation shapes of buildings and rigidity distribution. The standard is designated as 1.0, and the numerical value is lowered when structures become complicated. However, the standard for buildings with basements is set at 1.2.

T: Time index. When the seismic efficiency of buildings deteriorate with time, the value drops lower than 1.0.

6.3 REVERSE EFFICIENCY BASIC INDEX (Eo)

1) Outline

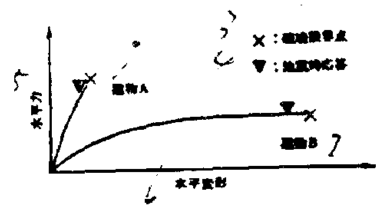
As can be diciphered by reading the Kanji characters, it is the value which is used as a basis to express the efficiency reserved in buildings. It was previously mentioned that the seismic efficiency of existing RC buildings was broad in width. Likewise, the width is, as also mentioned before, created due to the diversity of reserve strength and ductility of buildings. The Eo index is a yardstick to evaluate the seismic efficiency reserved in buildings, using strength and ductility reserved in buildings. This will be explained using a simple chart.

Figure 7 is a conceptual chart which indicates the relationship between horizontal force and deformation when the horizontal force is applied to RC buildings.

Figure 7. Relationship Between Horizontal Force and Horizontal Deformation of RC Buildings

key-1. building A

2. building B
3. x: destruction limit point
4. ▼: response during an earthquake
5. horizontal force
6. horizontal deformation



There are, however, all kinds of properties in RC buildings. The two types of buildings, Building A amd Building B shown in Figure 7, are examples of buildings with relatively simple properties. Building A is a RC building with many walls, a considerably high strength and a low ductility. Building B is a rahmen structure with a small number of walls, a not-so-high strength and substantial ductility. When these buildings are exposed to an earthquake, buildings will be safe if the maximum deformation is contained within the destruction limit point marked ▼ in the chart. If not, the buildings will be damaged by the earthquake. According to various research to date, buildings with many walls need a lot of strength for them to be able to meet the above conditions since they are low in ductility. Also, it is understood that the rahmen structures must be ductile since they do not have a very high strength.

The reserve efficiency basic index (Eo index herebelow) is established in order to provide a common yardstick to measure the seismic efficiency of buildings with many walls and rahmen structures in consideration of these different properties. In short, it can be expressed by,

Eo = (yardstick for strength) x (yardstick for ductility)

These strength and ductility yardsticks are termed Strength Index (C) and Firmness Index (F) in this diagnostic standard. Three types of summary calculation from the simple calculation used in the first diagnostic method to the somewhat detailed calculation required in the third diagnostic method are furnished.

The above examples cited are very simple, but real buildings are very complex, and the Eo index calculation will not be that easy.

Let us look at a slightly more complex example.

In Figure 8, conditions of a rahmen structured building with a small number of walls under a horizontal force are illustrated. Specifically, when a horizontal force is gradually applied, walls break at point (a). This building is not completely destroyed at this point. Once the horizontal resistance is lowered, the remaining rahmen elements start to resist the horizontal force if the deformation is allowed to continue. The building will not collapse to Rahmen Destruction Point (b).

According to the seismic diagnostic standards, the Eo index of a building such as this can be obtained using the following approach.

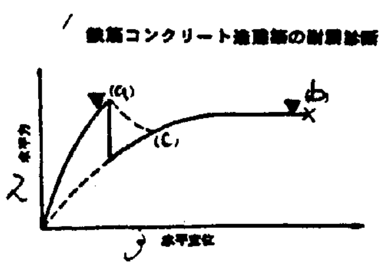
First, assuming that the building is supported only by walls, the Eo index will be obtained disregarding the rahmen. This is designated as E1. On the contrary, now assuming that the building is supported only by the rahmen, the Eo index is obtained and designated as E2. Next, the square root of the sum of the squares of these values is designated as Eo of the building.

specifically,

$$E_0 = \sqrt{E_1^2 + E_2^2}$$

The Eo index thus obtained is naturally is smaller than (E1+E2). Precisely, it is is evaluated lower than the simple sum of the seismic efficiency when the building is only supported by the walls and the seismic efficiency when the building is only supported by the rahmen.

Figure 8. Properties of Buildings with Rahmens and Walls



key-1. seismic diagnosis of RC buildings
2. horizontal force
3. horizontal displacement

The response of the buildings mixed with walls and rahmens must be very complicated. This approach was adopted in view of the fact that it was sometimes risky, according to the results of research to date, to consider the simple sum of the efficiency of buildings with walls only and the efficiency of buildings with rahmens only as the efficiency of the entire building.

Now, the relationship between the Eo index of the seismic diagnostic standards and seismic design methods or seismic diagnostic methods described in Section 5 will be explained in brief.

Normally, the earthquake response volume of buildings is obtained through the following procedure.

(1) Estimation of the size and properties of input seismic motions.
(2) Estimation of the restoring force characteristics of buildings, ie, strength, deformation ability and hysteretic characteristics (proper period and weight are included).
(3) Estimation of earthquake response volume, ie, response acceleration, response rate and response displacement.

The seismic desing of Literature 8 and the seismic design for super high rise buildings described in Section 4, and the seismic diagnosis of Literature 13 described in Section 5, are performed through this procedure, although there may be some differences in the precision of calculations.

To simplify the discussion, the above described procedure is summarized and shown in the part titled Method A in Figure 9 by representing the input seismic motion by Seismic Intensity (Kg), restoring force characteristics by Yield Seismic Intensity (ky) and deforming ability required of buildings by Response Displacement (δ). In the same figure, the part titled Method B is equivalent to the concept of the design seismic intensity for the first design in Literature 5, the concept of the required yield strength in Literature 20 and the concept of the seismic diagnosis in Literature 11. Also, Eo index value of the seismic diagnostic standards will be readily noticed to be equivalent to the part titled Method C. All of these methods commonly use the response volume of buildings relative to a certain seismic motion as the standard, and they are somewhat equivalent to fixing two of the three elements, kg, ky and δ, and calculating the reamining one element.

Figure 9 Relationship Between Seismic Design Method and Seismic Diagnostic Method

	Method A	Method B	Method C
	Earthquake Response calculation	Required yield strength	Reserve efficiency basic index
seismic motions (ground motion seismic intensity kg)	◎	◎	Up to what intensity can a building endure?
strength of building (yield seismic intensity ky)	◎	How much required ?	◎
Deforming ability of building (response displacement δ)	How much required ?	◎	◎

note ◎ = conditions(fixed)

? = result (answer to be obtained)

The reserve efficiency basic index is not the size itself of kg but the value equivalent to the elastic response shearing coefficient.

Incidentally, the relationship among these three elements is usually a non-linear issue, and it does not necessarily assure that the remaining one element can be calculated at the same accuracy no matter which two elements are established. The author of this paper believes that Method A is most reliable from the point of accuracy, and Method B and Method C are more practical. Also, Method B, Method C and Method A are suitable for the first design of the seismic design, for the summary calculation of the seismic diagnosis and for detail examination of seismic efficiency both in seismic design and seismic diagnosis respectively.

In the following, the calculation of the Eo indexes of the proposed seismic diagnostic standards for the first and second diagnoses will be summarized.

2) Eo Index for the First Diagnosis

Eo is calculated using Equation (2). However, when the building being diagnosed contains columns with ho/D below 2, another method shall be used. ho is the clear height of columns and D is the sectional depth and length of columns.

$$E_o = \frac{n+1}{n+i}(C_W + \alpha_1 C_C) \times F_W \quad \cdots\cdots (2)$$

where, n: number of stories in a building

i: story of the objective floor

First story is designated as 1, the highest of the story is designated as n.

C_W: Strength index of walls, calculated using Equation (3).

Cc: Strength index of columns, calculated using Equation (4).

α_1: 0.7, however when $C_W = 0$, it is designated as 1.0

F_W: Firmness index of walls, designated as 1.0. When there are no walls, firmness index of columns is used, which is also 1.0 for the first diagnosis.

Strength indexex are obtained by the following summary calculations using the wall ratio and column ratio.

$$C_W = \frac{\tau_{W_1}}{w} \times a_{W_1} + \frac{\tau_{W_2}}{w} \times a_{W_2} + \frac{\tau_{W_3}}{w} \times a_{W_3} \quad \cdots\cdots (3)$$

$$C_C = \frac{\tau_C}{w} \times a_C \quad \cdots\cdots (4)$$

where, a_{W1}: Wall ratio of walls with columns on both sides relative to the total floor area $A_{W1}/\Sigma Af$ (cm^2/m^2), a_{W2} and a_{W3} are wall ratios of walls with a column on one side and of walls without columns relative to the total floor area respectively.

A_{W1}: Total sum of the wall area with columns on both sides which affect the objective direction of the story concerned (cm^2).

ΣA_f: The total area of the building above the story concerned (m^2)

rW_1, rW_2, rW_3: Final average shearing stress of walls with columns on both sides, walls with a column on one side and walls without columns, and it is designated as 30 kg/cm^2, 20 kg/cm^2 and 10 kg/cm^2 respectively.

ac: Column ratio of independent columns to the total floor area ($cm^2/m2$)

τc: 10 kg/cm^2, however, 7 kg/cm^2 when ho/D is above 6.

w: Total weight of the building above the story concerned (weight of building itself + earthquake carry load)/ΣA_f(kg/cm^2). When not particularly calculated, it is designated as 1,200 kg/m^2.

To sum up, the first diagnostic method attempts to evaluate the seismic efficiency only by the summary calculation value [what is in ( ) in Equation (2)] of the reserve strength expressed by the shearing strength coefficient during wall destruction or column yield. Even when Rahmen framework of a high firmness is contained, the effect of cuctility after its yield is disregarded. Therefore, **if this** method is applied to pure Rahmen structures, the Eo index will be evaluated excessively low.

3) Eo for the Second Diagnosis

Assuming that beams have enough strength, the strength index is obtained from the final bending strength and final shearing strength of columns and walls. When columns and walls that yield to a bending force are contained, the effect of these are taken into consideration.

The following is the procedure.

a) Bending strength and shearing strength of each vertical element are calculated, and destruction mode and final reserve shearing force of each element are obtained. The classification of the destruction mode is from Table 5.

b) Firmness index F of each element is calculated (later to be discussed).

c) Gathering the elements with firmness index F values that are close, they are divided into 3 maximum groups. The groups with the smallest F index value to the largest index value are designated as Group 1, Group 2 and Group 3, and the firmness index of each group is designated as F1, F2 and F3.

d) Strength index of each group, C1, C2 and C3 are the total sum of the final shearing force of each group expressed by the shearing force coefficient.

e) Eo index shall be either of the larger values calculated by Equation (5) and Equation (6). However, if extremely brittle columns and shearing destruction columns are contained, or the building has maldistributed walls, special handling is proposed.

$$E_0 = \frac{n+1}{n+i}\sqrt{E_1^2 + E_2^2 + E_3^2} \quad \cdots\cdots (5)$$

$$E_0 = \frac{n+1}{n+i}(C_1 + a_2C_2 + a_3C_3) \times F_1 \quad \cdots\cdots (6)$$

Where, $E_1 = C_1 \times F_1$

$E_2 = C_2 \times F_2$

$E_3 = C_3 \times F_3$

a_2 and a_3 are from Table 6 and Table 7.

Firmness index F is from Table 7. However, the firmness index of bending columns and bending walls is from Equation (7) and Equation (8) respectively.

$$F = \phi\sqrt{2\mu - 1} \quad \cdots\cdots (7)$$

where μ: final plasticity ratio, calculation method is indicated.
(However, $1 \leqq \mu \leqq 5$)

$\phi : \dfrac{1}{0.75(1 + 0.05\mu)}$

$$\left.\begin{array}{ll} \text{When } {}_wQ_{su}/{}_wQ_u \leqq 1.3, & F = 1.0 \\ \text{When } {}_wQ_{su}/{}_wQ_u > 1.4, & F = 2.0 \end{array}\right\} \quad \cdots\cdots (8)$$

However, interims are interpolated on a straight line.
where, wQsu: final shearing strength of walls
wQu: reserve shearing force of walls during final moment (bending yield)

Table 5. Classification of Vertical Elements by Destruction Mode for Second Analysis

destruction mode	definition
bending columns	Bending yield preceeds shearing destruction.
bending walls	Bending yield preceeds shearing destruction.
shearing columns	Shearing destruction preceeds bending yield, however, extremely brittle columns are excluded.
shearing walls	Shearing destruction preceeds bending yield.
extremely brittle columns	Columns with ho/D below 2, shearing destruction preceeds bending destruction.

Table 6. Values of a_2 and a_3 in Equation (6)

Group 1 / Group 2 or Group 3	extremely brittle columns	shearing columns or walls
bending columns	0.5	0.7
bending walls	0.7	1.0
shearing columns or walls	0.7	——

Table 7. Firmness Index F for Second Diagnosis

destruction mode	Firmness Index F
bending columns	1.27-3.2 note) From Equation (7)
bending walls	1.0-2.0 From Equation (8)
shearing columns	1.0
shearing walls	1.8
extremely brittle columns	0.8

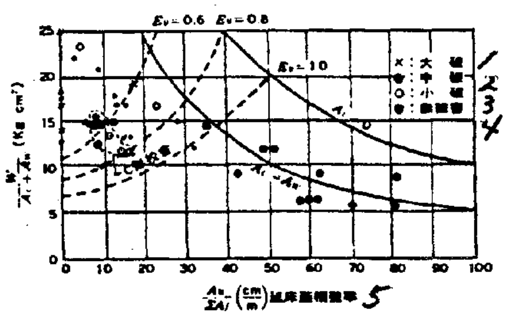
note) Under some conditions it becomes 1.0.

6.4 Examples of Application

Examples of this diagnostic method applied to the damaged and undamaged buildings during the 1968 Tokachi Offshore Earthquake will be given.

Figure 10 arranged damaged and undamaged buildings during the 1968 Tokachi Offshore Earthquake using the wall ratio and the column ratio (Literature 12). The dotted lines are the Eo index values [values of Equation (2)] for the first diagnostic method, drawn on the graph.

Figure 10. Relationship Between Quantity of cross-beam direction walls and columns and Earthquake Damage (Eo index value for the first diagnosis was drawn on a graph in Literature 12)



key-1. major damage
2. moderate damage
3. minor damage
4. no damage
5. ratio of walls to total floor area

However, when C_W and Cc were calculated, $w = 1,000$ kg/cm^2 was assumed similarly to Literature 12. Also, when calculating the wall ratio in the graph, rW_1 was discounted to 20 kg/cm^2 to average them out since all types of walls, walls with columns on both sides, walls with a column on one side and walls without columns, were included. Incidentally, Ac is treated slightly differently since circumferential columns of walls are not accounted into the column sectional area. From the graph, when the Eo value was above 1.0, the buildings were not damaged. Buildings which received major damage almost all indicated an Eo value lower than 0.6. As a whole, the relationship between the levels of damage and Eo values was clearly recognized.

Figure 11 indicates the results when the second diagnostic method was applied to the 1968 Tokachi Offshore Earthquake cases. Although all of the values indicated in the figure are values from the ground floor, the I_s value of buildings incurred major damage was 0.4-0.5, and the buildings were not damaged when this value was 0.6-0.7.

From all these results, although it may be somewhat ~~far~~-fetched, when assuming an earthquake about the level of the 1968 Tokachi Offshore Earthquake, a tentative curve can be drawn pertaining to the seismic efficiency of buildings, using the I_{SO} values as shown in Figure 12.

Figure 11. Relationship Between I_s Index Values for the Second Diagnosis and the 1968 Tokachi Offshore Earthquake Damage (from Literature 16)

key-1. 2nd diagnosis, I_s index value
2. Misawa Business High School(addition)
3. Hachinohe Specialized High School (N wing)
4. University of Hakodate (addition)
5. Hachinohe Library
6. Misawa Business High School (A wing)
7. Gonohe Elementary School (C wing)
8. Nejiro Elementary School (C-shaped school building)
9. cross-beam direction
10. span
11. major damage
12. moderate damage
13. undamaged
14. symbol

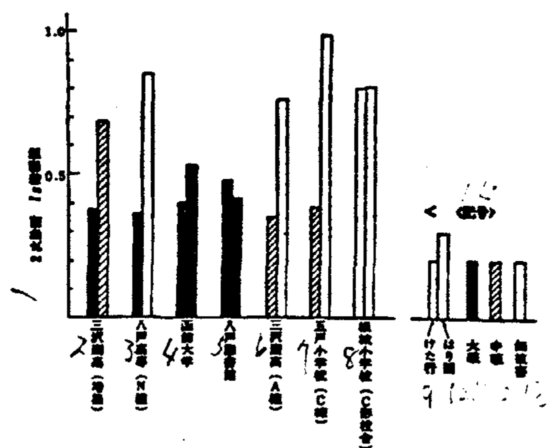
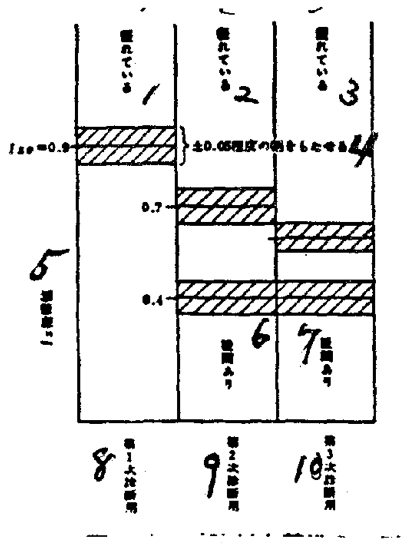


Figure 12. One Example of Seismic Judgement Standard

key-1. superior
2. superior
3. superior
4. slack of 0.1 is given
5. I_s index value
6. questionable
7. questionable
8. for the first diagnosis
9. for the second diagnosis
10. for the third diagnosis



6.5 How to Use the Seismic Judgement Standard

It was previously mentioned that the diagnostic standards proposed this time are comprised of the first, second and third diagnostic methods. The basic idea is the same for all of them, but there are differences in mathmatical difficulty, the details of which increase with the order from the first to the third. A brief description of the degree of difficulty will be given as follows.

First Diagnostic Method: Strength is calculated using only the sectional area of walls and columns. The simplest calculation is used.

Second Diagnostic Method: Calculation of final strength of columns and walls is required. It is however, simpler than the currently used structure calculation.

Third Diagnostic Method: Calculation level is equivalent to the currently used structure calculation. The amount of calculations are not great but knowledge relating to final strength and earthquake response are somewhat required.

As described above, more details are added as the diagnosis advances to a higher stage, and it is probably correct to assume that the credibility of the results also increases. When given a building, this is how the standards shall be used: (1) First, diagnose the building briefly using the first diagnostic method. (2) If a high I_s index value is obtained as a result, the diagnosis will be discontinued. (3) If the result of the first diagnosis indicates a not-so-high I_s index, the second diagnostic method shall be applied, and so forth. Of course, it is possible to apply immediately the second or the third diagnostic method. Also, when the result of the third diagnostic method applied indicated questionable earthquake resistivity, a reinforcement plan can be immediately implemented. This is one way to to the job, but it is also feasible to reinvestigate jointly using more detailed analysis such as oscillation analysis or full on-site-investigation.

7. POST SCRIPT

The above is a description of seismic efficienty of existing reinforced concrete buildings and seismic diagnostic methods for them in connection with seismic design methods.

There are many unclarified points concerning the properties of earthquakes and the earthquake response of buildings. However, it seems earthquake science has finally entered into a stage where seismic designs and seismic diagnoses with respect to the dynamic properties of buildings during earthquakes can be somehow practicalized. It is desirable that these methods will be applied to general building designs and diagnoses as soon as possible. Finally, I would like to close this paper by describing undauntedly but fully the seismic safety for reinforced concrete buildings.

(1) Buildings with many seismic walls will be probably safe under the currently used design method (buildings with wall ratio to the total floor space 30 cm^2/m^2 on the ground floor).

(2) Buildings with small number of walls that fall into any of the following conditions must be designed with great precaution, for instance, paying heed to the

dynamic properties during earthquakes, and you must be sometimes prepared for an increase of labor required for designs (naturally affect the design fee) and an increase of structure units required for buildings or a necessity to install more walls following the design alternation. All existing buildings must be diagnosed for earthquake resistivity.

(a) Buildings with a large span (above 6m)

(b) Buildings with a complex shape (Those which have stories with ventilation openings on the ceiling and with partially reduced floor space)

(c) Pilotis structures

(d) Buildings with shortened columns due to lower sectional walls

(e) Buildings built prior to the 1968 Tokachi Offshore Earthquake (possibly of shearing destruction due to the scarcity of hoops and columns)

(f) Recently built buildings if they are designed using the values obtained by inflating the shearing force which is equivalent to the design seismic intensity and using the minimum 1.5 as the inflation coefficient, as a design shearing force to be used to check the shearing strength of major columns, in stead of the equivalent shearing force when hinges yield at the upper and lower ends of columns or at neighboring beams.

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PART 3 SUMMARY OF SEISMIC DIAGNOSTIC STANDARDS FOR EXISTING REINFORCED CONCRETE BUILDINGS

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1. BACKGROUND FOR DRAWING STANDARD

Medium and low rise reinforced concrete buildings began to spread extensively throughout Japan as earthquake proof and fire proof buildings since after the Kanto Major Earthquake in 1923. On the other hand, around the same time, Professor Toshikata Sato advocated a seismic intensity method which cleverly monitored the effect of seismic motions on buildings as a method for seismic designs. Afterwards in Japan, the seismic design method based upon the seismic intensity method spread, and many highly earthquake resistive buildings , even from the world's standard, have been built. Nevertheless, it is also a fact that some reinforced concrete buildings which used seismic designs were damaged more than anticipated by major earthquakes which occurred after the Kanto Major Earthquake, for example, the 1948 Fukui Earthquake, 1964 Niigata Earthquake, 1968 Tokachi Offshore Earthquake and 1974 Oita Earthquake. This implies that buildings with structural styles which cannot be fully guaranteed to be safe under the rules of the conventional seismic intensity method, have also come forward.

Reflecting upon these earthquake damages, from the standpoint of the latest knowledge of earthquake proof engineering which enabled the construction of super high rise buildings, it is disclosed that the width of the seismic efficiency is broad from highly earthquake resistive buildings to a small number of buildings with questionable seismic safety, in spite of the fact that they were all designed according to more or less the same seismic design techniques.

Particularly, the finding that the damage to the low rise reinforced concrete buildings was outstanding during the 1968 Tokachi Offshore Earthquake, provoded and promoted various studies relating to the seismic design methods which took into consideration the dynamic behavior during earthquakes, for reinforced concrete buildings. Part of the results of the studies were adopted in the Building Standard Law Enforcement Ordinance and Reinforced Concrete Structure Calculation Standards of Architectural Institute of Japan, and are offered for practical use.

As above, the lessons learned from past earthquake damages in conjunction with the progress of the sciences, has contributed to the progress of seismic design methods. However, the presend condition cannot yet be called fully satisfactory in in studies related to seismic safety of existing buildings constructed before these experiences were put into use.

Especially, in recent years, in connection with the prediction of earthquakes in Kawasaki and shore off Tokai, the concern for the safety of buildings, storage for dangerous objects and urban facilities under the possible earthquakes, is growing larger. With these backgrounds, the Guidance Section, Housing Bureau of the Ministry of Construction planned to formulate seismic diagnostic standards for existing buildings and seismic improvement design guidelines, and contracted the projects to formulate seismic diagnostic standards and seismic improvement design guidelines for medium and low rise reinforced concrete buildings in 1976 to the Japan Special Building Safety Conter (Foundation). The center installed two work sections for the selection of the members for policy making as shown in the separate table and for preparation of proposals, and started to proceed with the task.

Thus, the projects were completed in March 1977, and the results were titled "Seismic Diagnostic Standards and Improvement Design Guidelines for Existing Reinforced Concrete Buildings, with Comments" and published by the Center. Also, courses relating to this subject were given at 5 sites throughout Japan including Tokyo.

Detailes of the methods concerned can be found in the above described publication. In this paper, the focus of the discussion will be the basic concept of the seismic diagnosis which is incorporated in the above described seismic diagnostic standards.

2. POLICY FOR PREPARATION OF DIAGNOSTIC STANDARDS

The preparation of this standard started with the primary objective to diagnose seismic efficiency of many buildings as fast as possible. For this purpose, the diagnostic methods are divided into three types from the first diagnosis which secreens buildings with questionable seismic safety by using the simplest possible

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calculations to the third diagnosis which assesses rather accurately the seismic safety of buildings by using somewhat complicated calculations. Screening technique divides the diagnostic method into three types with different level of accuracy. Also, the purport of the standards is to present a technique to express seismic efficiency reserved in buildings by continuous numeric values. Judging of the safety of buildings or the necessity of reinforcement are the in the hands of the users of the standards concerned based upon the numerized seismic efficiency. This deligation of responsibility is supported by the understanding in that the level of the seismic efficiency considered safe varies depending upon the use, importance factor and degree of seismicity at the construction site of the respective buildings, even there are two buildings having the same level of seismic efficiency. However, in reality, it is not possible to use the standards concerned without having some sort of criteria to judge what level of seismic efficiency is required to secure tentative safety against a certain level of earthquakes. In the appendix, therefore, numerical calculation results of the seismic efficiency indexes obtained in accordance with the standards concerned, in regard to the damaged and undamaged buildings exposed to the 1968 Tokachi Offshore Earthquake are indicated as a reference material for rendering judgement.

3. COMPOSITION OF DIAGNOSTIC STANDARDS

In the standards concerned, seismic safety of buildings is expressed by the following two indexes.

I_S (Seismic Index of Structure)

I_N (Seismic Index of Non-Structural Elements)

The I_S index expresses dynamic earthquake resistivity of buildings, and is expressed by the four sub-indexes indicated in Equation (1)

$$Is = E_0 \times G \times S_D \times T \quad \cdots\cdots(1)$$

Eo (Reserve Efficiency Basic Index): This index expresses seismic efficiency of the structure when a building with a good structural plan is built well on standard ground and terrain, and is obtained from a calculation based upon the reserve yield strength and deformation ability of the building.

G (Ground Motion Index): This is an index which expresses local characteristics of ground motions that affect primarily the foundation of a building, such as the relationship between the properties of the building and the amplification level of local ground motions on the lot where the building is constructed and the types of ground. At the present stage, it is difficult to evaluate G, and 1.0 is the tentative index used in these standards.

SD (Shape Index): It is an index which takes into consideration, by check list system, the effects on seismic efficiency of the structure plan related factors which are difficult to be evaluated by calculation such as shapes of a building in plane and elevation and rigidity distribution. It is a kind of reduction coefficient to Eo, and the standard value is 1.0. The numerical value will be lowered if there is some insufficiency in the structure plan.

T (Time Index): This is an index for the evaluation of the effects on the structural seismic efficiency of structural deterioration which develops with time, such as, cracks, deformation and degeneration of materials. It is also a kind of reduction coefficient to Eo, and can be obtained from simple investigation by check list system.

The I_N index indicates safety against damages that incur by the dropping of nonstructural elements such as finishing materials on theexterior of the buildings, and is calculated independently from the I_S index. The relationship between the structural deformation and the nonstructural elemet deformation ability during earthquakes is taken into account when computating this index. Specifically, in the case of the pure Rahmen structure which is anticipated to deform greatly during earthquakes, a high I_N index cannot be obtained unless the nonstructural elements that accompany it have a sufficient deformation ability.

4. CONCEPT OF RESERVE EFFICIENCY BASIC INDEX Eo

The Eo index in principle has two sub-indexes, and it is obtained from the combination of strength index C which expresses reserve yield strength of a building in the form of shearing strength coefficient and the firmness index F which expresses properties of deformation of the building.

1) Eo INDEX FOR THE FIRST DIAGNOSIS

In the first diagnosis, only vertical elements (columns and walls) are viewed as seismic elements that constitute a building, and these elements are classified into 3 types as indicated in Table 1. The Eo index is computated from either Equation (2) or Equation (5) based upon the summary calculation of respective C index and F index of the elements. Incidentally, the Eo index is calculated for each story and for each direction of thebuilding.

(1) Building Without Extremely Short Columns (ho/D $\geqq$ 2.0)

$$E_0 = \frac{n+1}{n+i} (C_w + \alpha_1 C_c) \times F_w \quad \cdots\cdots (2)$$

where, n: number of stories in a building.

i: story of the objective floor

First story is designated as 1, the highest of the story is designated as n.

C_W: Strength index of walls, calculated using Equation (3).

C_C: Strength index of columns, calculated using Equation (4).

d_1: 0.7, however, when C_W 0, it is designated as 1.0.

F_W: Firmness index of walls, designated as 1.0. When there are no walls, firmness index of columns is used, which is also 1.0 for the first diagnosis.

Strength index can be obtained by the following summary calculations if the horizontal sectional area of vertical elements is known.

$$C_w = \frac{\tau_{w_1}}{w} \times a_{w_1} + \frac{\tau_{w_2}}{w} \times a_{w_2} + \frac{\tau_{w_3}}{w} \times a_{w_3} \quad \cdots\cdots (3)$$

$$C_c = \frac{\tau_c}{w} \times a_c \quad \cdots\cdots (4)$$

where, aW_1: Wall ratio of walls with columns on both sides relative to the total floor area $A_{W1}/\sum Af$ (cm^2/m^2), a_{W2} and a_{W3} are wall rations of walls with a column on one side and of walls without columns relative to the total floor area respectively.

A_{W1}: Total sum of the wall area with columns on both sides which affect the objective direction of the story concerned (cm^2).

$\sum A_f$: The total area of the building above the story concerned (m^2)

rW_1, rW_2, rW_3: Final average shearing stress of walls with columns on both sides, walls with a column on one side and walls without columns, and it is designated as 30 kg/cm^2, 20 kg/cm^2 and 10 kg/cm^2 respectively.

ac: Column ratio of independent columns to the total floor area (cm^2/m^2)

τc: 10 kg/cm^2, however, 7 kg/cm2 when ho/D is above 6.

w: Total weight of the building above the story concerned (weight of building itself + earthquake carry load)/$\sum A_f$ (kg/cm^2). When not particularly calculated, it is designated as 1,200 kg/cm^2.

Table 1 Classification of Vertical Elements for the First Diagnosis

TITLE	DEFINITION
columns	independent columns with ho/D above 2
extremely short colums	independent columns with ho/D below 2
walls	include incidental walls not contained in Rahmen

note) ho: Column inside dimensions, which shall be shortened if lower sectional walls or drop walls are present by the length of the respective walls

D: sectional length and depth of columns

(2) Buildings With Extremely Short Columns

$$E_0 = \frac{n+1}{n+i} \left(C_{SC} + \alpha_2 C_W + \alpha_3 C_C \right) \times F_{SC} \quad \cdots\cdots(5)$$

where, Csc: C index of extremely short columns. It is obtained from equation (4). However, it is designated as $\tau c \fallingdotseq 15$ kg/cm^2.

d_2: 0.7

d_3: 0.5

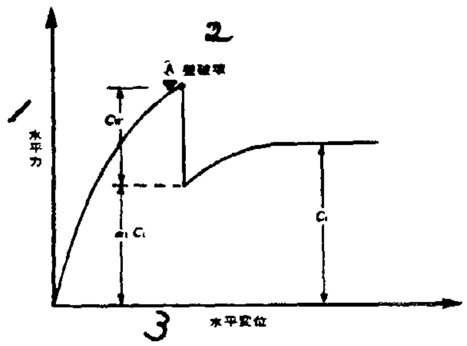
Fsc: Firmness index of extremely short columns.

To sum up, the Eo index is obtained through the following steps: vertical elements are classified into several typical element groups (extremely short columns, columns, walls with columns on both sides.....); average shearing stress at the final strength of each element group is established based upon the existing experimental data; reserve yield summary calculation value is obtained by multiplying the above average shearing stress by the horizontal sectional area of elements that belong to the element group concerned; and the strength index of each element group obtained from the weight of the building above the story concerned is added. However, thepurpose of the first diagnosis is the screening of buildings unlikely to have seismic problems among numerous buildings. Therefore, the evaluation shall be desirably on the safe side. In the standards concerned, the evaluation on the safe side is intentionally made by charging the destruction of the element group with the least deformation ability among all above element groups as the destruction of the building.

Therefore, d_1, d_2 and d_3 in Equation (2) and Equation (5) are a kind of strength discount coefficient which is used to be on the safe side. Specifically, as shown in Figure 1, in case of a building having both wall elements with comparatively poor deformation ability and wall elements with comparatively rich deformation ability, d1 is the coefficient which indicates what percent of the horizontal force of the final strength the columns share when the walls break, and d_1 is fiexed as 0.7 in these standards. Similarly, d_2 and d_3 are the coefficients that ciscount the final strength of walls and columns based upon the destruction (point B in Figure 3) of the extremely short columns. The standards designated them as $d_2 \fallingdotseq 0.7$ and $d_3 \fallingdotseq 0.5$.

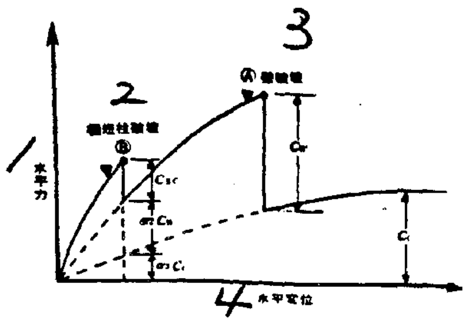
However, in buildings with a small number of extremely short columns, the evaluation obtained may appear to be too much on the safe side if the Eo index is calculated based upon the destruction of extremely short columns as shown in Figure 2. In such a case, Eo was calculated by using Equation (2) without paying attention to extremely short columns. The Eo index, that is, the Eo index that corresponds to the Point Ⓐ in Figure 2, will do. Generally, as the Eo index of buildings with extremely short columns, either of the larger values from Equation (5) and Equation (2) in disregard of extremely short columns, can be used.

Figure 1



key-1. horizontal force
2. wall destruction
3. horizontal displacement

Figure 2



key-1. horizontal force
2. extremely short column destruction
3. wall destruction
4. horizontal displacement

Also, generally in buildings with elements of a small ho/D value, such as extremely short columns, the yield strength after the maximum yield strength declines drastically. Therefore, even when the buildings may have only a small number of extremely short columns, if these columns are essential elements to support the vertical force (This type of elements are called Type 2 elemets in the standards), the destruction of extremely small columns may expedite the local collape of buildings. Consequently, when the extremely short columns are of Type 2 elements, the Eo index must be calculated from Equation (5) which is based upon the destruction of extremely short columns.

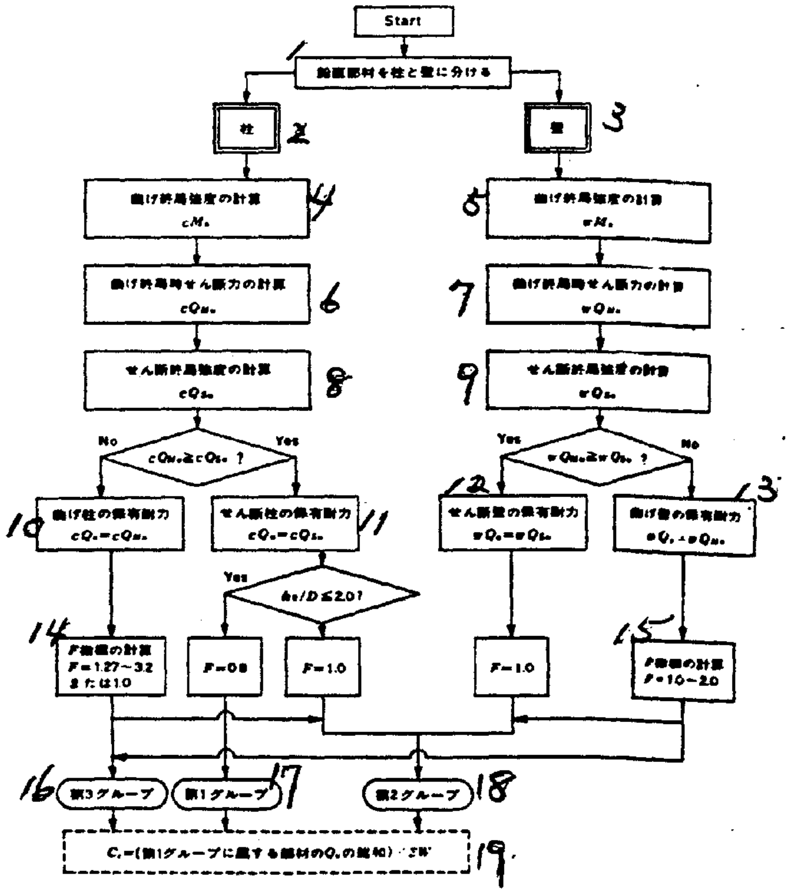
2)

(1) Index Calculation Method

In the second diagnostic method, the Eo index is calculated from Equation (6) and Equation (7), using the strength index obtained from final reserve shearing force of columns and walls, assuming that beams are sufficiently strong.

In the following, the Eo index calculation procedure will be indicated. Also, Figure 3 indicates a flow chart for calculation of strength indexes in the second diagnostic method.

Figure 3. Strength Index Calculation Procedure for Second Diagnosis



key-1. vertical elements are divided into columns and walls

2. columns
3. walls
4. calculation of bending final strength
5. calculation of bending final strength
6. calculation of shearing force at final bending
7. calculation of shearing force at final bending
8. calculation of shearing final strength
9. calculation of shearing final strength
10. reserve yield strength of bending columns
11. reserve yield strength of shearing columns
12. reserve yield strength of shearing walls
13. reserve yield strength of bending walls
14. calculation of F index, $F \doteq 1.27-3.2$ or 1.0
15. calculation of F index $F \doteq 1.0-2.0$
16. Group 3
17. Group 1
18. Group 2
19. Ci (total sum of Qu of elements in Group 1) / ΣW

Eo Index Calculation Procedure

a) The bending and shearing final strength of each vertical element is calculated, and then the destruction mode and final reserve shearing force are obtained. The destruction mode if from Table 2.

b) Firmness index F of each element is calculated (later to be described).

c) Gathering the elements with firmness index F values that are close, they are divided into 3 maximum groups. The groups with the smallest F index value to the largest index value are designated as Group 1, Group 2 and Group 3, and the firmness index of each group is designated as F1, F2 and F3. The smaller the number of groups, the better are the results.

d) Strength index of each group, C1, C2 and C3 are the total sum of the final shearing force of each group expressed by the shearing force coefficient.

e) Eo index shall be either of the larger values calculated by Equation (5) and Equation (6). However, if extremely brittle columns and shearing destruction columns are contained, or the building has mal-distributed walls, special handling is proposed.

$$E_0 = \frac{n+1}{n+i}\sqrt{E_1^2 + E_2^2 + E_3^2} \quad \cdots\cdots(6)$$

$$E_0 = \frac{n+1}{n+i}(C_1 + \alpha_2 C_2 + \alpha_3 C_3) \times F_1 \quad \cdots\cdots(7)$$

where, $E_1 = C_1 \times F_1$

$E_2 = C_2 \times F_2$

$E_3 = C_3 \times F_3$

a_2 and a_3 are from Table 3

Firmness index F is from Table 4. However, the firmness indexes of bending bending columns and bending walls are from Equation (8) and Equation (9).

Table 2. Classification of Vertical Elements by Destruction Mode for Second Analysis

destruction mode	definition
bending columns	Bending yield preceeds shearing destruction.
bending walls	Bending yield preceeds shearing destruction
shearing columns	Shearing destruction preceeds bending yield, however, extremely brittle columns are excluded.
shearing walls	Shearing destruction preceeds bending yield.
extremely brittle columns	Cplumns with ho/D below 2, shearing destruction preceeds bending destruction.

Table 3. Values of a_2 or a_3 in Second Diagnosis

Group 2 or Group 3	Group 1	
	extremely brittle columns	shearing columns shearing walls
bending columns	0.5	0.7
bending walls	0.7	1.0
shearing columns, shearing walls	0.7	—

Table 4. Firmness Index F for Second Diagnosis

destruction mode	firmness index F
bending columns	1.27–3.2 note) from equation (7)
bending walls	1.0–2.0 from equation (8)
shearing columns	1.0
shearing walls	1.8
extremely brittle columns	0.8

note) Under some conditions it becomes 1.0.

Firmness Index of Bending Columns

$$F = \phi\sqrt{2\mu - 1} \quad \cdots\cdots (8)$$

μ: Final plasticity ratio of bending columns which can be calculated from a separately prescribed equation using the ratio of final shearing force of columns and reserve shearing force at final bending and the average shearing stress level at final bending. However, a condition is stipulated in that the maximum and minimum values are set at 0.5 and 1.0 respectively regardless of the calculation results of μ, and F is 1.0 regardless of the value of μ.

$$\phi : \frac{1}{0.75\ (1+0.05\,\mu)} \quad \cdots\cdots (9)$$

Firmness Index of Bending Walls

When $_wQ_{su}/_wQ_u \leqq 1.3$, $F = 1.0$ ....(10)

When $_wQ_{su}/_wQ_u \geqq 1.4$, $F = 2.0$

However, interims are interpolated on a straight line.

Where, wQsu:final shearing strength of walls

wQu: reserve shearing force of walls during bending yield

(2) Concept of Firmness Index F

According to the studies by Newmark and his associates, the maximum response plasticity ratio μ during earthquakes of a system which has perfect elastic plastic type restoring force characteristics (yield strength Cy, Yield point displacement δy) can be calculated by the following equation based upon the elastic response shearing force coefficient C_E of the system.

$$C_E/C_y = \sqrt{2\mu - 1} \quad \text{..................................(11)}$$

The objective medium and low rise reinforced concrete buildings of the standards concerned are short period structures which can be regarded to have a virtually constant (around 3 times the ground motion acceleration) elastic response shearing force coefficient C_E. Therefore, when a quantity equivalent to C_y is designated as the C index and the quantity equivalent to $\sqrt{2\mu-1}$ is designated as the F index, seismic efficiency of buildings of a single destruction mode or of single vertical element is given in Equation (12).

$$E_0 = C \times F \quad \text{..(12)}$$

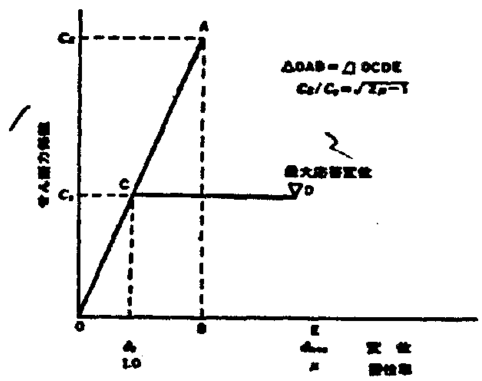
Nonetheless, the restoring force characteristics of actual concrete buildings are very comples and marked with a reduction of rigidity in association with cracks in theconcrete and a reduction of rising rigidity by the repeated loading.

When the results of the response analysis of mono mass system having degrading tri-linear type restoring force characteristics which represent restoring characteristics of bending destruction type reinforced concrete buildings relatively well, were handled in the same way as Equation (11) introduced by Newmark, it was discovered that the relationship between Cy and μ can be expressed by summary equation (13).

$$C_y = 0.75 \times \frac{1 + 0.05\,\mu}{\sqrt{2\mu - 1}} \qquad \cdots\cdots (13)$$

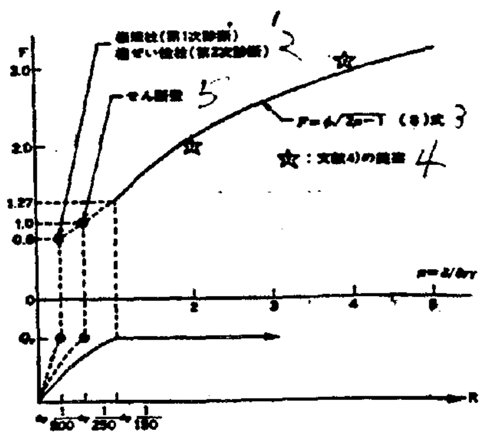
The firmness index F of bending columns is the inverse number of Equation (13). An assumingly relatively adequate values are also set as firmness indexes of other elements in the lihgt that sufficient experimental data is not available and the complicated calculations my stray from thepurpose of the standards, although, to be proper, the final plasticity ratio μ of elements shall be used as a basis to determine these values.

Figure 4. Response of Elastic and Plastic Systems



key-1. shearing force coefficient
2. maximum response displacement
3. displacement
4. plasticity ratio

Figure 5. Relationship Between Final Plasticity Ratio and F Index of Various Elements



key-1. extremely short columns (First diagnosis)
2. extremely brittle columns (Second diagnosis)
3. equation (8)
4. proposal in Literature (4)
5. shearing walls

(3) Eo Index of Buildings Composed of Elements With Various Qualities of Firmness

General buildings are composed of elements with various qualities of firmness, and the evaluation of their seismic efficiency is not easy. In the following, the basic concept of Equation (6) and Equation (7) proposed in the standards to obtaine Eo indexes of these buildings will be explained.

For an example, let us think of two buildings such as below.

Building I: Building composed only of shearing columns of $F_1 \doteq 1$, and $C_1 \doteq 0.9$

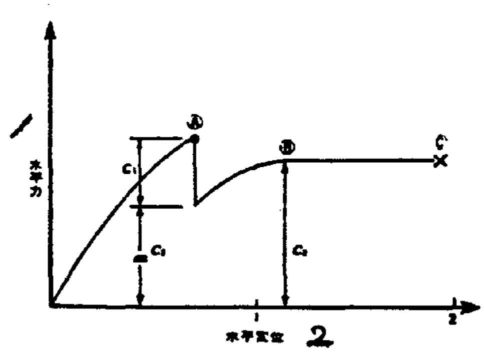
$$E_{01} = C_1 \times F_1 = 0.9$$

Building II: Building composed only of bending columns of $F_2 \doteq 2$, and $C_2 \doteq 0.45$

$$E_{02} = C_2 \times F_2 = 0.45 \times 2 = 0.9$$

According to the concept (Equation 12) described in (2), the Eo indexes of the two are equal, and both are 0.9. Next, let us suppose there is Building III which has walls with the same properties as Building I (ie $F \doteq 1.0$) and walls with the same properties as Building II ($F \doteq 2.0$). The relationship between the horizontal force and the horizontal displacement of the building such as this can be expressed as in Figure 6. Specifically, shearing walls breakdown at point A, and benind columns yield at point B and reach final deformation at point C.

Figure 6. Relationship Between Horizontal Force and Horizontal Displacement of Buildings Made of Shearing walls and Bending Columns



key-1. horizontal force
2. horizontal displacement

Now, the question is at what mixing ratio of C_1 (reserve shearing force of columns only/weight of building) and C_2 (reserve shearing force of walls only / weight of building) Building III can be evaluated to have the same seismic efficiency as Building I or Building II. Specifically, a limit curve relative to the combination of C1 and C_2 or E_1 and E_2 which is considered to have the same seismic efficiency as Building I or Building II may be drawn (in plane C) where C_2 is placed on the axis of ordinates and C_1 is placed on the axis of abscissas (Figure 7-a) or (in Plane E) where $E_2 \approx C_2 \times F_2$ is placed on the axis of ordinates and $E_1 \approx C_1 \times F_1$ is placed on the axis of abscissas.

For instance, one way to obtain this limit curve is to analize response of various C_1 and C_2 combinations, and seek the C_1-C_2 combination where the response of Building III can be below the final plasticity ratio of bending columns, that is, the C_1-C_2 combination where the response plasticity ratio will become practically the same as Building I.

Figure 7-a (Plane C)

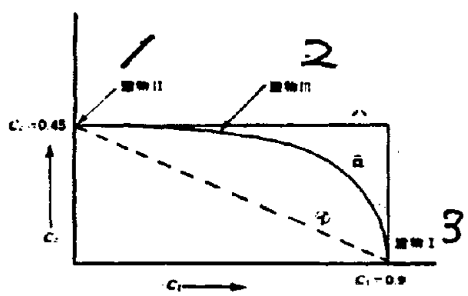
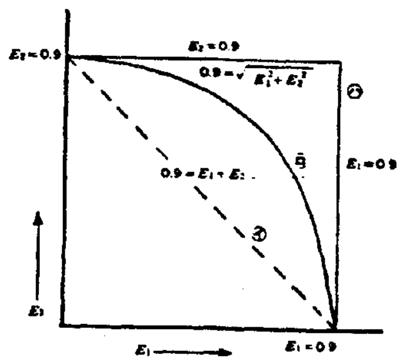


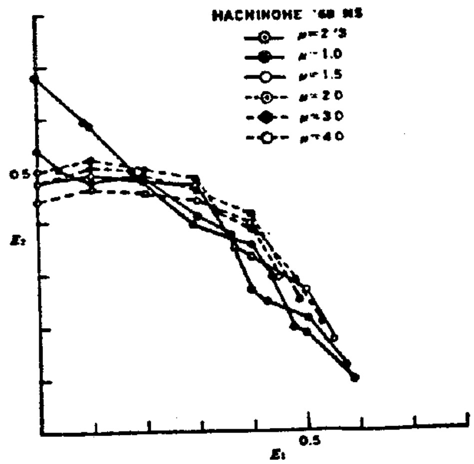
Figure 7-b (Plane E)



key-1. Building II
2.Building III
3. Building I

Figure 8 is an example where the points with equal maximum response plasticity ratio were connected using the results of response analysis of recorded seismic motions of various C_1-C_2 ratios in an oscillation system having a degrading tri-linear type bending column restoring force characteristic and an origin directional type wall restoring force characteristic. From the chart, the line where the maximum response plasticity ratio (maximum response displacement/yield displacement) becomes fixed, seems to be approximated by substantially a circle on Plane E.

Figure 8. Example of Limit Curves of Various C_1 and C_2 Combinations on Plane E

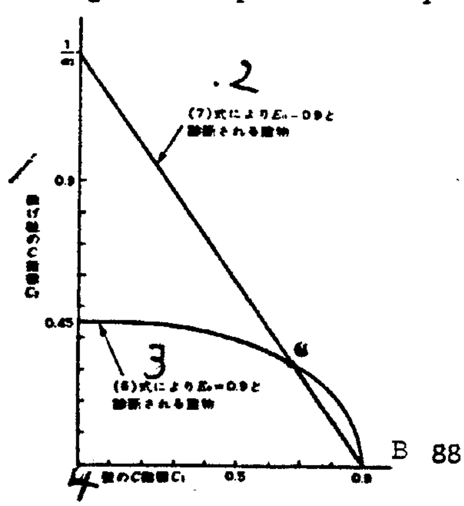


Equation (6) in the standards express this statement, and attempts to stretch it to a case with 3 types of vertical elements.

Equation (7) attempts to express the Eo index based upon the destruction of elements poorest in deformation ability, and its basic concept is the same as the Eo index for the first diagnostic method.

Figure 9 expresses the relationship between Equation (6) and Equation (7) as the Eo index of buildings with the previously described two types of structural elements (shearing walls and bending column, in this case $C_3=0$)

Figure 9 Comparison of Equation (6) and Equation (7)



key-1. C index of bending columns C_2

2. Buildings diagnosed as Eo 0.9 from Equation (7)

3. Buildings diagnosed as Eo 0.9 from Equation (6)

4. C index of walls C_1

3) Eo INDEX IN THE THRID DIAGNOSTIC METHOD

The third diagnostic method is different from the second diagnostic method in that the reserve yield strength of each element is calculated based upon the mechanism of frameworks taking into account the strength of beams and the relief of walls from the foundation. As a result, the following 3 types of destruction modes are considered as the destruction modes of columns and walls on each story besides the destruction modes considered in the second diagnostic method.

Columns Governed by Bending Beams	F = 3.0
Columns Governed by Shearing Beams	F = 1.5
Revolving Walls	F = 3.0

After obtaining the destruction mode of each element, reserve sheaing force and firmness index corresponding to the descturction mode, the calculational procedure for the Eo index will be the same as in the second diagnostic method.

5. SHAPE INDEX S_D

This index attempts to evaluate the effect on earthquake resistivity of the status of the dynamic balance of an entire building, which is essentially difficult to be calculated. It has meaning as a correction factor to the Reserve Efficiency Basic Index Eo.

1) APPLICABLE ITEMS

For the first diagnostic use, the following items can be checked.

* Items Related to Plane Shape: a) formality, b) ratio of side length, c) constriction, d) expansion joint, e) size of ventilation, f) maldistribution of ventilations, g) other special shapes

* Items Related to Elevation Shape: h) presence of basement, i) eveness of layer height, j) presence of pilotis, k) other special shapes

For the second diagnostic Use, the following items shall be checked in addition to the above described items.

l) eccentricity of the center of gravity and the center of rigidity on the plane, m) weight and rigidity ratio of upper and lower stories

2) INDEX COMPUTATION METHOD

Each of the above described items are divided into three grades. With a simple check of the plane view and the sectional view, G_i(1.0, 0.9, 0.8) of each item can be determined. For example, referring to Item b) of the ratio of the side length, $b =$ long side/short side ≤ 5, $5 < b \leq 8$, $8 < b$ will be grades as G_i 1.0, 0.9 and 0.8 respectively. Also, the degree of the influence of each item on the seismic efficiency is prescribed as the Range Adjusting Factor R_i. S_D index is calculated by the following equation using G_i and R_i.

$$S_D = C_a \times C_b \times \cdots\cdots\cdots\cdots\cdots C_n$$

$$C_i = [\,1 - (1 - G_i)\, R_i\,]$$

Incidentally, this index is calculated only once for one building in the first diagnosis, but Item l) and Item n) are calculated separately for each story and each direction in the second diagnosis. In the third diagnosis, the index used for the second diagnosis shall be used as it is.

6. TIME INDEX T

This is an index used to evaluate the effects on the structural yield strength of the degeneration and deterioration of materials occurring with time or of cracks developed due to insufficient designs and construction works, and one value is used for one building both in the first and second diagnoses.

1) FIRST DIAGNOSIS

Score points from 1.0 to 0.7 are given to each item such as deformation, cracks in walls and columns, past fire accidents, use of buildings, age of buildings, condition of finish by a simple history research and inspection of buildings. The minimum value is used as the time index for the first diagnosis.

2) SECOND DIAGNOSIS

The range of structural cracks, deformation, degeneration and aging that affects a building is inspected side by site (floor, major beam, wall, column), and the time index is calculated according to the following method.

$$T = (T_1 + T_2 + \cdots\cdots + T_n) / N$$
$$T_i = (1 - P_{si})(1 - P_{ti})$$

where, Tt: Time index of i story

N: number of stories inspected

Psi: total minum points of structural cracks and deformation of the i story

Pti: total minus points of degeneration and aging of the i story

7. NONSTRUCTURAL ELEMENT SEISMIC INDEX I_N

This index is used to evaluate the danger factor due to the destruction and fall particularly of external walls of nonstructural elements during earthquakes. The diagnostic method is sub-divided into first, second and third diagnostic methods, and all of them are computated for each wall surface and each story.

The make-up of the sub-indexes for the computation of I_N index is as follows.

- 非構造部材耐震指標（I_N）
  - 構法指標（B）
    - 変形追従指数（f）
      - 主体構造じん性グレード（g_S）
      - 非構造部材変形グレード（g_N）
    - 実態指数（t）
      - トラブル歴のグレード（g_H）
      - 経年数のグレード（g_Y）
  - 面積指数（W）—（壁面長さ l_i、高さ h_i、基準階高 h_s）
  - 影響度指数（H）
    - 環境指数（e）
    - 抑制指数（c）

key-1. nonstructural element seismic index I_N, 2. tectonical index (B) 3. area index (W), 4. influence index (H), 5. deformation following index (f), 6. fact index (t), 7. (length of wall surface li, height hi, standard story height hs), 8. environment index (e), 9. control index (c), 10. main structure rigidity grade (gs), 11. nonstructural element deformation grade (g_N), 12. trouble history grade (g_H), 13. time grade (g_Y)

Among the previously described indexes, gs g_N, g_H, g_Y, e andc are given as a constant in the separate table. Furthermore, f is shown as a matrix of gs, g_N, t, g_H and g_Y. Other indexes are calculated according to equations, and eventually, the index I_N is calculated from the following equation (26).

$$I_N = 1 - \frac{\Sigma B \cdot W \cdot H \cdot l_i}{\Sigma l_i}$$

$$= 1 - \frac{\Sigma \overbrace{\{f + (1-f)\,t\}}^{B} \cdot \overbrace{\left(0.5 + \frac{0.5\,h_i}{h_s}\right)}^{W} \cdot \overbrace{(\Sigma e \cdot c)}^{H}\, l_i}{\Sigma l_i} \quad \cdots\cdots\cdots\cdots \quad (26)$$

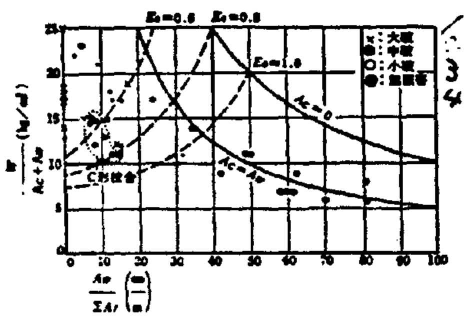
8. APPLICATIONAL EXAMPLES

Figure 10 compiled the results of seismic diagnosis of damaged and undamaged buildings during the 1968 Tokachi Offshore Earthquake, using the wall ratio and the column ration (Literature 9). The dotted lines are Eo index values (values from Equation (2)) plotted on the chart from the first diagnostic method.

However, in calculating C_W and C_c, $w \doteq 1{,}000$ kg/m^2 was assumed as in (Literature 9). Also, since the calculation of the wall ratio in thechart takes into account all of the walls with columns on both sides, walls with a column on one side and walls without columns, J_{W1} was discounted to 20 kg/cm^2, and $a_{W2} \doteq a_{W3} = 0$ was used for averaging the calculation results. Incidentally, thediagnostic standards did not take the columns fitted to the walls into account, and the treatment of Ac is therefore slightly different. From the chart, it is clarified that buildings with an Eo value of above 1.0 were not damaged, and the majority of buildings with major damage indicated an Eo value above 0.6. Figure 11 is the second diagnostic method results applied to buildings exposed to the 1968 Tokachi Offshore Earthquake.

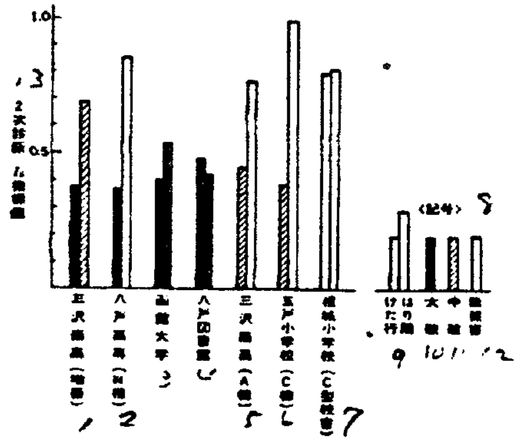
The value indicated in the chart are all from the values obtained on the ground floor. The I_s value of buildings with major damage was in the level of 0.4-0.5, and the buildings indicating an I_s value of 0.6-0.7 were not damaged. Incidentally, in this applicational example, the T index was designated as 1.0 for the computation.

Figure 10. Relationship Between the Quantity of Cross-beam Walls and Columns and Earthquake Damages (Eo index value for the first diagnosis was plotted on a chart in Literature 12)



key-1. major damage
2. moderate damage
3. minor damage
4. no damage

Figure 11. Relationship Between the Second Diagnostic I_s Value and Damage



key-1. Misawa Business High School (addition)
2. Hachinohe Specialized High School (N wing)
3. University of Hakodate
4. Hachinohe Library
5. Misawa Business High School (A wing)
6. Gonohe Elementary School
7. Nejiro Elementary School (C shaped School Building)
8. symbol
9. cross-beam span
10. major damage
11. minor damage
12. no damage
13. second diagnostic I_s index value

9. POST REMARKS

In the above context, a summary of the diagnostic standards formulated at this time and applicational examples were introduced. In conclusion, I_s and I_{so} values will be related, which are the standard used to evaluate the safety of the objective buildings based upon the structural seismic index I_s obtained from these diagnostic standards.

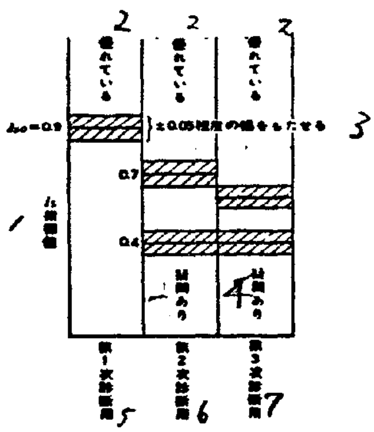
Figure 12 indicates one of the criteria of I_{so} assuming an earthquake of about the same seismic intensity as the 1968 Tokachi Offshore Earthquake, although it may appear somewhat far-fetched, based upon the applicational example.

When the I_s value is above 0.9 in the first diagnosis, it can be considered that the building is safe against an earthquake at a level of the Tokachi Offshore Earthquake. However, the buildings which indicated an I_s value below 0.9 in the first diagnosis canoot be judged yet, and the second diagnosis shall be desirably administered. Building with an I_s value above 0.7 in the second diagnosis can be very well considered safe. Also, the seismic safety of buildings with an I_s value below 0.4 is questionable, and it is recommended to enforce reinforcement measures from the seismic improvement design guidelines formulated simultaneously with these standards or conduct a more detailed investigation.

Of course, the adequacy of the above values must be examined in the days to come from various angles, for instance, accumulation of many more applicational examples. For more rational use of these standards, it is desirable to accumulate I_s data relating to various buildings and to verify I_s values from the possible future earthquake damages (of course, it is wished it will never happen).

Incidentally, the standards incorporated many concepts and techniques of various seismic diagnostic methods and seismic designs which have been already proposed. These are listed at the end as Literature for Reference.

Figure 12. An Example of I_{so} Vale



key-1. I_s index value

2. superior
3. a slack of ± 0.05 is given
4. questionable
5. for first diagnosis
6. for second diagnosis
7. for third diagnosis

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PART 4. HOW TO APPLY SEISMIC DIAGNOSTIC STANDARDS

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PREFACE

This material is a supplementary explanation of "Seismic Diagnostic Standards for Existing Reinforced Concrete Buildings" a summary of which was prsented in Part 3.

The author of this paper and others have participated in the project "Earthquake Control Measures for Reinforced Concrete Buildings" undertaken by the SPRC Research Committee established within the Japan Specail Building Sefety Center (Foundation) commissioned by Shizuoka Prefecture. As a part of the project, we worked for the computer programming of the above described seismic diagnostic standards, and have now completed the assignment in connection with the first and second diagnoses. This program was the result of our effort to render faithful programming to the original standards. However, when computarizing, it was necessary to add and develop algorithms for the part which the original standards entrusted to the engineering judgement of theoperators of the diagnoses.

This material extracted the part relating to the added and developed algorithms among the materials for a computer program guide, and emphasis of the explanation is on the part which can be utilized even if the diagnosis will be made using manual calculations.

1. FIRST DIAGNOSTIC I_s VALUE COMPUTATION METHOD

The I_s value for the first diagnosis is computated in accordance with the seismic diagnostic standards. However, the following operations which were not included in the diagnostic standards will be added.

1) The Eo value is corrected depending upon the strength of concrete and the variety of concrete. Specifically, thediagnostic standards estimate the concrete compression strength to be 200 kg/cm^2, but in this program, the following corrections are made by the strength of concrete and the variety of concrete.

(1) Regular concrete

$$Eo \doteq Eo\ (\text{standard}) \times \frac{Fc}{200}$$

whereas, Fc: concrete compression strength (kg/cm^2)

(2) Light Concrete

$$Eo \doteq Eo\ (\text{standard}) \times \frac{Fc}{200} \times d$$

whereas, d: Type 1 and Type 1 of light concrete is 0.9
Type 3 and Type 4 of light concrete is 0.8

2) When inputting the wall opening circumference ratio, it is possible to deal with walls with openings. The C_W of walls with openings can be calculated from the the following equation.

$$Cw = \frac{30}{w} \times aw_1 \times \frac{Fc}{200} \times (1-p)$$

where, $aw_1 = Aw_1 / \Sigma Af$

$Aw_1 = t \times \ell w_1$

Fe ~ concrete compression strength

In case of the light concrete, the discount ratio is multipled.

$P =$ equivalent opening circumference ratio $= \sqrt{\dfrac{\text{opening area}}{\text{wall face area}}}$

3) Eccentricity is calculated just as in the second diagnosis.

When eccentricity e (from diagnostic standard table 10) is larger than 0.15, this will be printed on the diagnostic card.

As described in the comments of the diagnostic standards, if the first diagnosis is applied to buildings with extremely large excentricity, the seismic index might be evaluated on the danger side. Therefore, when the diagnostic card indicates that the eccentricity is too large, it is necessary at least to carry out the second diagnosis.

2. SECOND DIAGNOSTIC I_s VALUE COMPUTATION METHOD

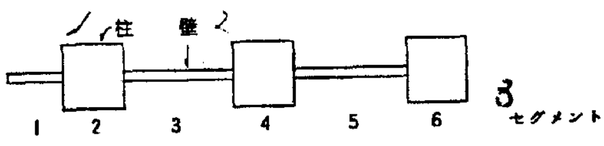
The second diagnostic I_s value is also computated, in principle, in accordance with the diagnostic standard procedure as in the case of the first diagnosis. However, since the original diagnostic standards were prepared assuming that the calculation would be made manually, partial correction and addition, or adoption of an original algorithm was necessary for the computer programming.

In the following, the essential points will be related.

1) DEFINITION OF VERTICAL ELEMENTS

In this program, elements composed of columns and walls are called vertical elements, and rectangular sectional elements such as columns and walls that comose of one vertical element is called a segment.

Figure 1. Example 1: Vertical Element Composed of 6 Segments



key-1. column
2. wall
3. segment

Figure 2. Example 2: Vertical Element Composed of 1 Segment

key-1. single independent column

If the list number of each segment is fed into the computer, separating segments for walls from segments for columns, they are automatically joined and composed into one vertical element within the program.

2) SIMPLIFICATION AND CLASSIFICATION OF VERTICAL ELEMENT SECTIONS

A program screen simplifies and classifies vertical elements composed of many segments into the following 6 types and an incidental wall for the convenience of computating sectional strength.

(1) Vertical Element Composed of Columns and Walls

a) Seismic walls with three or more columns are, for instance simplified as follows.

* All vertical reinforcement in the center column and the walls shall be equal.

*For the wall opening circumference ratio, the average value of the opening circumference ratio of each wall shall be simplified.

* For the wall hw, the average value of hw of each wall shall be simplified.

b) This simplification process will be disregarded when the total projection of the wall is less than 45 cm.

(2) Incidental Walls

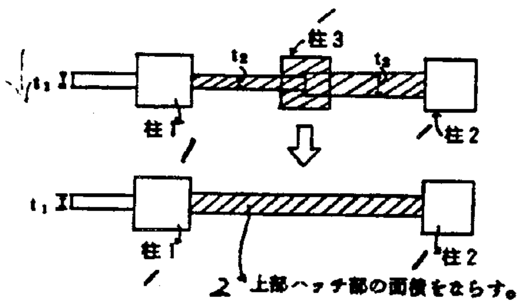
Walls without columns on either side are considered incidental walls even they are on the rahmen line. However, when these walls are treated as walls with openings, they shall be excluded from this category.

Figure 3. Sectional Types

断面タイプ番号 (NWR)	断面形状	名称
1	セグメント数：3	両側そで壁付柱又は中央柱付壁
2	セグメント数：2	片側そで壁付柱又は片側柱付壁
3	セグメント数：1	単独柱
4	セグメント数：5	両側そで壁付耐震壁
5	セグメント数：4	片側そで壁付耐震壁
6	セグメント数：3	独立耐震壁

key-1. sectional type number, 2. sectional shape, 3. title, 4. number of segments
5. column with walls on both side, or center column with walls
6. column with a wall one one side, or wall with a column on one side
7. single independent column, 8. seismic wall with wing walls on both sides
9. seismic wall with a wing wall on one side, 10. independent seismic wall

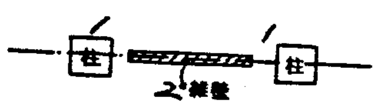
Figure 4. Simplification of Continuous Seismic Walls



key-1. column
2. Area of the upper hatch was filled.

Figure . Definition of Incidental Wall

key-1. column
2. incidental wall



3) COMPUTATION OF FINAL BENDING MOMENT (Mu)

The final bending moment will be computated using the complete plasticity theory upon which Equation (11) and Equation (12) of the diagnostic standards in based. (Sub-program EVA and ST2)

The following are the main assumptions:

Assumption 1: The 6 sectional types indicated in the previous paragraph are divided into a maximum of 7 sectional pieces as shown in Figure 6. (Incidental walls are the same as the sinle independent columns).
All the reinforcing steel contained in each sectional piece is equally distributed in the piece (However, the reinforcing steel in the center of the column in sectional types 1,2 and 3 are disregarded).

Assumption 2: It is supposed that the axial force that affects the columns will affect the center of the column in sectional types 1,2 and 3, and that the total axial force concentrates on the location 1/2 of the distance between the center of the right and left columns in sectional types 4,5 and 6.

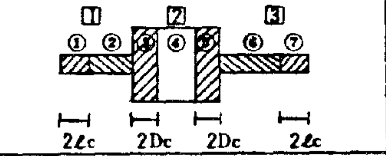
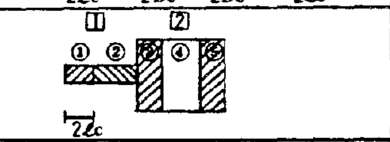
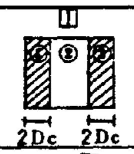
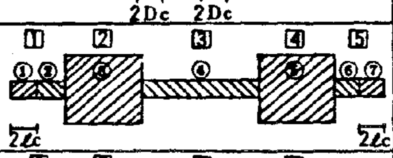
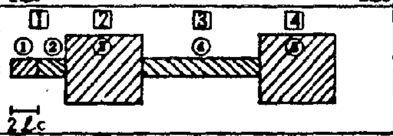
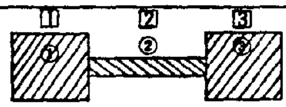
Assumption 3: Excluding sectional type 3 (including incidental walls), the average value of the final bending moment when the bending moment works on an element separately from both left and right directions, is designated as the final bending moment of the vertical element.

Assumption 4: When the upper and lower (capital and base in case of a column) sectional properties are different, the final bending moment average is considered the final bending moment of the element.

The location of neutral axis and the final bending moment can be calculated as below from the above assumptions.

For the sake of explaining symbols, the seismic wall in Figure 7 shall be used.

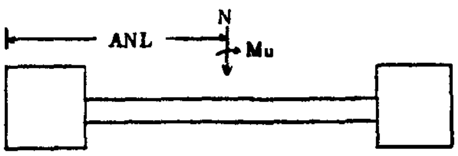
Figure 6. Drawing of Sectional Pieces

Sectional type		number of segment	sectional piece number	note
1		$\boxed{3}$	7	reinforcing steel in 4 is disregarded
2		$\boxed{2}$	5	same as above
3		$\boxed{1}$	3	reinforcing steel in 2 is disregarded
4		$\boxed{5}$	7	
5		$\boxed{4}$	5	
6		$\boxed{3}$	3	

ℓc: Distance from the center of the reinforcing steel at the wall end to the concrete edge

Dc: Distance from the column tensile reinforcement center to the concrete edge

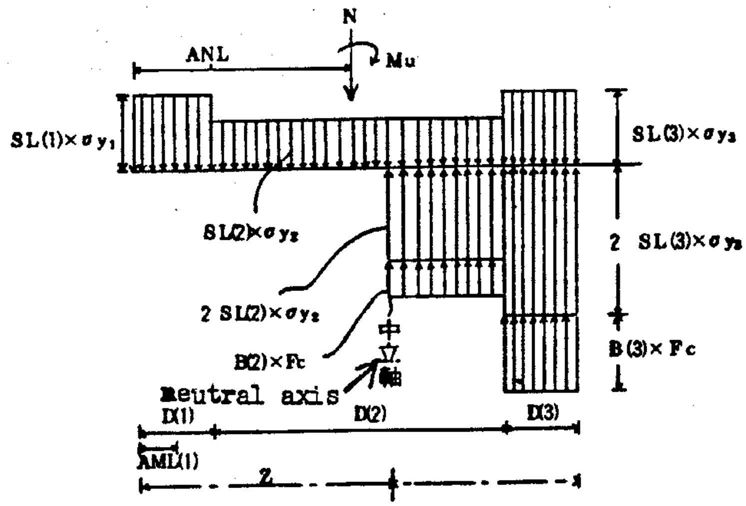
Figure 7. Seismic Wall Final Bending Moment Calculation Example



axial directiona force N (kg)

bending moment M (kg-cm)

sectional piece number	1	2	3	
width	B (1)	B (2)	B (3)	cm
depth	D (1)	D (2)	D (3)	cm
vertical reinforcement	SL (1)	SL (2)	SL (3)	cm²/cm
location of center of gravity	AML (1)	AML (2)	AML (3)	cm



Equation for Determination of Neutral Axis: When the neutral axis is in the Piece Number K

$$\sum_{I=1}^{N} SL(I) + D(I) + \sigma yI + N = (2\,SL(K) + \sigma yK + B(K) + Fc) \times \left(\sum_{I=1}^{K} D(I) - z\right)$$

$$+ \sum_{I=K+1}^{N} (2\,SL(I) \times D(I) \times \sigma yI + B(I) + D(I) \times Fc) \qquad \cdots\cdots\cdots\cdots \ (1)$$

Equation For Determination of Final Moment:

$$Mu = (2\,SL(K) + \sigma yK + B(K) + Fc)\left(\sum_{I=1}^{K} - D(I) - z\right)\left(\sum_{I=1}^{K} D(I) + z\right) \times \frac{1}{2}$$

$$+ \sum_{I=K+1}^{N} \{(2\,SL(I) \times D(I) \times \sigma yI + B(I) + D(I) + Fc) \times AML(I)\}$$

$$- \sum_{I=1}^{N} (SL(I) + D(I) + \sigma yI \times AML(I)) - N \times ANL \qquad \cdots\cdots\cdots \ (2)$$

However, in sectional types 1,2 and 3, the middle stage reinforcement of columns (reinforcement in the sectional piece, not marked with diagonal lines in Figure 6) were disregarded. This treatment is given in consideration of the adjustability of Equation (10) and Equation (11) of the diagnostic standards.

4) CALCULATION OF SHEARING FINAL STRENGTH (Qsu)

The shearing final strength Qsu was calculated using either one of the following equations in accordance with the sectional type shown in Table 1 by giving following corrections and additional restrictive conditions to Equation (13) and Equation (14) of the diagnostic standards

(1) Corrections-Additions to Standard Equation (13)

$$Qsu = \left(\frac{0.053Pt^{0.23}\,(180+Fc)}{M/(Q\cdot d)+0.12} + 2.7\sqrt{Pw\cdot s\sigma wy} + 0.1\sigma_0\right) \times b \times j$$

Standard Equation (13)

however, $1 \leqq M/(Q\cdot d) \leqq 3$

corrections

a) Single independent columns were omitted and $M/(Q\cdot d)$ was corrected to h/D.

where, h: ho or hw indicated in Table 1

D: Sectional depth indicated in Table 1

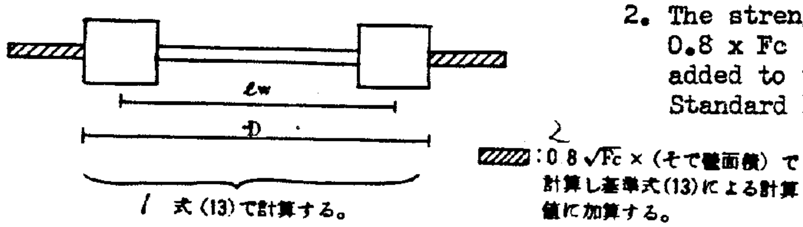
b) In the case of sectional types 4,5 and 6, either of the larger one between 0.8D or ℓw is adopted for j.

additions

a) When Pt(%) is 0.1 or below, 0.1 is used.

b) Wing walls in sectional types 4 and 5 shall be separately treated only when the shearing strength is calculated. The strength shall be calculated using $0.8 \times \sqrt{Fc}$ (area of wing walls), and added to the strength of seismic walls (area enclosed by columns)

Figure 8. Handling of Wing Walls (Sectional Types 4 and 5)



key-1. calculate using Equation (13)

2. The strength is calculated by 0.8 x Fc (wing wall area) and added to the calculation value from Standard Equation (13)

(2) Addition to Standard Equation (14)

$$Q_{su} = 0.8\sqrt{F_c}\left(\frac{\ell_w}{h_w}\right)\Sigma A + 0.5\left\{P_w \cdot \sigma_{wy} + P_s \cdot \sigma_{sy}\ \frac{t\,(\ell_w - P)}{b\,D}\right\} \times b_1 D + 0.1\,N \qquad \text{Standard Equation (14)}$$

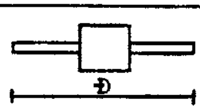
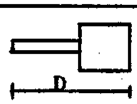
<u>addition</u>

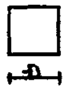
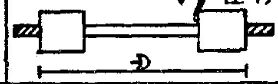
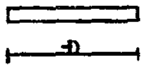
a) In case $h_w/\ell_w \leq 1$, Standard Equation (14) is used assuming $h_w/\ell_w \simeq 1.0$.

(3) The Relation of Sectional Types to the Adopted Shearing Strength Equation and Height

The diagnostic standards prescribe, "Final shearing strength of walls with a column on one side and walls without columns shall be computated using either Equation (13) or Equation (14) according to the shape and the arrangement of the reinforcements". However, in this program, the shearing strength equation and height to be used are designated as follows according to the sectional type and the height of the story (hs)/D.

Table 1. Shearing Strength Equation and Height

1 断面タイプ	2 形　状	hs/D	3 開口周比	4 高さ 5 注2)	6 せん断強度式
1		1.5 以上 7	−	ho	基準式(14) 9
		1.5 未満 8	−	hw	基準式(14) 9
2		1.5 以上 7	−	ho	基準式(14) 9
		1.5 未満 8	−	hw	基準式(13) 10

3		—	—	ho	基準式(13) 10
4、5、6	// 注1) 	—	12 0.4以下	hw	基準式(13) /10
雑 /3 壁		—	12 0.4以下	hs	基準式(13) 10

note 1) Hatch is considered separately.

note 2) 1/2 of the height becomes the height of the inflection point

ho: column inside height

hw: Input of the wall height into the program is given in terms of how many layers the objective wall continues to rise. The program converts this to hw. The uppermost layer of the continuous layer wall (including one layer wall) will be 2hw.

hs: height of story

5) CALCULATION OF SHEARING FORCE DURING BENDING STRENGTH (Q_{Mu})

Assuming that 1/2 of ho, hw or hs in the table of the previous paragraph was the inflection point height, final bending moment Mu was divided by the inflection point height to obtain this value.

6) DESTRUCTION TYPE AND FIRMNESS INDEX

The types and indexes were obtained in accordance with the same method as in the diagnostic standards.

7) GROUPING OF VERTICAL ELEMENTS

Grouping of vertical elements and computation of the Eo index were performed in accordance with the following method. This method was newly developed as a method suitable for computer programming and yet respectful of the intent of the diagnostic standards.

(1) Grouping to 11 Groups (Grouping-0$_{zero}$)

First, each vertical element is classified into 11 groups in accordance with F·index values.

Grouping-0 tries to carry out a safe side grouping by keeping the digital F value to the smaller side.

F values of bending element groups are virtually geometrical progression of $\sqrt{1.27}$ ($\fallingdotseq 1.13$). Therefore, the Eo value may be sometimes evaluated low by maximum 13% in this operation.

Table 2 Grouping-0

グループ番号	グループのF値	各グループに属する鉛直部材のF値	破壊タイプ
1	0.8	F = 0.8	極ぜい性部材
2	1.0	1.0≦F＜1.13	せん断柱・壁・曲げ壁
3	1.27	1.13≦F＜1.4	曲げ柱・曲げ壁
4	1.4	1.4≦F＜1.6	〃
5	1.6	1.6≦F＜1.8	〃
6	1.8	1.8≦F＜2.0	〃
7	2.0	2.0≦F＜2.3	〃
8	2.3	2.3≦F＜2.6	曲げ柱
9	2.6	2.6≦F＜2.9	〃
10	2.9	2.9≦F＜3.2	〃
11	3.2	F = 3.2	〃

key-1. group number, 2. F value of group, 3. F value of vertical elements which belong to each group, 4. destruction type, 5. extremely brittle element
6. shearing column /wall/bending column, 7. bending column/bending wall
8. bending column

(2) Grouping to One Group (Grouping-I)

Grouping-I computates the Eo value by gathering groups which were grouped into a maximum of 11 groups from the operation in the preceeding paragraph into one group. The Eo value obtained (=E1M) here will be shown in the following paragraphs.

This Eo value is compared to the Eo values obtained from Grouping-II and Grouping-III (E2M, E3M respectively), and the maximum value of the three will be adopted as a final result.

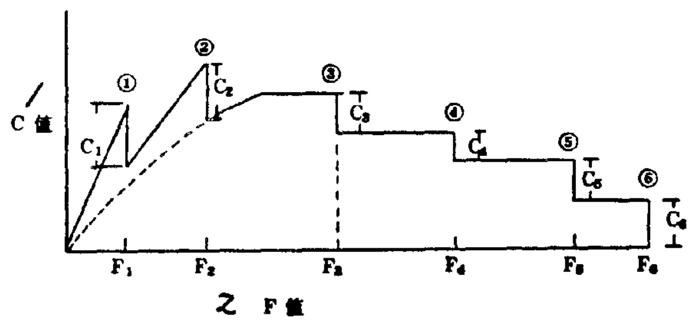
The grouping policy of Grouping-I is as follows:

Explaining this by the example in Figure 9, first, it is assumed that F values of all groups are equal to F_1, and Eo is computated by the following equation which is thecorrected version of Diagnostic Standard Equation (5).

$$E_0 = \frac{n+1}{n+i}\{C_1 + \alpha_2 C_2 + \alpha_3 (C_3 + C_4 + C_5 + C_6)\} \times F_1$$

Eo obtained from this results will be the Eo which took the extremely brittle property into account if the Group 1 is represented by extremely brittle elements.

Figure 9. Example of Grouping-0 result



key-1. C value, 2. F value

Next, in line with the same purpose, elements with an F value lower than the F value (F_2) of the Number group will be omitted. All F values of groups with an F value larger than F_2 are assumed to be equal to F_2. Subsequently, the

Eo value is obtained from the following equation.

$$E_o = \frac{n+1}{n+i}\{C_2 + \alpha\ (C_3 + C_4 + C_5 + C_6)\} \times F_2$$

The similar operations will be repeated sequentially in respect to the No. 3 Group and the No. 4 group and so forth to calculate Eo. The maximum Eo value obtained was designated as the Eo value (E1M) when grouping into one.

(3) Grouping to Two Groups (Grouping-II)

The maximum 10 types of groups excluding the extremely brittle elements were grouped spontaneously into 2 groups, and the Eo value was calculated in accordance with the diagnostic standard equation (4). The maximum Eo value obtained from the all possible combination of 2 groups was designated as the E value (E2M) when grouping into two groups.

$$E_o = \frac{n+1}{n+i}\sqrt{E_A^2 + E_B^2}$$

where, $E_A = C_A \times F_A$

$E_B = C_B \times F_A$

C_A, C_B = C value of spontenious 2 groups

F_A, F_B F value of spontenious 2 groups

For instance, in case of the Figure 9, if all the elements were grouped into No. 2 group and No. 4 group, the following shall be observed.

$$F_A = F_2\ ,\qquad F_B = F_4$$

$$C_A = C_2 + \alpha \times C_3$$

$$C_B = C_4 + C_5 + C_6$$

(4) Grouping to 3 Groups (Grouping-III)

Using the same procedure as in theprevious paragraph, the maximum 10 types of groups excluding the extremely brittle elements were grouped spontaneously into 3 groups, and the Eo value was obtained from the following equation. The maximum Eo value obtained from the all possible combinations of 3 groups is designated as E3M.

$$E_0 = \frac{n+1}{n+i}\sqrt{E_A^2 + E_B^2 + E_C^2}$$

where, E_A, E_B, Ec: E value of each group when the to groups were grouped spontaneously into 3 groups.

For example, if all the elements were grouped into No. 2, No. 3 and No. 5 groups in Figure 9, the following shall be observed.

$$F_A = F_2,\quad F_B = F_3,\quad F_C = F_5$$
$$C_A = C_2,\quad C_B = C_3 + C_4,\quad C_C = C_5 + C_6$$
$$E_A = C_A \cdot F_A,\quad E_B = C_B \cdot F_B,\quad E_C = C_C \cdot F_C$$

(5) Final Results

From above, the Eo value, when taking extremely brittle elements into account, can be obtained by the operation of Grouping-I. The Eo value when extremely brittle elements are disregarded, will be the maxium value among E1M, E2M and E3M obtained from Grouping-I, Grouping-II and Grouping-III.

The diagnostic standards divide vertical elements into the maximum 3 types of groups by some sort of method, and adopt either one of the larger Eo values obtained from Standard Equation (4) or from Standard Equation (5) which grouped the elements into one group using the No. 1 Group as a standard. The method adopted in this program envelopes and extends the method used in the diagnostic standards. This method examines all possibilities when grouping elements into one, two and three groups on the safe side, and selects the maximum Eo value.

Incidentally, in this program, whether or not the extremely brittle elements are type 2 structurral elements will not be checked. When there are extremely brittle elements involved, both Eo values when these elements are taken into account and when they are disregarded will be displayed. Which Eo value shall be adopted will be up to the judgement of the examiners.

This is primarily because it is nearly impossible for the examiners to judge all of the vertical elements at the input data preparation stage, and it is extremely difficult at the present state to establish an algotithm for the programming of this judgement.

PART 5 SEISMIC JUDGEMENT INDEX VALUE (E_T) AND ITS CONCEPT

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ONE IDEA CONCERNING SEISMIC JUDGEMENT INDEX

1. PREFACE

It is predicted that there is a possibility of an ocean type major earthquake larger than the Kanto Major Earthquake, which may occur along the coast of Tokai by the end of this century. Shizuoka prefecture is preparing control measures to minimize the damage. As a part of the efforts, it was decided to diagnose the earthquake resistance of existing reinforced concrete buildings based upon the Seismic Diagnostic Standards for Existing Reinforced Concrete Buildings (Japan Special Building Safety Center). Therefore, it is necessary to have seismic judgement index values to evaluate the seismic indexes obtained relative to the imaginary Tokai Earthquake.

According to the studies conducted to date, once the location of the hypocenter and magnitude of the earthquake are determined, the intensity of the seismic motions at each locality can be tentatively computated and seismic judgement index values can be obtained. However, it is doubtful that at the present condition, the location of the hypocenter and the magnitude can be accurately estimated. In addition, it is possible that local earthquakes may occur like the Izu Oshima Nearsear Earthquake which occurred in 1978 and damaged eastern Izu. Therefore, it is difficult to think that the seismic efficiency of buildings can be evaluated by determining the size of seismic motions from a specific imaginary earthquake.

Accordingly, it is thought more practical first to determine the size of the seismic motions based upon past earthquake statistical data and give buildings a standard seismic efficiency, in stead of merely evaluating the seismic efficiency of buildings against the imaginary Tokai Earthquake, and then to take a position to cope with the gigantic earthquake currently at issue along the coast of Tokai.

Shizuoka prefecture has promoted various studies since 1971 in the name of basic research for earthquake control measures, and it is our objective to decide on the seismic judgement index vslues in compliance with "Seismic Diagnositc Standards" using the results of these studies. However, the final decision making must take into account the prefectural administrations since the seismic efficiency to be reserved in buildings will be affected in many ways by the administrative decisions, for instance, measures taken after the earthquake, reinforcing costs and construction costs.

Incidentally, this theory was elaborated as a result of rearranging the report, "Examination of Earthquake Input, Part 1. Seismic Diagnosis of Existing Buildings, Earthquake Control Plan for Reinforced Concrete Buildings" (Japan Sepcial Building Safety Center).

2. TABLE OF SEISMIC JUDGEMENT INDEX VALUES

1) FIRST DIAGNOSIS

Standard Seismic Judgement Index Values $1.0 \times C_G \times C_I$

Imaginary Tokai Earthquake Seismic Judgement Index Values $1.6 \times C_G \times C_I$

where, C_G and C_I : same as in the second diagnosis

2) SECOND DIAGNOSIS

Seismic Judgement Index Values E_T

E_T $C_G \times C_I \times Es$..........(1)

C_G : terrain index, established by terrain conditions, and one of the values Table 1 is selected.

Table 1 C_G

general terrain	precipice	non-concordant layer	local height
1.0	1.25	1.25	1.25

Footnote 1

C_I : Importance factor $C_I \geqq 1.25$

Es : Seismic judgement basic index

$$E_s = C_{TG} \times \alpha_{TG} \quad \cdots\cdots\cdots\cdots \quad (2)$$

C_{TG}: This index is either one of the smaller values selected from Table 2 and Table 3, first by classifying the destruction mode of vertical element groups which greatly affects the judgement into the bending destruction mode and shearing destruction mode, subsequently by using ground predominant period T_G and the number of the layers of a building. However, when the destruction mode is not classified, the shearing destruction mode is used.

Footnote 1 : An importance factor C_I of important public buildings such as hospitals is designated as $C_I \geqq 1.25$. $C_I \geqq 1.25$ is equivalent to $\alpha_{TG} = 0.45$ within 40 km distance from the epicenter of the imaginary Tokai Earthquake. Incidentally, since the seismic judgement index value is the value to examine whether a building will be destroyed, it is necessary to give $C_I \geqq 1.25$ in correspondence to the importance factor of the important buildings which must be protected against any damages more than slight damages.

Table 2. Relation of Predominant Period to C_{TG}

脚注 2

卓越周期	曲げ、せん断破壊形
To = 0.3 秒	3.6
0.4	3.1
0.5	2.8
0.6	2.5
0.7	2.4
0.8	2.2

key-1. predominant period
2. bending, shearing destruction modes
3. seconds
4. footnote 2

Table 3. Relation of Number of Layers to C_{TG}

層数	曲げ破壊形	せん断破壊形
1	3.1	3.5
2	2.7	3.0
3	2.5	2.8
4	2.4	2.6
5	2.3	2.5
6	2.2	2.5

key-1. number of layers
2. bending destruction mode
3. shearing destruction mode

α_{TG} : Standard ground seismic intensity 0.23

: Imaginary Tokai Earthquake ground seismic intensity (Table 4)

Table 4. Epicentral distance and ground seismic intensity

epicentral distance	α_{TG}
25 ~ 40 Km	0.36
40 ~ 50	0.33
50 ~ 60	0.30
60 ~ 70	0.27
≧ 70	0.23

The above results are tabulated in Table 5.

Footnote 2 : The predominant period shall not be mechanically considered an equivalent for ground type.

Table 5-1. Es when α_{TG} =0.36

( )bending destruction type

卓越周期 T_G 1 / 層数 N 2	0.3 sec	0.4 sec	0.5 sec	0.6 sec	0.7 sec	0.8 sec
1	1.25 (1.00)	1.10 (1.10)	1.00 (1.00)	0.90 (0.90)	0.85 (0.85)	0.80 (0.80)
2	1.10 (0.95)	1.10 (0.95)	1.00 (0.95)	0.90 (0.90)	0.85 (0.85)	0.80 (0.80)
3	1.00 (0.90)	1.00 (0.90)	1.00 (0.90)	0.90 (0.90)	0.85 (0.85)	0.80 (0.80)
4	0.95 (0.85)	0.95 (0.85)	0.95 (0.85)	0.90 (0.85)	0.85 (0.85)	0.80 (0.80)
5	0.90 (0.85)	0.90 (0.85)	0.90 (0.85)	0.90 (0.85)	0.85 (0.85)	0.80 (0.80)
6	0.90 (0.80)	0.90 (0.80)	0.90 (0.80)	0.90 (0.80)	0.85 (0.80)	0.80 (0.80)

key- 1. predominant period
2. number of layers

Table 5-2 Es when $\alpha_{TG}=0.33$

( ) bending destruction type

卓越周期 To (1) / 層数 N (2)	0.3 sec	0.4 sec	0.5 sec	0.6 sec	0.7 sec	0.8 sec
1	1.15 (1.05)	1.05 (1.05)	0.90 (0.90)	0.85 (0.85)	0.80 (0.80)	0.75 (0.75)
2	1.00 (0.90)	1.00 (0.90)	0.90 (0.90)	0.85 (0.85)	0.80 (0.80)	0.75 (0.75)
3	0.90 (0.85)	0.90 (0.85)	0.90 (0.85)	0.85 (0.85)	0.80 (0.80)	0.75 (0.75)
4	0.85 (0.80)	0.85 (0.80)	0.85 (0.80)	0.85 (0.80)	0.80 (0.80)	0.75 (0.75)
5	0.85 (0.75)	0.85 (0.75)	0.85 (0.75)	0.85 (0.75)	0.80 (0.75)	0.75 (0.75)
6	0.85 (0.75)	0.85 (0.75)	0.85 (0.75)	0.85 (0.75)	0.80 (0.75)	0.75 (0.75)

key-1. predominant period, 2. number of layers

Table 5-3 Es when $\alpha_{TG}=0.30$

( ) bending destruction type

卓越周期 To (1) / 層数 N (2)	0.3 sec	0.4 sec	0.5 sec	0.6 sec	0.7 sec	0.8 sec
1	1.05 (0.95)	0.95 (0.95)	0.85 (0.85)	0.75 (0.75)	0.70 (0.70)	0.65 (0.65)
2	0.90 (0.80)	0.90 (0.80)	0.85 (0.80)	0.75 (0.75)	0.70 (0.70)	0.65 (0.65)
3	0.85 (0.75)	0.85 (0.75)	0.85 (0.75)	0.75 (0.75)	0.70 (0.70)	0.65 (0.65)
4	0.80 (0.70)	0.80 (0.70)	0.80 (0.70)	0.75 (0.70)	0.70 (0.70)	0.65 (0.65)
5	0.75 (0.70)	0.75 (0.70)	0.75 (0.70)	0.75 (0.70)	0.70 (0.70)	0.65 (0.65)
6	0.75 (0.65)	0.75 (0.65)	0.75 (0.65)	0.75 (0.65)	0.70 (0.65)	0.65 (0.65)

key-1. predominant period, 2. number of layers

Table 5-4 Es when $\alpha_{TG}=0.27$

( ) bending destruction type

卓越周期 /To 層数 N 2	0.3 sec	0.4 sec	0.5 sec	0.6 sec	0.7 sec	0.8 sec
1	0.95 (0.85)	0.85 (0.85)	0.75 (0.75)	0.70 (0.70)	0.65 (0.65)	0.60 (0.60)
2	0.80 (0.75)	0.80 (0.75)	0.75 (0.75)	0.70 (0.70)	0.65 (0.65)	0.60 (0.60)
3	0.75 (0.70)	0.75 (0.70)	0.75 (0.70)	0.70 (0.70)	0.65 (0.65)	0.60 (0.60)
4	0.70 (0.65)	0.70 (0.65)	0.70 (0.65)	0.70 (0.65)	0.65 (0.65)	0.60 (0.60)
5	0.70 (0.60)	0.70 (0.60)	0.70 (0.60)	0.70 (0.60)	0.65 (0.60)	0.60 (0.60)
6	0.70 (0.60)	0.70 (0.60)	0.70 (0.60)	0.70 (0.60)	0.65 (0.60)	0.60 (0.60)

key-1. predominant period, 2. number of layers

Table 5-5 Es when $\alpha_{TG}=0.23$

( ) bending destruction type

卓越周期 To 層数 N	0.3 sec	0.4 sec	0.5 sec	0.6 sec	0.7 sec	0.8 sec
1	0.80 (0.70)	0.70 (0.70)	0.65 (0.65)	0.60 (0.60)	0.55 (0.55)	0.50 (0.50)
2	0.70 (0.60)	0.70 (0.60)	0.65 (0.60)	0.60 (0.60)	0.55 (0.55)	0.50 (0.50)
3	0.65 (0.60)	0.65 (0.60)	0.65 (0.60)	0.60 (0.60)	0.55 (0.55)	0.50 (0.50)
4	0.60 (0.55)	0.60 (0.55)	0.60 (0.55)	0.60 (0.55)	0.55 (0.55)	0.50 (0.50)
5	0.60 (0.55)	0.60 (0.55)	0.60 (0.55)	0.60 (0.55)	0.55 (0.55)	0.50 (0.50)
6	0.60 (0.50)	0.60 (0.50)	0.60 (0.50)	0.60 (0.50)	0.55 (0.50)	0.50 (0.50)

key-1. predominant period, 2. number of layers

3. SEISMIC JUDGEMENT INDEX VALUE COMPUTATION PROCEDURE

Assuming a one story building of an ideal mono-mass system without spiral grains, the response of the building can be obtained by determining (1) the size and the properties of the input seismic motions and (2) properties of the building (period, type and yield strength of restoring force characteristics, damping).

Now, as for (1), the properties of the seismic motions is assumed and the size is given as Variant α, and as for (2), the type of the restoring force characteristics and damping are assumed, and the period and yield strength are given as Variants T and Ky respectively. Here, if the period (T) is considered as a specific value, and the response can be expressed by Plasticity Ratio μ, there are only three variants, μ, α and ky. They can be all related by the respnse analysis. For instance, when the size of the seismic motions is established, there develops a specific relation between μ and ky of the building in that if one is established the other is automatically established. When deformation μ of the building is established, ky is the yield strength necessary to curtail the response of the building to μ, ie, the required yield strength. If μ and ky of the building are established, the size of the seismic motions α can be established.

In the case of "Seismic Diagnostic Standars", the limit of μ of a building can be determined from the properties of elements and yield strength of ky of the building from strength index (C). Therefore, the size of the seismic motions α which the building can withstand can be given. Reserve Efficiency Basic Index Eo of the "Seismic Diagnostic Standards" expresses the relation of ky and μ in the form of $Eo \simeq C \times F$ by introducing Firmness Index F which can be determined from μ, relating one to one to the size of seismic motions α. Deriving from the meaning of equation $Eo \simeq C \times F$, the size of the seismic motions is determined, and Eo is obtained from a specific μ and a ky. This value obtained will be the judgement value corresponding to the Eo value and is applicable to all cases.

Actual buildings are multi-layered, and plane and sectional characteristics of buildings are different in addition to the changes with time. "Seismic Diagnostic Standards" have various ingenuities to cope with these factors. These ingenuities appear in the following equation which computates Structural Seismic Index I_S.

$$I_s = E_o \times G \times S_D \times T$$

$$E_o = \frac{n+1}{n+i} \times C \times F$$

where, $(n+1)/(n+i)$ is the coefficient that makes a multiple-layer building equivalent to a mono-layer building, SD(shape index) is the coefficient which primarily corrects plane and sectional weakpoints, and T (time index) is the coefficient that introduces changes with time.

From the meaning attached to this equation, if a seismic judgement index value is established relative to specific values of coefficient $(n+1)/(n+i)$, SD and T, this seismic judgement index value can be applicable to all cases. As will be related later, since an ideal mono-mass system's required yield strength spectrum is used when determining the seismic judgement index value, the seismic judgement index value established is evidentally for the case when $(n+1)/(n+i) = 1.0$, SD= 1.0 and T $\approx$ 1.0.

Incidentally, the coefficient G in the previously described equation is the ground motion index, and this coefficient is introduced in the "Seismic Diagnostic Standards". The main purpose of this coefficient here is to temper terrian index C_G and response magnification factor C_{TG}. However, at the present stage, it is intentionally avoided to change the seismic judgement index to a great extent in association with the relationship between the predominant period of ground and a building.

4. EXAMINATION OF THE INPUT SEISMIC MOTIONS

1) CHARACTERISTICS OF INPUT SEISMIC MOTIONS AND THE STANDARD VALUE OF ITS SIZE

There is no clear answer as to what could be the yardstick to measure the size of an seismic motion. Since the object of the study here is low or medium rise reinforced concrete buildings, the characteristics of the seismic motions will be provided and the size of the acceleration will be given as the yardstick. The characteristics of the seismic motions adopted here are similar to the characteristics of seismic waves which have been comparatively well known, TAFT (E.W.) ELCENTRO (N.S), HACHINOHE (N.S), etc. The predominant wave of these seismic waves is in the vicinity of 0.4 sec., however, in this section, we take a position that the pr period changes, as will be described later, by some groung dypes.

When evaluating standard seismic efficiency, the 1968 Tokachi Offshore Earthquake which damaged comparatively many reinforced concrete buildings will serve as a criterion. The size of the seismic motions of this earthquake shall be evaluated to be predominant period$\fallingdotseq$0.4 sec and 0.23g ($\alpha_{TG} \fallingdotseq 0.23$). It will not be too inconsistant to think that this value fluctuated by ground conditions and terrain conditions. This value seems an appropriate fundamental value even based upon the past earthquake statistical data (Literature 1).

Also, the seismic judgement index value I_{so} prescribed in the "Seismic Diagnostic Standards" based upon this earthquake, corresponds relatively well to the standard seismic judgement basic index value relative to $\alpha_{GT} \fallingdotseq 0.23$ in Table 5-5.

There is a so called B type artificial seismic wave in Literature 2, which is created as an imitation of the above described seismic wave. Since the relationship of the properties of a building and its yield strength (required yield strength) to be reserved relative to this artificial seismic wave group is illustrated, the following examination will be conducted employing this artificial seismic wave group.

Incidentally, in this artificial seismic wave group, the following is the relationship among the average maximum acceleration, average maximum velocity and the average maximum deformation when the predominant period is 0.4 sec.

Average Maximum Acceleration	Average Maximum Velocity	Averate Maximum Deformation
0.23 g	32 kine	24 cm

Assuming that the energy of each frequency component of this artificial seismic wave group is constant, and that the predominant period is changed by changing the time axis, the acceleration is inversely proportional to the square root of the predominant period (footnote 3, Literatures 2 and 3).

For example, the ratio of the average maximum acceleration value and the predominant period of ground at this time (Literature 4) will be,

predominant period	maximum acceleration value
0.2 sec	0.33 g
0.4 sec	0.23 g
0.5 sec	0.21 g
0.8 sec	0.17 g

Incidentally, the ground which has these predominant periods will be substantially from Type 1 to Type 4 in terms of ground types. However, machanically relating one to the other will be risky in view of the fact that the scope of the predominant period is rather wide even among the same ground types. Also, the increase in the seismic magnitude or the oscillation is said to lengthen the predominant period according to some studies (Literatures 3 and 4), but this factor will not be considered in this section.

2) SIZE OF IMAGINARY TOKAI EARTHQUAKE

Designating the maximum acceleration on the ground (Type 2 ground) with a predominant period of 0.4 sec as 0.45 g (α_{TG}=0.45), double the input seismic motion standard value 1.23 g, let us evaluate this maxiumum acceleration relative to an earthquake of an magnitude of $M \fallingdotseq 8$ in terms of the epicentral distance.

According to the assumption given in the above, it is granted that the maximum acceleration amplitude appears on the ground with the shortest predominant period. If the predominant period $T_G \fallingdotseq 0.2$ sec is given to the objective group of this type, the maximum acceleration 0.45g ($\alpha TG \fallingdotseq 0.45$) of the ground with a predominant period of 0.4 sec is equivalent to 0.63 g on the objective ground. Now, if $h_G \fallingdotseq 0.2$ is given as an equivalent damping common logarithm of the ground, the magnification of the propagation function will be 2.5 times (Footnote 4). Consequently, the maximum acceleration at the foundation will be o.25g, and the evaluation of this value in accordance with Literature 5 from Figure 1 will obtain an epicentral distance $\Delta = 25$ km (Footnote 5). Similarly, the size of the standard seismic motion 0.13g is 70 km in terms of the epicentral distance.

Footnote 3: This thinking is euqivalent to the fact that the filter characteristics are similar but only the predominant period has changed, when it is assumed that a wave form (white noise) having a constant power spectrum density entered the foundation and a wave form filtered by the characteristics of the ground will appear on the surface.

Footnote 4: According to Literature 3, oscillation characteristic G(T) of the ground is 2.2 folds when

$$G(+) = \left[\left\{ 1 - \frac{T}{T_0} \right\}^2 + \left\{ \frac{0.2}{\sqrt{T_0}} \left(\frac{T}{T_0} \right) \right\}^2 \right]^{\frac{1}{2}} \quad T = T_0 = 0.2 \text{ sec}$$

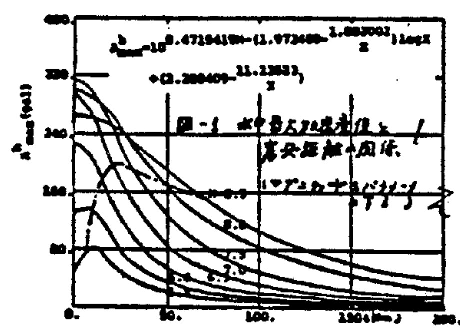
and 1.5 folds when

$$G(+) = \left[\left(\frac{1+d}{1-d} \right)^2 \left\{ 1 - \frac{T}{T_0} \right\}^2 + \left\{ \frac{1.5}{\sqrt{T_0}} \left(\frac{T}{T_0} \right)^2 \right\}^{-\frac{1}{2}} \quad \tau T = T_0 = 0.2 \text{ sec}$$

In either case, the amplification ratio is proportional to $\sqrt{TG}$, and acceleration amplitude of the foundation is inversely proportional to the period. Therefore, the acceleration amplitude of the surface is inversely proportional to $\sqrt{TG}$.

Figure 1. Maximum Acceleration Value and Epicentral Distance (Literature 5)

key-1. relationship of horizontal maximum acceleration and epicentral distance
2. magnitude is used as a parameter
* The value of the abscissa where it crosses the one dot broken line indicates the hypercentral depth.
* considered as an average value



According to Literature 3, the previously described 0.45 g (T_G≑0.45 sec) is evaluated as 60-80 km in terms of epicentral distance from the concept of the oscillation characteristic of the ground. Referring to Literature 5 and Literature 3, the relationship between the epicentral distance and the maximum acceleration on the ground with a predominant period of 0.4 sec was undauntedly assumed as follows.

epicentral distance	maximum acceleration	α_{TG} *
25 ~ 40 km	0.45 g	0.45
40 ~ 50 km	0.33 g	0.33
50 ~ 60 km	0.30 g	0.30
60 ~ 70 km	0.27 g	0.27

* Input seismic intensity

For the sites within theepicentral distance of 25 km, there are still many more points unclarified, but tentatively they will be considered the same as the sites within the epicentral distance of 25-40 km. (Footnote 6)

Incidentally, according to the assumption adopted here, the ratio of the foundation and the maximum acceleration of the surface when the maximum acceleration of the foundation is fixed, will be 2.5 folds at predominant period T_G≑0.2, 1.8 folds at predominant period T_G≑0.4, 1.7 folds at predominant period T_G≑ 0.5 and 1.4 folds at predominant period T_G≑0.8 (Footnore 7).

If the magnitude is 7.0, the maximum acceleration 0.45 g (α_{TG}=0.45) is 15 km in terms of the epicentral distance. Incidentally, the acceleration studies so far concerns the surface acceleration, but this is designated as the value which is loaded on a building and expressed as a seismic intensity.

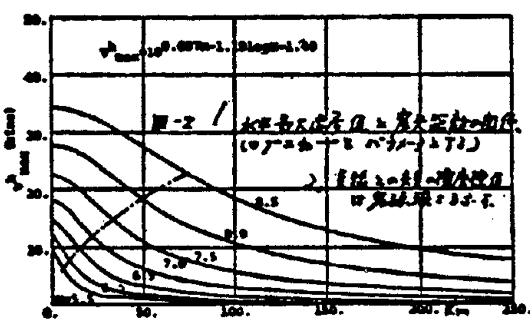
Footnote 5: This is the value for hard ground, but designated as the value for the foundation considered here. Accordingly, if the ground is hard, the input seismic intensity can be reduced.

Footnote 6: If an ability to destroy a building with a comparatively short period to a great extent is attributable to velocity, the maximum velocity at M=8 will be 28 kine on the foundation and 70 kine on the surface (Literature 5)

The four types of earthquakes from A type to D type perceived in Literature 4 correspond to from a giant earthquake to a local earthquake, and the average maximum acceleration on the surface when the power spectrum density on the foundation is constant (0.126 ft^2/sec^3) will be as follows.

	TG = 0.4 sec	TG = 0.2 sec
DType	0.25 g	0.35 g
CType	0.33 g	0.45 g
BType	0.42 g	0.59 g
AType	0.45 g	0.63 g

Supplementary Figure



key-1. relationship of horizontal maximum acceleration and epicentral distance
2. The value of the abscissa where it crosses the one dot broken line indicates the hypercentral depth

Also, when the maximum acceleration is constant at 0.45 g (TG= 0.4 sec), the average maximum velocity and maximum deformation are obtained as in the following Table.

	average maximum velocity	average maximum deformation
DType	36 kine	5.2 cm
CType	40 kine	9.0 cm
BType	66 kine	48.0 cm
AType	69 kine	46.0 cm

Therefore, when the destruction ability of seismic waves is viewed in terms of velocity, the seismic motion of B type earthquake imagined here seems to be also a very large one. However, the ratio of velocity and acceleration varies by the seismic waves, and it is also a fact that the ratio similarly varies when the predominant period is changed (Literatures 3 and 5)

Footnote 7: The ratio of the maximum accelerations of the foundation and the surface from Literature 4 will be an average 1.8, 1.7 and 1.4 folds on Type 2 ground ($TG \fallingdotseq 0.4$ sec), Type 3 ground ($TG \fallingdotseq 0.5$ sec) and on Type 4 ground ($TG \fallingdotseq 0.8$ sec) respectively, when the maximum acceleration of the foundation is designated as 0.15 g. The correspondability appears to be good when considering that the input wave form on the foundation has a peak at 0.4 sec.

5. PROPERTIES OF BUILDINGS AND REQUIRED YIELD STRENGTH

By roughly dividing the properties of buildings into those which are susceptible to bending destruction and those which are susceptible to shearing destruction, the required yield strength of the respective types will be discussed.

1) BENDING DESTRUCTION TYPE

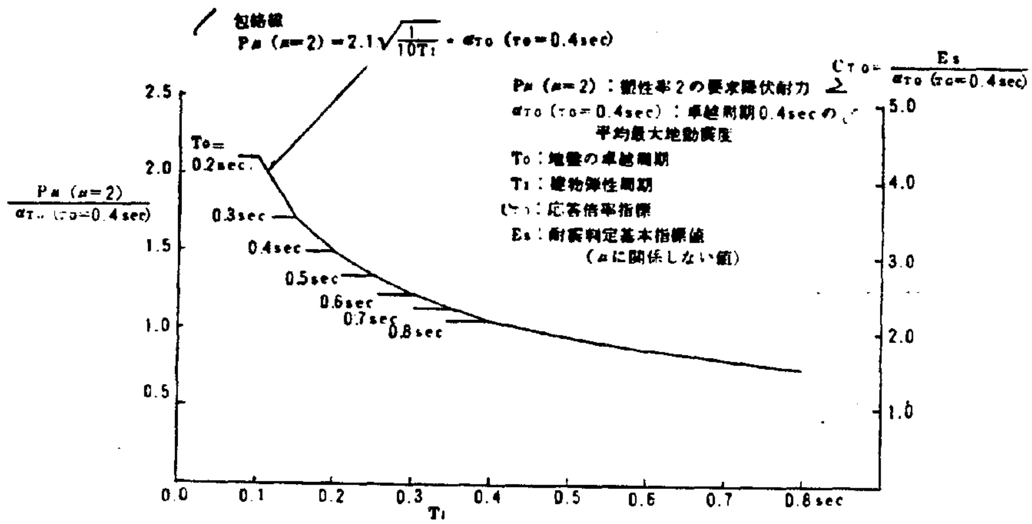
The DTRI model is adopted as the restoring force characteristic. This mo del is not complex, but expresses comparatively well theproperties of the bending destruction type reinforced concrete buildings.

The required yield strength spectrum relative to the previously described artificial earthquak group with a predominant period $TG \fallingdotseq 0.4$ sec which used this DTRI restoring force model is indicated as a ratio to the size of the seismic motions in Literature 2. In the literature, required yield spectrum is indicated relative to each of the 5 types of artificial earthquake groups and each of the 3 types of restoring force characteristic forms. However, in view of the fact that the effect by the difference in types of artificial earthquake groups was insignificant (Footnote 8), the yield strength spectrum of a 95% probability obtained from using a B type input wave form and using a $Py \fallingdotseq 3$ Pc, $T_2 \sqrt{2T_1}$ restoring force characteristic form which contributes to the maximum required yield strength, is adopted when determining the

Seismic judgement index values. Where, P_c is crack yield strength, P_y is a seismic intensity relative to the yield bearing strength, T_2 is a period obtained from yield point rigidity and T_1 is an elastic period. Also, the equivalent ductility damping obtained from the hysteretic area is $hj \fallingdotseq 0.11$. The required yield spectrum relative to $\mu = 2$ at this time, is a case of the ground with a predominant period $TG \fallingdotseq 0.4$ sec indicated in Figure 2. Examining this spectrum, it was found that the ratio of the size of the seismic motion and the required yield strength is inversely proportional to the square root of the period (T_1) of the building. Also, if the ratio of the predominant period of the ground and the period of the building is fixed even when the predominant period of the ground is changed, it is drawn that the input seismic wave form corrected the time axis by the ratio of the predominant period as mentioned when the input seismic motion was assumed. Consequently, the raio of the size of the seismic motion and the required yield strength will be similar to the case with the predominant period $TG \fallingdotseq 0.4$. On the other hand, similarly as mentioned when the input seismic motion was assumed, the square root of the predominant period of the ground and the size of the seismic motion are inversely proportional. Evidentally, when the axis of the ordinates expresses the ratio of the yield strength and the size of the seismic motion with a predominant period $TG \fallingdotseq 0.4$ sec, the ratio of the apparent size of the seismic motion and the required yield strength can be changed by the predominant period (corresponding to the period of the building) of the ground. By these manuevers, the ratio of the size of the seismic motion with a predominant period TG 0.4 sec and the required yield strength was plotted in Figure 2 against the predominant period of the respective grounds by varying the predominant period of the ground. The envelope of the chart is uniformal because the fact that the ratio of the size of the seismic motion and the required yield strength is proportional to the square root of the period of the building and the fact that the size of the seismic motion is inversely proportional to the square root of the predominant period (corresponding to the period of the building) of the ground, mutually check one another out. Incidentally, according to the assumption that the period of the building relative to the yielding point rigidity is considerably close to the required yield strength spectrum in the "Seismic Diagnostic Standards", yielding point regidity and the required yield strength relative to $\mu \fallingdotseq 2$ and 4 are plotted in Figure 3 at a probability of 85% (Literature 6). The value of the yield strength from Figure 3 takes the upper limit value of Figure 3 considering that the energy absorption by the hysteresis of the building is small. Evidentally, when the absorption of energy by the hysteresis of the building is large, the yield strength to be reserved in the building can be lowered.

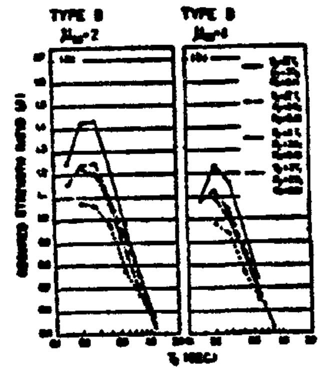
Footnote 8: When the predominant characteristics (predominant period, propagation function) of the ground is uniformal, the maximum destruction can be used as one of the yardsticks to measure the destruction force. However, as to the absolute value of the maximum acceleration, please refer to Footnote 6. Also, by moving from the A type to the D type, errors of accuracy in response spectrum increase. With a probability of 95%, the destruction force of the D type sometimes becomes stronger than the others.

Figure 2. Required Yield Strength and Response Magnification Index in Bending Destruction Type Buildings with a Plasticity Ratio 2



key-1. envelope, 2. required yield bearing strength at a plasticity ratio 2, 3. average maximum seismic intensity at a predominant period of 0.4 sec, 4. predominant period of the ground, 5. building elastic period, 6. response magnification index, 7. seismic judgement basic index value (value not related to μ)

Figure 3. Yielding Point Period and Required Yield Strength
(Literature 6)



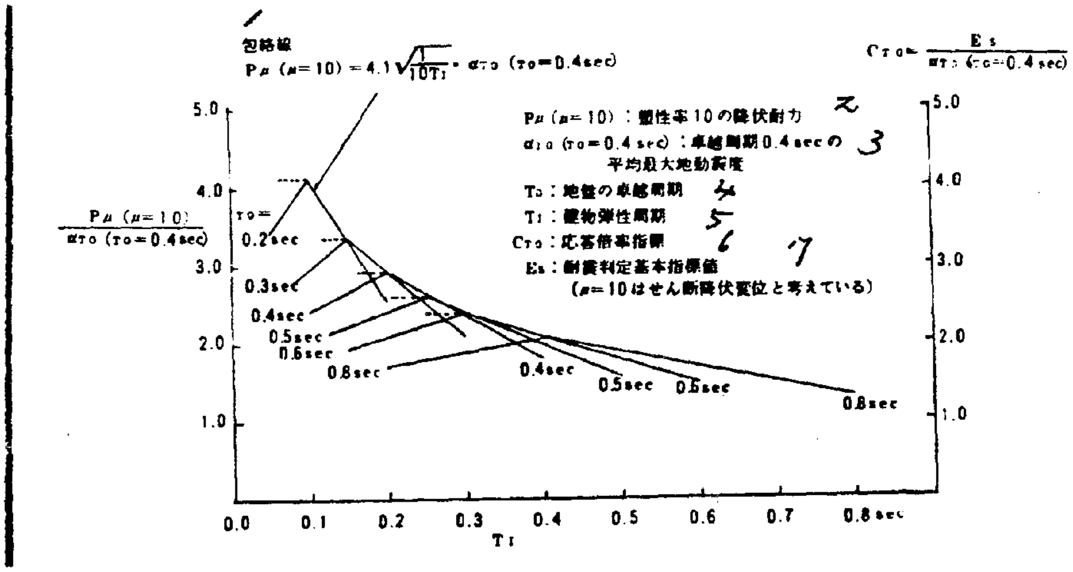
2) SHEARING DESTRUCTION TYPE

The restoring force characteristic is generically called the origin directive type.

"Seismic Diagnostic Standards" allow a deformation slightly larger than the deformation at the time of the shearing yield for the shearing destruction type buildings. Now if the ratio of the size (αTG) of the seismic motion with a predominant period TG $\fallingdotseq$ 0.4 sec and the required yield strength during yielding is computated using the same method as in 1) and using the same conditions as in Literature 2— artificial earthquake: B type, Py $\fallingdotseq$ 1.9 pc, $\mu \fallingdotseq 10$ ($\mu \fallingdotseq 1.0$ relative to yielding point deformation) and probability : 95%, it will be as shown in Figure 4. The ratio of the side of the seismic motion and the required yield strength becomes a maximum at 1/2 of the predominant period of the ground. At both sides of this point, the ratio drops roughly on a straight line. The envelope of the response magnification index takes the same form as the bending destruction type, but will hardly vary by the types of earthquakes just as in 1) where the envelope was affected by the changes in the size of the earthquake motion due to the predominant period of the ground. (Footnote 9)

Footnote 9: The destruction force of D type drops more compared to the bending destruction type.

Figure 4. Required Yield Strength and Response Manification Index of Shearing Destruction Type at a plasticity of 10



key-1. envelope, 2. required yield bearing strength at a plasticity ratio of 10, 3. average maximum seismic intensity at a predominant period of 0.4 sec, 4. predominant period of the groun, 5. building plastic period 6. response magnification index, 7. seismic judgement basic index value ($\mu \fallingdotseq 10$ is considered to be the shearing yield displacement).

6. SEISMIC JUDGEMENT BASIC INDEX

1) SEISMIC JUDGEMENT BASIC INDEX E_s FOR STANDARD EQRTHQUAKE MOTION

(1) Method for Calculation of Seismic Judgement Basic Index

After obtaining the ratio of the required yield strength to the size of the earthquake motion from Figure 2 and Figure 4, the response magnification index can be obtained from multiplying the ratio by Firmness Index. The response magnification index obtained from multiplying the required yield strength in Figure

2 by the Firmness Indes value $F \doteq 2.1$ at a plasticity ratio of 2, and the response magnification index obtained from multiplying the required yield strength in Figure 2 by a firmness index of 1.0, can be read on the right side axis in Figure 2 and Figure 4 respectively. If this response magnification index is multiplied by the input seismic intensity, a seismic judgement basic index can be obtained.

In the following, the seismic judgement basic indexes for the first and second diagnoses will be established.

(2) First Diagnosis

Since the response magnification index is thr function of a period of a building, the period is estimated. Incidentally, the objective building is of a shearing destruction type because the shearing destruction type gives a larger seismic judgement basic index than the bending destruction type. "Seismic Diagnostic Standards" prescribe that the element angle is 1/500 level at the wall shearing yield point, the destruction point of the extremely short column. Now, if a one layer building can endure a seismic intensity of R, the period at the destruction point (Yield point period) beill be

$$T = 0.2\sqrt{S / R} \cdots\cdots\cdots\cdots \quad (3)$$

S is a story height of 3.3m, and 330/500—0.66 cm. On the assumption that $R \doteq 0.9$ the period at the destruction point is 0.172 sec, and theelastic period will be around 0.1 sec. Therefore, the response magnification index at 0.1 sec can be found from Figure 4 as C_{TG} 4.1. From $\alpha_{TG} \doteq 0.23$, the seismic judgement basic index Es becomes $0.31 \times 0.23 \doteq 0.94$. This value is designated as 1.0 which is larger than the maximum seismic judgement basic index 0.8 obtained in the second diagnosis in (2), and relates to the reliability of the both diagnostic methods.

(3) Second Diagnosis

a) Bending Destruction Type

The seismic judgement basic index E_S is as follows.

$$E_s = 4.3\sqrt{1/10 \cdot T} \quad \alpha_{T0} = 0.7\sqrt{1/5 \cdot T} \text{ (however, } T = \frac{TG}{2} \text{ when } T \leqq \frac{TG}{2} \quad \cdots\cdots (4)$$

TG is thepredominant period of the ground, and the results from "Ground and Geological Survey" (Literature 4), a reaearch resport commissioned by Shizuoka prefecture, will be used.

"Seismic Diagnostic Standards" prescribed that the element angle is 1/50, the level athe column bending yield point. If the inside emasurement is 2.8m, 1.9m is obtained. The yield point period is given from Equation (1).

$$T = 0.61 \text{ sec} \qquad C = 0.2$$

$$T = 0.50 \text{ sec} \qquad C = 0.3$$

$$T = 0.41 \text{ sec} \qquad C = 0.45$$

Evidentally, it is not likely that an elastic period will be below 0.2 sec. The elastic period of the building is assumed as follows.

$$\begin{array}{lll} T = 0.2 & & N = 1 \\ T = 0.2\sqrt[4]{3(N-1)} \ \text{sec} & \text{N: number of stories} & N \geqq 2 \end{array} \quad \cdots\cdots\cdots\cdots\cdots (5)$$

However, this will not apply when judgement is given based upon the precisely calculated elastic period and yield point period. Also, T does not express a normal elastic period of a building, but it is a period determined from a bending yield type vertical elemet group which gives an important effect on the seismic judgement basic index (Footnote 10).

The period of a multilayer structure is given on the short side, which has a meaning in that the concentration of deformation in a specific layer is prevented. In the"Seismic Diagnostic Standards", this point is, as previously described, taken into account as a coefficient $(n+1)/(n+1)$ when calculating Eo. However, learning from Literature 8, a value more on the safe side shall be given.

b) Shearing Destruction Type

The seismic judgement basic index Es is as follows.

$$E_s = 4.1\sqrt{1/10\cdot T}\ \alpha_{T0} = 0.8\sqrt{1/7\cdot T_1}\ \text{(however, } T = \frac{TG}{2}\text{ when } T \geq \frac{TG}{2}\text{)} \ldots\ldots (6)$$

Incidentally, for the seismic judgement basic index, an envelope on the safe side within the range of $T > \frac{G}{2}$ is used. Even when $T < \frac{TG}{2}$, the seismic judgement basic index shall not be dropped but will be the dotted line as indicated in Figure 4. Therefore, it becomes the same form as in the bending destruction type.

Considering the difference in reliabilities of the first and the second diagnoses and the ratio of the periods of the shearing destruction type building and the bending destruction type building in the second diagnosis, the period of the shearing destruction type one story building used in the 2nd diagnoses is estended $\sqrt{2}$ times compared to the one used in the first diagnosis, and the elastic period of the building is assumed as follows.

$$\begin{array}{lll} T = 0.14\ \text{sec} & N = 1 & \\ T = 0.14\sqrt[4]{3(N-1)}\ \text{sec} & N \geq 2 & \end{array} \quad \ldots\ldots\ldots\ldots (7)$$

T does not express a normal elastic period of a building, but is a period determined from a shearing yield type vertical element group which gives an important effect on the seismic judgement basic index (Footnote 10).

c) Compound Destruction Type Combining Bending and Shearing Destruction Types

Es of this building is determined taking into consideration whether or not the destruction mode of the vertical element that gives an important effect on the final seismic judgement basic index will be bending destruction or shearing destruction. When difficult to judge the mode, the calculation will be made based upon the shearing destruction mode.

Footnote 10: In the philosophy of "Seismic Diagnostic Standards", there is a point of view that the shortening of the period of a building raises the yield strength of the building.* It is is supposed that the yield strength of a building is inversely proportional to the square root of the building, the seismic judgement basic index values can be determined by a specific period when the number of the layers of the building is the same, since the seismic judgement basic index values are not related to the period of the building.

*As an extreme view point, rigidity and yidld strength are propotional. According to this way of thinking, the seismic judgement basic index must be determined at a point where the period of a building is as long as possible.

Also, from these maneuvers, first the yield deformation of a building is established, then the plasticity ratio is computated from that point.

2) SEISMIC JUDGEMENT BASIC INDEX E_s FOR AN IMAGINARY TOKAI EARTHQUAKE MOTION

(1) Allowable Lower Limit Value of Seismic Judgement Basic Index Es

This seismic judgement basic index can beobtained similarly to the seismic judgement basic index for the standard eqrthquake motion at αTG=0.45. For the imaginary Tokai Earthquake, the lower limit value of the seismic judgement basic index will be sought because of the view described below. Incidentally, this will not be applied to important buildings.

a) Bending Destruction Type

Now, allowing a large damage, a deformation of $\mu \simeq 4$ level is assumed. Precisely, allowing larger damage and allowing a total deformation up to 200%, the required yield strength will be lowered approximately by 20% according to Literature 2. If firmness index F is fixed, it is possible to allow a seismic judgement basic index which is low by 20% as a lower limit value. (Footnote 11).

b) Shearing Destruction Type

In the light that the "Seismic Diagnostic Standards" set a condition that al allowable deformation is double the deformation during yielding, supposing this condition is applied to the total range of period, it is possible similarly to allow the seismic judgement basic index which is low approximately by 20% as the lower limit value.

From above, it is permissible to seek a seismic judgement basic index by designating apparently $\alpha TG \simeq 0.36$ when the size of the earthquake motion is really $TG \simeq 0.45$ (Footnote 12).

(2) First Diagnosis

$$1.0 \times \frac{0.36}{0.23} = 1.57$$

Es Shall be established as 1.6

(3) Second Diagnosis

Views of (1).a) and b) are adopted only when a building is in the vicinity of the hypercenter.

Footnote 11: This value can be considered not to be directly corresponding to the plasticity ratio in "Seismic Diagnostic Standards", but to the absolute displacement. However, such a position may cause some problems when the period of a building is long, ie, when the predominant period of the ground is long. Also, from another view, it can be thought that the period of the building is lengthened by $\sqrt{2}$ times.

Footnote 12: Up to now, surface acceleration is considered to mean the input acceleration of a building. However, there may be cases where the surface acceleration can be directly discounted by the effect of the interaction of the ground and the building or by allowing the destruction of the ground to be taken into account. (Literature 8)

7. SEISMIC JUDGEMENT INDEX VALUE E_T

There have been many cases to date where the destruction force of an earthquake motion was augumented by terrain conditions such as a precipice, a non-

-concordant layer and a local height. Here, this factor is given as C_G and incorporated in the seismic judgement index value.

Also as menticned in 6 , 2) (1) titled Allowable Lower Limit Value of Seismic Judgement Basic Index Es, this value is calculated by assuming large damage to a building and an importance factor is introduced into the calculation because of the reason to be later described.

Therefore, the final seismic judgement index value E_T will be

$$E_T = C_G \times C_I \times E_S$$

Incidentally, the earthquake motion on a ground with a sharp propagation function and a large predominancy will not be much larger in its effectiveness than the earthquake motion with a small predominancy, when a building is allowed to receive large damage to an extent (Literature 2). Nevertheless, when only slight damage is allowed, the effect of such an earthquake motion cannot be ignored. Therefore, this effect must be added to the importance factor when dealing with an important building which should not be damaged more than slightly.

8. CONCLUSION

The seismic judgement index values were calculated for both cases using a standard size earthquake motion and a tentatively imaginary Tokai Earthquake size earthquake motion. However, there are still many engineering problems to be solved at present for us to make decisions about these values. Especially, earthquake motions, grounds and interactions of buildings with grounds have many more unclarified points. Specifically, citing some of the countless problems, there is a question as to the characteristics and the size of the earthquake motions in a super earthquake such as the one imagined here, a question if characteristics of an earthquake motion change when the magnitude of the earthquake changes, a question as to the relation of these unclarified points to the characteristics of an earthquake, and a question as to how much of the earthquake motion on the surface affects the buildings. However, I believe it is not yet the stage when all these can be answered clearly.

Consequently, all the values established here must be considered provisional.

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Collection of Summaries of Scientific Speeches Presented At The Convention Of The Architectural Institute Of Japan

(Tokai) October, 1976

PROBABLISTIC STATISTIC RESEARCH ON NONLINEAR RESPONSE SPECTRA OF REINFORCED CONCRETE STRUCTURES

(Part 1, Spectra Relative to Artificial Earthquakes)

Regular Member Masaya Murakami

1. PREFACE

A general philosophy when working with seismic design and seismic judgement, is to improvise a way to hold down the structural damages to a minimal level against a moderate earthquake and to prevent the structures from collapsing or totally collapsing against a major earthquake. In this concept, there are hidden thoughts that it is not possible to design structures totally free of damages in an area with a high earthquake risk factor due to the economical restraints and that the damage to the structures designed with adequate seismic considerations will limit the response values within a certain range by the absorption of energy which is associated with the damage. On the other hand, uncertainty of earthquakes are high in every aspect. The characteristics of earthquake motions fluctuate a lot even when a building site is set up. Additionally, the characteristics of the objective structures also contain fluctuations.

Thus, it is necessary to take a probablistic statistical approach when damages are predicted by means of earthquake analytical techniques. This study pays attention to the effect of the fluctuant properties of earthquake motions on reinforced concrete structures, but disregards the fluctuation of the characteristics of the structures. In order to perform the probablistic statistic seismic response analysis, it is necessary to presume suitable statistic models for anticipated earthquake motions. In this study, non-stationary filtered white noise which is used by many research workers, is adopted. This is not a perfect model, but it reflects the major statistic characteristics of earthquake motions which have been observed to date. Use of a model for seismic response analysis is thought to allow a more practical prediction than use of specific actually measured earthquake records. Precisely, in the study, 20 models were made for each of the 5 types of oscillation systems which imitated reinforced concrete structures. Response spectra relative to Period T_1 which corresponds to initial rigidity was sought. Furthermore, based upon the concrete numerical values from the previously described seismic design idea proposed by Dr. Umemura (Literature 1) and following the Literatures 2 and 3 where the yield strength required of the structures was calculated, the study reveals that the yield strength required of the structures characterized with a restoring force which exhibits 4 types of bending yield, depends on the yield point regidity and the equivalent ductile damping obtained from the hysteretic area of that time.

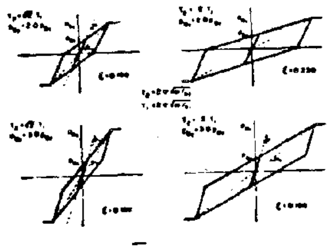
2. ARTIFICIAL EARTHQUAKES

Describing the outline of the method for making 20 models for each of the 5 types of artificial earthquakes (A,B, Bo_2,C and D), white noise was made non-stationary by using the 4 types of time intensity functions proposed by Jennings and associates. Ground characteristics were given through the response of a mono-mass system (period: 0.4 sec, damping constant: 0.6, however, damping constant 0.2 for Bo_2) and the long period components were removed through the response of a mono-mass system (period: 7 sec for A,B and Bo_2 types, period: 2 sec for D and C types, damping constant : $1/\sqrt{2}$). Finally, axial correction was made in accordance with the method of Berg and Housner (Literatures 2 and 3). The earthquake motions were aimed at earthquakes D,C, B and A in order of magnitudes from large to small, and B type earthquake motion corresponds to the earthquake group which included the N-S component of the 1940 El Centro Earthquake. Incidentally, the yield strength of structures was made non-dimensional by the average maximum acceleration of each type of earthquake motion and the mass of the oscillation system.

3. STRUCTURE MODELS

The restoring force characteristics of the structure models were 4 types of D-TRI restoring force characteristics which express bending yield mode. Figure 1 indicates the equivalent ductile damping constant which can be obtained from the parameter values of these 4 restoring force characteristics, yielding point rigidity and hysteretic area. Incidentally, the restoring force characteristics reduce rigidity at a ratio of the sum of the positive and negative directional maximum displacements and the displacement which is two times the yield displacement, when the response displacement exceeds the yield displacement. However, the value corresponding to the yield point rigidity after the fall of rigidity and the hysteretic area, will always be constant. (Figure 1)

Figure 1

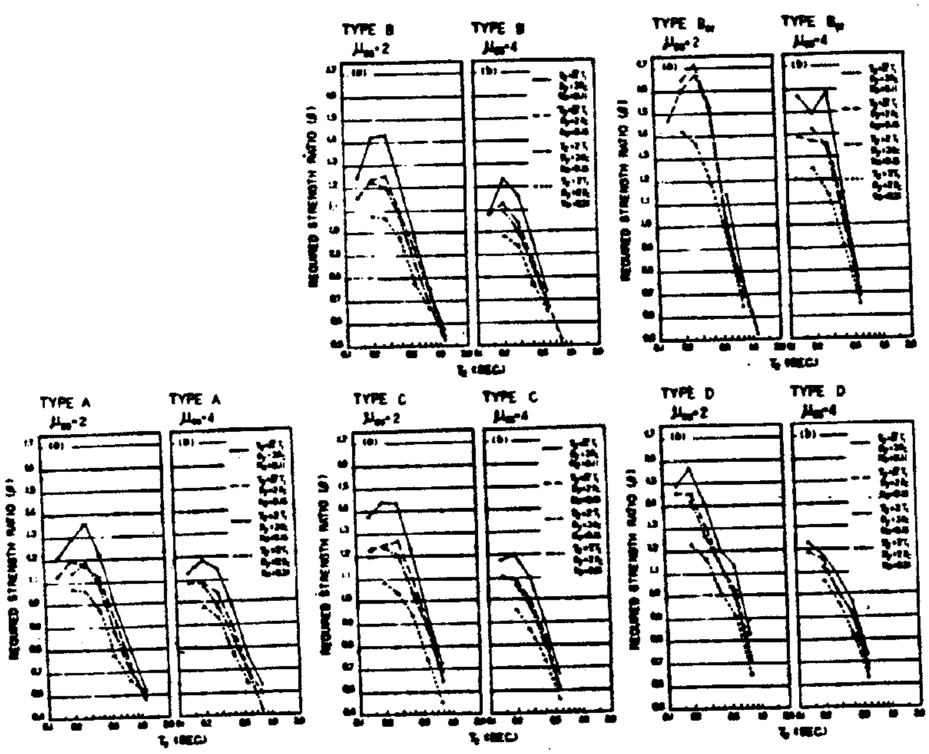


4. RESULTS AND REVIEW

A series of reference materials are arranged by non-dimensional yield strength ratio and plasticity ratio. Figure 2 indicates the yield strength ratio at which the 4 types of models suffered from 5 types of artificial earthquakes will satisfy the idea of the seismic design at a 85% probability, relative to period T_2 which is equivalent to the yield point rigidity. The procedure used to plot the chart will be introduced in Part 2 of the report.

Incidentally, the previously described idea of a seismic design is for a plasticity ratio of 2 for a moderate eqrthquake (maximum acceleration 0.3 g) and a plasticity ratio of 4 for a major earthquake (0.45 g). Therefore, Figure 2 indicates the yield strength ratio required to maintain structure models within a plasticity ratio of 2 and a plasticity ratio of 4 at a 85% probability. The values obtained by multiplying these yield ratios by 0.30 and 0.45 respectively are the yield strength required of the structures. In both cases of elasticity ratio 2 and elasticity ratio 4 in each graph, it is shown that the response is governed by the equivalent ductile damping constant. Also, assuming that the ground predominant period is 0.4 sec, the probablistic maximum deformation is given when the period which corresponds to the yield point rigidity after the fall of rigidity approaches to 0.4 sec. In the case with a plasticity ratio of 2, the response reaches the maximum when the period T_2 of the structures is $1/\sqrt{2}$ of the ground predominant period, ie, 0.288 sec, while in the case with a plasticity ratio of 4, the maximum response is seen when T_2 is 1/1 of the predominant period, ie, 0.2 sec. Likewise, the Bo_2 type earthquake which contains many 0.4 sec period components remarkably affects structures with a period shorter than the predominant period, and it demands larger yield strength. Furthermore, these analytical results can be also used when the predominant period changes if a period ratio relative to structures is taken into account.

Figure 2



5. CONCLUSION

Results of the probablistic statistic analysis of spectra revealed:

1) In reinforced concrete structures, the response value is governed by the equivalent

ductile damping constant which is obtained from the yield point rigidity and the hysteretic area, 2) Results contrary to common sense such as seen in the specific earthquake response values, ie, the results which dictate that the larger the yield strength or larger the damping, larger the response value, will no longer be observed, 3) Structures with a period shorter than the ground predominant period indicate larger response, 4) In the idea of the seismic design, limit of the plasticity ratio and the limit of the yield strength have the same measning. Difference between the equivalent ductile damping 0.11 and 0.24 contributes to the difference in the required yield strength by a maximum 30%, which is rather a small value compared to the ratio of approximately 2 times in the case of the required yield strength obtained from the stationary oscillation response.
Before closing this paper, I would like to mention that this study owes much to the joint research (Literatures 2 and 3) conducted with Professor Penzien at University of California (Berkeley) when the author took part in the Japan/US Cooperative Research "Earthquake Engineering Stressing Earthquake Resistivity of School Buildings" cosponsored by the Japan Society for the Promotion of Science and National Science Foundation of America. I would like to express my appreciation to all who were involved.

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University of Chiba

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Part 6 ESTABLISHMENT OF SEISMIC JUDGEMENT INDEX VALUES (E_T)

IN

SHIZUOKA PREFECTURE

Construction Section, Housing Department
Shizuoka Prefecture

B143a

ESTABLISHMENT OF SEISMIC JUDGEMENT VALUES (E_T) IN SHIZUOKA PREFECTURE

As a result of the seismic diagnosis, the judgement of the structural seismic index (I_S) has been made conventionally referring to a judgement index (I_{SO}) which is based upon the Tokachi Offshore Earthquake. However, in appreciation of the later investigation results concerning the magnitude of predictable earthquakes, epicentral distance and terrain and ground conditions, the seismic judgement index values (E_T) for Shizuoka Prefecture will be established as follows.

JUDGEMENT OF FIRST OR SECOND DIAGNOSTIC STRUCTURAL SEISMIC INDEX (I_S)

The judgement of the structural seismic index (I_S) will be made in accordance with Equation (1), using Structural Seismic Index (I_S) computated in Chapter 3 of the Seismic Diagnostic Standards for Existing Reinforced Concrete Buildings (published by Japan Special Building Safety Center under the editorial supervision of Building Guide Section, Ministry of Construction) and using Seismic Judgement Index Values (E_T) established in the following Section 1 or Section 2.

$$I_S \geq E_T \quad \cdots\cdots\cdots\cdots (1)$$

I_S: seismic index of structure

E_T: seismic judgement index value (from Section 1 or 2)

1. SEISMIC JUDGEMENT INDEX VALUES FOR FIRST DIAGNOSIS (E_T)

The computation of the seismic judgement index values for the first diagnosis will be performed in accordance with Equations (2) to (4), using the number of building layers, the contour of building sites and the degree of building destruction.

$E_T = 1.10 \times C_G \times C_I$ ··················· (2) for 1–2 layers

$E_T = 1.00 \times C_G \times C_I$ ··················· (3) for 3–4 layers

$E_T = 0.90 \times C_G \times C_I$ ··················· (4) for 5–6 layers

C_G: Contour of Building Site (from 3–2)

C_T: Degres of Building Destruction (from 3–3)

2. SEISMIC INDEX VALUES FOR SECOND DIAGNOSIS (E_T)

The computation of the seismic judgement index values for the second diagnosis will be performed in accordance with Equation (5), using basic seismic index values computated from epicentral distance, ground type, number of building layers, mode of building destruction, contour of building site and degree of building destruction.

$$E_T = E_S \times C_G \times C_I \quad \cdots\cdots\cdots\cdots\cdots\cdots (5)$$

Es: basic seismic index value (from 3–1)

C_G, C_I: Same as for the first diagnosis

3–1. Es: Basic Seismic Index Values

the values are obtained from Tables 2–1 to 2–5 according to districts by epicentral distance, buildings by ground type, buildings by number of layers and buildings by mode of destruction.

(1) Districts will be selected by the location of buildings on the district map in Figure 1.

(2) Ground types will be selected from Table 1 after survey of building sites.

(3) Number of layers of buildings will be confirmed.

(4) For the destruction mode of buildings, the examinor must judge whether the

buildings are of the shearing destruction type or the bending destruction type in the process of the diagnoses.

(5) The numerical values obtained at the point where the ground type and the number of layers meet one another will be designated as basic seismic index values (Es).

Table 1 Ground Type

type	predominant period	geological feature/stratum
Type 1	0.3 sec	The ground surrounding the building concerned is composed of rocks, hard gravel and others mainly consisting of the stratum made before the tertiary period.
Type 2	0.4 sec	The ground surrounding the building concerned is composed of gravel, hard clay mixed with sand, loam and others mainly consisting of a diluvium, or a more than 5m thick alluvium consisting of pebbles or gravel.
Type 3	0.5-0.6 sec	Standard ground which does not belong to any of the Type 1, Type 2 and Type 4 districts but belongs to alluvium mainly consisting of sand, clay with sand, clay and dirt.
Type 4	0.7-0.8 sec	Extremely soft ground which belongs to one of the following. 1) A more than 30m deep alluvium consisting of humus soil dirt andothers similar to these (including banking) 2) Reclaimed land which meets the following conditions. A) Land reclaimed by filling marsh and muddy sea. B) Land filled with trash and garbage, mud and others which belong to these soft and weak soil. C) Land filled approximately more than 3 m deep. D) Land reclaimed approximately less than 30 years ago.

Table 2-1 Es Values for District A

key-1. upper line: shearing destruction mode
2. lower line: bending destruction mode
3. ground by type
4. predominant period
5. number of layers
6. type 1
7. type 2
8. type 3
9. type 4

1 上段：せん断破壊形
2 (下段)：曲げ破壊形

地盤種別 3 / 卓越周期 4 / 層数 5	第 1 種 6 0.3 sec	第 2 種 7 0.4 sec	第 3 種 8 0.5 sec	第 4 種 9 0.7 sec
1	1.10 (1.10)	1.10 (1.10)	1.00 (1.00)	0.85 (0.85)
2	1.10 (0.95)	1.10 (0.95)	1.00 (0.95)	0.85 (0.85)
3	1.00 (0.90)	1.00 (0.90)	1.00 (0.90)	0.85 (0.85)
4	0.95 (0.85)	0.95 (0.85)	0.95 (0.85)	0.85 (0.85)
5	0.90 (0.85)	0.90 (0.85)	0.90 (0.85)	0.85 (0.85)
6	0.90 (0.80)	0.90 (0.80)	0.90 (0.80)	0.85 (0.80)

Table 2-2 Es Values for District B

key- same as in Table 2-1

1 上段：せん断破壊形
2 (下段)：曲げ破壊形

地盤種別 3 / 卓越周期 4 / 層数 5	第 1 種 6 0.3 sec	第 2 種 7 0.4 sec	第 3 種 8 0.5 sec	第 4 種 9 0.7 sec
1	1.05 (1.05)	1.05 (1.05)	0.90 (0.90)	0.80 (0.80)
2	1.00 (0.90)	1.00 (0.90)	0.90 (0.90)	0.80 (0.80)
3	0.90 (0.85)	0.90 (0.85)	0.90 (0.85)	0.80 (0.80)
4	0.85 (0.80)	0.85 (0.80)	0.85 (0.80)	0.80 (0.80)
5	0.85 (0.75)	0.85 (0.75)	0.85 (0.75)	0.80 (0.75)
6	0.85 (0.75)	0.85 (0.75)	0.85 (0.75)	0.80 (0.75)

Table 2-3 Es Values for District C

key-same as in Table 2-1

/ 上段：せん断破壊形
2(下段): 曲げ破壊形

地盤種別 3 / 卓越周期 4 / 層数 5	第 1 種 6	第 2 種 7	第 3 種 8	第 4 種 9
	0.3 sec	0.4 sec	0.5 sec	0.7 sec
1	0.95 (0.95)	0.95 (0.95)	0.85 (0.85)	0.70 (0.70)
2	0.90 (0.80)	0.90 (0.80)	0.85 (0.80)	0.70 (0.70)
3	0.85 (0.75)	0.85 (0.75)	0.80 (0.75)	0.70 (0.70)
4	0.80 (0.70)	0.80 (0.70)	0.80 (0.70)	0.70 (0.70)
5	0.75 (0.70)	0.75 (0.70)	0.75 (0.70)	0.70 (0.70)
6	0.75 (0.65)	0.75 (0.65)	0.75 (0.65)	0.70 (0.65)

Table 2-4 Es Values for District D

key-same as in Table 2-1

/ 上段：せん断破壊形
2(下段): 曲げ破壊形

地盤種別 3 / 卓越周期 4 / 層数 5	第 1 種 6	第 2 種 7	第 3 種 8	第 4 種 9
	0.3 sec	0.4 sec	0.5 sec	0.7 sec
1	0.85 (0.85)	0.85 (0.85)	0.75 (0.75)	0.65 (0.65)
2	0.80 (0.75)	0.85 (0.75)	0.75 (0.75)	0.65 (0.65)
3	0.75 (0.70)	0.75 (0.70)	0.75 (0.70)	0.65 (0.65)
4	0.70 (0.65)	0.70 (0.65)	0.70 (0.65)	0.65 (0.65)
5	0.70 (0.60)	0.70 (0.60)	0.70 (0.60)	0.65 (0.60)
6	0.70 (0.60)	0.70 (0.60)	0.70 (0.60)	0.65 (0.60)

Table 2-5 Es Values for District E

key-same as in Table 2-1

1 上段：せん断破壊形
2(下段): 曲げ破壊形

地盤種別 3 卓越周期 4 層数 5	第 1 種 6 0.3 sec	第 2 種 7 0.4 sec	第 3 種 8 0.5 sec	第 4 種 9 0.7 sec
1	0.70 (0.70)	0.70 (0.70)	0.65 (0.65)	0.55 (0.55)
2	0.70 (0.60)	0.70 (0.60)	0.65 (0.60)	0.55 (0.55)
3	0.65 (0.60)	0.65 (0.60)	0.65 (0.60)	0.55 (0.55)
4	0.60 (0.55)	0.60 (0.55)	0.60 (0.55)	0.55 (0.55)
5	0.60 (0.55)	0.60 (0.65)	0.60 (0.55)	0.55 (0.55)
6	0.60 (0.50)	0.60 (0.50)	0.60 (0.50)	0.55 (0.50)

3-2 C_G: Contour of Building Site (Terrian Index)

C_G is selected from Table 3 according to the contour of the building sites. In this case, precipice is judged in accordance with Caluse 10 of Shizuoka Prefecture Building Standard Regulations.

Table 3

contour of building site	precipice	place where supporting ground is extremely tilted	local height	others
C_G	1.25	1.25	1.25	1.0

3-3 G_I: Degree of Destruction of Buildings (Importance Factor)

G_I is selected from Table 4 depending upon the level of the allowable damage to the buildings by an earthquake.

Incidentally, prefectural government buildings, hospitals, buildings which serve as a disaster control base, or a place of evacuation and rescue activities during an emergency shall be desirably rated 1.25 in terms of G_I.

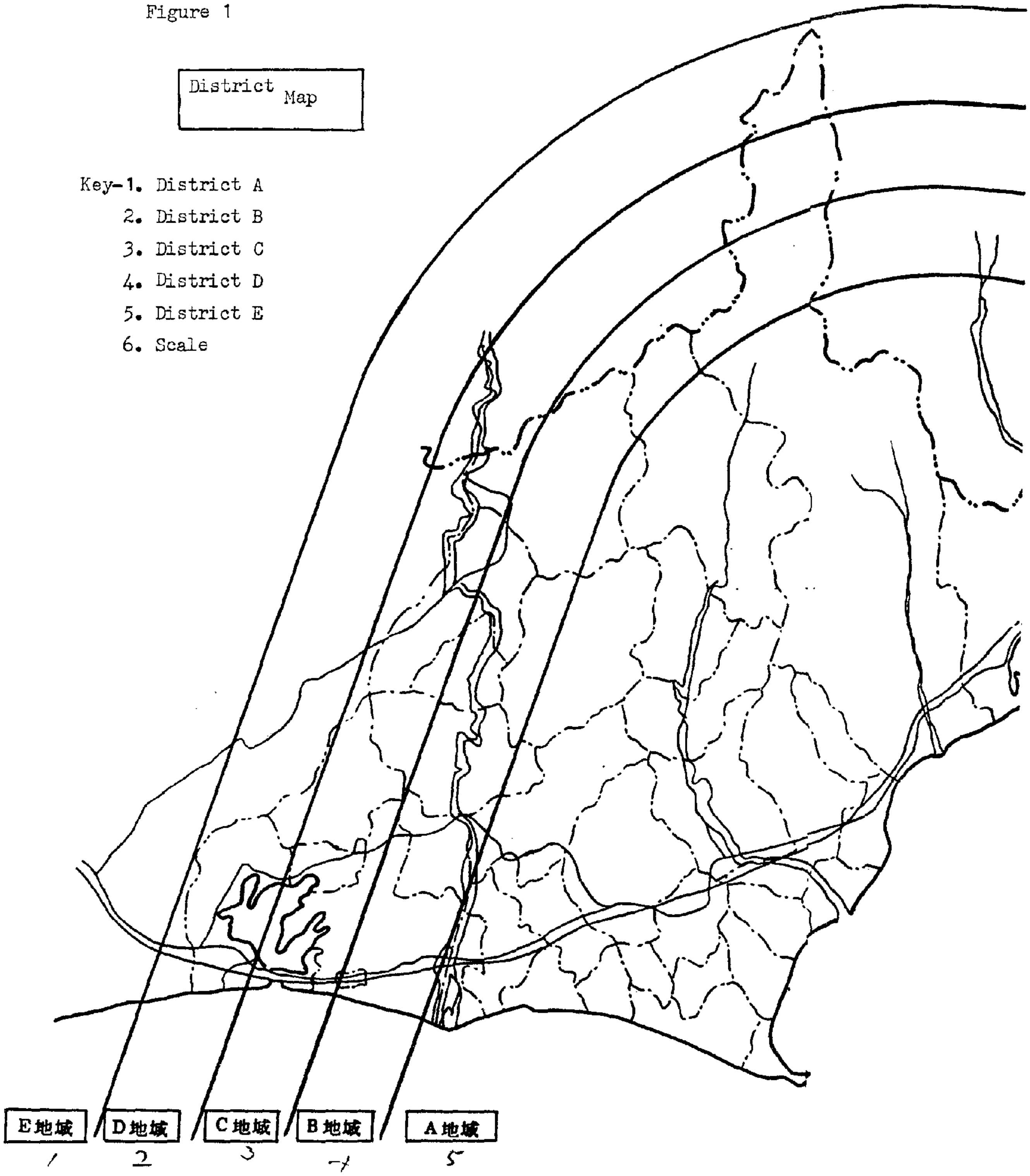
Table 4

degree of destruction of buildings	Buildings suffer only slight damages during earthquakes, and usable afterwards	Buildings suffer considerable damage but will not collapse
G_I	1.25	1.0

Figure 1

District Map

Key–1. District A
2. District B
3. District C
4. District D
5. District E
6. Scale



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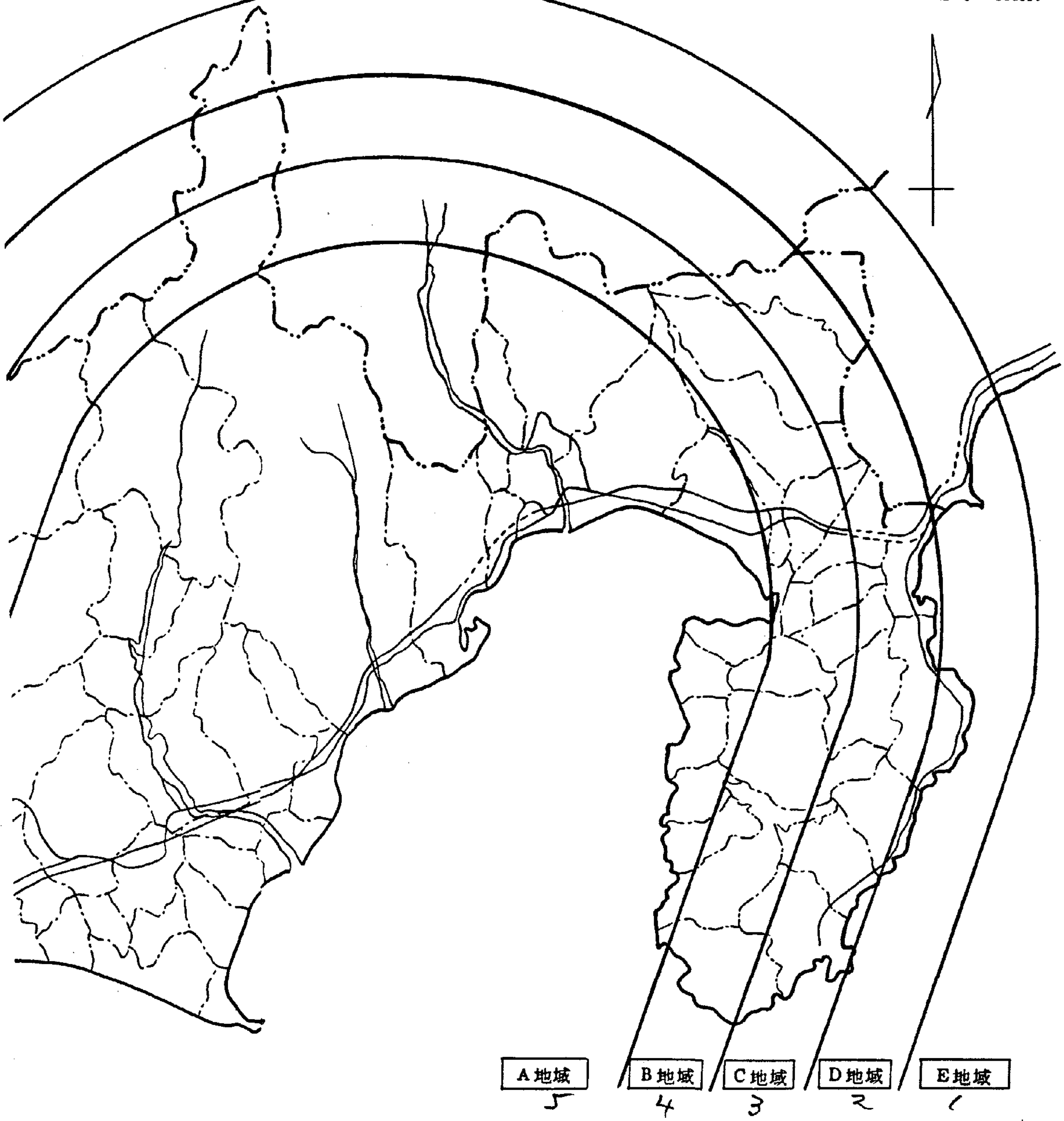

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Methods for Determining Earthquake-Resistance and Earthquake-Proof Designs of Pre-Existing Steel-Skeleton Construction.

Urban Housing Section, Shizuoka Prefecture

Introduction.

On August 7, 1979, in accordance with special legislation for the prevention of damage due to large-scale earthquakes, Shizuoka Prefecture was designated as an area to be subjected to reinforcement against these damages.

Accordingly, the prefecture, as well as each city, town, and village, formulates plans for emergency procedures to minimize damage and organize shelters, based on the policies stipulated by the National Government. Furthermore, those in charge of special buildings housing an unspecified but large number of people or plants employing more than 1,000 workers or handling dangerous substances are required to prepare important emergency plans on such things as disaster drills. These disaster prevention plans are to be formulated in 1979.

As for the measures concerning earthquake damage to buildings, the criteria for determination of earthquake-resistance of wooden and steel-reinforced concrete structures and guidelines for earthquake-proofing designs have already been determined and some are already put into practice.

On the other hand, studies on earthquake-resistance of steel-skeleton structures have also been actively conducted. In June 1978, the National Government issued a standard for evaluation of earthquake-resistance of pre-existing steel-skeleton structures.

As stated earlier, Shizuoka Prefecture has been placed in an unique position with its entire area designated to be subjected to reinforcement according to the special legislation for prevention of damage caused by large-scale earthquakes. Accordingly, the prefectural government recognizes the need to improve,at the earliest date, the earthquake-resistance of steel-skeleton structures within the prefecture. It also recognizes the need to pay attention to special buildings which house an unspecified but large number of people and those to be used for disaster prevention, evacuation, and rescue activities.

In view of the above-stated situations, the prefectural government considers it necessary to formulate methods for evaluation and improvement of earthquake-resistance of pre-existing steel-skeleton structures. Subsequently, these methods were created under the guidance of Professors Tsutomu Kato and Kosuke Akiyama of Tokyo University.

This booklet was based on the "Standard for Determination of Earthquake-Resistance of Pre-Existing Steel-Skeleton Structures and Guidelines for Their Improvement" (published by the Japanese Society for Prevention of Architectural Damage due to Disasters) which was issued by the National Government. It describes the methods for determining the earthquake-resistance of pre-existing steel-skeleton structures within the prefecture and methods for their improvement.

As to the descriptions of the formats and operations of the input data in a computerized system, a separate booklet was prepared by the Japan Telegraph and Telephone Public Corporation which kindly cooperated with Shizuoka Prefecture for determination of earthquake-resistance of steel-reinforced concrete buildings.

It is sincerely hoped that, by applying these methods, earthquake-resistance of steel-skeleton structures will be greatly improved.

December 1, 1979.

TABLE OF CONTENTS

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Methods for Determining Earthquake-Resistance and Earthquake-Proof Designs of Pre-Existing Steel-Skeleton Structures, Shizuoka Prefecture. - December 1, 1979

Part I. Methods for determining earthquake-resistance of medium- to low-building.

Chapter 1. Basic items.

1. Basic principles.

The present methods apply to determination of earthquake-resistance of structural components of pre-existing steel-skeleton construction of medium- to low-height.

The determination of earthquake-resistance of the structural components in this literature is expressed in indices (numerical figures) and are intended to determine the possibility of collapse of the structure in case of earthquakes.

2. Criteria of application.

The architecture to which the present methods apply is, as a rule, defined as steel-skeleton structures which have eave heights measuring less than 31 meters.

However, buildings with large spanned arches such as gymnasiums and those buildings which are complexes of steel-reinforced and steel-skeleton steel-reinforced structures or those makedly different from ordinary steel-skeleton construction in their scale and form are excluded from these criteria.

3. Preliminary investigation.

A preliminary investigation is conducted to determine if the methods are applicable.

4. Determination of earthquake-resistance of the structure.

The earthquake-resistance of the structure is determined by applying equation (1) (shown below) using structural index VR evaluated in 4.1 and earthquake input index VI determined in 4.2.

$$VR > VI \qquad (1)$$

4.1. Structural earthquake resistance index VR.

Structural earthquake-resistance VR is evaluated by equation(2) applying the index showing the volume of energy absorbed by the entire structure of the respective floor before its collapse.

$$VR = Q \times V \times S \qquad (2)$$

where Q : quality index (according to Section 1, Chapter 2)

V : standard earthquake-resistance index (according to Section 2, Chapter 2)

S : form index (according to Section 3, Chapter 2).

Standard earthquake-resistance index V is evaluated for the direction of beams and girders on each floor but quality index Q and form index S are independent of the floor and direction.

If quality index Q is unsatisfactory, the determination procedure is interrupted.

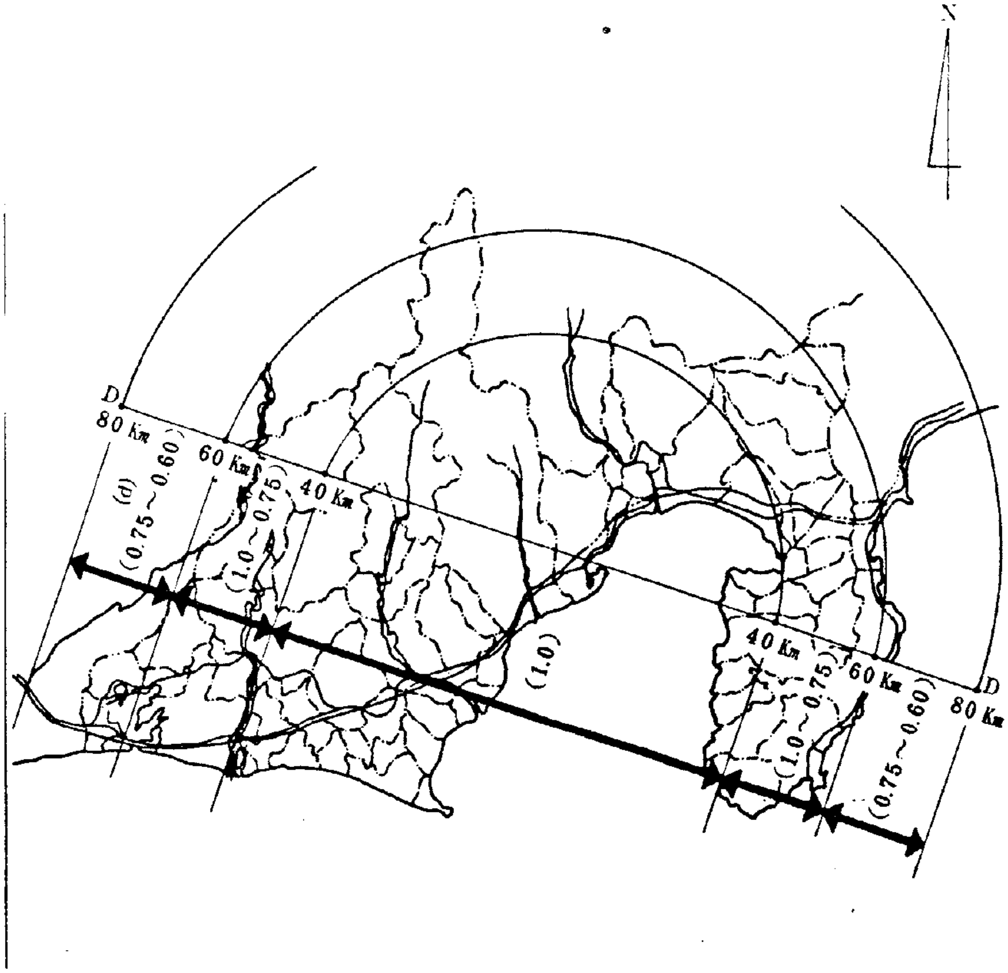
4.2. Earthquake input index VI.

Earthquake input index VI is an index indicating the size of the input energy of the earthquake and corresponds to the basic volume of the input energy, ground type, and the extent of the seismic activity as follows:

$$VI = d \times V10$$

d : a coefficient for distance (km) from the seismic source and shown as a numeral in Figure-1. $D \geq 40$km is found on the direct line corresponding to the specific value of D.

Figure 1. The value of d.



V10: Value shown in Figure-2 for each ground type (1, 2, 3, or 4).

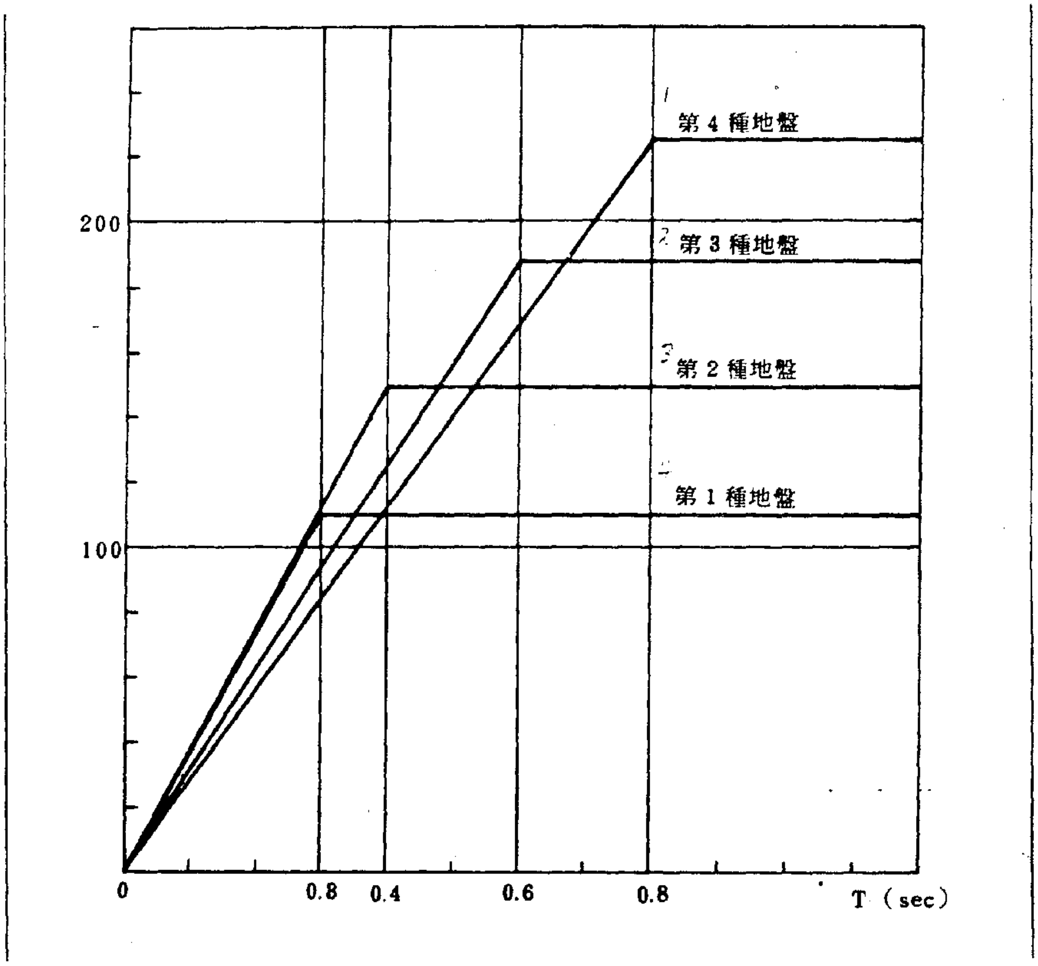


Figure-2. Value for V10

T indicates the primary specific period of the building.

1. ground type 4, 2. ground type 3, 3. ground type 2, 4. ground type 1.

1. The present part applies only to buildings used for purposes such as business offices. For buildings with large spanned arches such as gymnasiums, refer to the "Methods of Evaluation of Earthquake-resistance of Buildings such as Gymnasiums."
2. Index V1 is computed from the distance the building is from the seismic source, the ground type, and the primary specific period of the same building.

3. The primary specific period of the building is computed applying formula (1), (2), or (3), according to the structural form of the building:

i) for a pure rigid structure, $T = RT$ (sec)

$$\begin{aligned} N < 5 &: RT = 0.2N \\ N \geq 5 &: RT = 0.57 + 0.087\ N \end{aligned} \qquad (1)$$

ii) pure strut structure : $T = BT$ (sec)

The pure strut structure is one which resists horizontal forces with an axial strut (hereafter called strut) alone. BT is computed using the following equation:

$$BT = 0.71\ RT \qquad (2)$$

iii) Rahmen-strut mixed structure : $T = MT$ (sec)

The Rahmen-strut mixed structure is defined as that with the Rahmen and strut coexiting in a single skeletal plane or one where the two structures are arranged in parallel.

$$MT = \frac{RT}{\sqrt{2 - \bar{r}}} \qquad (3)$$

(codes) N: the number of floors in the building.

$\bar{r}$: the average of the Rahmen ratio r of each floor.

(Notes) Equations (1), (2), and (3) are based on those shown in B'1, Chapter 2 of the "Standard for Determination of Earthquake-Resistance of Pre-existing Steel Skeleton Structure" (edited by Ministry of Construction, Department of Housing, Section for Constructive Instruction).

4. The ground type is determined on the basis of the results of drilling investigations applying Figure-3 Table for ground types (1) and (2) (shown at the end of the volume) and Table 1, Table for Ground Types.

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表1　地盤種別表（建設省告示第1074号）

種別	地質・地層
第1種	地盤が当該建築物の周囲にわたって、岩盤、硬質砂れき層、その他主として第3紀以前の地層によって構成されているもの
第2種	地盤が当該建築物の周囲相当の範囲にわたって砂れき層、砂混り硬質粘土層、ローム層、その他主としてこう積層によって構成されているもの又は厚さがおおむね5m以上の砂利層もしくは砂れき層の沖積層によって構成されているもの
第3種	第1種、第2種及び第4種のいずれにも属さない区域の基準的な地盤で砂層、砂まじり粘土層、粘土層、泥土など主として沖積層に属するもの
第4種	著しく軟弱な地盤で、つぎのいずれかに該当するもの 1）腐植土、泥土、その他これに類するもので構成されている沖積層（盛土も含む）で、その深さがおおむね30m以上のもの 2）つぎの各条件に適合する埋め立て地 A）沼沢、泥海などを埋め立てた場所であること B）埋め立て土が、ごみ、泥土、その他これらに類する軟弱土であること C）埋め立て深さが、おおむね3m以上あること D）埋め立てられてからおおむね30年を経過してないもの

Table 1. Table for Ground Types (Ministry of Construction Announcement 1074).

1. types, 2. ground texture, 3. ground layer, 4. type 1, 5. the ground around the building is composed of rocks and a gravel layer, or composed mainly of a ground layer dated prior to the third century, 6. type 2, 7. the ground under and around the building is composed of a gravel layer, a hard clay layer, mixed with gravel, or a loam layer; mainly composed of a diluvium formation; or composed of a gravel layer or its alluvial layer measuring more than 5 m in thickness. 8. type 3, 9. the standard ground type of the area not belonging to type 1, 2, or 4; it is mainly composed of an alluvium formation of sand, clay mixed with sand, clay, and mud, 10. type 4, 11. markedly soft ground belonging to one of the following: 1) humus soil, mud, or an alluvial layer composed of these (including banking); its depth measures approximately 30m or more; 2) reclaimed land meeting the following conditions; A) land reclaimed from muddy seas, shallow lakes, or swamp, measuring over 30m in depth, B) a land fill composed of refuse, muddy soil, or soft earth, C) the depth of the reclaimed land measures more than 3m in depth, D) the time lapse since reclamation is less than 30 years.

Chapter 2. Computation of structural earthquake-resistance index VR.

Section 1. Evaluation of quality index Q.

1. General evaluation.

2. Quality index Q (hereafter called index Q) is used to evaluate the quality of the structural frame and the effects of its aging. It is based on an actual survey.

2. The actual survey is based on a survey of the "important items" and the "general items".

3. The survey of the "important items" includes an evaluation of these items which are considered to have extremely significant effects on resistance and deformation characteristics of the structural frame. If even a single item in the survey fails to meet the standard, index Q is deemed "unsatisfactory". Otherwise, the survey result is included in the computation of the standard earthquake-resistance index V described in Chapter 2, Section 2.

4. The survey on the "general items" includes those items which are considered to affect the general quality of the structural frame. The comprehensive result of this survey is used to rate index Q into 4 classes (1.0, 0.9,0.8, and unsatisfactory).

Quality index Q (hereafter abbreviated to index Q) is an index to evaluate the quality affecting the resistance or deformation characteristics or the effects of aging of the structur al frame. The value assigned to the index is determined by the results of a survey of the structural frame.

The structural frame of the building is observed directly while various tests are performed. Based on these studies, an actual investigation is conducted.

(Important items) i) A survey is made to observe if the conditions of the general architectural frame, parts, and connecting sections differ greatly from the specification of the architectural plan. If the discrepancies from the architectural plans are within a certain standard range, the survey results are reflected in the computation of standard anti-seismic resistance index V but the evaluation of index Q is not taken into consideration. If, however, the discrepancies are large, index Q is expressed as "unsatisfactory", while the content of the survey is detailed in the final evaluation and the survey process is interrupted. ii) The purpose of the second evaluation is to observe if the resistance of each part of the structural frame has been much reduced due to the construction method or aging process. If the resistance of each structural part observed in the actual investigation has not been much reduced from what is sufficient to be earthquake-resistance, the data are handled in the manner of i). If, however, the resistance of each structural part does not meet a certain standard, index Q is deemed "unsatisfactory" and the evaluation process is interrupted, while the fianl determination is based on the quality of the part rather than the anti-seismic evaluation of the frame of its structural dynamics.

The "general items" concern the general qualities of the structural frame. Many items by nature cannot simply be quantified from the survey results.

For this reason, the present evaluation process is based on a technological determination comprising the entire test results and gives a final numerical rating.

In the evaluation of index Q which is determined from the result of the study of the "general items", however, the structural frame which deviates greatly from the standard quality is treated separately. In such an instance, no numerical value is given but, instead, is simply judged "unsatisfactory".

Computation of index V appears unnecessary for the structural frame with its Q index judged "unsatisfactory": the anti-seismic determination based on such an index lacks reliability or is even meaningless in some instances. Therefore, no numerical value is given to the Q index. In such instances, the anti-seismic determination process is interrupted but resumed after appropriate treatment.

2. Actual survey.

2.1. Survey items.

The survey items includes the following 8.

1) General shaft construction,
2) column materials,
3) beam materials,
4) shaft strut materials,
5) column-beam connections,
6) welded joint,
7) fastener joint
8) column base and foundation

The above 8 items are investigated in an actual survey.

The shaft struct materials in this section include light materials such as round steel and angle steel. The survey involves these materials as well as their gusset plates and connections to columns and beams.

When large members such as H bars or steel pipes are used as struts, their members and joints are evaluated by the method used for column materials and their end connections by the methods for column-beam connections. The column joints with drastic changes in the cross sections are also evaluated according to the method of evaluation of the column-beam connections.

For welded joints, the welded sections of the beam ends, diaphragms, and column ends around the column-beam connections are the subjects of investigation. With the fastener joints, the survey centers around the beam and column joints.

2.2. Sites of investigation.

The objects of investigation are 3 column-beam connections and their peripheries and 3 base sections which are located as far apart as possible within the building. With a building with diagonal members, these locations must include those end connections. However, the sites of investigation of 1) general frame work specified in 2.1 involve, as a rule, the entire structural frame.

Evaluation of the general frame work is highly significant and forms the basis of computation for index V. As a rule, it pertains to the entire structure. As to the remaining items of the investigation, 3 areas are selected in each building to investigate the column-beam connections and their peripheries as well as the column base sections.

If it becomes evident that sections of the steel frame were, for some reason, produced at several different iron foundries independently, 3 sites must be selected for the part produced by each manufacturers.

As to the specific site of investigation of the column-beam connections and their peripheries, the focus will be on a specific column-beam connection as well as parts of the column and beam materials at its periphery. In a structural frame with diagonal members to add resistance to earthquakes and wind, areas including the connections of these member end sections must be selected as sites for investigation.

Similar considerations must be made in the investigation of the column base sections.

Therefore, investigations of the items shown in 2.1 must be made at each location within the building. The individual site for each item of investigation is called a part of the investigation.

Thus a part of the investigation constitutes a specific object of investigation for each item such as column material, beam material, and connection panels.

When 3 beam materials are investigated at a single site, there are 3 parts to the investigations where tests are conducted according to 2.3. If, however, 2 of the 3 beam materials have identical cross sections and are considered to be in a similar condition, an investigation may be limited to only one out of these two. If the cross sections of the materials differ from each other, one the other hand, all are investigated

It is desirable that the sites of investigation are selected as far away as possible from each other. In this instance, distances may be measured vertically as well as horizontally. In reality, however, selection of these sites may be restricted in buildings which are being occupied. Selection may be made to suit the conditions in these instances.

The anti-seismic determination is made based on the assumption that the drawings of each building are available. It is more efficient if the sites of investigation can be pre-selected from these drawings before the actual investigation takes place.

2.3. The content of investigation.

At each site of investigation, a part of investigation is established for each item described in 2.1. Items described in Tables 1 to 9 are investigated at these parts.

These items are classified as "important matters" and "general matters".

2.4. Individual evaluation of index Q.

Individual evaluation of the "important matters" and "general matters" is made in the following manner at each part of investigation:

(1) The "important matters"

The"important matters" are evaluated according to the test standard described in Tables 8 to 14 for each item. If the individual evaluation does not rate as high as D, the result is included in the computation of the standard anti-seismic index described in Chapter 2, Section 2 and excluded from the evaluation of index Q.

(2) The "general matters".

The results are evaluated comprehensively and each item is rated as to A, B, or C according to the standard described in Tables 8 to 14. Each item of investigation is studied on the subjects described in Tables 1 to 7.

The "important matters" investigated are based on the following two aspects:

i) whether the conditions of the members and connections differ much from the original drawings,

ii) whether the resistance of each part of the structural frame is much reduced due to the method of construction and aging process.

The specific subjects of evaluation designated as "important matters" for each item of investigation are shown below. i) and ii) of the note section indicate that the respective item of investigation corresponds to i) or ii) of the test criteria given above.

1 調査項目	2 重要事項	3 備考
1) 軸組全般	5けた行方向およびはり間方向柱材間隔	i)
	6階高	i)
	7部材の欠落	i)
	8軸組筋違材の位置	i)
9 2) 柱材、3) はり材	10断面の形状、寸法	i)
	11腐食による板厚の減少	ii)
12 4) 軸組筋違材	13断面の形状、寸法	i)
	14関連接合部の耐力	i), ii)
	15腐食による断面の減少	ii)
16 5) 柱はり接合部	17ダイアフラムの有無と形状、寸法	i), ii)
6) 溶接継目	19溶接継目の種類	i), ii)
	20有効のど断面積	ii)
21 7) ファスナー継手	22位置と継手耐力	i), ii)
23 8) 柱脚	24アンカーボルトの有効断面積の総和	i), ii)
	25腐食による板厚の減少	ii)

1. items of investigation, 2. important matters, 3. notes, 4. 1) general frame structures, 5. spacing of the columns in the direction of the girders and beams, 6. floor height, 7. defects of members, 8. positions of the diagonal member materials, 9. 2) column material; 3) beam material, 10. form and dimension of the cross section, 11. reduction in the panel thickness due to corrosion, 12. 4) diagonal member material, 13. form and dimension of the cross section, 14. stress of the associated connections, 15. reduction in cross section due to corrosion, 16. 5) column-beam connection, 17. presence or absence of the diaphragm and its form and dimension, 18. 6) welded joints, 19. types of welded joints, 20. effective throat cross section, 21. 7) fastener joint, 22. position and joint stress, 23. 8) column base, 24. sum of the effective cross section of the anchor bolt, 25. reduction of the plate thickness due to corrosion.

As shown above, the investigation of the "important matters" is mainly quantitative and requires determination of the dimensions of the parts of the structural frame such as the members and connections.

The investigation of the "general matters" is conducted to observe the general condition (qualities) of each part of the structural frame and its content mainly consists of qualitative evaluation.

2.5 shows the basic concept of the specific content of each item of investigation. In the actual investigation, "Appendix 1, Form for the Results of the Investigation" is used to record the results.

As a rule, the investigation is conducted on all the prescribed items. Even when circumstances do not permit this, no item should be omitted unnecessarily.

Determination of index Q is based on individual evaluations which are obtained, as a rule, from each test item at each test site. For instance, if one column and 2 beams are investigated at a single site and the process is repeated at 3 sites, the test results are 90% complete. Similarly, test results of several test objects are obtained on other test items.

According to the test results from each test site, the individual evaluations are ranked from A to D. Based on individual evaluations of all test items and all test sites, index Q of the structure is obtained from Table 15 as a comprehensive evaluation.

If the test result for a single important matter of these test items fails to meet a certain quality standard, index Q of the entire structural frame is deemed "unsatisfactory". The standard of determination in this instance is based on the following: 1) deviation of each part of the structural frame remains within 10% of the original dimension indicated in the drawing and 2) the test data measure up to 70% of both the value uniformly projected at the designing stage and yield strength of each connection.

If the test results on the "important matters" fail to meet these standards, a value, D, is given to that item which failed. This results in indication that quality index Q of the entire structure is "unsatisfactory".

If the test results on the "important matters" meet these standards, they are included in the computation of index V but are not used in the determination of index Q. Thus if an individual evaluation, D, on the "important matters", is not given to every item at every test site, individual test results of the "general matters" are evaluated comprehensively to derive index Q.

The standard for the individual evaluation of the "general matters" is all based on the question of the final stress of the architecture in a major earthquake. Compared with standards such as "Guidelines for Steel Frame Construction Technology", "Standard for Steel Frame Construction Quality", and "Guidelines for Construction of Designs with High Strength Bolt Connections",all of which have been promulgated by the Japanese Society of Architecture and are intended to set standards for normal environments, these requirements may appear somewhat lenient. It should be kept in mind that the requirements of the present evaluation are for major disasters such as earthquakes.

In determining index Q based on the individual evaluation on the "general matters", the general quality of the structure is considered extremely poor if the number of sections with grade C exceeds 1/4 of the total number of the sections evaluated. In such an instance, index Q becomes "unsatisfactory" and no quantitative points are given. The present method of evaluation is based on the assumption that the number of test sections on individual test items are well balanced.

Therefore, if the number of test sections on individual test items is extremely large or small, it must be adjusted prior to evaluation of index Q. The standard number of test section on individual item are shown below (with the number of test sites - 3):

test items	number of test sections
column material	3
beam material	6
frame diagonal member material	3 - 6
column-beam connection	3
welded joint	6 - 12
fastener joint	3 - 6
column base	3

The contents and methods of evaluation of each test item and details of evaluation.

1) General frame structure.

表1　軸組全般の検討事項　〔重要事項〕

検討事項	検討内容
1）軸組基準寸法 イ）けた行方向柱材間隔 ロ）はり間方向柱材間隔 ハ）階高	イ）、ロ）、ハ）の各項目について実測値の設計図書で示された数値に対する比の値の最大値 $_1r$ および最小値 $_2r$ を求める
2）部材の欠落	柱、はり、軸組筋違材など主要な構造材の欠落の有無と、その位置を調べる

表8　軸組全般の個別評価（表1の検討）

〔重要事項〕	
検討事項と検討結果の状況	個別評価
1）軸組基準寸法 イ）、ロ）、ハ）の各項目において、いずれかの項目でも $_1r \geq 1.1$ または $_2r \leq 0.9$ の場合 2）部材の欠落 主要な構造材が欠落している場合	D

1. Table 1, Items of evaluation of the general frame structure (Important matters), 2. evaluation items, 3. contents of evaluation, 4. 1) standard dimensions of the structure, a) column spacing in the direction of the girders, b) column spacing in the direction of the beams, c) floor height, 5. the maximum $_1\gamma$ and the minimum $_2\gamma$ are computed in the ratio of the actual dimensions on items a), b), and c) to those shown in the drawing, 6. missing members, 7. absence of major components such as columns, beams, and frame structure is ascertained and the location of the missing portion is investigated, 8. Table 8. Individual Evaluation, 9. (Important matters), 10. test items and conditions observed in the test, 11. individual evaluation, 12. 1) standard structural dimension, $_1\gamma \geq 1.1$ or $_2\gamma \leq 0.9$ in a), b), or c); 2) absence of a member; absence of a major component.

The tests on the general frame structure involve column spacing in the direction of the girders and beams, floor height, presence or absence of major structural components, and locations of frame diagonal members. Major deviations from the original drawings are detected in these tests.

These test items concern deformability and stress of the building and are used as the bases of computation of index V. Therefore, they are all termed "important matters" and the tests involve the entire architectural structure.

The major thrust of the procedure is to create a plane drawing of each floor and a structural drawing of the major parts. Plane drawings and structural drawings may be copied from the originals with the dimensions of each part indicated.

In some buildings, dimensions for such things as column spacings and floor height may differ considerably from the original drawings. Even when the drawings appear to be in satisfactory order these dimensions must be confirmed at the site. These dimensions can usually be determined accurately over the internal and external decorating materials without exposing the components. An accurate measurement must be made to determine index Q (important matters) as well as to compute index V.

The presence of earthquake-proofing elements such as structural diagonal members placed within the architecture is important in the computation of index V. In older buildings, these may sometimes be removed (though they existed at the time of construction) or moved to another location. Investigation must be carried out carefully in these instances.

The test data are entered in "Appendix 1, Form for Test Data (NO.3).

Coding or numbering of the column lines in the directions of the girders and beams on the plane drawing is convenient in actual investigations. The test sites used in the actual investigation of the column-beam connections and column bases are also entered in the above-mentioned form.

Example 1.

(Test results) A 3-story building used for business offices was investigated. Column spacing in the direction of the girders measured $1\ell = 6.52$m while the drawing showed $0\ell = 5.80$m. Other column spacings and floor heights differed 20-30mm from the drawings but the deviations were insignificant. Subsequently, the following was computed for test item 1), a) standard structural dimension:

$$1\gamma = 1\ell/0\ell = 652/580 = 1.12 > 1.1$$

The individual evaluation was rated D.

(Treatment) It was evident that the dimensions of this building deviated from the original drawings. However, the deviation was limited to the span. It was decided that this fact was to be entered clearly in the final evaluation and the evaluation process was continued. It was also decided that the Rahmen with the deviated span would be used for the location of the actual examination of the column-beam connections and their peripheries. Computation of Index V will be computed based on the results of this examination. By this process, individual evaluation D concerning the general structural frame is removed and may be disregarded in the subsequent total evaluation of Index Q for this building.

2) Column and beam materials.

表2　柱材およびはり材の検討事項

〔重要事項〕

検討事項	検討内容
1）部材の断面諸係数 イ）断面積 ロ）断面係数	イ)、ロ）について実測値の設計図書に示された数値に対する比の値 r を求める

〔一般事項〕

検討事項	検討結果の分類		
	(a)	(b)	(c)
イ）　フランジ部分断面欠損（補強なし）	なし	15 %未満	15 %以上
ロ）　ウエブ部分断面欠損（補強なし）	なし	30 %未満	30 %以上
ハ）　仮設用溶接	なし	3ヶ所以内で短いビードあり	4ヶ所以上で短いビードあり
ニ）腐食（板厚の減少）	ほとんどなし	10 %以下	10 %を超える

表9　柱材およびはり材の個別評価（表2の検討）

〔重要事項〕

検討事項と検討結果の状況	個別評価
1）　部材の断面諸係数 イ)、ロ）のいずれの項目でも　$r \leq 0.7$　の場合	D

〔一般事項〕

検討結果の状況	個別評価
(c）がなく、（b）が1つ以下の場合	A
{（c）の数 + 0.5 ×｛（b）の数｝≧ 1.5 の場合	C
上記以外の場合	B

1. Table 2. Test items for column and beam materials.
2. (important matters)
3. evaluation items
4. content of evaluation
5. 1) section modulus of members.

A) area of the cross section

B) coefficient of the cross section

6. γ, the ratio of the test result to the figure shown in the drawing is computed on A) and B).

7. (General matters)

8. evaluation items

9. classification of the test results

10. A) defects in the cross section of the flange section (unreinforced)

11. none

12. 15% or less

13. more than 15%

14. defect in the cross section of the web section (unreinforced)

15. 30% or less

16. more than 30%

17. C) temporary welding

18. in 3 or less locations with short beads

19. inmore than 4 locations with short beads

20. corrosion (with reduction in plate thickness)

21. hardly present

22. less than 10%

23. more than 10%

24. Table 9. Individual evaluation of the column and beam materials (evaluation of Table 2).

25. (important matters)

26. Evaluation items and description of the condition

27. individual evaluation

28. 1) section modulus of members

$\gamma \leq 0.7$ in both A) and B)

29. (general matter)
30. conditions observed
31. individual evaluation
32. no (c) and 1 or less (b)
33. the number of (c)'s + 0.5 x $\{$the number of (b)'s$\}$ ≥ 1.5
34. none of the above

The test contents and test methods for the column and beam materials are described together. These test items are intended to examine whether the forms and dimensions conform to those in the drawings and, if they do not, what forms have replaced the original.

Therefore abbreviated drawings--mainly the cross sections--are entered in the report form. In this instance, it is advisable that the form and dimensions of the cross section are copied from the drawing ahead of time and the actual dimensions of each section are added to these. It is also necessary that the conditions and positions of defects in the cross sections and temporary welding be recorded in abbreviated drawings when these features are noted.

Dimensions are determined using a regular steel tape, slide, or micrometer. In the determination of the plate thickness of the materials with closed cross sections, such as a web plate of a steel H-beam, boxed cross section, rectangular cross section with a middle support, and steel pipe, an ultrasonic thickness gauge is used. Refer to "thickness Determination Using a Portable Pulse Reflector-Type Ultrasonic Thickness Gauge-NDIS 2408-77" by Japanese Society of Non-Destructive Examination.

The numerical values for the section modulus of the member are expressed in an appropriate unit as shown in the cross section list.

The test is conducted using "Appendix 1, Form for the Test Results (Nos. 5 and 6)".

In the "general matters", the results of the tests are rated as (a), (b), or (c). In some items, assignment of a clear-cut numerical value is not possible and the results are expressed in general outlines.

A) and B). Defects in cross sections.

The defects are those not expected in the design stage. These were caused on the flange or web sections due to procedures such as installation of pipes and yet no subsequent reinforcement was made.

C) Short bead welding.

Short beads tend to occur in welding temporary and secondary sections. It is known that, if the bead length is less than 25-40mm, the thermally affected portion of the welded metal or the main material hardens due to rapid cooling, causing a defect and subsequently affecting adversely the pliability of high-tensile steel or thick plates used as the main material. In the present test, the upper limit of a short bead is set at 25mm. If short beads measuring less than 25mm are found (in 1/3 the length at the end of the member in most instances), the following ratings are given: (b) for 3 locations; (c) more than 4 locations; and (a) for none.

If the entire member length is investigated, the number of the sites of examination may be increased.

Corrosion.

Corrosion is another phenomenon of aging and deterioration of steel-skeleton buildings. Rapidity of rusting of the iron frame exposed to the atmosphere is often observed and corrosion constitutes a common problem affecting the structural strength of buildings such as industrial installations. Steel skeletons such as those used in office buildings--the subject of the present investigation--is, on the other hand, located indoors and the rusting process takes place rather slowly so development of serious problems is somewhat rare. Problems of corrosion affect the structural members adjacent to the exterior portion from which the finishing material became detached or rain water seeped in, or the column bases were exposed to a moist atmosphere. In the regions close to the sea shore, reductions in plate thickness up to 4-5mm in 10 years are known to happen and special care in examination is needed.

Example 2. Column materials.

Figure 1 shows the test results for a rectangular cross section with a support of an H-type steel with a reinforcement plate as an example of a column. The flange, web, and reinforcement plate thickness represent the portion of the column base without a reinforcement plate. They aere determined by using an ultrasonic thickness gauge and a standard gauge. However, determination of the web thickness is not possible when the reinforcement plate is welded for the entire length of the H-type steel. In such instances, the value listed in the drawing was used. When the length of fillet welding of a reinforcement plate to an H-type steel flange is too small, the plate does not move together with bending of the H-type steel in the direction of the weak axis. This should be taken into consideration in the test.

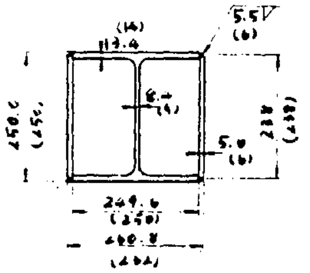
Figure-1. Example of a test on a column material.

調査結果記入用紙（№ 5）		柱材
柱の位置	2階 Y_1 通 X_2 通交点　記号 C_2	
断面構成	充腹材　圧延鋼単材・形鋼単材に補強・溶接組立材・その他（　　）	
	非充腹材　ラチス形式・帯板形式・有孔板形式・その他（　　）	

断面詳細（溶接・綴材等も記入）

設計断面：H − 250 × 250 × 9 × 14

補強PL − 6・



$$_0Z_x = \left(10800 + \frac{1.2}{12}\,23.8^3\right) / 12.5 = 972$$

$$_1Z_x = \frac{1}{12}\left(25^4 - 24.1 \times 22.3^3 + 1.12 \times 23.8^3\right) / 12.5 = 923$$

$$_0Z_y = \left(3650 + 2 \times 23.8 \times 0.6 \times 12.8^2\right) / 13.1 = 636$$

$$_1Z_y = \left\{ \frac{1}{12}\,25^4 - \left(\frac{22.8}{12} \times 12.1^3 + 12.1 \times 22.3 \times 6.47^2\right) \times 2 + 2 \times 23.8 \times 0.56 \times 12.8^2 \right\} / 13.0 = 596$$

◎柱脚部高さ 300cm にわたり補強PL　片側欠損

（　）内は設計値

重要事項		設計値	実測値	γ
1）断面諸係数	断面積	$_0A = 92.18 + 2 \times 23.8 \times 0.6 = 120.7$ cm²	$_1A = 25 \times 1.84 \times 2 + 22.8 \times 0.84 + 2 \times 23.8 \times 0.56 = 112.4$	$_1A / _0A = 0.93$
	断面係数	$_0Z_x = 972$ cm³	$_1Z_x = 923$	$_1Z_x / _0Z_x = 0.95$
		$_0Z_y = 636$ cm³	$_1Z_y = 596$	$_1Z_y / _0Z_y = 0.94$

一般事項　検討事項	(a)	(b)	(c)
イ）フランジ部分断面欠損（補強なし）	なし	15 %未満	15 %以上
ロ）ウェブ部分断面欠損（補強なし）	なし	30 %未満	30 %以上
ハ）仮設用溶接	なし	3ケ所以内で短いビードあり	4ケ所以上で短いビードあり
ニ）腐食（さびの発生による板厚の減少）	ほとんどなし	10 %以下	10 %を超える

Q指標の個別評価	
A	(c)がなく(b)が1つ以下の場合
C	〔(c)の数 + 0.5 × {(b)の数}〕≧ 1.5
D	〔重要事項〕で $\gamma \leq 0.7$ の場合
B	上記以外の場合

1. form to enter the results (No.5)
2. position of the column
3. second floor, intersection of Y1 and X2, code C2
4. composition of the cross section
5. filled material: rolled steel, reinforced cast steel, welded assembly,
   others (   )
6. non-filled material: lattice form, lattice tie, perforated plate,
   others (   )
7. details of the cross section (include welding and binding materials)
8. cross section in the drawing
9. reinforcement
10. reinforced to the ;to the height of 300cm at the column base, one side missing
11. (   )shows the figure in the design
12. important matters
13. designed value
14. actual figure
15. coefficients of cross section
16. area of the cross section
17. section modulus
18. general matters
19. items of evaluation
20. A) defective cross section of the flange section (unrefinforced)
21. none
22. less than 15%
23. over 15%
24. B) defective cross section of the web section (unreinforced)
25. less than 30%
26. more than 30%

27. C) temporary welding

28. short beads at 3 locations or less

29. short beads at 4 locations or more

30. D) corrosion (reduction in the plate thickness due to development of rusting)

31. almost none

32. less than 10%

33. more than 10%

34. individual evaluation of index Q

35. non (c) and (b) is 1 or less

36. $\left[\text{the number of (c)'s} + 0.5 \times \left\{\text{the number of (b)'s}\right\}\right] \geq 1.5$

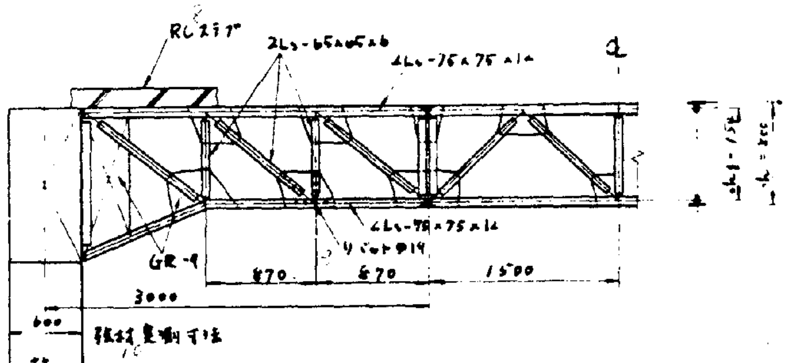
37. (important matters) $\gamma \leq 0.7$

38. none of the above

Figure-2. Example of a test of a beam.

調査結果記入用紙（№ 6）		はり材
はり位置	1階　5通A－B間のはり	記号　G_1
断面構成	充複材　圧延形鋼単材・溶接組立材・その他（　　）	
	非充複材　有孔ばり　組立材　（トラス材・ラチスばり）・その他（　　）	

断面詳細（溶接・継材等も記入）



$_1h = 786\,cm$　　$_1hg = 738\,cm$

$_1A = 15.45\,cm^2$　　$_0Zx = 16.56 \times 2 \times 87.7^2 \times 2 / 40 = 2350$

$_0A = 16.56$　　$_1Zx = 15.45 \times 2 \times 86.9^2 = 2 / 39.3 = 2140$

◎ 不良リベット（心ずれ、だんご）が若干みられる。

重要事項		設計値	実測値	r
1）断面諸係数	断面係数	$_0Zx = 2350$	$_1Zx = 2140$	$_1Zx / _0Zx = 0.91$
	ウェブ断面積	$_0Aw =$	$_1Aw =$	$_1Aw / _0Aw =$

一般事項	検討事項	(a)	(b)	(c)
イ）フランジ部分断面欠損（継手以外）		なし	15 %未満	15 %以上
ロ）ウェブ部分断面欠損（設備用貫通孔などで補強のないもの）		なし	30 %未満	30 %以上
ハ）仮設用溶接		なし	3ケ所以内で短かいビードあり	4ケ所以上で短かいビードあり
ニ）腐食（さびの発生による板厚の減少）		ほとんどなし	10 %以下	10 %を超える

Q指標の個別評価		
	Ⓐ	(c)がなく(b)が1つ以下の場合
	C	{(c)の数＋0.5×｛(b)の数｝}≧1.5
	D	［重要事項］で $r \leq 0.7$ の場合
	B	上記以外の場合

1. form to enter the test results (No.6)   Beam material
2. beam location
3. the first floor, line 5 between A and B   Code G1
4. composition of the cross section
5. filled material: rolled steel, cast assembly, others (   )
6. unfilled material: perforated beam, assembly material (truss, lattice), others (   )
7. details of the cross section (includes welding and binding materials)
8. RL slab
9. rivet
10. actual dimension of the chord member
11. column
12. a few bad rivets (off-centered or rounded) were noted
13. important matters
14. figures in the drawing
15. actual values
16. 1) coefficient of the cross section
17. section modulus
18. area of web cross section
19. general matters
20. evaluation items
21. A) defective cross section of the flange section (except the joint)
22. none
23. more than 15%
24. more than 15%
25. B) defective cross section of the web section (perforated for installation but not reinforced)
26. less than 30%

27. more than 30%

28. C) temporary welding

29. short beads in 3 locations or less

30. short beads at 4 locations or more

31. D) corrosion (reduction in the plate thickness due to rusting)

32. almost none

33. less than 10%

34. more than 10%

35. individual evaluation of index Q

36. no (c)'s and (b) 1 or less

37. $\left[\text{number of (c)'s} + 0.5 \times \{\text{the number of (b)'s}\}\right] \geq 1.5$

38. (important matter) $\gamma \leq 0.7$

39. none of the above

3) Column-beam connecting section.

表3　柱はり接合部の検討

[　重　要　事　項　]	
検　討　事　項	検　討　内　容
パネル板有効体積	設計値に対する実測値の比の値 r を求める

[　一　般　事　項　]			
検討事項	検　討　結　果　の　分　類		
	A	B	C
ダイヤフラム	十分なダイヤフラムがあり溶接施工も正しく行われている	・ダイヤフラムがあるが溶接が貧弱 ・不十分なダイヤフラムしかない	ダイヤフラムがない。またはあってもフランジで板厚以上の芯ずれがある。

表10　柱はり接合部の個別評価（表3の検討）

[　重　要　事　項　]	
検討事項と検討結果の状況	個　別　評　価
パネル板有効体積 $r \leq 0.7$　の場合	D

[　一　般　事　項　]	
検討事項と検討結果の状況	個　別　評　価
十分なダイアフラムがあり、溶接施工も正しく行われている	A
ダイアフラムがあるが溶接が貧弱、又は不十分なダイアフラムしかない	B
ダイアフラムがない。又はあってもフランジ板厚以上の芯ずれがある	C

1. Table 3. Evaluation of the column-beam connecting section
2. (important matters)
3. evaluation items
4. evaluation contents
5. effective volume of the panel plate
6. the ratio of the actual dimension, , to that of the drawing is determined.
7. (general matters)
8. evaluation items
9. rating of the results of the evaluation
10. diaphragm
11. the diaphragm is sufficient and welding was performed correctly.
12. the diaphragm exists but is insufficient with poor welding techniques.
13. the diaphragm is absent or exists but its eccentricity entends beyond the thickness of the flange plate.
14. Table 10. Individual evaluation of the column-beam connecting section (evaluation of Table 3)
15. (important matters)
16. evaluation items and results
17. individual evaluation
18. effective volume of the panel plate $\gamma \leq 0.7$
19. (general matters)
20. evaluation items and results
21. individual evaluation
22. the diaphragm is sufficient and welding was performed correctly
23. diaphragm exists but welding technique is poor or the diaphragm is insufficient
24. The diaphragm is absent or exists with its eccentricity extending beyond the thickness of the flange plate.

The items of evaluation of the column-beam connection section include two concerning the panel plate and the diaphragm. These are evaluated to observe whether there are any factors which are unfavorable in their dynamic composition.

The effective column of the panel plate is the basis on which computations of the stress of the panel section is performed and evaluation of its deviation from the drawing is important. If reinforcement of the panel plate is indicated in the drawing, especially, examination of the actual reinforcement as indicated is highly significant.

The form of the diaphragm and the details of the connecting sections are also important in the evaluation but evaluation of each detail is often difficult. Presence or absence of the diaphragm and the condition of its attachment are roughly observed in the (general matters).

The test of the column-beam connecting section is performed using (Appendix 1, Form to Enter the Test Results (Nos. 6-7), Column-Beam Connecting Section 1-2). Most of the data entered in Form No. 6 are from the investigation on site. The areas around the column-beam connecting section, presence or absence of defects in the cross section, and methods of reinforcement of the panel are entered in the drawing on Form No.6. The drawing may be annotated while checking the list at the bottom of the form.

It is recommended that the following drawings be copied ahead of time from the original. Actual dimensions and plate thicknesses should be entered on the copy while measuring to increase efficiency in the investigation and avoid omission of any items.

The areas around the column-beam connecting section are often characterized by complex structures. Besides the cross sections, 3-dimensional drawings, and plane drawings entered on the form for recording, supplementary drawings may be added to include every detail. In these drawings, the relationships such as those of the upper and lower columns, beams, perpendicular to each other, and levels of upper and lower flanges of the beam, and diaphragms should be clearly delineated.

Each dimension and plate thickness necessary for preparation of the drawings may be determined by normal means in most instances. The presence of a diaphragm in a closed section such as columns with a box section or box sections assembled by welding a flange to the weak axis of an H-type steel is determined using ultrasonic thickness gauges and other instruments.

Example 4.

Figure 3 shows the results of a test on a column-beam connecting section which includes a column with a box section--an H-type steel with a reinforcement plate--and an H-type steel beam. In this example, the height of the beam running in the X-direction differs from that running in the Y-direction by 50mm; and a diaphragm is missing at the flange site under the beam in the X-direction (H-300 x 150 x 6.5 x 9). However, at the end of the beam running in the X-direction, a cover plate measuring 6mm in thickness is fillet-welded with a throat depth of 4mm; and the beam together with the flange is butt-welded to the column flange.

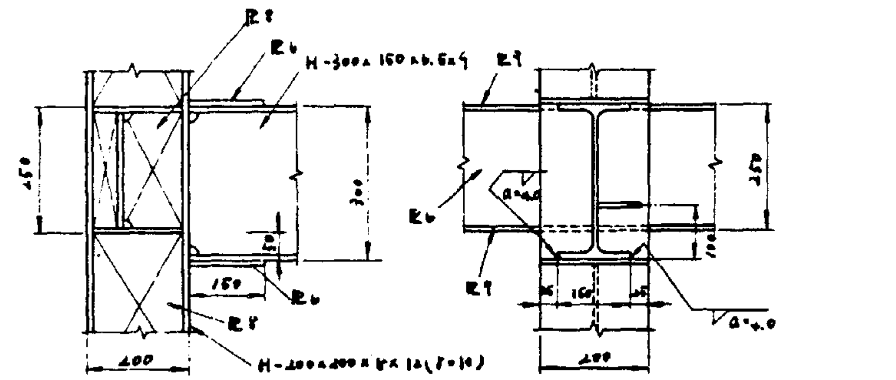
Figure-3. Example of a test of the column-beam connecting section.

1 調査結果記入用紙（№6）　　　　柱はり接合部(1)

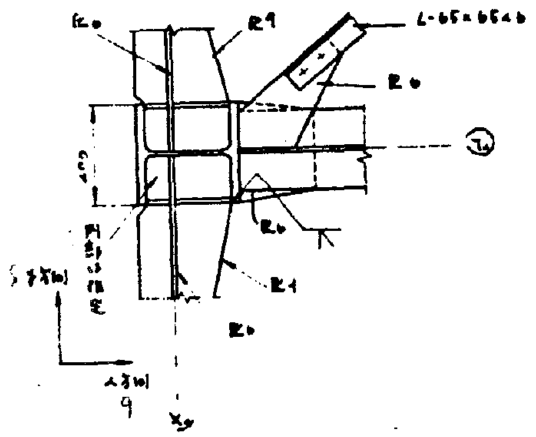
2 位　置	3 4階～5階間　X_2通り－Y_2通り

4 詳細図

5（断面図、立面図）Y_2 通り（ x方向）　（断面図、立面図）X_2通り（ y 方向）



6 平面図



10 チェックリスト

- ☑ パネル板厚 11
- ○☑ パネル補強の方法（板厚）12
- ☑ ダイアフラムの位置・寸法 13
- ○☑ パネル断面欠損の有無 14
- ☑ ダイアフラム断面欠損の有無 15
- ☑ 溶接継目の種類と寸法 16
- ☑ スカラップの有無 17

○印のものは推定による値 18

	A	Ⓑ	C
19 Q 指標の個別評価	20 十分なダイヤフラムがあり、溶接施工も正しく行われている	21 ・ダイヤフラムがあるが溶接が貧弱 ・不十分なダイヤフラムしかない	22 ダイヤフラムがない。またはあってもフランジ板厚以上の芯ずれがある。

1. Form to enter the test results (No. 6) Column-beam connecting section(1
2. location
3. the 4th to 5th floor, X2 to Y2
4. detailed drawings
5. (cross section and 3-dimensional drawings), Y2 line (X-direction)
   (cross section, 3-dimensional drawing) X2 line (Y-direction)
6. plane drawing
7. interior is estimated
8. Y-direction
9. X-direction
10. check list
11. panel plate thickness
12. method of reinforcement of the panel (plate thickness)
13. location and dimension of the diaphragm
14. presence or absence of defective aross section of the panel
15. presence or absence of defective cross section of the diaphragm
16. types and dimensions of welded joints
17. presence or absence of scallops
18. O marks an assumed value
19. individual evaluation of index Q
20. the diaphragm is sufficient and welding is performed correctly
21. the diaphragm exists but is insufficient with poor welding technique.
22. the diaphragm is absent or exists but its eccentricity extends beyond
    the flange plate thickness.

4) Welded joints.

表4　溶接継目の検討事項

〔重要事項〕	
検討事項	検討内容
1）設計図書との対応 イ）突合わせ溶接を指示された溶接部	a．断続すみ肉溶接となっているか否か b．連続すみ肉溶接となっている場合はその脚長
ロ）連続すみ肉溶接を指示された溶接部	a．断続すみ肉溶接となっている場合はその溶接長および脚長 b．連続すみ肉溶接となっている場合はその脚長

〔一般事項〕			
検討事項	検討結果の分類		
	(a)	(b)	(c)
1）設計図書との対応 イ）突合わせ溶接を指示された溶接部	指示通り	連続すみ肉溶接、ただし脚長が0.5×（突合わせる母材の材厚）以上あり	
ロ）連続すみ肉溶接を指示された溶接部	指示通り	連続すみ肉溶接で脚長が不足。ただし脚長は0.5×（指定寸法）以上あり	a．連続すみ肉溶接で脚長が0.5×（指定寸法）未満 b．断続すみ肉溶接
2）突合わせ溶接継目 イ）溶接終始端の処理	エンドタブ内で処置	回し溶接で処理	(a)、(b)以外
ロ）フランジの溶接に伴うウェブの処理	スカラップを設けるか開先加工に工夫を施す		特に処理せず
ハ）余盛寸法	1／4×（板厚）以上。または10 mm以上	のど厚を確保	のど厚不足
ニ）外観検査	問題なし	やや問題あり	かなり悪い
ホ）内部欠陥長さ	フランジ板厚未満	(a)、(c)以外	2×（板厚）以上
3）連続すみ肉溶接継目 1）はり端フランジ継手 イ）ウェブの処理	スカラップを設ける		特に処理せず
ロ）溶接終始端の処理	回し溶接で処理	回し溶接せず	
2）その他の継手 イ）ウエブの処理	スカラップを設ける	特に処理せず	
ロ）溶接終始端の処理	回し溶接で処理	回し溶接せず	

表11　溶接継目の個別評価（表4の検討）

〔重要事項〕	
検討事項と検討結果の状況	個別評価
1）設計図書との対応 イ）設計図書において突わせ溶接を指示された溶接部が断続すみ肉溶接となっている場合、および連続すみ肉溶接となっていて、かつその脚長が0.5×（突合わせる母材の板厚）未満の場合	D
〔一般事項〕＊1）	
検討結果の状況	個別評価
(c)がなく、(b)がn＊2）以下の場合	A
〔(c)の数＋0.5×｛(b)の数｝〕≧nの場合	C
上記以外の場合	B

＊1　調査部位を構成するすべての溶接継目に関する検討結果により個別評価を行う
＊2　n：調査した溶接継目の数

1. Table 4. Items of Evaluation of a Welded Joint

2. (important matters)

3. evaluation items

4. contents of evaluation

5. 1) correspondence with the drawings.

   a) section to be butt-welded

6. a. whether it is block fillet-welding

   b. weld thickness when it is continuous fillet-welding

7. b) section to be continuous fillet-welding

8. a. weld length and thickness when it is block fillet-welding

   b. weld thickness when it is continuous fillet-welding

9. (general matters)

10. items of evaluation

11. rating of the results of evaluation

12. 1) correspondence with the drawing

    a) section marked for butt-welding

13. corresponding with the drawing

14. continuous fillet-welding but the weld thickness exceeds 0.5 x (thickness of the base metal to be butted)

15. b) section marked for continuous fillet-welding

16. correspondence with the drawing

17. weld thickness is insufficient in continuous fillet-welding but the thickness is in excess of 0.5 x (specified size)

18. a. weld thickness is 0.5 x (specified size) or less in continuous fillet-welding

    b. block fillet-welding

19. 2) butt-welding joint

    a) treatment of the edges of welding

20. treated in the end tab

21. treated by circular welding

22. other than (a) or (b)

23. b) treatment of the web associated with flange welding

24. created scallops or applies a certain design in beveling

25. no treatment

26. c) shrinkage allowance

27. in excess of 1/4 (plate thickness) or over 10mm

28. maintains the throat thickness

29. throat thicness insufficient

30. d) examination of the external appearance

31. no problems

32. somewhat problematic

33. fairly poor

34. e) length of the internal defect

35. insufficient flange plate thickness

36. other than (a) or (b)

37. over 2 x (plate thickness)

38. 3) continuous fillet-welding joint

    1) beam end flange joint

        a) web treatment

39. creation of scallops

40. no treatment

41. b) treatment of the edges of the weld

42. treated with circular welding

43. no circular welding

44. 2) other joints

        a) treatment of the web

45. creation of scallops

46. no treatment

47. b) treatment of edges of the weld

48. treatment with circular welding

49. no circular welding

50. Table 11. Individual Evaluation of Welded Joints (evaluation of Table 4)

51. (important matters)

52. evaluation items and description of the results

53. individual evaluation

54. 1) correspondence with the drawing

    a) section specified to be butt-weld in the drawing is, in reality, block fillet-weld or continuous fillet-weld and its thickness is 0.5 x (plate thickness of the base metal to be butted) or less

55. (general matters)

56. description of the results

57. individual evaluation

58. no (c)'s and (b)'s is less than n * 2)

59. [the number of (c)'s + 0.5 x {the number of (b)'s}] ≥ n

60. * 1:an individual evaluation is made based on the results of observation of all the welded joints composing the test location.

    * 2: h: the number of welded joints investigated

The items listed in Table 4 are investigated at each welded joint selected as the test sites.

In the test of the column-beam connecting section and its periphery, the beam end connection, column end konnection, diaphragm connection, and gusset plate connection encompass units of a single test site. The test site is described in the following: i) in a column-beam connection using H-type steel for the column and beam materials, (as shown in Figure 4) there are 2 beam connections and one diaphragm connection in the direction of the strong axis of the column and 2 gusset plate connections in the direction of the weak axis of the column. In this instance, the upper and lower beam flange welded joints (A) and the web welded joints (B) are considered to form a set for a test site.
ii) At the diaphragm connection, the column web and column flange welded joints (C and D) compose a set for a test site.
iii) As to the gusset plate connection, a welded joint (E) at the column web position and another welded joint (F) at the upper and lower diaphragm positions are located at both sides of the colúmn web. These These constitute each test site. When the Rahmen structure has a weak axis direction of the column and a bracket is formed by extending the diaphragm and web-gusset plate, (as shown in Figure 5) the beam connecting section in the direction of the column weak axis replaces the diaphragm and gusset plate connections shown in Figure 4 as a test site.
iv) When the beam on the side of the weak axis of the column is directly welded to the column web and a steel plate is inserted in the section corresponding to the difference between the internal dimension of the beam flange end and the column material, as shown in Figure 6, the welded joints corresponding to C, D, and G at the upper and lower flange positions are considered to form a set as a test site for a diaphragm connection.

v) When a box-shaped section with a reinforcement--composed of H-type steel with a cover plate--is used as a column material, as shown in Figure 7, the number of welded joints increases (corresponding to H, I, J, and K) over that seen at the column-beam connection indicated in Figure 5.

These welded connections are collectively studied and considered to form a single test site. These are also called column end connecting sections.

At these column-beam connecting sections, investigation of the the beam and diaphragm connection welded joints in the direction of the weak axis is impossible by ordinary means.

(The important matters) In this section of the test, each welded joint which composes a test site is investigated to observe whether its type and dimension are according to those specified by the drawings.

If an important welded joint which is specified to be butt-welded by the drawing is in reality a continuous fillet-welding, the construction technique is considered to be poor and such a finding is clearly entered in the final report.

It should be considered, however, that the present process is for the determination of anti-seismic resistance of pre-existing buildings and such undesirable construction methods have often been practiced in steel-skeleton construction. If the thickness of continuous fillet-welding is over 0.5 x (plate thickness of the base metal to be butted), the finding is included in the computation of index V and the examination is continued.

If the weld thickness is less than 0.5 x (plate thickness of the base metal to be butted), the joint strength is less than 0.7 times the yield resistance of the base metal to be butted even with fillet-welding on both sides. Thus the quality as well as the resistance strength of the welded connection is inadequate. Therefore index Q in the individual evaluation is rated D and the examination process is interrupted.

If a joint specified to be butt-welded by the drawing is a continuous fillet-weld for reasons similar to those described above, index Q in the individual evaluation is rated D and the testing procedure is also interrupted.

(General Matters) First, the welded joints composing the test sites are investigated as to their types, thickness of the beads, and their relationship to the drawings.

Next, evaluation items 2) are investigated if the joints are found to be butt-welds. If fillet-welding is used, the joints are classified into the following categories according to their dynamic properties:

i) beam end flange joint, frame diagonal member joint, etc., to which the mainly normal stress is exerted; and

ii) joints to which mainly shearing stress is exerted.

These joints are investigated on evaluation of item 3).

As a rule, the above processes are performed on all the welded joints composing the test site where possible. The individual evaluation of each test site is based on the complete test results. Therefore, the test results on these joints should be entered in the records.

The following factors should be considered when evaluating items 2) and 3) on the butt- and fillet-weld joints:

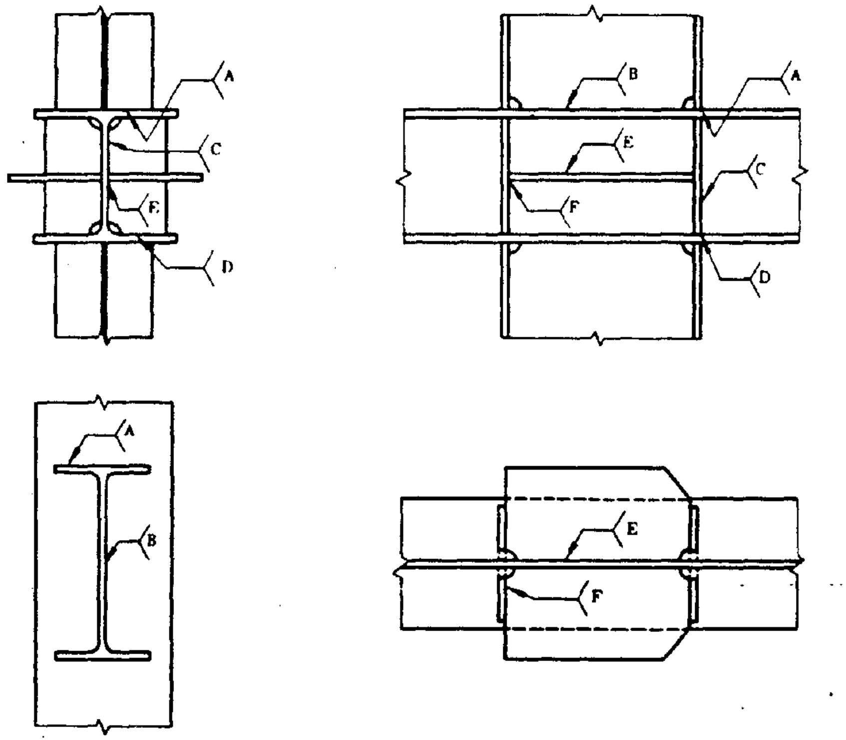
a) several factors which are believed to affect the stress and deformation capacity of the joint but the extent of their effects are difficult to quantify.

b) factors which may affect the welded joints but, due to a limited number of the test samples, the extent of the effects is difficult to generalize.

The above should be taken into consideration when evaluating the quality of the joints.

The minimum number of welded joint test sites at each location should be 2 at the beam end connection (one each at the beam end connection on the strong axis side and weak axis side of the column), one at the diaphragm connection, and another at the column end connection if it exists.

Figure-4.



図－4

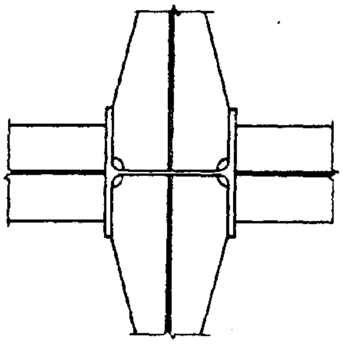


Figure-5

図－5

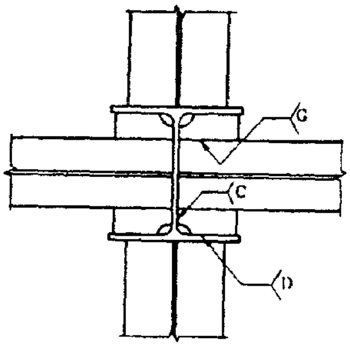


Figure-6

図－6

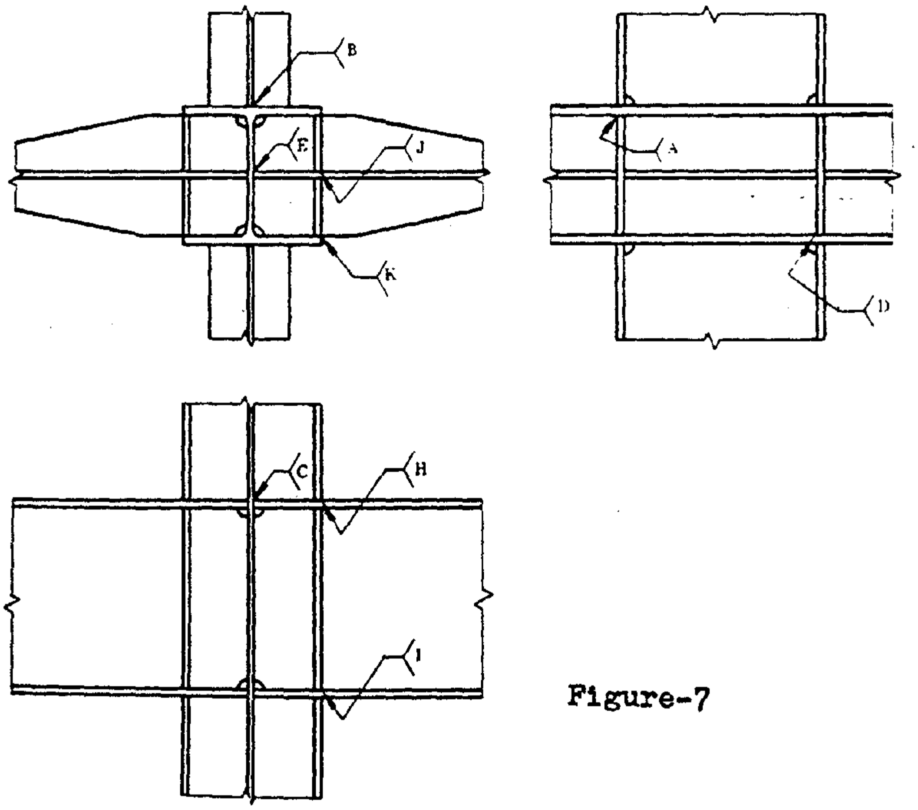


Figure-7

The "Appendix 1, Form to Enter the Test Results (No.*) and (No.9)" is used in the investigation of the welded joints. The details of the investigation are described in the following according to the items on this form.

i) Enter the test sites and the names of the locations.

ii) In the investigation of the welded connections, a determination is made of the welded joint to be investigated is butt-welded or fillet-welded.

Prior to the on-site investigation, the drawing (namely, detailed drawings of the connections) or a structural computation if the detailed drawing is unavailable, are examined to determine the type of each welded joint and the findings are entered in (   ) on the form.

Presence or absence of the end tab and backing metal are used as a check point when determining the types of welded joints by external examination.

As shown in Key Table-1, the presence of both the end tab and backing metal indicates a butt-welded joint ; while the absence of these shows a strong possibility that the joint is fillet-welded.

Key Table-1. Determination of the types of welded joints (estimated)

エンドタブ	裏当金	判定（目安）
有	有	突合わせ溶接（レ形グループ溶接）
有	無	突合わせ溶接（裏ハツリ溶接）[1)]
無	有	突合わ溶接[1)]
無	無	すみ肉溶接[2)]

注1）溶接ビード幅が狭い場合はすみ肉溶接の可能性がある。

注2）はりウェブにスカラップがあり、溶接ビード幅が適正な（広い）場合は突合わせ溶接の可能性が大きい。

1. end tab
2. backing metal
3. determination (estimation)
4. present
5. absent
6. butt-welding (grouped welding)
7. butt-welding (back chipping welding)1)
8. butt-welding 1)
9. fillet-welding 2)
10. Note 1) When the welded bead with width is small, it is most probably fillet-welding.

    Note 2) When the web is scalloped and the bead width is appropriate (wide), it is most probably butt-welding.

Wh at is shown in the above Key-Table-1 is merely an estimate. Ideally, the determination should be made by probing the inside of the welded metal using an ultrasonic instrument. In this method, if an echo showing a defect is detected for the entire length of the welded line, it is judged to be fillet-welding.

If the type of welded joint is found to differ from that specified in the drawing, index Q is determined from the test results. These results are also reflected in the computation of index V.

iii) The base metal types of the columns, beams, diaphragms, and frame diagonal members and the grade of the welding rods used are noted in the design references and installation outlines. If these are unknown, various stresses of the welded joints are computated based on the values of base steel SS41.

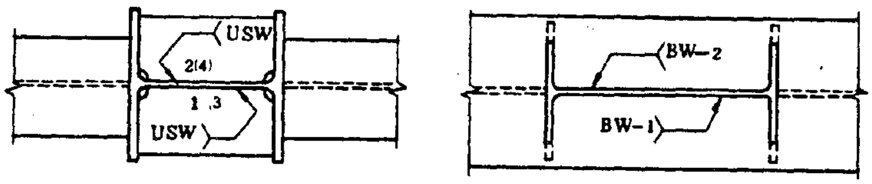
iv) The effective throat thickness and the effective length of welding of the joints composing the connections being investigated are computed.

Figure-8 shows a rigid connection of an H-type steel beam to the strong axis of an H-type steel material as an example to be entered on the form described above. The form to enter the details of thw welded joint composing the column-beam connection is shown in Figure-9.

UF-0 and 1 and LF-0 and 1 correspond to i of column 2 (types of welded joints) of the form; beam end flange joint BM is ii, beam end web joint US-0 and 1 are ii; diaphragm-flange joint USW and LSW are iv, diaphragm web joint (refer to Figure-8).

The method of computation of the effective throat thickness (a_e) and the effective length of welding (b_e) are shown below.

The throat thickness of thw welded joint and the fillet size are derived from an average of the respective welding lines. If the forms of the welding beads are not uniform, the number of sites where measurements are taken (as shown in Figure-9) is increased appropriately and the average of these measurements is computed.



Notes: O: outside

I: inside

U: upper side

L: lower side

S: stiffener (diaphragm)

F: flange

W: web

B: beam

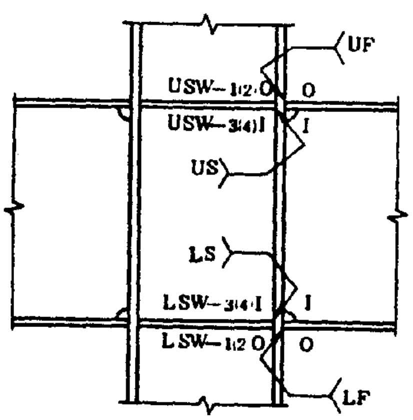
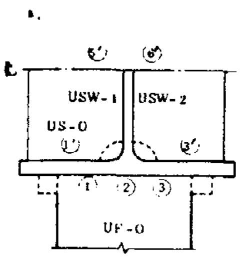


Figure-8.

Figure-9.



UF−O

	サイズ	のど厚
①		
②		
③		
3 平均値		

US−O

	サイズ	のど厚
①'	1	2
③'		
3 平均値		

USW

	サイズ	のど厚
⑤'	1	2
⑥'		

	a	b
USW−1		
USW−2		

(mm)

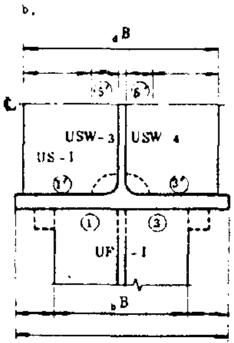
a_e = a（突合わせ溶接）or

4 　= MIN｛S/$\sqrt{2}$,a｝（すみ肉溶接）

b_e = (mm)

a_e = (mm)

b_e = (mm)



UF−1

	サイズ	のど厚
①		
③		
3 平均値		

a_e = (mm)

b_e = (mm)

US−1

	サイズ	のど厚
①'	1	2
③'		
3 平均値		

a_e = (mm)

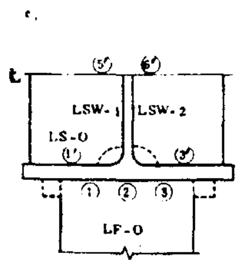
b_e = (mm)

USW

	サイズ	のど厚
⑤'	1	2
⑥'		

	a	b
USW−3		
USW−4		

(mm)



LF−O

	サイズ	のど厚
①	1	2
②		
③		
3 平均値		

a_e = (mm)

b_e = (mm)

LS−O

	サイズ	のど厚
①'	1	2
③'		
3 平均値		

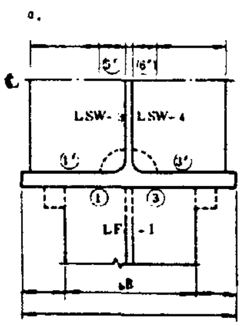
a_e = (mm)

b_e = (mm)

LSW

	サイズ	のど厚
⑤'	1	2
⑥'		

	a	b
LSW−1		
LSW−2		



LF−1

	サイズ	のど厚
①	1	2
③		
3 平均値		

a_e = (mm)

b_e = (mm)

LS−1

	サイズ	のど厚
①'	1	2
③'		
3 平均値		

a_e = (mm)

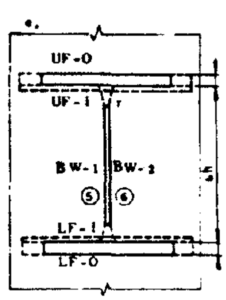
b_e = (mm)

LSW

	サイズ	のど厚
⑤'	1	2
⑥'		

	a	b
LSW−3		
LSW−4		

(mm)



5 柱およびはり部材のサイズ

6 柱	
7 は　り	

BW

	サイズ	のど厚
⑤	1	2
⑥		

	a	b
BW−1		
BW−2		

(mm)

8 〔注〕突合わせ溶接の場合：aはフランジ（又はダイアフラム）等の材厚とし、
Sは補強すみ肉のサイズとする。

図－9

1. size
2. throat thickness
3. average
4. a_e = a (butt-welding) or
   = MIN S/ 2,a (fillet-welding)
5. sizes of the column and beam materials
6. column
7. beam
8. (nots) butt-welding: a is thickness of the flange (or a diaphragm) and S is the size of the reinforcing fillet.

a) For a butt-welded joint.

The effective throat thickness (a_e) is set at the thickness of the base metal.

The effective length (b_e) of the weld is figured from the material used as the base metal to be butted when the edges of the weld are treated in the end tab or sufficient circular welding is performed.

When treated with methods other than those above, 2 a_e is subtracted from the weld length. In butt-welding without scallops in the web of the beam end flange joint, a complete welding effect is not expected unless special processes are applied to the beveling and backing metal of the beam flange. Therefore, (btw + br) of the entire width is subtracted from be.

btw is the web thickness and br is the radius of curvature of the beam fillet section.

The reinforcing fillet welding of a butt welded joint at the T and cross joints is effective in preventing cleavage in the direction of the plate thickness. The size (S) of such a weld is determined and incorporated in the evaluation of index Q.

b) Continuous fillet-welded joints.

The size and throat thickness of fillet welds are determined at each site using a medium gauge. S1 and S2 shown in Figure-10, Size, are measured and a smaller figure is used.

Figure-10.

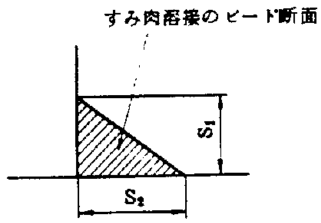


図 — 10

1. Cross section of the bead in fillet-welds.

The average of the size and throat thickness determined at various sites on the welding line are called s and a, respectively, and the smaller of 0.7s and a becomes the effective throat thickness (a_e).

The effective length (b_e) of weld is the width of the material to be welded in complete boxing; and the weld length minus 2 X (the size of the fillet) in other types of welding.

If there are no scallops in the beam web and diaphragms and the fillet weld is applied to the entire periphery, the following computations are performed:

A) inside of the beam and flange joint (UF-1, LF-1)

be = bB - (btw + br)

B) beam end web joint (BW-1, BW-2)

be = bh - br

C) diaphragm-flange joint

be = dB - (ctw + cr)

where bB. dB, and bh are the dimensions shown in Figure-9; while ctw, br, and Cr are beam web thickness, column web thickness, radius of curvature of the fillet of the beam member, and radius of the curvature of the fillet section of the column member, respectively.

The effective throat cross section of the welded joint (Ae) is computed in the following manner:

Ae = ae x be

Breaking axial strength, wPB and two breaking-shearing strengths wQB's are computed for each welded joint, based on the type of steel for the base metal observed in item iii; and using the effective throat thickness (ae) and effective length of welding (be) of the welded joint investigated in item iv.

In the following instances, however, index Q is rated D regardless of the weld length and thickness:

A) The welded joint specified to be a butt-weld on the drawing is found to be a continuous fillet-weld with an effective throat thickness (ae)(investigated in item iv) that measures less than 0.5 x (thickness of the base metal to be butted);

B) If the joint specified to be butt-welded is found to be a block fillet-weld during the test of itm ii, other connections are also inspected by similar methods to determine the types, effective throat thickness, and lengths of their welded joints.

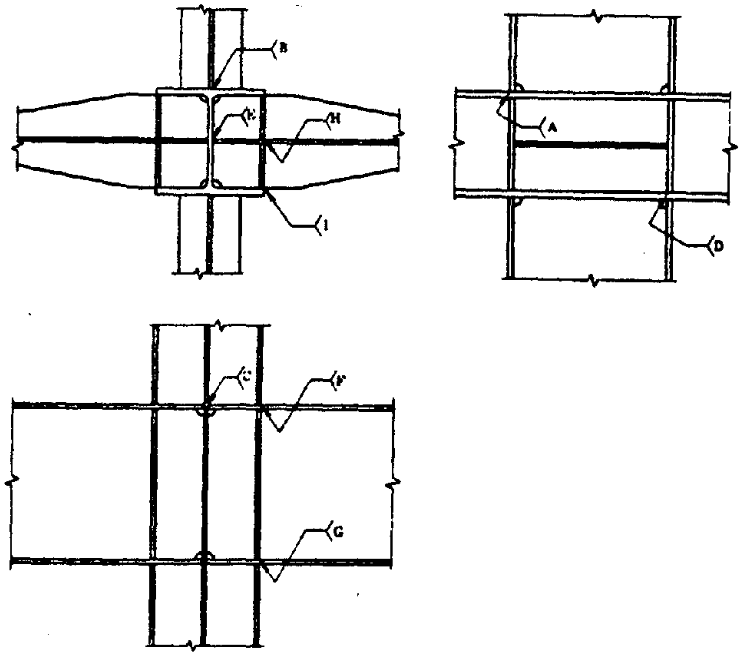
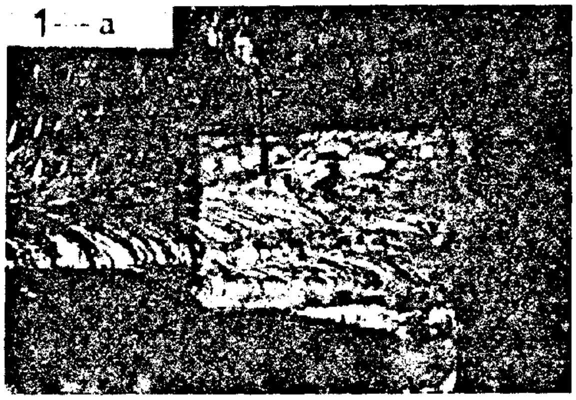


Figure-11.

For instance, at the column-beam connection shown in Figure-11 (where an H-type steel beam is bound with a rigid connection to an H-type steel box column with a cover plate), types and effective throat thickness and lengths must be investigated on all the joints in the section corresponding to A-I. However, the welded joints corresponding to C, D, and E cannot be investigated by an external inspection (determination of the type of welded joint corresponding to D is possible using an ultrasonic flaw detection method). The ultrasonic flaw detection method is needed for accurate determination of welded types of joints corresponding to F, G, H, and I. These welded joints are comprehensively evaluated based on the results of the ultrasonic flaw detection method, external observations of A and B (F, G, H, and I as well), details of the drawings, details of the construction outlines, and the conditions of the construction.

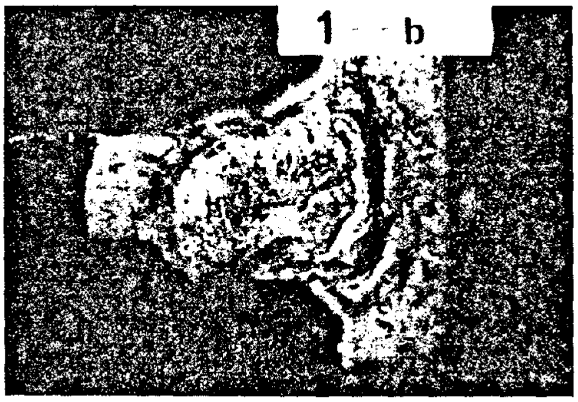
v) The findings on each welded joint are rated as (a), (b), or (c) at each item of evaluation for respective joint types. These ratings are necessary to determine index Q and must be entered on the form.

The item entitled "treatment of the edges of welding" describes the construction conditions of the butt- and continuous fillet-welded joints. The former may be (a) treatment of the end tab, (b) in boxing, and (c) in the manner shown in Figures-12 (a), (b) and (c); the latter may be (a) treatment in boxing or (b) without boxing but in the manner shown by Figures-13 (a) and (b). Figure 13 (a) shows an example in which scallops are created for web treatment associated with beam and flange welding.



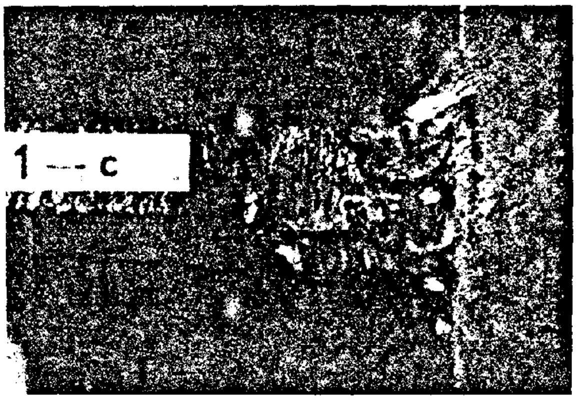
(a)

(a) end tab treatment



(b)

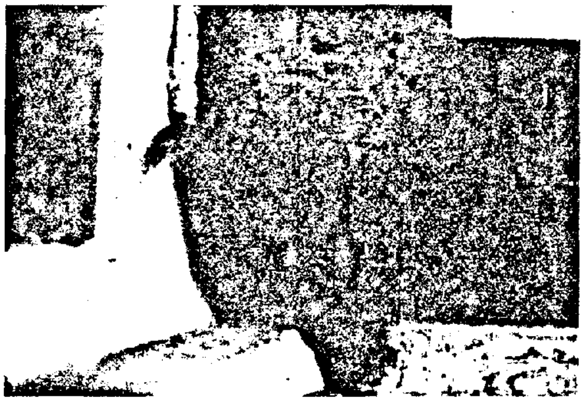
(b) boxing treatment



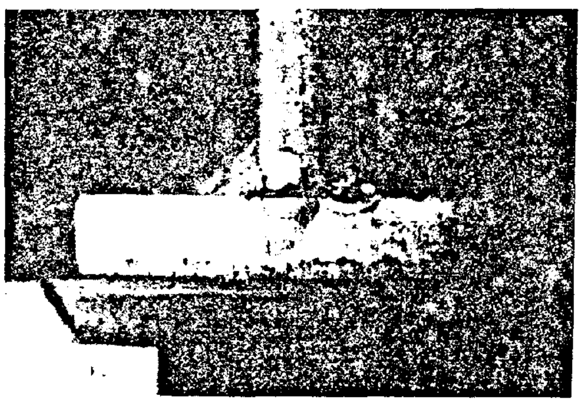
(c)

(c) an example of
welded edge treatment
(butt-welding)

Figure-12. Examples of welded edge treatment
(butt-welding)



(a) treatment in boxing

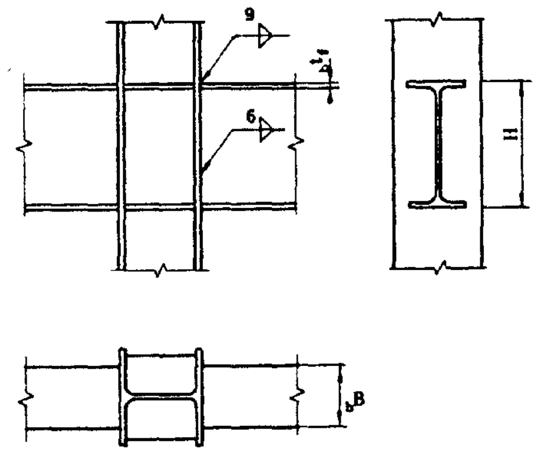


(b) without boxing

Figure-13. Examples of welded edge treatment
(fillet-welding)

Example 5.

The beam end welded joint shown in Figure-14 is evaluated according to the results of the investigation.



* column H-300 x 300 x 10 x 15 (r =18)

* beam H-600 x 200 x 11 x 17 (γ = 22)

* steel SS41

for computation of stress

$\sigma Y = 2.64$ t/cm3

$\sigma B = 4.51$ t/cm3

Figure-14

(The results of the investigation) The following was discovered from the results of the investigation:

1. Continuous fillet-welding was applied to both sides of the beam end flange joint. The welding thickness was S=9mm (the architectural design specifies butt-welding).
2. Continuous fillet-welding was also applied to both sides of the beam end web joints. The thickness of welding was S=6mm (the architectural design specifies S=11mm).
3. Scallops were absent at the beam web (the design calls for scallops

measuring b = 35mm).

4. The column and beam sections meet the design specifications.

i) Preliminary computations.

A) Beam end flange joint:

effective throat thickness ae = 0.7S = 0.7 x 0.9 = 0.63cm

effective length of the weld

exterior be = 20cm

interior be = bB - (btw + br)

= 20 - (1.1 + 2.2) = 16.7 cm

B) Beam end web joint

effective throat thickness ae = 0.7 S = 0.42cm

effective length of weld be = bh - br = (60-3.4) - 2.2

= 54.4 cm

ii) Evaluation of bending resistance.

A) Maximum bending moment of the beam end MB

$$MB = (Ae)\ \text{flange} \times H \times \frac{1}{\sqrt{3}}\, \sigma B$$

$$+ 2\ (ae)\ \text{wed} \times \frac{(be)2\ \text{wed}}{4} \times \frac{1}{\sqrt{3}}\, \sigma B$$

= 0.63 (20 + 16.7) x 60.0 x 0.577 x 4.51

+ 0.5 x 0.42 x (54.4)2 x 0.577 x 4.51

= 3610 + 1617 = 5227 t·cm

B) Total plastic moment of the beam MY

MY = 1.15 x Z x σY

= 1.15 x 2590 x 2.64 = 7863 t.cm

C) Stress increase rate at the joint bγm

$$b\gamma m = \frac{MB}{MY} = \frac{5227}{7863} = 0.66$$

iii) Evaluation of the shearing resistance.

A) The maximum shearing resistance of the beam end QB

$$QB = (2\ ae \times ba)\frac{1}{\sqrt{3}}\sigma B$$

$$= 2 \times 0.42 \times 54.4 \times 0.577 \times 4.51 = 118.9\ t$$

B) shearing resistance of the beam QY

$$QY = (H - btf)\ btw \times \frac{1}{\sqrt{3}}\ Y$$

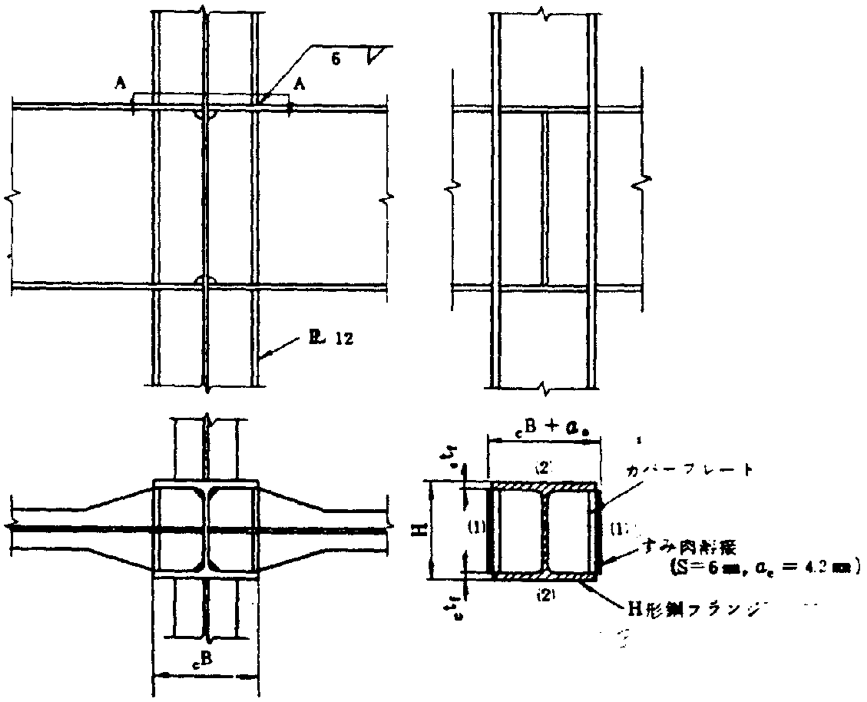
$$= 58.3 \times 1.1 \times 0.577 \times 2.64 = 97.7\ t$$

C) stress increase rate at the joint $b\gamma s$

$$b\gamma s = \frac{QB}{QY} = \frac{118.9}{97.7} = 1.22$$

Example 6.

The bending resistance of the column end welded joint is evaluated according to the test results when a box column, a steel H-column to which a reinforcement plate (a cover plate), is welded, is subjected to a bending stress along a weak axis (as seen in Figure 15):



* column H-300 x 300 x 10 x 15 (r =18)

  cover plate 2 - H 12

* beam H - 600 x 200 x 11 x 17 (r =22)

* steel SS41

  for computation of the resistance σY = 2.6[illegible] t/cm2,

  σB = 4.51 t/cm2

Figure-15

1. cover plate, 2. fillet-weld, 3. H-steel flange.

(Results of the investigation) The following were determined as the results of the investigation:

1. The column has a closed H-type cross section with a cover plate; but continuous fillet-welding is applied on one side of the column cover plate and the beam flange (with box welding); and the welding thickness was S=6mm (butt-welding was specified in the plan).
2. The dimensions of the column members, beam members, and cover plates conformed to the plan.
3. The building is low and the column axial strength need not be considered.

i) preliminary computation.

Column end flange joint

effective throat thickness ae = 0.7 S = 0.42 cm

effective weld length be = H - 2 ctf = 30 - 3 = 27cm

ii) evaluation of the bending resistance

The bending resistance of the column end joint in the column weak-axis direction is determined by : (a) the bending resistance determined at the welded section of the column cover plate and (b) the bending resistance borne by H-steel flanges (2) (Figure 24. cross section A-A).

A) The maximum bending moment in the direction of the weak-axis determined at the welded section of the column cover plate wMB

The axial tensile strength of the welded column cover plate section, wPB, is:

$$wPB = Ae \times \frac{1}{\sqrt{3}} \sigma B$$

$$= 0.42 \times 27 \times 0.577 \times 4.51 = 29.5 \text{ t}$$

therefore, the maximum bending moment is shown in equation (1):

$$wMB = wPB \times (cB + ae)$$

$$= 29.5 \times (30 + 0.42) = 897 \text{ t.cm} \quad (1)$$

B) The maximum bending moment, MBf, in the direction of the column weak axis, borne by H-steel flanges (2) is shown by equation (2):

$$MBf = 2 \frac{ctf \times cB2}{4} \sigma B$$

$$= 0.5 \times 1.5 \times 30^2 \times 4.51 = 3044 \text{ t.cm} \qquad (2)$$

wPB does not reach the yielding axial strength of the column cover plate, cPY = 85.5 tons. Therefore, equation (3) is adopted to express the maximum bending moment, MB, in the direction of the column weak axis.(note)

MB = MAX (wMB, MBf) = 3044 t.cm

(note) When wPB exceeds the yielding axial force of the column cover plate, cPY (85.5 tons), the following is used: MB=wMB+MBf

C) When the contribution by the H-steel web is ignored, the total plastic moment in the weak column axis direction, MYy, is expressed by the following:

$$MYy = 2 \frac{ctf \times cW2}{4} \sigma Y + 27 \times 1.2 \times (30-1.5) \times \sigma Y$$

$$= 1782 + 2438 = 4220 \text{ t.cm}$$

D) The column resistance increase rate, cτm, at the column end (Figure 15, cross section A-A):

$$c\tau m = \frac{MB}{MYy} = \frac{3044}{4220} = 0.72$$

5) Fastener joint.

1 表5　ファスナー継手の検討事項

2〔　重　要　事　項　〕	
3 検　討　事　項	4 検　討　内　容
1）接合部耐力 5 はり継手、柱継手についての部材強軸方向の終局曲げモーメントM_{Bx}	M_{Bx}について実態調査に基づく値M_{Bx}の設計図書に示された値${}_oM_{Bx}$に対する比の値γを求める。6

7〔　一　般　事　項　〕			
8 検　討　結　果	9 検　討　結　果　の　分　類		
	(a)	(b)	(c)
1）接合部の状況〔I〕 10 イ）設計図に指示された以外のファスナーの使用	11 使用せず	12 不明または0.15 n* 以内使用	13 0.15 n* を超えて使用
14 ロ）ファスナー孔があるがファスナーの入っていない所があるか	ない 15	全孔数の15 %以内に入っていない 16	全孔数の15 %を超えて入っていない 17
18 ハ）ファスナーの本数は設計図通りであるか	設計図通り 19		設計図と異なる 20
2）接合部の状況〔II〕 21 イ）接合面の密着	22 肌すきなし	23 僅かな肌すきあり	24 明らかに肌すきあり
ロ）腐食 25	ほとんどなし 26	かなりさびている 27	全面にわたってひどくさびている 28

* n：接合部の全ファスナー数 29

30 表12　ファスナー継手の個別評価（表5の検討）

31〔　重　要　事　項　〕	
32 検討事項と検討結果の状況	33 個　別　評　価
34 1）接合部耐力　　$\gamma \leq 0.7$ の場合	D

35〔　一　般　事　項　〕	
36 調査結果に対して、接合部の状況〔I〕では(c)に4点、(b)に2点、接合部の状況〔II〕では(c)に2点、(b)に1点の評価をそれぞれ与えてこの評点により下記にしたがって評価を行う。	
37 検討事項と検討結果の状況	38 個　別　評　価
39 1）接合部の状況〔I〕、2）接合部の状況〔II〕ともそれぞれ評点の小計が2点以内の場合	A
40 1）接合部の状況〔I〕、2）接合部の状況〔II〕ともそれぞれの評点の小計が4点以上、または両者を通じて評点の合計が6点以上の場合	C
41 上記以外の場合	B

1. Table 5. Items of evaluation for fastener joints.

2. (important matters)

3. items of evaluation

4. contents of evaluation

5. 1) resistance strength of the connection

   final bending moment, MBx, of the beam and column joints in the direction of the strong axis of the member.

6. γ, the ratio of MBx, the result of the actual investigation of MBx, to o MBx, that shown in the architectural plan, is computed.

7. (General matters)

8. The results of the investigation

9. rating of the results

10. 1) conditions of the joint (I)

    Use of fasteners other than those indicated in the plan.

11. not used

12. not known or less than 0.15 n* are used

13. more than 0.15 n* used

14. b) whether there are empty fastener holes

15. none

16. less than 15% of the holes are empty

17. more than 15% of the holes are empty

18. c) whether the number of fasteners corresponds to that specified by the drawing

19. the number match es that of the drawing.

20. differs from that of the drawing

21. 2) conditions of the joint (II)

    a) fit of the connection

22. no separation

23. slight separation

24. separation is evident

25. b) corrosion

26. almost none

27. a fair amount of rusting observed

28. extensive rusting on the entire surface

29. * n: the number of all the fasteners at the connection

30. Table 12. Individual evaluation of the fastener joint (evaluation of Table 5).

31. (important matters)

32. items of evaluation and the conditions observed

33. individual rating

34. 1) when the resistance of the connection is $\gamma \leq 0.7$

35. (general matters)

36. In the conditions of joint (I), values, 4 and 2, are given to (c) and (b), respectively; based on these values, the following ratings are assigned.

37. Items of evaluation and conditions observed

38. individual rating

39. 1) the figure assigned to the conditions of joint (I) and 2) to joint (II) are over 4; or the sum of the two is over 6

41. none of the above.

The items of evaluation of a joint with a fastener are roughly divided into 3, among which resistance of the joint belongs to the (important matters).

In the evaluation of the joint resistance, attention is focused on the possible deviation of the joint condition from the specification in the original plan.

In the evaluation of the (general matters), 1) methods of installation of the fastener is observed in the "condition of the joint (I)"; and 2) the general condition of the entire connection is studied in the "condition of the joint (II)".

The content of the observation of (I) is believed to be more significant than (II) in the structural resistance of the joint. Therefore, more weight is given to (I) in the individual evaluation of the joint.

"Appendix 1, Forms to Enter the Test Results (No.10) and (No.11)" are used to record the results of the evaluation of the joint fasteners. In form (No.10), the details of the joint section are drawn and appropriat columns are filled. The items to be entered are:

a) sections and their dimensions of the connected members

b) position of the joint

c) the form, dimension, and plate thickness of the splice plate.

d) positions of the bolts (pitch and distance from the edge)

e) types, diameters, and the number of fasteners

In the above items, the figures listed in the architectural plans are also entered in the form. The fastener hole diameter, unless measured by removing the fastener, is recorded as fastener diameter plus 2 mm.

The resistance of the connection is computed from the above items. The computations are made by using Equations (5), (6), and (7) of

Appendix 2-1 and those of (33) and (34) of Appendix 2-6.

In the evaluation of the connection resistance, both the value derived from the details of the connection indicated in the architectural plans and that based on actual examinations are computed and δ, the ratio of the latter to the former, is derived.

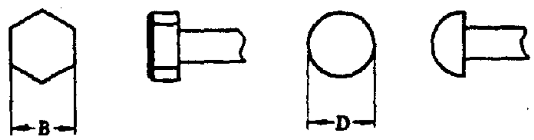
If the joint in question is mostly subjected to stress in the axial direction or to shearing forces, the resistance of the joint to these external forces is computed. Therefore, in the investigation of the fastener joint of a rigid axial bracing member, for instance, the tensile strength in the axial direction is evaluated in testing the resistance of both the connection and the member.

If the diameters of the high strength bolts, rivets, and other bolts are not known, determination of the diameter, D, of the round head section of the rivet or the width between two sides of the bolt head or the nut, B, and reference to Key Table-2 establish the diameters of these bolts and rivets.

The diameter of high strength bolts, H, will be used as its diameter, d, as the two are idnetical.

Key Table-2. The relationship between the (normal) diameter of the fastener and the width between two sides of the head section, B, or the head diameter, D.

[1]ファスナーの径(呼び)	[2]高力ボルトB(㎜)	[3]中ボルトB(㎜)	[4]リベットD(㎜)	[1]ファスナーの径(呼び)	[2]高力ボルトB(㎜)	[3]中ボルトB(㎜)	[4]リベットD(㎜)
M10		17	16	W $\frac{3}{8}$ (10φ)		17	16
M12	22	19	19	W $\frac{1}{2}$ (13φ)	21	21	21
M16	27	24	25	W $\frac{5}{8}$ (16φ)	26	26	26
M20	32	30	32	W $\frac{3}{4}$ (19φ)	32	32	30
M22	36	32	35	W $\frac{7}{8}$ (22φ)	35	35	35
M24	41	36	38	W 1 (25φ)	41	41	40
M27		41	39.5	W $1\frac{1}{8}$ (28φ)		46	45
M30		46	42.5	W $1\frac{1}{4}$ (32φ)		50	51



1. the diameter of the fastener
2. high strength bolt B
3. medium bolt B
4. rivet D

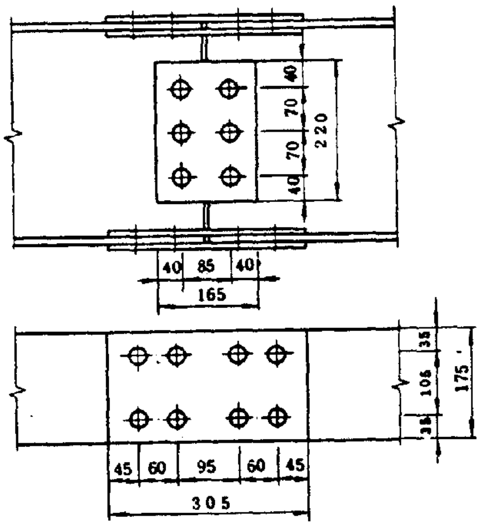
The results of the investigation of the connection conditions are entered on form (No.11). The investigation is mainly performed by external examination. The content of the specific findings are listed in form (No.11). The condition of the fastener opening cannot usually be determined unless the fastener is removed; but, if the dimension is inaccurate and the openings are marred from the use of a reamer or cut by flame, the evidences of such processes are evident and external observation may be sufficient. Extremely poor conditions may affect computation of the resistance strength of the connection. In such instances, investigation by detaching the fastener is necessary.

Even when the entire connection section is coated, a lapse of time and development of rust make the observation of the condition of the fitting surface of a high strength bolt extremely difficult. In such instances, the item may be omitted.

Example 7.

Example of deviation of a beam joint made of rolled H-steel H-350 x 175 x 7 x 11 (S S 41) is evaluated. Its design is shown in Figure-16.

Figure-16.



Flange connecting section

high strength bolt 4 - N 20 (F 10 T) (one side)

splice plate 1. 6mm x 305mm x 175mm

splice plate 2. 6mm x 305mm x 65mm, 2 plates

pitch p=60mm, distance from the end el=45mm, distance from the edge e2=35mm

Web connecting section

high strength bolt 3 - M 20 (F 10 T) (one side)

splice plate 6mm x 220mm x 165mm, 2 plates pitch p=70mm, distance from the end e1=40mm, distance from the edge, e2=40mm

(a. When the condition matches the original design)

The resistance of the connection is computed according to the details shown in Figure 16. There are no data concerning the bolt hole diameter and (bolt diameter + 2mm) was used in the computation.

$Pbf1 = 1PBf1 = Ae \times \sigma B = \{(17.5 - 2.2 \times 2) + 2\ (6.5 - 2.2)\} \times 0.6 \times 4.51$

$= 58.7$ t

$PBf2 = 1PBf2 = 0.75n \times m \times Ab \times b\sigma B = 0.75 \times 4 \times 2 \times 3.14 \times 11 = 207$t

$PBf3 = 1PBf3 = n + e1 \times t \times B = 4 \times 4.5 \times 1.1 \times 4.51 = 89.3$ t

$\therefore PBf = 1PBf = 58.7$ t

$PBw1 = 1PBw1 = Ae{*}1 \times \sigma B = [35 - 2\ (1.1 + 1.4) - 3 \times 2.2] \times 0.7 \times 4.51$

$= 73.9$ t

$PBw2 = 1PBw2 = 0.75\ n \times m \times Ab \times b\sigma B = 0.75 \times 3 \times 2 \times 3.14 \times 11 = 155$ t

$PBw2 = 1PBw2 = n \times e1{*}2 \times t \times \sigma B = 3 \times 4.0 \times 0.7 \times 4.51 = 37.9$ t

$\therefore PBw = 1PBw = 37.9$ t

Notes * 1: Considering the external force in the direction of the member axis, the effective cross section of the web section is taken in the direction perpendicular to the member axis. If H-steel is used, the figure derived by subtracting the bolt hole area from the web portion between the upper and lower web fillet edges and the smaller figure of the effective cross section of the splice plate in the same direction is used.

2* Considering the axial direction, the edge distance in the same direction is used in the computation of the web section resistance.

The maximum bending moment of the connecting section and the total plastic moment of the connected member are computed as follows:

$MBx = 1MBx = (1PBf + 1/8\ 1PBw) \times H = (58.7 + 37.9/8) \times 35 = 2220$ t.cm

$\therefore \gamma = 1.0$

(b. When the fastener used differs from the drawing)

When medium M20 bolts are used at the flange and web sections instead of high strength bolts, the values of 1PBf2 and 1PBw 2 are altered from the above computation.

$1PBf2 = 0.75n \times m \times Ab \times b\sigma B = 0.75 \times 4 \times 2 \times 3.14 \times 4.51$

$= 85.0\ t \geq 1PBf1 = 58.7\ t$

$1PBw2 = 0.75\ n \times m \times Ab \times b\sigma B = 0.75 \times 3 \times 2 \times 3.14 \times 4.51$

$= 63.7\ t > 0PBw2 = 37.9t$

Therefore, there is no change in the computation of the resistance of the connection. However, the rating of (A) of the general matters, the condition of the connection (1) becomes (c).

(c. When the position of the bolt hole is inaccurate and the distances from the end in both the flange and web are smaller than the figures given in the design).

e_1 of the flange is set at 30mm and that of the web at 35mm. Furthermore, actual flange web thicknesses of the beam are 10.2mm and 6.3mm. Considering the results of the computation of a, 1PBf3 and 1PBw3 are computed.

$1PHf3 = n \times el \times t \times B = 4 \times 3.0 \times 1.02 \times 4.51 = 55.2\ t$

$1PBw3 = n \times el \times B = 3 \times 3.5 \times 0.63 \times 4.51 = 29.8\ t$

Therefore, the following is deducted: 1PHf=1PBf3=55.2 t, 1PBw=1PBw =1PB w3=29.8 t.

$1MBx = (1PBf + 1/8\ 1PBw) \times H = (55.2 + 29.8/8) \times 35 = 2062\ t.cm$

$_{o}MBx = ({}_{o}PBf + 1/8\ {}_{o}PBw) \times H = (58.7 + 37.9/8) \times 35 = 2220\ t.cm.$

$\gamma = 1MBx/{}_{o}MBx = 2062/2220 = 0.93$

(d. When the flange joint takes the form of a single shear connection using a splice plate measuring 12 mm in thickness attached outside and a medium bolt M 20 is used as a fastener)

The distance from the edge and pitch are made identical to that used in a. Normal figures are used for all plate thicknesses.

1PBf1 = Ae x σB = (17.5 - 2.2 x 2) x 1.1 x 4.51 = 65.0 t

1PBf2 = 0.75 x n x m x Ab x bσB = 0.75 x 4 x 1 x 3.14 x 4.51 = 42.5 t

$\therefore$ 1PBf = 1PBf2 = 42.5 t

1MBx = (1PBf + 1/8 1PBw) x H = (42.5 + 37.9/8) x 35 = 1653 t.cm

γ = 1MBx/o MBX = 1653/2220 = 0.74

The medium bolt is replaced with a high strength bolt for improvement.

6) Diagonal bracing of the frame.

Table 6. Items of evaluation for diagonal bracing materials.

1〔重　要　事　項〕	
2 検　討　事　項	3 検　討　内　容
1）耐　力 4 イ）筋違材の有効断面における引張耐力 P_{B1} ロ）筋違材とガセットプレートとの接合部耐力　P_{B2} ハ）ガセットプレートの耐力　P_{B3} ニ）ガセットプレートと柱またははりとの接合部耐力 P_{B4} イ）からニ）の最小値を筋違材の引張最大耐力 P_B とする。	5 P_B について実態調査に基づく値 ${}_1P_B$ の設計図書に示された値 ${}_0P_B$ に対する比の値 r

6〔一　般　事　項〕			
7 検　討　事　項	8 検　討　結　果　の　分　類		
	(a)	(b)	(c)
1）部材の状況 9 イ）たわみ・ゆるみ	10 な　し	11 目につく程度	12 軸組筋違材として効果が少ない
13 ロ）腐食（断面積の減少）	ほとんどなし 14	10 %以下 15	10 %を超える 16

Table 13. Individual evaluation of the diagonal bracing material (evaluation of Table 6)

1〔重　要　事　項〕	
2 検討事項と検討結果の状況	3 個　別　評　価
1）耐　力 4 $r \leq 0.7$ の場合	D
5〔一　般　事　項〕	
6 検　討　結　果　の　状　況	7 個　別　評　価
(c)がなく、(b)が1つ以下の場合 8	A
〔(c)の数 + 0.5 ×〔(b)の数〕〕≧ 1 の場合 9	C
上記以外の場合 10	B

Table 6. Items of evaluation for diagonal bracing materials.

1. important matters
2. items of evaluation
3. contents of evaluation
4. 1) Stress
   A) tensile resistance, PB1, at the effective section of the bracing material
   B) Stress, PB2, of the bracing material and gusset plate connections
   C) stress, PB3, of the gusset plate
   D) stress, PB4, of the gusset plate and column or beam connections
   The minimum value among A) to D) is called the maximum tensile resistance, PB, of the bracing material.
5. the ratio of $_{1}PB$ derived from the actual investigation of PB to ^{0}PB indicated in the drawing becomes γ.
6. general matters
7. items of evaluation
8. rating of the results of investigation
9. 1) conditions of the member
   A) deflection and luxation
10. none
11. barely noticeable
12. ineffective as the bracing
13. B) corrosion (reduction in the cross section)
14. almost none
15. less than 10%
16. over 10%

Table 13. Individual evaluation of the bracing material (evaluation of Table 6)

1. important matters
2. items of evaluation and description of the conditions
3. individual rating
4. 1) stress
   if $\gamma \leq 0.7$
5. general matters
6. description of the conditions
7. individual rating
8. no (c) and less than 1 (b)
9. if $\left[\text{the number of (c)} + 0.5 \times \left\{\text{the number of (b)}\right\}\right] \geq 1$
10. none of the above.

The diagonal bracing materials investigated here are light materials such as round and angle steel. The evaluation includes not only the bracing materials but also the connections of the bracing to the gusset plate, and gusset plate to the column or beam.

When H-type steel or steel pipes are used for diagonal bracing, they are treated as if they are column or beam members. Their joints are evaluated according to the methods applied to the fastener joints and their connections with columns and beams according to those applied to the column-beam connections.

The stress of the bracing is included in the important matters. The stress discussed here is the ultimate stress of the above described items and is derived from equations (35) to (40) of Appendix 2-8 using the actual dimensions of each part. Therefore, the purpose of examination is to obtain the data necessary for these computations.

When the dimensions of each part are determined, the ultimate stress of the bracing is computed. In this computation, the tensile stress of the effective section of the bracing, stress of the connection of the bracing material and the strap metal, the stress of the connection of the strap metal and gusset plate, stress of the gusset plate itself, and stress of the connection of the gusset plate structure are computed. The minimum of these is called 1PB.

If the bolt hole diameter is not investigated, an axial diameter + 2mm is used. When 1PB is determined, a theoretical figure of the ultimate stress, 0PB, is derived from the figure on the drawing of the same section. From this γ=1PB/0PB is computed.

The stress is evaluated based on the deviation of the state of the shaft bracing, including the associated connections, from that shown in the drawings.

If, in the result of the investigation of the "Important Matters," is less than 0.7, the bracing in question is rated D in its individual evaluation.

If the rating is not D, the results of the investigation of the "Important Matters: are used in the computation of Index V.

In the examples in which round steel was used for bracing and the members were butt-welded near the turnbuckle (as in Figure-17), none has been found welded in a proper manner in the past investigations on earthquake damages. Such a method of welding is totally unreliable in stress resistance and the bracing is considered ineffective. Subsequently, a rating of D is given in the individual evaluation.

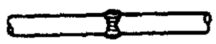


Figure-17

Among the "General Matters", the conditions of the members and corrosion are evaluated.

In the investigation of item A) deflection and luxation among the General Matters, the rating is decided in the following manners: luxation of the bracing is frequently observed in light tension bracing structures; when luxation exists in the bracing, such a structure not only fails to bear the initial horizontal force but may invite rupture of the bracing members and connections due to an impact of tensile force during an earthquake; with the angle of the bracing set at 45° , luxation of the bracing, e/ℓ, (deflection at the center of its length), corresponding to the between layer displacement angle of the frame of 1/500, is roughly computed to be 2.6/100. From this e/ℓ value, the following criteria are set for evaluation: (a) less than 1/100, (b)1/100 to 3/100, and (c) over 3/100.

"Appendix 1, Form to Enter the Results of Investigation (No.12)" is used.

The investigation includes the bracing materials as well as their end connections. Therefore, dimensions of each section, including those of the connections with the columns and beams, are determined and entered in the drawings. The dimensions shown in the drawings should also be included as much as possible.

Example 8.

(The results of investigation)

1. Conditions of the connections.

Figure-18 shows the results of the investigation of the methods of connections and cross sections of members. The comparisons between the actual dimensions and the values shown in the drawings are in Key Table-3.

No deflection or luxation of the bracing was noted. The steel type is S S 41.

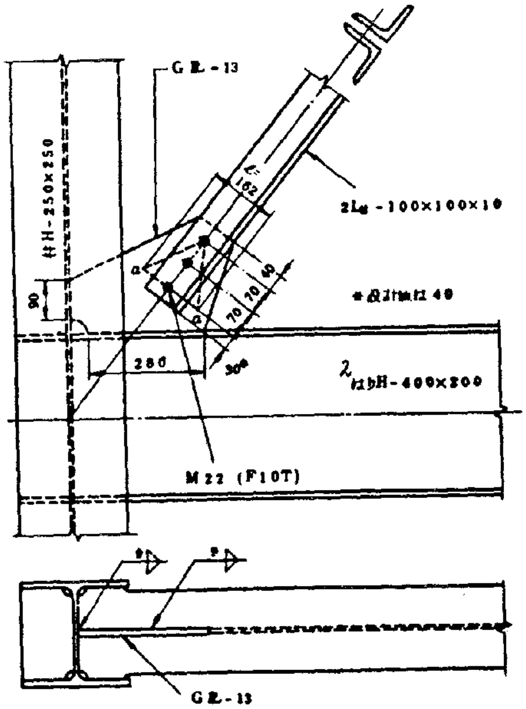


Figure-18.

1. * the value in the drawing is 40.

2. beam H-400 x 200

Key Table-3.

	1 設　計　値	2 実　測　値
筋違材断面 同断面積 A_g (cm²)	2 Ls − 100 × 100 × 10 2 × 19.0	2 Ls − 98 − 98 − 9.5 2 × 17.9
高力ボルト孔径 (cm)	2.85	5 2.4 (推定)
ガセットプレート厚 (cm)	1.3	1.27
7 ガセットプレートと骨組とのすみ肉溶接脚長 (cm)	0.9	0.9
8 高力ボルト 同ピッチ (cm) 同はしあき (cm)	8 − M 22 (F 10 T) 7.0 4.0	9 同　左 7.0 8.0
10 鋼　種	SS 41 (σ_B = 4.51 t/cm²)	

1. values of the drawings.

2. actual dimensions

3. cross section of the bracing

   area of the cross section Ag (cm2)

4. high strength bolt hole diameter

5. (estimated)

6. gusset plate thickness

7. fillet-welding thickness of the gusset plate and frame

8. high strength bolt

   pitch (cm)

   distance from the end (cm)

9. left

10. steel type

ii. Computation of the stress.

A) the maximal tensile force PB1 involving the effective cross section of the bracing.

designed value $0PB1 = Ae \times \sigma B = 2 \times (19.0 - 2.35 \times 1.0) \times 4.51 = 150$ t

actual value $0PB1 = 2 \times (17.9 - 2.4 \times 0.96) \times 4.51 = 141$ t

$\therefore \gamma = 1PB1/0PB1 = 141/150 = 0.94$

B) Stress, PB2, on the bracing and gusset plate connection

stress on the end section of the angle steel

designed value $0PB2 = n \times e \times t \times \sigma B \times 2 = 3 \times 4.0 \times 1.0 \times 4.51 \times 2 = 108$ t

where n : the number of fasteners

e : distance from the end

t : plate thickness

actual dimension : $1PB2 = 3 \times 3.0 \times 0.98 \times 4.51 \times 2 = 79.6$ t

$\therefore \gamma = 79.6/108 = 0.74$

Stress on the high strength bolt:

$_0P_{B2} = {_1P_{B2}} = n \times m \times 3/4\ A_b \times {_b\sigma_B} = 3 \times 2 \times 3/4 \times 3.80 \times 11$

$= 118\ t$

where n: the number of fasteners

m: the number of friction surfaces

Ab: area of the cross section of the fastener shaft

$_b\sigma_B$: tensile strength of the fastener

$\therefore\ \gamma = 1.0$

C) Stress, PB3, of the gusset plate.

evaluation at the a-a section of Figure-18.

designed value: $_0P_{B2} = \ell_e \times t \times \sigma_B = (16.2 - 2.35) \times 1.3 \times 4.51$

$= 81.2\ t$

actual value : $_1P_{B2} = (16.2 - 2.2) \times 1.27 \times 4.51 = 79.0\ t$

$\therefore\ \gamma = 79.0/81.2 = 0.97$

the end section:

$_1P_{B3} = 3 \times 4.0 \times 1.27 \times 4.51 = 68.7\ t$

D) Stress on the gusset plate and frame connection, PB4

$_0P_{B4} = {_1P_{B4}} = (28 + 9) \times 2 \times 0.7 \times 0.9 \times 4.51/\sqrt{3} = 121\ t$

$\gamma = 1.0$

7) Column Base.

Table 7. Items of evaluation for the column base

1〔 重 要 事 項 〕			
2検討事項	3 検討内容		
1）柱脚 4 アンカーボルトの有効断面積の総和	5 調和結果で得られた有効断面積の総和の設計図書で指示された有効断面積の総和に対する比の値 r を求める。		
6〔 一 般 事 項 〕			
7検討事項	8 検討結果の分類		
	(a)	(b)	(c)
1）柱脚 9 i）根巻きのある柱脚 イ）根巻きコンクリートのかぶり厚さが設計図書に比して不足していないか＊1	不足していない 10	11不足が 15 % 未満であるが、かぶり厚さ ≧ 5 cm	不足が 15 % 以上、またはかぶり厚さ ＜ 5 cm 12
13 ロ）根巻きコンクリートの高さが設計図書に比して不足していないか＊2	不足していない 14	不足が 15 % 未満であるが、高さ ≧ 柱径の 2 倍 15	不足が 15 % 以上、または高さ ＜ 柱径の 2 倍 16
17 ハ）コンクリートのひびわれ状況	ヘアクラック程度 18	やや大きい 19	非常に大きい 20
21 ニ）コンクリート上面での鉄骨柱のさび発生状況（板厚の減少）	ほとんどなし 22	10 % 以下 23	10 % を超える 24
ii）根巻きなしの柱脚 25 イ）有効なアンカーボルトの本数が設計図書と異なっていないか＊3	異なっていない、または異なっていても多くなっている。 26	不明、または 20 % 以内不足 27	20 % を超える不足がある 28
29 ロ）アンカーボルトの径が設計図書と異なっていないか	30異なっていない、または異なっていても太くなっている	不明、または 15 % 以内小さくなっている 31	15 % を超えて小さくなっている 32
ハ）柱脚部分のさびの発生 33状況（板厚の減少）	ほとんどなし 34	10 % 以下 35	10 % を超える 36

Table 14. Individual evaluation of the column base (evaluation of Table 7)

1. (important matters)
2. items of evaluation
3. contents of evaluation
4. 1) column base

   the sum of the effective surface area of the anchor bolt section
5. compute the ratio, γ, the sum of the effective areas of the sections derived from harmonic analysis to that indicated by the drawing
6. (general matter)
7. items of evaluation
8. rating of the results of the investigation
9. 1) column base

   i) column base with a footing

   A) whether the thickness of the concrete footing is sufficient according to the drawing specification *1
10. sufficient
11. insufficiency is less than 15% but the thickness is ≥ 5 cm
12. insufficiency is over 15% or the thickness is < 5 cm
13. B) whether the height of the concrete footing is sufficient according to the drawing specification *2
14. sufficient
15. insufficiency is less than 15% but the height $\geq$ x column diameter
16. insufficiency is over 15% or the height < 2 x column diameter
17. C) cracking of the concrete
18. hairline cracks
19. somewhat larger cracks
20. extremely large cracks
21. D) a rusting condition of the steel-skeleton above the concrete (reduction in plate thickness)

22. hardly recognized

23. less than 10%

24. over 10%

25. ii) column base without footing

A) whether the number of the effective anchor bolts differs from the drawing specification *3

26. the number does not differ or more bolts are used

27. not known or reduced by 20%

28. reduced by over 20%

29. B) whether the diameter of the anchor bolt differs from the drawing specification

30. meets the specification or the diameter is larger

31. unknown or the diameter is reduced by 15% or less

32. the diameter is reduced by more than 15%

33. C) rusting of the column base section (reduction is plate thickness)

34. hardly recognized

35. less than 10%

36. over 10%

*1, *2 : If the overlay thickness and the height are insufficient even when the actual dimensions meet the drawing specification, a rating of (b) is given.

*3 : If no nuts are attached to the anchor bolts or nuts are found to be ineffective, the anchor bolts are rated ineffective.

Table 14. Individual evaluation of the column base (evaluation of Table 7)

1. (Important Matters)

2. items of evaluation and an observation of the results

3. individual rating

4. 1) column base $\gamma \leq 0.7$

5. (General Matters)
6. observation of the results
7. individual rating
8. no (c)'s and more than 2 (b)'s
9. the number of [(c)'s + 0.5 x {the number of (b)'s}] $\geq$ 2
10. none of the above

The column bases are roughly divided into two types--those with footings and those without. When the footing exists, investigation is limited to an external examination due to difficulty of making repairs after chipping the concrete for investigation, as well as the possibility of causing serious damage such as cracking of the base slab in the process of chipping.

Therefore, the data from the drawing are entered for the section covered by concrete. The major items to be observed in the investigation are: the height of the concrete footing, thickness of the covering on the steel skeleton, conditions of corrosion of the steel skeleton column over the concrete surface, and cracks in the concrete.

Specific notes on each item of investigation are given below:

i) The thickness of the covering of the concrete footing.

Considering the possibility of contact of the column base portion with the earth and the distribution of the strutures, more than 7-8 cm is necessary for the minimum thickness of the covering. The present evaluation adopts a minimum covering thickness of 5 in accordance with the "Standard for Computation for Steel Skeleton-Steel Reinforced Concrete Structures" proposed by the Japan Society of Architecture.

ii) The height of the footing.

According to the results of past experiments, a foot height which is 3 times the diameter of the column is believed to be sufficient to provide rigidity and withstand stress forces. Columns measuring more than 200mm in diameter are often used and twice this figure, 400mm or over, should be sufficient as the height of the footing to maintain its rigidity and stress force. Therefore, the standard was set at the diameter of the column x 2.

iii) Cracking of the concrete footing.

If cracking is noted in the concrete footing, the extent of these defects are determined by the widths and lengths of the cracks. These findings are recorded on the drawings.

iv) The number and diameters of the anchor bolts.

Examination of columns with no footings is relatively easy and can be readily compared with the details in the drawings. An investigation includes the number of anchor bolts, their diameters, and the conditions of reinforcement applied to the columns and base plates.

The investigation is not limited to the quantity and diameters but includes a determination of the actual effectiveness of these bolts. If the thread section is unmarred, unusually clean, or projects more than usual, it is possible that a sham bolt was merely inserted later. A nut too large which does not fit the bolt may be used. Special attention must be paid to these conditions.

v) Corrosion of the column base.

If corrosion is noted in the column base section, rust is removed using a wire brush and reduction of the thickness of each section and the extent of corrosion are determined.

vi) Condition of reinforcement of the column base section.

The types of reinforcement format are entered on the form. If the reinforcement materials are joined by welding, whether by butt- or fillet-welding,it is noted in the Note Section and the size of the weld is examined. If the reinforcement is joined by rivets or bolts, its relationship to the architectural plan is recorded.

vii) Determination of each dimension.

Determination of the dimensions of parts such as column members, base plates, reinforcement rib plates, anchor bolts (and their positions), concrete footings is made using one or more of the following: a yard stick, tape measure, gauges.

Modification of the Data from the Investigation to Be Applied for Computation of Index V.

As a rule, the results of the investigation of the (Important Matters), unless D is given in the individual evaluation, are reflected in the computation of Index V. The basic concept of inclusion of these data and the methods of inclusion concerning each item of investigation are detailed in the following.

"Appendix 1, Form to Enter the Results of the Investigation, Summary of the Investigation for Determination of Index V" is used in this process.

If there is a discrepancy between the results of the investigation and the architectural plan specifications, the cause for such a discrepancy is investigated first; then it is determined whether such a deviation is localized or entensive affecting the entire structural frame.

If the discrepancy is localized, the information concerning the structural frame is partially modified. If necessary, the number of investigatinn sites can be increased. Wheh the results of investigation items--such as reinforcement of the panel section of the column-beam connection, presence or absence of the diaphragm and its connection, types of welding and its effective cross sectional areas, and installation of the column base--indicate possible involvement of the entire structural frame, the values indicated in the architectural plan are extensively modifiád, based on actual observations of these items, prior to computation of Index V. Considering factors such as the accuracy of the determination, errors at the time of construction, tolerance of the members, and accuracy in computation of Index V, deviations within ±15cm in the dimensions of the shaft parts and within 10% for other items are ignored and the values indicated in the architectural plan are used in the computation.

i) General frame structure.

The actual structural dimensions of items such as spacing between column members in the directions of girders and beams and floor heights a re used as they are in the computation.

If a defect exists in a major member, this fact and the condition of the frame around the member with such a defect are reflected in the computation.

ii) Functions of the cross sections of columns and bemas.

If the findings on the areas of the cross sections and cross section coefficients of the members differ from the architectural plan specifications, such findings are reflected in the computation. It is recommended that an average of the actual value and the architectural plan value be computed for each member, such as a column or beam, and used as the modifying coefficient.

iii) Column-beam connections.

The actual value of the connecting panel equivalent panel volume, Vp, is reflected in the computation of Index V. In this instance, the modification coefficient used in the computation is derived from the ratio of the actual value to that indicated in the architectural plan. The person in charge of the investigation decides, according to the condition of the structure, whether a modification coefficient is derived for each combination of column and beam or an average from all the data is used.

iv) Welded joints.

If the types of welded joints, effective throat thickness, and effective length of weld differ from those indicated on the architectural plans, the results of the investigation are reflected in the computation of Index V. The methods are shown below:

A) Types of welded joints.

If the result of the investigation shows that butt-welding which was stipulated on the architectural plan has been replaced by continuous fillet-welding even at a single joint, all the butt-welds shown on the plans is replaced by continuous fillet-welds for the computation of Index V.

B) Effective throat depth.

If the result of the investigation shows that butt-welding is applied, the plate thickness of the butted base member is used as the effective throat depth.

If application of continuous fillet-welding is found, an average of the throat depth of one site and that of the opposing site and the average of 0.7 x (the result of the measurement of welding thickness) are computed for each joint (such as beam and flange joint, or beam end web joint) and the smaller of the two is used as the effective throat depth.

C) The effective length of a weld.

The effective length of a weld is computed from the width of the material welded and the condition of welding of the joint (end treatment of the welded joint and presence or absence of scallops).

In many instances, butt-welding which was specified in the architectural plans has been replaced by continuous fillet-welding. In these instances, fillet-welding is applied normally in a single layer-single bath regardless of the thickness of the plate to be connected. Therefore, investigation of several welded joints will give the general thickness of the fillet-welds and subsequently the effective throat depth of the general structure. By applying this effective throat depth to the entire structure, the stress of the welded connections can be computed. Similarly, the general condition can be surmised from the stress of the effective length of welding (used in computation of stress) of several locations.

v) Fastener joints.

The joint stress to be used in Index V computations is modified according to the results of the investigation when deviations from the architectural plans exist in types, diameters, number of fasteners, forms and dimensions of splice plates, pitch, and distance from the edge, with subsequent changes in the joint stress. As to the method of modification, the average of the minimal value of γ (the raio of the joint stress derived from actual observations to that obtained from the architectural plans may be multiplied by all the joint stresses. Or, a ratio of the joint stress to the stress of the material to be connected may be computed first; then the joint stress may be computed from that of the base material using this ratio.

If the γ values vary over a wide range, the lowest figure is adopted. If the range is small, on the other hand, an average is used.

vi) Bracing materials.

The stress of the bracing is often determined at their connections. However, details of the connections are missing in many architectural plans. For this reason, the actual stress of the bracing (including its connections) becomes important in computing Index V.

The ratio of the minimal value among the ultimate stresses computed at various parts to the yielding stress of the bracing base material (without a defect in its cross section) may be determined; and an average or the minimal value at each site of investigation may be used to compute Index V.

vii) The column base.

If the details of the column base are found in the investigation, the stress on the column base is computed in a pinned or locked position using these details regardless of the specification of the plan. The results of the investigation give only a few data and those with the lowest conditions are used in the computation.

2.5. Total evaluation of Index Q.

Based on all the individual evaluations obtained in the actual investigation, Index Q of the structural frame is determined from Table 15. In the evaluation of Index Q, the results of investigation of the "Important Matters" have a higher priority.

Table 15. The standard of evaluation for on-site investigations.

1 各調査部位において得られた個別評価の分類	2 Q指標
3〔 重 要 事 項 〕	
4 イ) 各調査項目の〔重要事項〕で1つの調査部位においても個別評価Dが与えられた場合	5 不 可
6〔 一 般 事 項 〕	
7 イ) 全調査結果を通じてCがなくBの数が$N^*/8$以下の場合	1.0
8 ロ) 全調査結果を通じてCがなくBの数が$N^*/8$を超える場合	0.9
9 ハ) 全調査結果を通じてCの数が1以上かつ$N^*/4$以下の場合	0.8
10 ニ) 全調査結果を通じてCの数が$N^*/4$を超える場合	11 不 可

1. Classification of hhe individual evaluation obtained at each site of investigation
2. Index Q
3. "Important Matters"
4. A) among the "Important Matters", an individual rating of D is given to at least one site of investigation
5. unsatisfactory
6. the "General Matters"
7. A) no C's and the number of B"s is less than N*/3 for all test results
8. B) no C's and the number of B's exceeds N*/3 for all test results
9. C) the number of C's is more than 1 and less than N*/4 for all test results
10. D) the number of C's exceeds N*/4 for all test results
11. unsatisfactory

N* is the total number of test sites.

Evaluation of Index Q is based on the results of investigation of all the sites for each test item.

These test results are expressed as individual evaluations for each test site. Therefore, each site and the results of individual evaluations of each test item are summarized in "Appendix 1, Form for Entering Results of Investigation (No.15)" prior to total evaluation of Index Q.

In filling in this form, the number of test sites, ni, is recorded for each item so that the total number of test sites, N, is readily derived through equation $N=\sum ni$. Detailed evaluation is needed for the section which is rated D, whish will be cicled to call attention to it. If a single D is given in an individual evaluation, Index Q is labelled "unsatisfactory". When an appropriate treatment has been applied to correct the condition, this rating of D is removed and individual evaluations (A-C) derived from the observation of the "General Matters" are adopted instead.

If no site is rated D in the individual evaluations, the individual evaluation at all test sites are summarized and total evaluation of Index Q is (as shown in Table 15) is made.

The total evaluation is based on the numbers of A's, B's, and C's. These alphabetic ratings for each test site are added prior to the total evaluation. In the bottom row, the notes are entered which give comprehensive results on the quality and aging changes.

Section 2. Computation of standard anti-earthquake index V.

The standard policy for the computation.

(1) Computations are made on the girder spacing and beam directions in the building.

(2) The present investigation excludes buildings constructed with steel other than SS41, SM41, STK41, STKR41, SM50, SM50Y, STK50, and STKR50.

(3) In the evaluation of structural strength, 1.1 times the minimal tensile strength, the yielding point set by the JIS standard of the above-listed steels, is used. In the evaluation of deformation function, 1.2 times the minimal value set by the JIS standard is used as the yielding point of the same steels.

(4) If the building is L-, T-, or]-shaped on a plane, the form is broken into rectangles (or squres) and anti-earthquake index V is computed for each rectangle.

(5) Computation of the standard earthquake resistance index is by one of the following two methods:

A. according to "Standard for Anti-earthquake Resistance Investigations of Pre-Existing Steel-Skeleton Buildings (edited by Architectural Instruction Section, Department of Housing, Ministry of Construction Chapter 2, Section 2 (Computation of the Standard Earthquake Resistance Index V).

B. Computed by a computer using the program developed by Shizuoka Prefecture.

(6) The data sheet used in computation of V by a computer is prepared according to the architectu ral plan.

If items are to be modified for computation of V subsequent to the actual investigation (refer to Section 1), the data sheet is prepared using the appropriate figures.

Section 3. Form Index S.

1. General Topics.

Form index S (hereafter called index S) is a quantification of factors which have ill effects on antiseismic capacity (such as complexicity of the form) but which are not reflected in index V. It is intended to modify inex V.

Computation of antiseismic index V is designed with an assumption that the building has a standard form. Therefore, structural antiseismic index, VR, of a building with a complex form must be computed with a certain modification of index V. Index S is the modification coefficient in such instances.

Among the factors which affect the antiseismic properties, those already indluded in the computation of index V are excluded and the following 5 items are discussed.

A coefficient is given to each item so that, if its effect is to be included in the computation of index V, it can readily be removed from index S.

2. Computation of form index S.

Equation (1) is used in the computation of index S.

$S = a1 \times a2 \times a3 \times a4 \times a5 \qquad (1)$

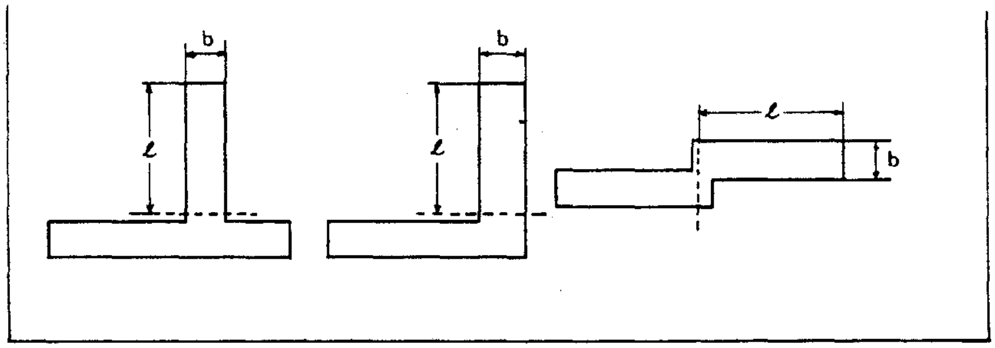
Figures are given in the following paragraphs for coefficients a1 to a5. If these figures are inappropriate, index S is set at 1.0. S is not less than 0.8.

Index S is expressed as the product of the 5 coefficients explained in the following. It is not below 0.8. The reason for setting the minimum at 0.8 is that the product of the coefficient is not very significant and simultaneous existence of many factors is unlikely.

2.1. Coefficient,al,for regularity of the form.

When the building plan shows an irregular form with an angle or a projection and the form ratio, ℓ/b, of the section forming an irregularity (such as a projection) is over 3:

$$al = 0.95 \qquad (2)$$

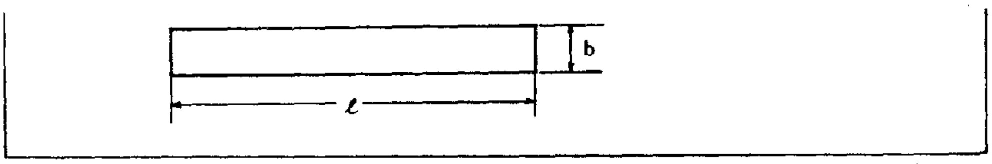


When the building plan shows a T- or L-form or has an indentation and irregularity, the form is divided into several regular forms and V index is computed for each section (as it has been stated in the paragraph describing the basic principle of index V computation in Section 2). The dynamic property of the steel-skeleton structure is characterized by a strong directionality and it is impossible to compute a single V index for an entire building which has a complex configuration on the plane. Instead, antiseismic property of each section of the building must be confirmed independently. On the other hand, though equipped with sufficient antiseismic property, behaviors of each section may be different during an earthquake. Therefore, VR of such a building should be different from those of buildings with regular forms. However, it does not seem necessary to reduce this index even further in spite of the multiplicity of the sections the building is divided into. It was decided that certain indentations and projections of a building can be ignored and the form regularity was limited to ℓ/b of 30%.

2.2. Coefficient, a2, for the ratio of two sides.

When the ratio of two sides of a rectangle, ℓ/b, is over 6,

$$a2 = 0.95 \qquad (3)$$



The following effects often become evident in the earthquake responses of buildings with rectangular planes:

a) the effects of deformation of the floor on the response,

b) the effects of varied seismic amplitudes and phases at various parts of the building foundation.

A reference describes the following:*

1) the antiseismic elements such as highly rigid bracings are coneentrated at both sides or at the center of the building,

2) a section with a number of floors different from the remainder exists at the center of the building,

3) a cection with reduced floor rigidity is located near the attachment of the bracing,

4) the antiseismic rigidity distribution is uneven in the direction of the height and the portion with a higher rigidity is located at the upper level.

When unfavorable conditions such as above exist, a dynamic analysis of such a building shows that the effect of the floor deformation becomes significant with the plane form ratio ℓ/b (ℓ: the length of the long side, b: the length of the short side) exceeding 6.

* Ko Jo "Study on Vibration Analysis by Two-dimensional Rearrangement of Points with Variable Properties in a Building", Hokkaido University Doctoral Thesis, March 1974.

2.3. Coefficient, $a3$, concerning the presence or absence of a collar brace.

When the foundation takes an independent form without a collar brace:

$$a3 = 0.9 \quad (4)$$

As seen in many instances of earthquake destruction, shifting of the foundation often causes damage even when the upper structure is sufficiently resistant to seismic forces. In general, the steel skeleton is relatively light and its force exerted on the foundation is small. This is particularly true in the case of low steel structure, foundations of which are often designed without much reinforcement. If the foundations are indpendent without connecting collar braces or foundation braces in steel-reinforced concrete buildings, they readily shift with a slight movement of the ground. Therefore, without collar braces or similar devices, damage due to shifting of the foundation is expected and the antiseismic properties of such buildings are believed to be reduced.

2.4. Coefficient, $a4$, concerning the stairwell.

When the area occupied by the stairwell exceeds 30% of the remaining area surround by two braces adjacent to each other on one floor or preceding or succeeding floor and the shafts running perpendicular to these braces:

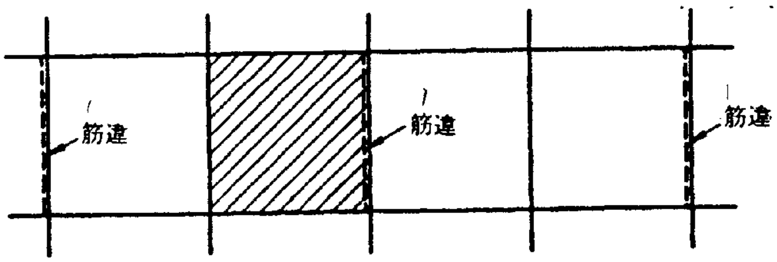
$$a4 = 0.95 \quad (5)$$

If a seismic force is expected to be transmitted through the floor to reduce its destructive force, such floor must have sufficient strength and rigidity. If a stairwell exists, on the other hand, both the strength and rigidity of such floor may be reduced. The area shown below (the area with oblique lines) --which is enclosed by the structures including the bracings (in a single floor or on the floor above or below) and supports running perpendicular to those braings--is expected to serve as a single unit in resisting the seismic force. If more than 30% of the area is used as a space for a stairwell, this resistance is believed to be somewhat reduced. Therefore, Index V is reduced accordingly.

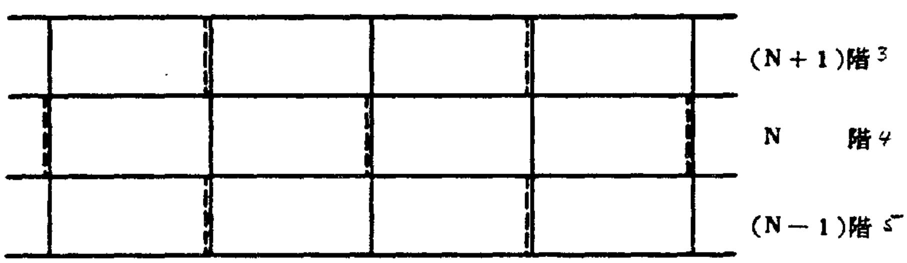
The space used for an elevator is considered to be a form of a stairwell. If an elevator core is created, however, such core is believed to constitute an antiseismic element and such stairwell is excluded from the present consideration. If the elevator is not equipped with such antiseismic elements, Index V is reduced according to the specification of the present section.

If the area used as a stairwell is extremely large, the portions on both sides cannot resist the seismic force as a single unit. Subsequently, each portion is evaluated independently for its antiseismic property.

The portion occupied by stairs only is not considered to be a stairwell.



2 N階平面図



6 軸組図

1. bracing
2. plane of the Nth floor
3. (N + 1)th floor
4. Nth floor
5. (N - 1)th floor
6. atructural plan.

2.5. Coefficient, a5, concerning the internal rigidity of the floor.

When the internal rigidity of the floor is dependent solely on the horizontal bracings of the steel skeleton:

$$a5 = 0.85 \qquad (6)$$

A steel-reinforced floor or a floor composed of concrete over a deck plate is considered to have sufficient strength and rigidity if the shear connector connecting the floor and the steel beams is effective. It is believed that, because of this floor, the bracings act as one unit to resist the seismic force. If precast block or ALC is used as the floor material, sufficient rigidity and strength cannot be expected from the floor and horizontal bracings become necessary.

Round bars and angle steel are normally used as horizontal bracings. Such horizontal bracings,however, lack in rigidity and strength when compared with steel-reinforced floor and a floor composed of concrete over a deck plate. Furthermore, ridigity of these bracings is even more reduced due to local deformation caused by vertical deflection of the member itself, luxation of the attachment, or a force. The strength of their connections are also insufficient in many instances. For these reasons, the horizontal bracings are insufficient to act as rigid plates. If the internal rigidity of the floor is to be maintained by the horizontal bracings alone, index V is reduced.

PART II: Diagnostic Method for Earthquake Resistance for Structures such as Gymnasiums, etc.

Chapter One:

1. Basic Policy

This method applies the existing method for earthquake resistance diagnosis for midsized and low steel framed buildings.

Here, the structural resistance to earthquake in this diagnosis will be expressed in terms of an index (numerical value), and a determination will be made as to whether or not the structure will be destroyed by an earthquake.

2. Range of Application:

This method is generally applicable to buildings with a height of 31 meters or less.

However, the large span structures such as gymnasiums, or ferroconcrete structures, or compound structures of ferroconcrete and other materials, even though they are steel framed structures, differ considerably in their scale and type, so they will not be included in the discussion now.

3. Preliminary Investigation:

In order to determine whether this method is appropriate, a preliminary investigation is necessary to see if the method would be applicable.

4. The method of figuring the structure's earthquake resistance involves the use of equation (1) and the structural earthquake resistance index V_R as evaluated in 4.1, and the earthquake input index V_I as determined in 4.2.

$$V_R > V_I \qquad (1)$$

4.1 Structural Earthquake Resistance Index V_R:

V_R, the structural earthquake resistance index is an index determined by equation (2) where the earthquake resistance of each floor's structural framing to the point where it would be collapsed, and that of the total quake energy absorbable by the whole structure in terms of the quantity of energy involved.

$$V_R = Q \times V \times S \qquad (2)$$

Here, Q: is the quality Index (according to Chapter 2, Part 1)

V: is the standard earthquake resistance index (according to Chapter 2, Part 2)

S: is the shape index (according to Chapter 2, Part 3).

The standard earthquake resistance index V is evaluated for each floor in the direction of the spans and in the beam direction, but there is no distinction in the Q, quality index, or the S, shape index, by the floor level of the structure. (eg. here, floor means 'story' as in a 6-story building).

When it is impossible to get a Q quality index, the diagnostic operations must be stopped.

4.2 Earthquake Input Index, V_I:

The Earthquake input index V_I is an index that determines the amount of earthquake energy input, and the input energy's basic quantity is determined as follows according to the class of geological foundation and by the level of earthquake activity.

V_I is determined by the following equation:

(3)

d: is a coefficient determined by the distance (in Km) from the earthquake epicenter, and when d is greater than or equal to 40 Km, the value of D is lineary interpolated.

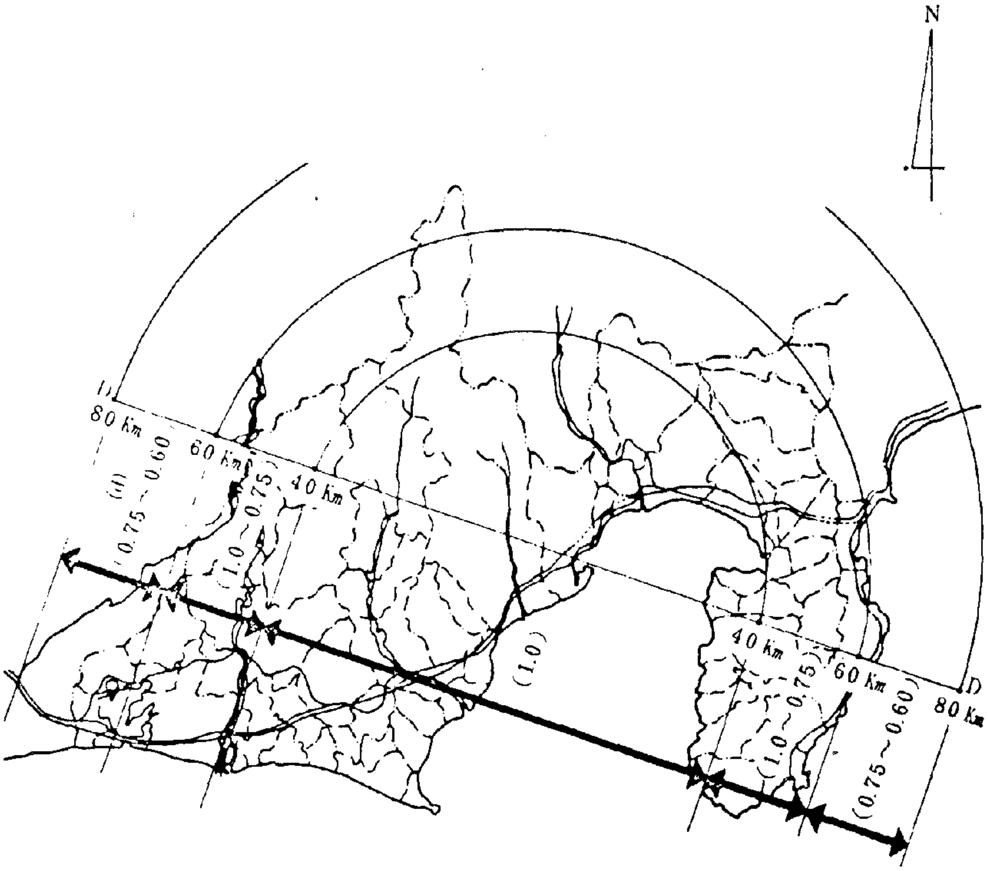


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Figure 1: The Value of d

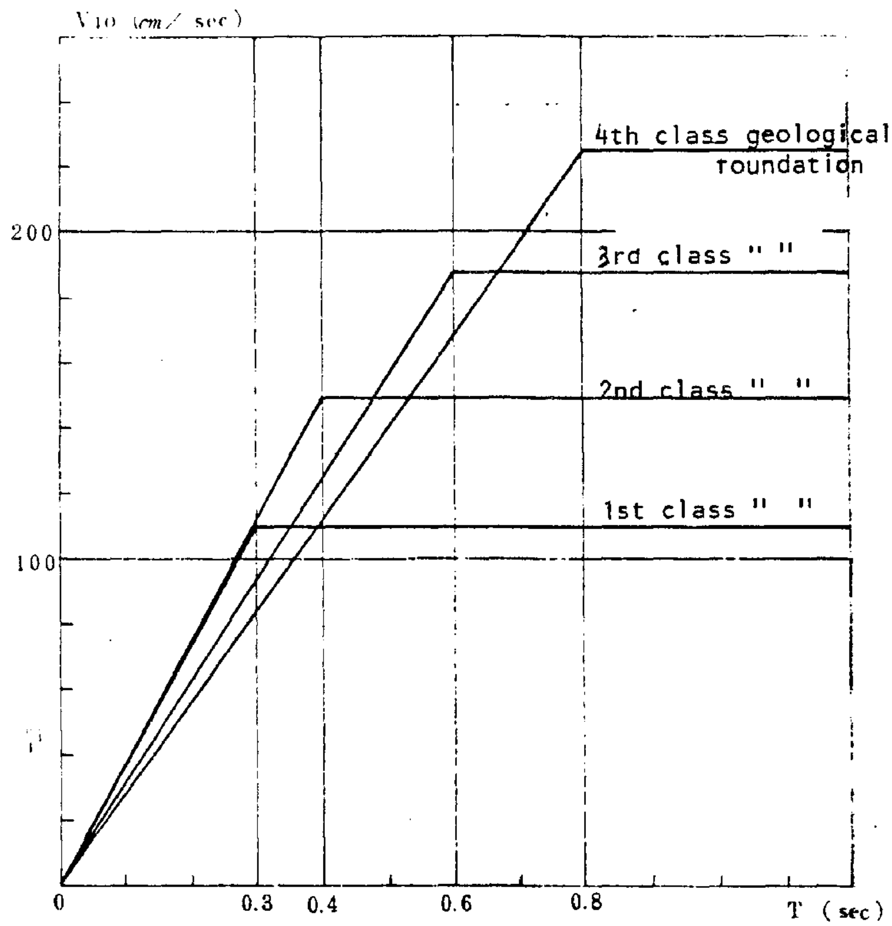


Figure 2: Value of V_{10} (Here, T is the structure's primary characteristic cycle)

D3

1 In this part, aside from gymnasiums, indoor exercise rooms, assembly halls, factories, warehouses, and stoors which have large spans are the types of buildings being considered.

2 The V_I index is calculated according to the distance from the earthquake epicenter of the subject building, the class of geological foundation it is built upon, and the structures primary characteristic period.

3 The structures primary characteristic period T is calculated according to the method described in this section Part 2, paragraph 2.2.3.

4 The type of geological foundation can be determined from Figure 3, Geological Classification Figures (1)(2) (at the end of this volume), or from Table 1's table of geological foundations which are used in conjunction with samples taken from borings in the vicinity of the structure.

Table 1: Table of Geological Foundations
(according to the Ministry of Construction's order number 10/4)

Type Foundation	Geological Texture and Strata
1st Class	The geological foundation, throughout the range of said building's period has a solid rock, hard type of sand and rock or other primary foundation formed in the Tertiary Period or before
2nd Class	The geological foundation, throughout the range of said building's period has a gravel, or sand and clay mixture, or a loam grade strata and it has a sand strata or sand and gravel strata which is more than 5 m thick.
3rd Class	This is a geological foundation not belonging to 1st, 2nd, or 4th classes which is basically a sand foundation, clay mixed with sand, clay, or mixed type of foundation.
4th Class	This is a markedly weak foundation which conforms to one or more of the following: 1) A depth of 30 m or more of humus soil, mixed soil, or other similar types of composite soil. 2) Land fill withic meets the following conditions: A) swamp, or mud-flats type of location. B) when the land fill material is trash, mud, or other similar types of weak fill. C) when the fill material has a depth of 3 m or more. D) when the fill has not been in place for 30 years.

CHAPTER 2: Computation of the Structural Earthquake Resistance Index V_R

Part 1: Evaluation of the Quality Index, Q:

In Section 1 (Method of Earthquake Resistance Diagnosis for low and Midsized Buildings) Chapter Two, Part 1 (Evaluation of Quality Index Q) are the basis for the computations.

Part 2: Computation of the Basic Earthquake Resistance Index V:

2.1 Basic Policy for the Computation:

1 It is computed both for the between span and in the direction of the cross-beams for the structure.

2 Structures using steels other than the following are outside the scope of this diagnosis: SS41, SM41, STK41, STKR41, SM50, SM50Y, STK50, STKR50.

3 For evaluating the strength, the JIS (Japan Industrial Standards) specifications for yield point, tensile strength, and the minimum values are multiplied by 1.1, and in the evaluation of capacity for deformability, the JIS minimum value for the yield point is multiplied by 1.2.

4 When the planar shape of the building is an L-shape, T-shape, or ⊐-shape, this shape is broken down into regular rectangles and the computations are carried out for the basic earthquake resistance index V for each of these rectangles.

5 The basic earthquake resistance index V computation may be accomplished manually.

2.2 Computation for the V index for Gymnasiums, etc.:

1 Range of Application:

This method of computation has the following structural types, and is for plain steel framed structures:

(span direction): pillar base pins are simple rigid frames

(cross beam direction) a simple tensile diagonal beam (strut) structure

2. Computation of the Basic Earthquake Resistance Index, V:

V is computed according to the following equations:

(span direction): $$V = 330\,\alpha T \qquad (1)$$

(crossbeam direction): $$V = 270\,\alpha T\sqrt{0.5 + B\eta} \qquad (2)$$

(symbols) α: yield shear force coefficient

$\alpha = Q/W$

Q: yield shear force (t)

W: Weight of building (t)

T: Characteristic period (seconds)

$B\eta$: magnification rate of the accumulative plastic deformation of diagonal beams.

3. Computation of the Characteristic Period, T:

T is computed according to the following equation:

$$T = 2\pi\sqrt{\frac{W}{gK}} \qquad (4)$$

(Symbols):

W: Weight of building (t)

g: acceleration of gravity 980cm/sec^2

K: spring constant (t/cm)

Computation of the spring constant, K, is done with the following:

(span direction): $$K = \frac{2E}{h^2} \cdot \frac{1}{\frac{1}{6Kb} + \frac{1}{3Kc}} \qquad (5)$$

(Symbols) E: Young's coefficient 2100 t/cm2

h: .Pillar height (cm)

$Kb = \Sigma kb$

$kb = Ib/\ell$: beam strength (cm^3)

$Kc = \frac{1}{2}\Sigma kc$

$kc = Ic/h$: pillar strength (cm^3)

Ib: beam cross-section secondary moment (cm^4) ℓ: beam's span (cm)

It: pillar cross-secton secondary moment (cm^4)

Σ: sum of all beams and pillars

(cross-beam direction) $$K = \Sigma \frac{E\,{}_BA\cos^3\theta}{B\ell} \qquad (6)$$

(Symbols): $_BA$: Diagonal beam cross-sectional area (cm^2)

E: Young's coefficient 2100 t/cm^2

θ: Angle of slope of the diagonal beams with respect to the horizontal plane

$_B\ell$: Interval between pillars including diagonal beams (cm)

I: Sum of all diagonal beams, however, diagonal beams on the compressed side are ignored.

4. Computation of the Yield Shear Force Q:

The computation of Q is according to the following equation:

(span direction) $$Q = \Sigma Mr/h \qquad (7)$$

(Symbols) M_r: moment of resistance at column (pillar) capital (t·cm)

H: column height (cm)

Σ: Sum of all column capitals (cap pieces)

Mr is the smallest value between M_1 thru M_6.

M_1: total moment of plasticity for comumn (t· cm)

However, it must not exceed the moment determined by the side buckling elasticity.

M_2: total moment of plasticity for beams (c · cm)

However, it must not exceed the moment determined by the side buckling.

Also, the load weight distribution effects must also be considered in M_2.

The side buckling elasticity moment is determined by the following equation:

$$M = 1.1 \text{ fb Zp} \qquad (8)$$

(Symbols) fb: as 2/3 of the side buckling distance, computed according to the steel structural design.

short-term permissible bending force (t/cm^3)

Zp: Plasticity cross-sectional coefficient (cm^3)

Effect of load distribution is as follows.

For 2-intersection rigid frames: When: $\xi < 0.5$ then $M_2 = oM_2$ (9)

$\xi \geq 0.5$ then $M_2 = oM_2 \times 2\ (\sqrt{2\xi} - \xi)$ (10)

For 3-intersection rigid frames: $M_2 = oM_2 - M_L$ (11)

(Symbols) oM_2: A computed M_2 ignoring the effects of load distribution (t cm)

M_L: moment at the end of the material due to vertical load (t cm)

ξ: Mw/oM2

$$Mw = \frac{w\ \ell^2}{8} \qquad (t\ cm)$$

w = beam load distribution (t cm)

ℓ = beam span (cm)

For the truss material, instead of M_1 and M_2, the following equation is used to compute the moment:

$$M = Hg \cdot Pc \qquad (11)\ (sic)$$

Symbols) Pc: short-term permissible compression for the truss (t)

Hg: distance between truss centers of gravity (cm)

For the truss stock, the M_1 and M_2 cannot exceed the value determined by the following:

$$M = (P_L \sin\theta - Q_L)\,h, \qquad M = \frac{(P_L \sin\theta - Q_L)\,\ell}{2} \tag{12}$$

(for columns) (for beams)

(Symbols)

P_L: short term permissible compression force for lattice (t)

θ: slope of lattice stock along the axis of the lattice material

QL: Stock end shear force from vertical load (t)

h: column height (cm)

ℓ: beam span (cm)

M_3: panels' total moment of plasticity (t cm)

M_3 is computed according to the following equation:

$$M_3 = 0.77\ Vp\ \sigma y \tag{13}$$

(Symbols)

Vp: Panel's effective volume (cm^3) , Vp = HbHctp

σy: yield stress (1.1 times the nominal value)(t/cm^2)

Hb: beam height (cm)

Hc: column height (cm)

tp: panel thickness

M_4: Material end moment according to a rip at the point of connection of the material.

M_4 is computed according to the following equation:

(for beams)

$$M_4 = \min\left(\frac{\ell}{\ell - 2\ell_o} M_B \cdot \frac{(Q_B - Q_L)\,\ell}{2}\right) \tag{14}$$

$$M_4 = \min\left(\frac{h}{h - \ell_o} M_B \cdot (Q_B - Q_L)\,h\right)$$

(for columns)

(Symbols)

ℓ: beam span (cm)

h: column height (cm)

ℓo: distance from connecting part to material (cm)

M_B: Break bend moment of the connecting part

For H-shaped beams, strength is computed according to:

$$M_B = 0.8 \cdot \frac{fPB}{fPY} \cdot MY \quad (15)$$

(symbols) fPB: break axial strength of connection including flange (t)

fPY: flange's yield axial strength (t)

MY: Total plasticity moment (t cm)

For the truss materials, the following equation is used:

$$MB = Hg\ P_{CB} \quad (16)$$

(Symbols) P_{CB}: break axial strength of connection including truss (t)

Hg: distance between centers of gravity of truss stock

Q_B: is the break shear force at the connection (t)

For the truss materials, the lattice materials break axial component perpendicular to the axial direction of the material is taken.

In the computation of the break axial force and the break sheer force, one follows the maximum resistance of the connection in "Appendix 2-1".

Q_L: Material edge shear force from vertical load (t)

M_5: The moment which determines breakage of the panel locally around its perimeter where it is attached (t · cm).

"According to Appendix 2 - 2 connected panel periphery's local breakage maximum moment (Figure 7)."

M_6: The column capital moment determined by the colum base's shear force (t cm)

$$M_6 = (0.4\ P + 0.75\ ne\ f\ A\ \sigma B)\ H \quad (17)$$

(Symbols) P: existing axial force (t), It is 0 under tension.

ne: effective anchor bolt number

when t is equal to or greater than 1.4d, then ne = n/2.

when t is less than 1.4d, then ne = n

t: base plate thickness (cm)

d: diameter of anchor bolts (cm)

σB: anchor bolt tensile strength (t/cm^2)

h: column height (cm)

n: the number of anchor bolts

(cross-beam direction) $Q = \Sigma bg$ (18)

(Symbols): Bg: horizontal resistance force of one set of diagonal beams (t)

$Bg = BP \cos\theta$

(Symbols): θ: angle of slope with respect to the horizontal of diagonal beams.

$BP = \min(BP_1, BP_2)$

BP_1: Yield axial force for one diagonal beam (t)

BP_2: Break axial force for one diagonal beam including connecting part.

According to "Appendix 2-8, break axial force for diagonal beam).

However, the axial force of the diagonal beam of the materials in the vicinity of the diagonal beam, and the axial force of the vertical load sum should not exceed the following values:

1) the short-term permissible compression force for the material (t)
2) the break axial force for the material connection (t)
3) the axial force caused by the uplift of the foundation for the columns (t).

5. The Accumulative Plastic Deformation Magnification Rate $B\eta$ for the Diagonal Beams:

$B\eta$ is computed according to the following equation:

$$B\eta = a + b\left(\frac{BP_2}{BP_1} - 1\right) \leq 25 \qquad (19)$$

Here, when BP_2/BP_1 is less than 1, then $B\eta = 1$.

Here, when determinining the axial force limitations of the peripheral materials, use $B\eta = 1$.

(Symbols) BP_1: Yield axial force for diagonal beams (t)

BP_2: Break axial force for diagonal beams including connecting part (t).

The values of a, and b are as follows:

Type of Steel	a	b
SS41 SM41	10	40
STK41 STKR41	0	50
SM50 SM50Y	10	40
STK50 STKR50	0	50

Part 3: Shape Index S:

Computations are based upon Section 1 (Earthquake Resistance Diagnostic Method for Low and Midsized Buildings), Chapter 2, Part 3 (Shape Index S).

SECTION THREE: Methods for Design Revisions for Earthquake Resistance:

Chapter 1: Basic Items:

1.1 Principals of Revision Design:

Design revisions are based upon the results of earthquake resistance diagnosis.

1.2 Establishing Goals for Design Revision:

When accomplishing design revisions, it is necessary to design goals to be achieved into these revisions, and then when the design has been completed, confirmation must be made according to earthquake resistance diagnosis to see whether these goals have been attained.

When the objectives are not reached, one must look carefully at the contents of the diagnostic method, and revise those parts which are inappropriate and accomplish a new diagnosis.

1.3 Policy for Design Revision:

While it is an accumulation of the earthquake resistance index V_R, quality index Q, basic earthquake resistance index V, and the shape index S, the revision is only concerned with the basic index V.

Chapter 2: Design Revisions Concerning the Basic Earthquake Resistance Index V:

2.1 Items for Discussion.

During the diagnosis, results for each story's Vi are investigated, and discussion of design revision must begin for the story of the building that has the smallest Vi.

2.2 Increasing the Yield Shear Force:

This involves increasing the yield shear force coefficient αi. At this time, the rigid frame's yeild shear force coefficient $R\alpha i$, and the diagonal beam portions yield shear force coefficient $B\alpha i$ are considered separately, and each is improved according to the following policies.

1 With regard to the Rigid Frame's Yield Shear Force Coefficient $R\alpha i$:

(a) With the aim of increasing the cross-section of the columns and the cross-section of the beams, flange plates and cover plates made of wave-plate are added.

However, in order to realize the desired level of strength in these additions, it is necessary to attach them in an appropriate way.

(b) If it is possible to newly install columns or axial frame groupings it is very effective, however, it is necessary to incorporate these with existing axial frames and attain a good connection with the floor slab in order to increase resistance to earthquake.

(c) Check the data sheets for when the earthquake resistance diagnosis was accomplished and see whether the joints and connections are sufficiently strong. If they are not, repair them so that their maximum moment is 1.2 times or more stronger than the total plastic moment of the material's cross section. See sections 3.1 and 3.2 for ways to repair and revise welded joints and bolt joints.

(d) If the diaphragm has fallen off or is insufficient, erect a sturdy one.

2 With regard to the Yield Shear Force Coefficient $B\alpha i$ for the Diagonal Beams.

(a) If the diagonal beam cross-section is insufficient, replace them with larger ones. Do not, however, use round steel stock.

(b) Investigate the data sheets for the initial earthquake resistance diagnosis, and when the highest strength of the diagonal beam connection is lower than the yield resistance of the diagonal beam cross-section, change out the connection portion so that its resistance is 1.2-times that of the yield resistance of the diagonal beam cross-section. See section 3.3 for methods of computing the effective cross section during design, computation of the resistance of weld junctions, bolt junctions, and design methods for gusset plates.

(c) If possible, increase the number of diagonal beams. At that time, make sure that the maximum resistance of the connecting part of the diagonal beams is 1.2 times greater than the yield resistance force for the diagonal beam cross-section.

(d) If the diagonal beams have undergone revision to increase their yield shear force coefficient, make sure that the columns, beams, and column bases in the vicinity of the diagonal beams have sufficient resistance.

2.3 Increasing the Accumulative Plastic Deformation Magnification Rate ηi.

(a) If the beams have insufficient resistance to side buckling, then midway side buckling preventors can be newly erected to prevent side buckling. See section 3.4 for one example of design for side buckling preventors.

(b) Investigate the data sheets made at the time of the earthquake resistance diagnosis, and with regard to those connections and junctions which have insufficient resistance, make their total plastic moment 1.2 times the maximum moment which can be transmitted to them.

2.4 Increasing the Energy Concentration Coefficient Φi.

When the energy concentration coefficient φi is insufficient on a certain floor of the building, the method in paragraph 2.2 should be used to increase the αi of the floors with a low Vi value. So far as possible, make the basic shear force coefficient distribution $\bar{\alpha} i$ as close as possible to that shown in Table 3.1.

Table 3.1: Basic Shear Force Coefficient Distribution $\bar{\alpha} i$.

N	J												
	1	2	3	4	5	6	7	8	9	10	11	12	13
1	1.0												
2	1.0	1.38											
3	1.0	1.20	1.65										
4	1.0	1.12	1.37	1.86									
5	1.0	1.10	1.25	1.53	2.00								
6	1.0	1.06	1.18	1.37	1.65	2.15							
7	1.0	1.05	1.15	1.28	1.47	1.76	2.27						
8	1.0	1.05	1.12	1.23	1.37	1.55	1.87	2.40					
9	1.0	1.03	1.10	1.18	1.30	1.45	1.65	1.95	2.45				
10	1.0	1.03	1.08	1.16	1.26	1.36	1.53	1.72	2.03	2.50			
11	1.0	1.03	1.07	1.13	1.23	1.32	1.43	1.60	1.78	2.10	2.60		
12	1.0	1.02	1.07	1.12	1.20	1.27	1.37	1.50	1.65	1.87	2.15	2.63	
13	1.0	1.02	1.06	1.11	1.17	1.23	1.33	1.43	1.56	1.72	1.93	2.24	2.70

N: Total Number of Floors in Builaing, J: Floor number

2.5 Increasing the Twisting Coefficient d:

Increase d by adding new diagonal struts when the d value is small because of other quake-resistance measures on the planar design.

CHAPTER 3: Methods of Revision and Repair:

3.1 Improving Weld Junctions:

1 When the corner welds in T-junctions and X-junctions have insufficient strength:

(a) When there is no scallop in the web end of the beam, place one in, and in the corner weld portion, and in the unwelded gouging in the material, reweld, or place a new weld so that it is completely welded (Figure-1.1)

(b) Also use a cover plate on the beam flange and weld the corner junction, and weld the junction between the cover plate and the column flange. (Figure 1.2)

(c) When the beam flange plate thickness is 12 mm or below, do additional welding so that the corner weld length is 1.3 times or more bigger than the the beam flange plate thickness. However, the corner weld goes around the total perimeter. When the repair methods of b) and (c) are used, when the original bead is dramatically poor, remove it with a grinder, etc., and reweld.

(d) When it is possible to newly install a triangular rib, reinforce this rib with a corner weld along its junction surfaces.

In this case, so that the installation of the triangular rib does not cause a large secondary stress, it is necessary to install a reinforcing plate. (Figure 1.3)

2 When Quake Resistance is insufficient because of insufficient weld cut in at butt joints:

(a) Gouge the butt joint and reweld so that there is a complete cut in by the weld (Figure 1.4)

(b) Change the design of the butt junction on both (or one) sides, with a metal plate.

Make a corner weld between this metal plate and the primary stock, or make a connection with high strength bolts to provide a friction held junction.

In order to get a close contact between the metal plate and the primary stock, eliminate any bulging welds before application of the plate. (Figure 1.5)

3 When the weld has dropped off:

(a) When it is possible to apply a beveler to the part where the weld should be, process the butt end and weld.

When this is difficult or impossible, reinforce using the methods described in 1, (b), and (c).

(b) Accomplish corner welds on all of the parts which should have a corner weld.

When accomplishing a corner weld is difficult, accomplish a butt weld where possible (for example, when the gap in a T-junction is too large).

4 When the weld has dropped off and it has already become impossible to reweld it:

Repair and reinforce the inside of the closed cross section where there is no place that can be welded or install a horizontal "hanch".

When these repairs are made using welding, consider the following:

*when welding, avoid if possible, an upward-looking posture, and instead use a downward-looking, horizontal, or standing posture.

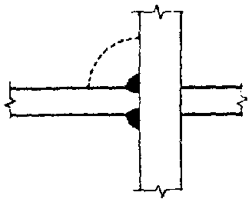
* Remove rust and paint from the areas to be welded.

* Accomplish sufficient pre-heating (see Table 1.1)

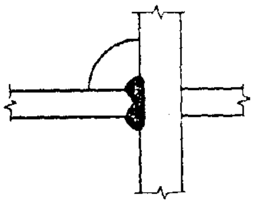
* Erect a scaffold so that the welding can be done with a stable footing, and take care to prevent fires and secondary fire hazards.

Table 1.1: Preheating Temperatures

Weight of Primary Stock (t)	Preheating Temperature
Less than 1 ton per 25mm	$50^{\circ}C$
Greater than or equal to 1 ton per 25 mm	$100^{\circ}C$



a. Before Repair



b. After Repair

Figure -1.1

Reinforce corner weld

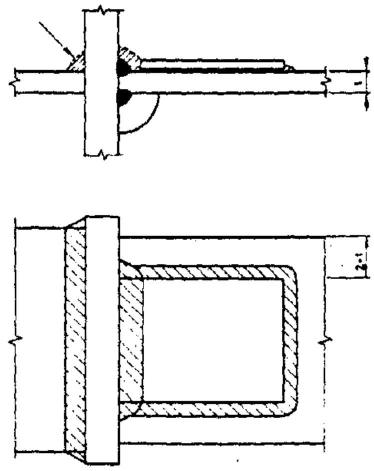


Figure 1.2

Reinforcing plate

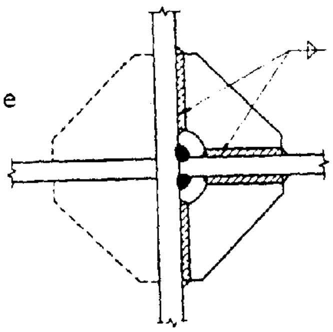


Figure 1.3

a. Before Repair

b. After Repair

Figure 1.4

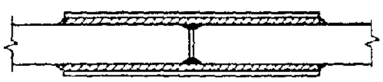


Figure 1.5

3.2 Methods for Repair and Revision of Bolt Junctions:

1 When the shear resistance of the bolts is insufficient:

When the shear resistance of the bolts is insufficient because regular bolts have been used, remove those bolts and replace them with high-strength bolts. However, if high strength bolts are being used, and their dimensions are such that 1 or less threads on the nut are engaged, change out the bolts for ones of correct dimensions.

2 When the pitch or the end edge distance are insufficient:

When the pitch or the end edge distance are insufficient, repairs are sometimes possible by replacing the joint plate with a thicker one. When the end distance between the materials and the pitch is poor, strength can be increased by attaching some material to the joint plate by means of a corner weld.

In this instance, most of such corner welds cut into the the upper flange of the floor plate, so in many cases this is difficult to do, so one will have to work with the web joint or with the lower flange joint in most cases. Only in cases where the corner weld reinforcement of the web or lower flange is insufficient is it necessary to try to reinforce the upper flange.

In a corner weld for a combination joint, as Figure 2.1 indicates, both the flange and the web joints are welded at the intersection and in axial direction of the material. The former weld need only be made if the latter is insufficient. At the web joint, so long as the joint plate is not way too small, the flange gets in the way and in many cases the weld in the axial direction of the stock becomes impossible. Since in most cases, at the flange joint, the joint plate covers the whole width of the flange, when a weld is made in the axial direction of the material, it is accomplished after gouging the flange side (see Figure 2.2-C).

3 When the joint strength is insufficient:

In this case, a combination joint such as discussed in 2 above can be accomplished using a corner weld to secure the needed strength.

3.3 Repair and Revision of the Diagonal Beam Junction:

1 When the diagonal beam cross-section is insufficient:

When the diagonal beam material has an insufficient cross-sectional area, exchange it for material which does and make the joints according to "Joints for Diagonal Beams."

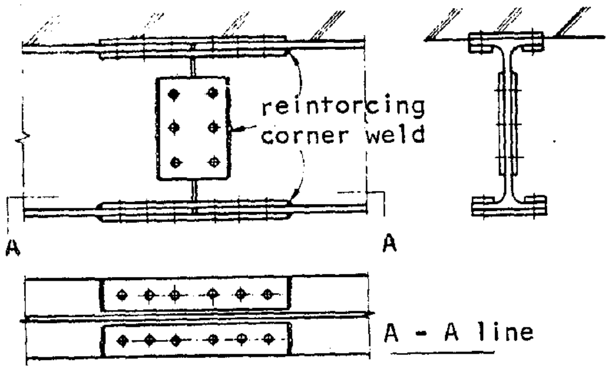


Figure 2-1

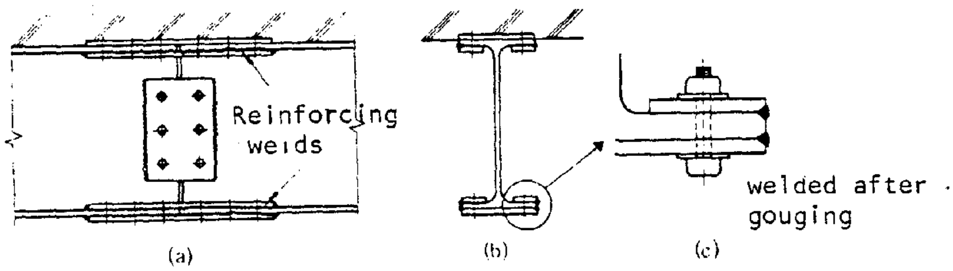


Figure 2-2

2 When the strength of the diagonal beam joint is insufficient:
The repair or reinforcement objectives are as follows:

a) $An/bA \geqq 0.75$ Where: $An = bA - (he + d)\ b^t$ (3.1)

: effective cross-sectional area

bA: Cross-sectional area (cm^2) of diagonal beam stock

d: Fastener hole diameter (cm)

b^t: Plate thickness of the diagonal beam stock (cm)

he: the range (cm) where the junction strength due to eccentricity can be eliminated from the computation, this is determined by Table 3.1.

Table 3.1: Value of he.

n	1	2	3	4	5
angle iron	gh-t	0.7gh	0.5gh	0.33gh	0.25gh
U-shaped steel	gh-t	0.7gh	0.4gh	0.25gh	0.2gh

gh: height of protrusion of diagonal beam. See Figure 3.1.
t: See Figure 3.1-b.

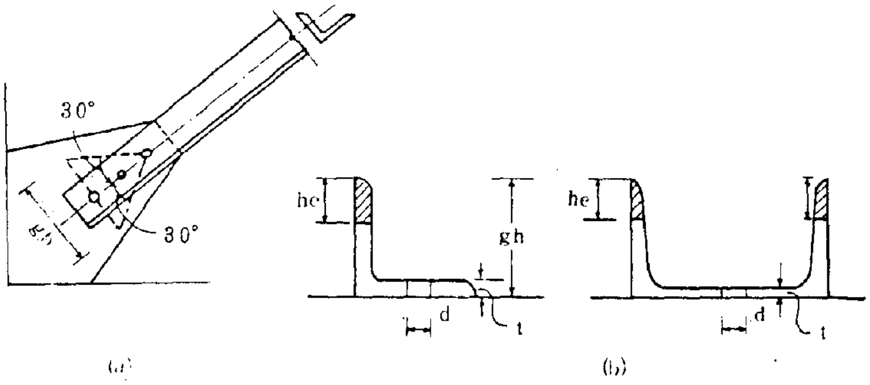


Figure 3.1

(b) According to the computation of the maximum quake resistance for joints (3.4) (3.5), and (3.6) and that for the joint strength of a gusset plate junction (3.7), the resistance minimum values P1, P2, P3, and P4, or P = Min must be repaired so that they satisfy the following equation:

$$P \geqq 1.33\ {}_{B}PY = 1.33\ bA\sigma Y \qquad (3.3)$$

The equations for the maximum resistance to quake for P_1, P_2, P_3 and P_4 are:

$$P_1 = 0.75 \cdot n \cdot m \cdot fA \cdot f\sigma B \qquad (3.4)$$

$$P_2 = n \cdot e \cdot t \cdot \sigma \qquad (3.5)$$

$$P_3 = gAn \cdot \sigma B \qquad (3.6)$$

Here,

bPY: Yield axial force of diagonal beam (t)

bA: Cross-section area of diagonal beam (cm^2)

fA: Axial cross sectional area (cm^2) for fasteners (bolts, rivets)

$gAn = (gb - \alpha) \cdot gt$: gusset plate's effective cross-sectional area (cm^2)

gt: gusset plate thickness (cm)

gb: gusset plate effective width (cm). (See Fig. 3.1)

e: end edge distance in direction of stress (cm)

σY: Yield stress on diagonal beams (t/cm^2)

$f\sigma B$: Tensile strength of fasteners (t/cm^2)

σB: Tensile strength of diagonal beam material and gusset plate (t/cm^2)

Equation for computing the joint strength with gusset plate:

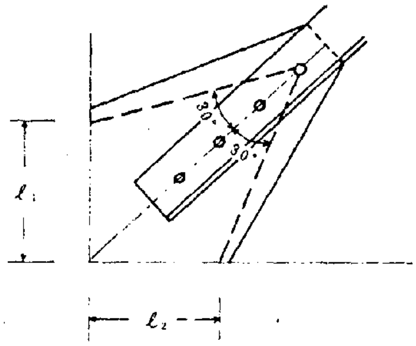
$$P_4 = tA \cdot \sigma B \qquad (3.7)$$

Here, tA: Effective cross-sectional area of the junction weld, (see Figure 3.2)

For a joint with two-sided corner welds:

$$P_4 = \frac{1}{\sqrt{3}} wA \cdot \sigma B \qquad (3.8)$$

Here, wA: Effective cross-sectional area of corner weld. See Figure 3.2



$tA = (\ell_1 + \ell_2)t$

$wA = 2\ (\ell_1 + \ell_2)\ a$

t: gusset plate thickness

a: corner weld throat thickness (cm)

Figure 3-2

c) When the diagonal beam is angle-iron or flat plate, and the number of fasteners is insufficient:

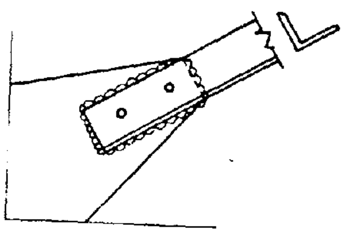
i For 1 fastener:

change out the diagonal beam and gusset plate and reattach according to "standard junctions."

ii When there are two fasteners:

a For angle-iorn, make a combination joint regarding:

$L\text{-}65^2 \times 6$, $L\text{-}75^2 \times 6$, $L\text{-}90^2 \times 6$ (see figure 3.3), other types of angle iron steel are the same as for i.



Note: Corner weld is around the whole perimeter, before welding, change out fasteners to high strength bolts (F10T)

Figure 3.3

b For flat plate, use a combination joint.

iii When there are 3 or more fasteners, use a combination joint.

iv Other:

When the measures in a) thru c) fail to sufficiently provide the quake resistance needed, the diagonal beam stock and gusset plates must be replaced.

d) When the quake resistance of the gusset plate is insufficient:

As Figure 3.4 shows, the size of the gusset plate should be such that it can encompass a regular polygon shape comprised of the first fastener hole at the apex with the last fastener hole's center being at the base of a line which runs perpendicular to the stress line. At this time, the effective cross-sectional area of the gusset plate will be considered as being along the a - a cross-sectional line. If this area is insufficient, the gusset plate must be replaced with a larger one, or a cover plate must be placed atop the gusset plate and corner welded around its whole perimeter (See Figure 3.5).

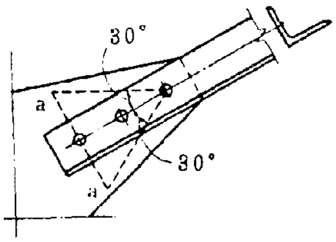


Figure 3.4

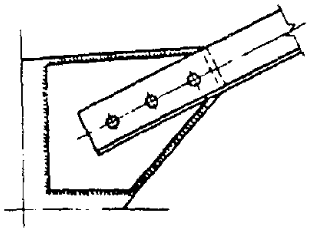


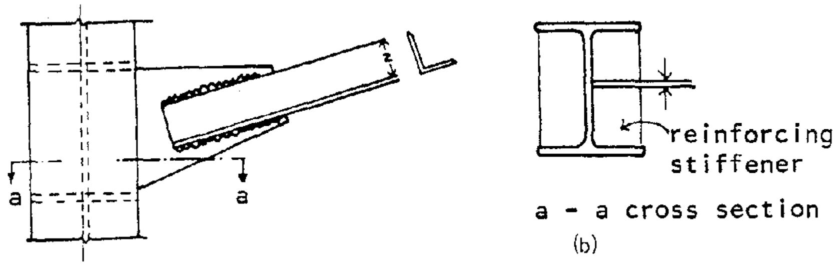
Figure 3.5

e) When the weld joint strength is insufficient:

When the weld between the diagonal beams and gusset plate, or between the gusset plate and the axial frame is insufficient, it can be gouged and rewelded, or it can be strengthed by additional corner welding until the required base length has been attained.

3 When there are insufficient countermeasures to localized stress:

a) Reinforcing ribs or stiffeners can be added to columns and beams to prevent the generation of localized stress. (See figure 3.6)



(a)

(b)

Figure 3.6

b) When new diagonal beams must be installed:

In order to increase the yield shear force of the building floor, improve the yield shear force distribution, or to reduce twisting deformation, it is necessary to increase the V index. The addition of new diagonal beams is one effective method. However, with regard to the structure's functional restrictions, this often involves considerable difficulty.

Accordingly, the new addition of diagonal beams involves much prior discussion from both the design and construction viewpoints, and in constructions, it is necessary to have sufficient quality control for the junction areas.

The following points are those which must be considered in general:

i Horizontal and vertical mating must be considered for the diagonal beams.

ii The addition of axial force on the columns and beams, stress on the foundation, and increased strength required of the column and beam junctions must be considered.

iii If it is necessary that certain points in the materials do not have axial centers which match, one has to consider the added stress due to eccentricity.

iv In construction, on-site welding must be avoided as much as possible in favor of using high strength bolt connections at the junctions.

v Localized deformation can be prevented by adding reinforcing ribs and stiffeners, etc., to beams and columns, but during construction, the primary stock must not be damaged.

3.4 Repairs to prevent side buckling:

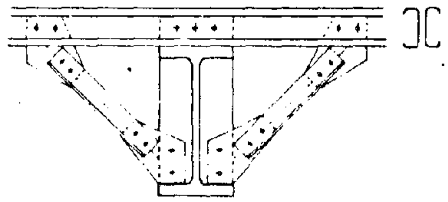


Figure 4.1

APPENDIX

APPENDIX 2: Computations for Quake Resistance for Beams, Columns, and Junctions:

2-1 Maximum Resistance for Junctions:

(1) Butt junctions (Figure 1)

$$_wP_B = A\sigma_B$$

$$_wQ_B \doteq \frac{1}{\sqrt{3}} A\sigma_B \qquad (1)$$

(2)

(Symbols) σB: Tensile strength of primary stock (t/cm^2)

A = ae (cm) x be (cm)

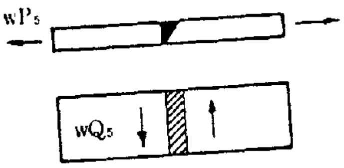


Figure 1

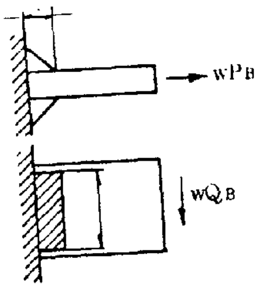


Figure 2

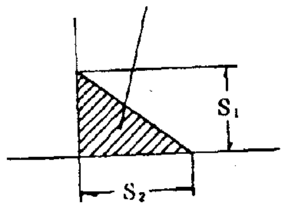


Figure 3

The effective thickness (ae) is the thickness of the primary stock at the butt junction.

Effective weld length (be) is the dimensions of the primary stock at the butt junction provided that the ends of the bead have been end tabbed or that there has been a sufficient weld around on them. With regard to edge processing other than that just mentioned, it is the value of the weld length minus 2 ae. With regard to butt welds where no scrap has been placed on the web area by the beam flange junction, so long as there has not been beveling on the flange on the beam or metal application to the back side, a complete welding cannot be assured, so for the be, (btw + br) is subtracted from the total width. Here, the btw is the beam lip thickness, and br is curve radius of the fillet part of the beam.

(2) Corner welding (Figure 2):

$$_wP_B = \frac{1}{\sqrt{3}} Ae\sigma_B \qquad (3)$$

$$_wQ_B = \frac{1}{\sqrt{3}} Ae\sigma_B \qquad (4)$$

D23

(Symbols) σB: Tensile strength of the primary stock (t/cm^2)

Ae: Effective throat cross-sectional area

= ae (cm) x be (cm)

The size of the corner welding and the throat thickness is measured at each place using a naka gage.

Here, the size is the S_1 and S_2 shown in Figure 3, and the smaller value is used.

The effective thickness (ae), when the average value for the size s and throat thickness a, is the smaller of the value 0.7s or the value of a.

The weld's effective length (be) when there is a complete weld-around is the width of the stock in question, or, when there is no such weld around the value of 2 x (size of the corner material) should be deducted from the length of the weld. However, for beam webs and diaphragms, etc, where there is no scrap and a corner weld around the total perimeter, the following should be used:

i) Inside of the joint on the beam end flange:

be = bB - (btw + br)

ii) Beam end web joint:

be = dB = (ctw + cr)

Here, bB, DB, and bhr are those dimensions shown in Figure 4b, btw is the beam lip thickness, ctw is the column web thickness, br is the beam stock's and cr are the column stock's fillet curve radius.

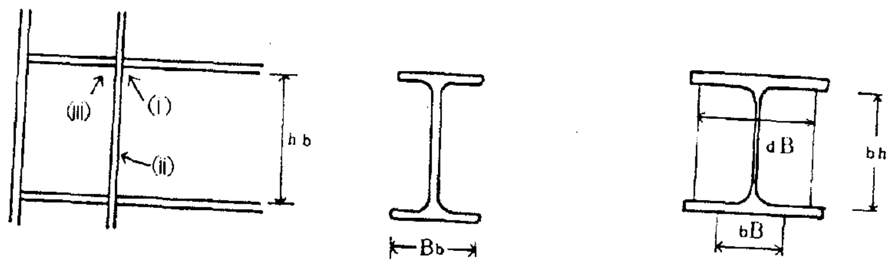


Figure 4a

Figure 4b

2 Bolt (center bolts, high stength bolts) and Rivit Connections' Maximum Axial Force fPB and Maximum shear force fQB (Figure 5)

D24

1) The maximum resistance $_5P_B$ on junctions subject to axial force is the minimum value selected from P1, P_2, or P_3.

$$P_1 = A_n \sigma_B \qquad (5)$$

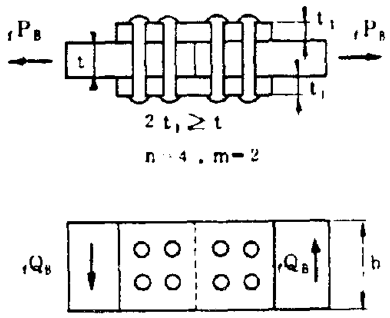
$$P_2 = 0.75\, n \cdot m \cdot {}_fA \cdot {}_f\sigma_B \qquad (6)$$

$$P_3 = n \cdot e \cdot t \cdot \sigma_B \qquad (7)$$

2) The maximum resistance fQB on junctions subject to shear force is the minimum value selected from Q_1 or Q_2.

$$Q_1 = \frac{1}{\sqrt{3}} A_n \sigma_B \qquad (8)$$

$$Q_2 = 0.75\, n \cdot m \cdot {}_fA \cdot {}_f\sigma_B. \qquad (9)$$



bolt tensile strength	
F 8T	8.8 (t/cm²)
F 10 T	11.0 (t/cm²)
F 11 T	12.1 (t/cm²)

Figure 5

(Symbols):

An: Effective cross-sectional area of the Junction, in the figure (b-2d) t (cm^2)

fA: axial cross-sectional area (cm^2) for fasteners (bolts, rivets)

fσB: Tensile strength of the fastener material (t/cm^2)

σB: Tensile strength of the junction material

D25

n: Number of fasteners

m: Number of surfaces of fasteners subject to shear (m = 2 in the case of the figure)

d: Fastener hole diameter (cm)

t: Plate thickness of the material to be joined (cm)

e: Distance to end in the direction of the stress (cm)

3 Maximum Tensile Resistance of Round Steel Cut by Bolts or Screws (round steel diagonal beams, or anchor bolts, etc):

$$P_B = A_e \sigma_B \quad (10)$$

$$A_e = 0.75 fA \quad (11)$$

(Symbols): fA: Axial cross-sectional area of bolts or round steel (cm^2)

σB: tensile strength of bolt or round steel (t/cm^2)

4 Tension junctions using high-strength bolts (split-T junctions)'s maximum resistance (Figure 6):

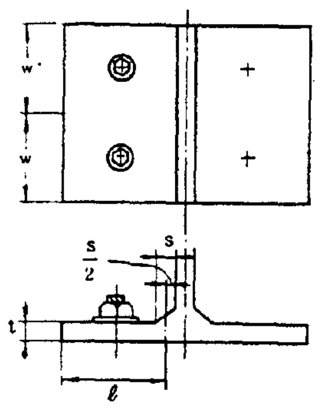


Figure 6

The maximum resistance of the bolt row on one side of a T flange is determined according to equations (12) - (14). When there is no dramatic difference in bolt strength and number on each side of the T flange, the resistance of the junction is the sum of the resistance of each side.

D26

When $\frac{2.5\,wt^2}{\ell}\,\sigma_B \leq B_r$

Then $P_B = \frac{1.47wt^2}{\ell}\,\sigma_B$ (12)

When $\frac{wt^2}{\ell}\,\sigma_Y \leq B_Y < \frac{2.5wt^2}{\ell}$

Then $P_B = 0.6\ n \cdot B_T$ (13)

When $B_r < \frac{wt^2}{\ell}\,\sigma_Y$

Then $P_B = n \cdot B_r$ (14)

(Symbols):

- σY: Yield point of the T-flange material (t/cm^2)
- σB: Tensile strength of the T-flange material (t/cm^2)
- n: Number of bolts on one side of the T flange
- BY: High stength bolts' yield strength (t/cm^2)
- Br: High strength bolts' tensile strength (t/cm^2)

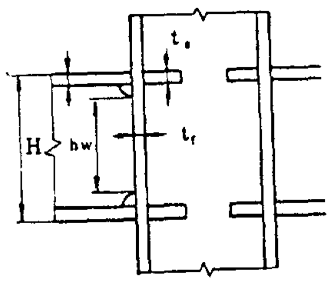
See Figure 6 for w, t and ℓ

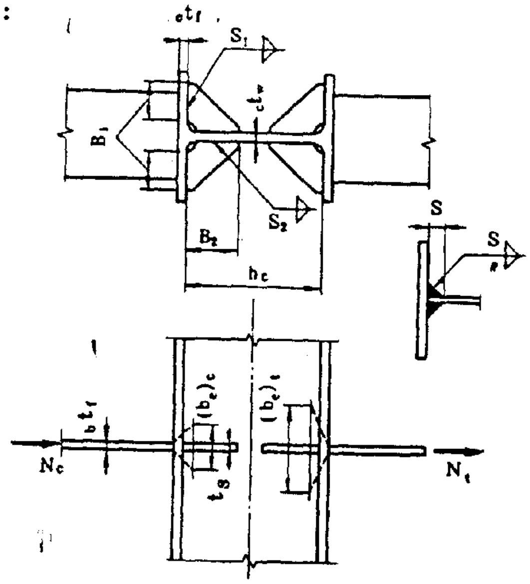
5 Maximum Shear Force for Junction:

The computation is made only with respect to the web part junction for shear force on the strong axial direction side, and on the weak axial direction side, it is made for only the flange part junction. The web and flange maximum shear resistance is computed as WQB or TQB according to the connection method used.

2-2 Localized Breaking Along the Perimeter or Attached Panels Determined by Maximum Moment on the Ends of the Stock (Figure 7)

Figure 7: Strong Axial Direction:





(compression side) Tension side)

γ: curve radius for fillet on H-shaped column stock

(Note) For the case of a through diaphragm: B2 - 0.5me

D27

1 When an H-shaped Cross-sectioned column is Subject to strong Axial Bending:

$$M_{BX} = P_B H + \frac{1}{4} P_{BW} \cdot hw \qquad (15)$$

(Symbols): $P_B = N_T$ or N_c

FBW: Maximum axial strength (t) in the beam web weld joint

hW: Weld length in the beam web (cm)

σY: Yield point of the material (t/cm^2)

σB: Tensile strength of the material (t/cm^2)

Nt and Nc are determined as follows:

(1) When Nt is the tensile stress of the beam flange's stress:
Nt is the tensile resistance of the column web portion Nt_1 and the colum weld joint's tensile resistance Nt_2 in sum:

$$N_t = N_{t1} + N_{t2} \qquad (16)$$

However:

$$N_{t1} = ctw\,\{btf + 5\ (ctf + r)\} \cdot \sigma_B \qquad (17)$$

When the column is a weld assembled H-cross-sectional shape piece, the column flange and column weld is a butt weld and in the equation above r = 0.7 ctw.

When there is a column assembly with a corner weld, the equation (18) below is used: $N_{t1} = 1.4\,S\ (btf + 5\,ctf)\dfrac{\sigma_B}{3}$ (18)

$$N_{t2} = MIN\left(2B_1 \times 0.7S_1 \times \frac{\sigma_B}{\sqrt{3}},\ B_1 \times t_2 \times \sigma_B,\ 4B_2 \times 0.7S_2 \times \frac{\sigma_B}{\sqrt{3}},\ 2B_2 \times t_2 \times \frac{\sigma_B}{\sqrt{3}}\right) \qquad (19)$$

On the right hand side of equation (19) in terms: 1 and 2, the first term unnessary when the diaphragm/flange junction is a butt junction weld, but they express the final resistance of that diaphragm/weld junction. Also, on the right hand side, terms 3 and 4 express the diaphragm/web junctions final strength, but when the diaphragm/web junction is a butt weld junction, the third term is unnecessary.

(2) When the beam flange's Stress is a compression stress, Nc is:

$$N_c = N_{c1} + N_{c2} \qquad (20)$$

However

$$N_{c1} = ctw\,\{btf + 2\ (ctf + r)\} \cdot 1.2\,\sigma_Y \qquad (21)$$

$$N_{c2} = MIN\left(1.2\,B_1 \times ts \times \sigma_Y,\ 4B_2 \times 0.7S_2 \times \frac{\sigma_B}{\sqrt{3}},\ 2B_2 \times ts \times \frac{\sigma_B}{\sqrt{3}}\right) \qquad (22)$$

D28

When there is a butt weld junction between the diaphragm and the web, the second term in the upper equation on the right side is unnecessary.

As figure 8 shows, with a column beam connection, there are instances where there is a band plate. In this case, where there is a band plate, the flange's stress as the point of connection where it is transmitted to the bannel, may be considered to correspond to the band plate's cross-sectional area Abe which corresponds to its effective width.

$$A_{be} = 2b_e \times t_b \qquad (23)$$

Here, tb is the thickness of the pand plate,

be = btt + 5ctf.......where the beam flange receives the tensile force

be = btf + 2tt.........where the beam flange receives a compression force

2 Column beams receiving a weak axial bend which are of an H-shaped cross-section: When the resistance of the connection is determined by the diaphragm (Figure 9)

$$M_{uy} = N_y H \qquad (24)$$

However,

$$N_y = MIN(2B_3 \times t_s \times \frac{\sigma_B}{\sqrt{3}}, 4B_3 \times 0.75 S_1 \times \frac{\sigma_B}{\sqrt{3}}) \qquad (25)$$

When there is a butt weld at the diaphragm and the web, the second term above is unnecessary.

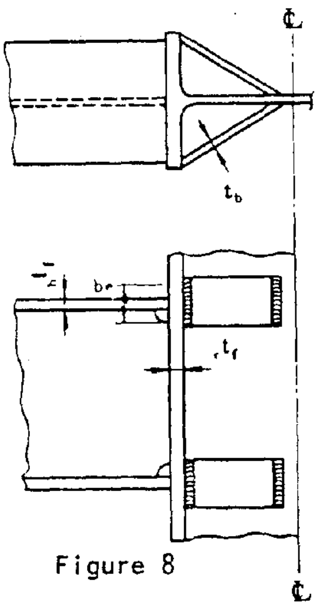


Figure 8

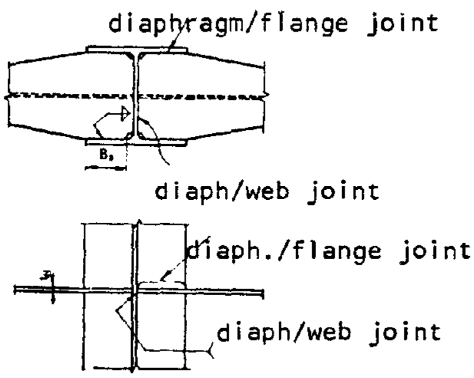


Figure 9: Weak axial direction

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3 When stress is determined by the breakage of a diaphragm outside of the beam/column connection area:

$$M_{BX} = \sqrt{2}\, B_m\, t_d\, H \sigma_B \tag{26}$$

(Symbols) Bm: External diaphragm minimum width (cm)

td: External diaphragm thickness (cm)

σB: Diaphragm's tensile strength (t/cm^2)

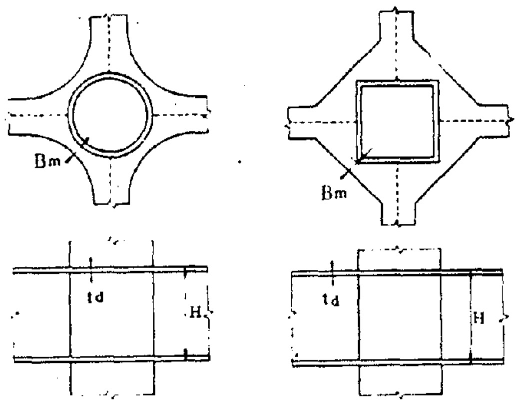


Figure 10

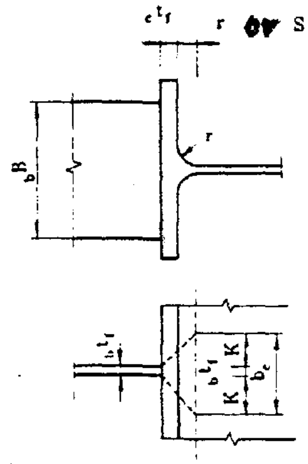


Figure 11

4 Maximum moment, MBX, when there is no diaphragm:

$$M_{BX} = 1.2\, {}_ct_w\, ({}_bb_f + 2K) \cdot H \sigma_Y \tag{27}$$

However: K = ctf + r (hot-rolled H-shaped column)

ctf + s (for weld-assembled H-shaped column cross-section)

s: the leg length of the corner weld between column flange and column web.

2-3 Dealing with Beams Having Hances:

As Figure 12 shows, when there is an intersection C where there is a hance opening for the material end which directly joins:

D 30

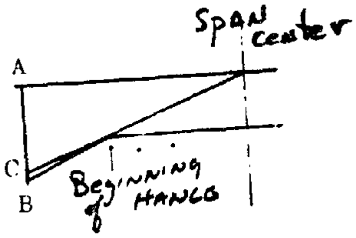


Figure 12

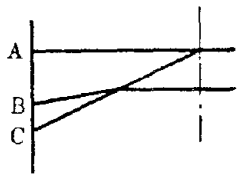


Figure 13

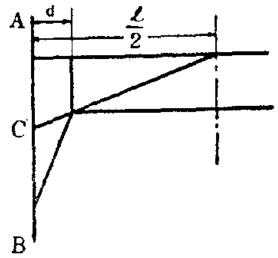


Figure 14

1. When AC is greater than or equal to AB (Figure 13)

i) When the material has an H-shaped cross-section, a box-shaped cross-section, steel pipe, of an ⊟-shaped cross section, the dimensions at the end of the stock are used and entered on data sheet 2. (For the ⊟-shaped stock, the reinforcing plate is ignored, and the dimensions are as they would be for an H-shape).

ii) In cases other than i) above, use the dimensions of the end of the stock, and enter them on data sheet 3.

2 When AC is less than or equal to AB (Figure 14):

Use the cross-sectional dimensions at the point of the hance opening and enter them on data sheet 3.

However, the total plasticity moment Mp computation must be done using the following equation (28):

$$M_P = \frac{\ell}{\ell - 2d} \cdot M_P' \qquad (28)$$

Here, Mp' is the total plasticity moment (t cm) at the point of the hance opening, and ℓ and d are the distances shown in Figure 14 (cm).

2-4 Cross-Sectional Properties of Special Cross-Sectioned Materials:

1 The Minimum cross-section secondary radius is computed according to the following equations.

1) For ⊥-shaped materials:

D3/

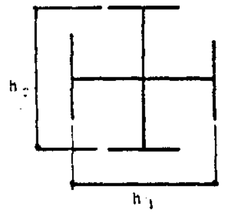


Figure 15

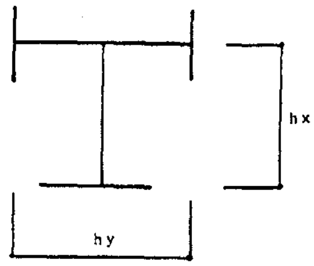


Figure 16

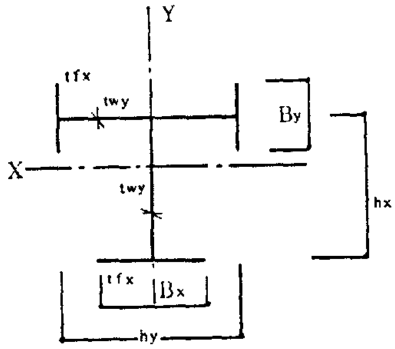


Figure 17

The minimum cross section secondary radius is $i = min, (0.3\ h_1, 0.3\ h_2)$

2) For ⊤ -shaped materials:

The minimum cross-section secondary radius is $i = min\ (0.3hx, 0.3hy)$

2 The total plasticity moment for ⊤ -shaped stock can be determined by the following equations:

1) Total plasticity moment around X axis:

$$M_P = \left(A_{fx} + \frac{A_{wx}}{2}\right) h_x \sigma_y \qquad (29)$$

here

$$A_{fx} = B_x \cdot t_{fy}$$

$$A_{wx} = h_x \cdot t_{wx}$$

2) Total plastic moment around Y axis:

$$M_P = \left(A_{fy} + \frac{A_{wy}}{4}\right) h_y \sigma_y \qquad (30)$$

here

$$A_{fy} = B_y \cdot t_{fy}$$

$$A_{wy} = h_y \cdot t_{wy}$$

2-5 Cross-Sectional Computations for τ, kp, kd:

According to the cross-sectional shape, for H-shaped stock, one assumes a strong axis and a weak axis, which are computed according to the methods for H-shaped stock (see previous section for the standards for evaluating earthquake resistance fore existing steel framed buildings).

1 For ╋ -shaped cross-sectional stock----assumption of an H-shaped cross-section strong axis.

1) Thickness ratio d/tw, b/tf

Use the numerical values in Figure 18 for the computation.

D32

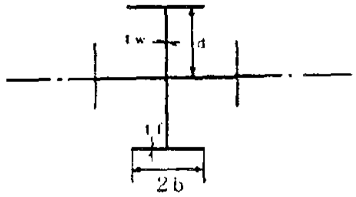


Figure 18

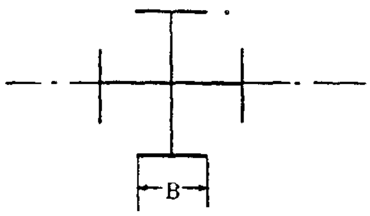


Figure 19

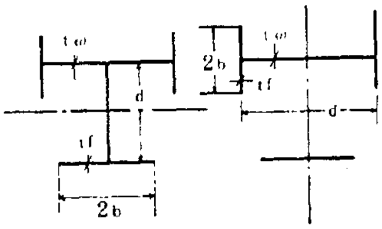


Figure 20

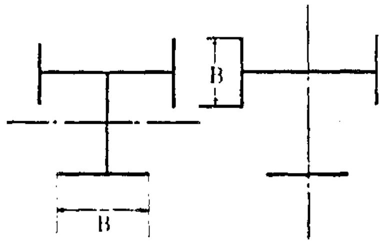


Figure 21

2) The thinness-length ratio for the weak axis λy:

$$\lambda_y = 0.2B \qquad (31)$$

However, the length is shown in B in Figure 19.

3) Axial force ratio p/py:

p is the axial force from the vertical load

py is the total cross-sectional value for area which is computed.

2 'I' -Shaped material......suppose a reinforced H-shaped material.

1) The width-thickness ratio d/tw, b/tf is determined in the direction of use according to Figure 20.

2) Thinness-length ratio λy:

$$\lambda_y = 0.2B \qquad (32)$$

However, the length is shown in B in Figure 21.

3) Axial force ratio p/py:

e same as in a)

D33

2-6 Joints and Bevels: Quake Resistance for Special Cross-Sectioned Materials

1 Maximum Tensile resistance:

Computed according to 2-1 of the Appendix

2 Breakage Moment:

According to the cross-sectional shape, a strong axis for H-shaped materials and a weak axis are supposed, and from Appendix 2-1, PBf and PBW are computed:

Here: PBf is the junction's maximum resistance between the flange and a hypothetical part.

PBW: is the junctions's maximum resistance between the web and a hypothetical part.

At this time, the breakage moment, MB is:

i) When a strong axial bend is received:

$$M_B = (P_{Bf} + \frac{1}{8} P_{BW}) H \tag{33}$$

H: distance between the flange and a hypothetical part (cm)

ii) When a weak axial bend is received:

$$M_B = \frac{1}{2} P_{Bf} B \tag{34}$$

3 Break Shear Force:

Just as for the breakage moment, a strong and weak axis is assumed for the H-shaped material, and according to the methods detailed in Appendix 2-5, 5 (Maximum shear force Q_B for the Junction), the calculations are completed.

2-7 Effective Volume of Panels:

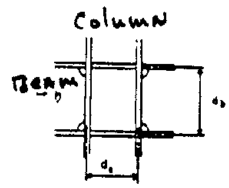
These are determined according to the Table on the next page.

Table 1

Panel Type	V_p computation
H-shaped strong axial beam	$V_p = d_b \times d_c \times {}_c t_w$
□-shaped beam strong axis	$V_p = d_b \times d_c \times (2{}_c t_f' + {}_c t_w)$
H & -shaped weak axis columns	$V_p = 2 d_b \times d_c \times {}_c t_f$
cross shaped columns	$V_p = \phi \times d_b \times d_c \times {}_c t_w$ $\phi = \frac{\alpha^2 + 2.6 (1 + 2\beta)}{\alpha^2 + 2.6}, \quad \alpha = \frac{d_b}{{}_c b_f}, \beta = \frac{{}_c A_f}{{}_c A_w}$ ${}_c A_f = {}_c b_f \times {}_c t_f, \quad {}_c A_w = d_c \times {}_c t_w$
Compressed cross-section col. (internal/external type diaphragms)	$v_p = 2 d_b \times d_c \times {}_c t_f$
Round columns	$v_p = \frac{1}{2} {}_c A \times d_b$

The (v) of columns is shown for standard shapes. For those which are not of standard shapes, appropriate adaptions are inferred. When the panel has tabler plate reinforcement, the effective rate is 70% of the strength of the reinforcement

Symbols:
ctf: colmun flange thickness
cbf: column flange width
ctw: panel thickness
cA: total area for column
ctf': flange thickness in weak axial direction (cover plate thickness)



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2-8 Break Axial Force for Diagonal Beams (struts):

The maximum resistance Pu is the smallest value among the following P_1 thru P_3 equations:

$$P_1 = A_n \sigma_B \tag{35}$$

$$P_2 = 0.75 \cdot n \cdot m \cdot {}_f A \cdot {}_f \sigma_B \tag{36}$$

$$P_3 = n \cdot e \cdot t \cdot \sigma_B \tag{37}$$

Here:

fA: axial cross-sectional volume of the fastener (bolt/rivit) (cm^2)

d: Fastener hole diameter (cm)

e: Edge distance in direction of stress (cm)

he: The range of deductable cross-section according to computations of eccentritity (see Figure 22);

However, he is according to Table 2:

Table 2

n	1	2	3	4	5
Ш-shaped	dh − t	0.7 gh	0.5 gh	0.33 gh	0.25 gh
U-shaped	dh − t	0.7 gh	0.4 gh	0 25 gh	0.2 gh

For the case of flat steel, he = gh - t (t: steel thickness)

gh: height of protrusion of diagonal beam stock

n: number of fasteners

m: number of fasteners receiving shear force

t: thickness of material being joined (cm)

bt: thickness of stock for diagonal beam (cm)

σt: Maximum stress on connecting material (t/cm^2)

fσB: fastener material's maximum stress.

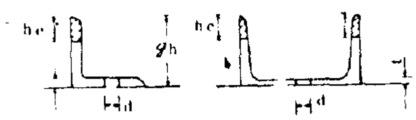

Figure-22

2. Maximum Resistance of Gusset Plates:

The maximum resistance P_4 of gusset plates is determined by the following equation:

$$P_4 = {}_gA_n \cdot \sigma_B \qquad (38)$$

The gusset plate must be 9mm or greater in thickness, here:

gAn = (gb-d)·gt: gusset plate's effective cross-sectional area (cm^2)

gh = gusset plates effective width (cm) (see Figure 23)

gh = gusset plate plate thickness (cm)

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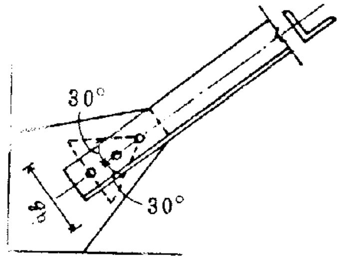


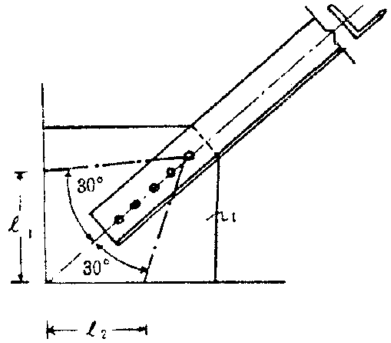


Figure 23 Figure 24

3. Gusset Plate's and Frame's junction Resistance (P_5)

For a butt welded joint, this is determined according to equation (39):

$$P_5 = tA\sigma B \tag{39}$$

Here, tA is the effective cross-sectional area of the butt weld (see Figure 24)

For a corner weld on both faces for the junction, it is determined using equation (40):

$$P_5 = \frac{1}{\sqrt{3}} \, wA\sigma B \tag{40}$$

Here, wA: effective cross-sectional area of corner weld (see Figure 24)

$tA = (\ell_1 + \ell_2)\, gt$

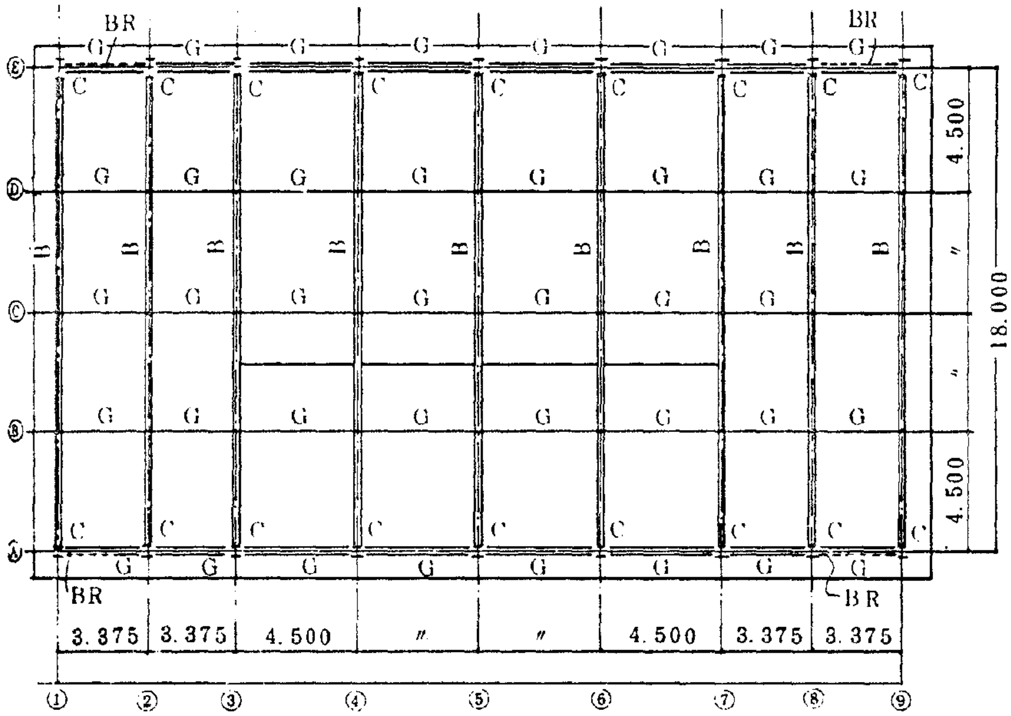
$wA = 2\,(\ell_1 + \ell_2)\, a$

gt: gusset plate thickness

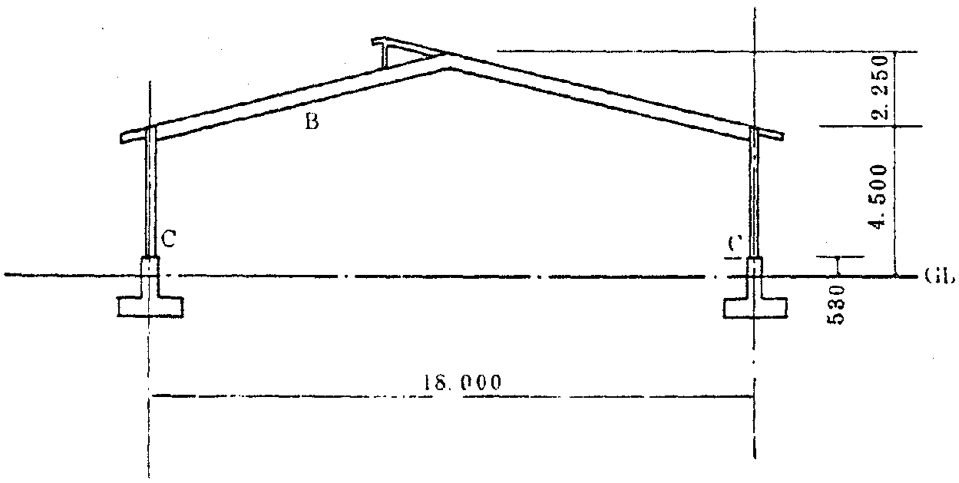
a: throat thickness of corner weld.

Appendix 3: Computation of the V Index for Gymnasiums:

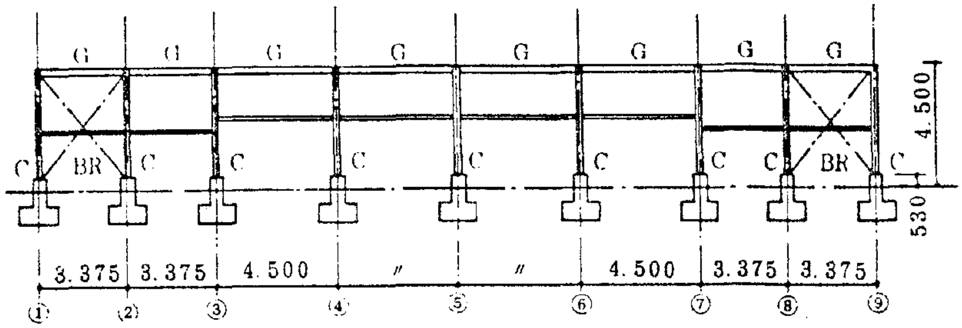
1. Outline of Building:



Configuration of R-floor -------axial frame brace (BR)

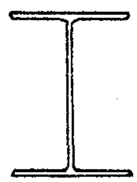


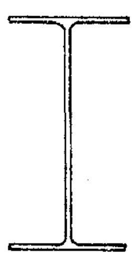
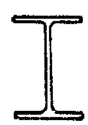
(2) thru (8) through axial frame diagram



(A), (E) Through Axial diagram

Main Cross-sectional List:

C	BR
	
H − 244 × 175 × 7 × 11 B.PL. − 16 ANCHOR BOLT 2 − M22	22φ

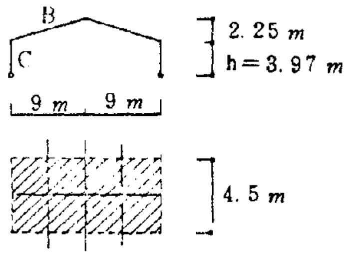
B	G
	
H − 350 × 175 × 7 × 11	H − 150 × 100 × 6 × 9

For SS41 steel, no special characteristics

2. Computation of the V Index in the span Direction:

2.0 Standard Computation:

Construction Model:



Weight of Building:

Roof $0.065 \times 31.5 \times 20.0 = 41.0\,t$

Walls $0.07 \times 18.0 \times \left(\frac{4.5}{2} + \frac{2.25}{2}\right) \times 2 = 8.5\,t$

Cross-walls $0.07 \times 31.5 \times 4.5 / 2 \times 2 = 9.9\,t$

59.4t

For the upper ½ of column height, the total weight of the building is computed

W = 59.4t computed for the whole building

2.1 Computation of Spring constant K:

Strength of column Kc (H-244 x 175 x 7 x 11)

$I_x = 6120\ cm^4$

$k_c = 6120 / 397 = 15.42$

$K_c = \frac{1}{2} k_c \times 2 \times 9 = 138.8\ cm^3$

Kc = 138.8 cm^3

Strength of beams Kb (H-350 x 175 x 7 x 11)

$I_x = 13600\ cm^4$

$k_b = 13600 / 1800 = 7.56$

$K_b = k_b \times 9 = 68.0$

Spring constant K

$$K = \frac{2E}{h^2} \cdot \frac{1}{\frac{1}{3K_c} + \frac{1}{6K_b}} = \frac{2 \times 2100}{397^2} \cdot \frac{1}{\frac{1}{3 \times 138.8} + \frac{1}{6 \times 68.0}} = 5.49\ t/cm$$

K = 5.49 t/cm

2.2 Individual periodicity T Computations

$$T = 2\pi \sqrt{\frac{W}{gK}} = 2 \times 3.14 \times \sqrt{\frac{59.4}{980 \times 5.49}}$$

$= 0.660$ 秒

= 0.66 seconds

D40

2.3 Computation of Yield Shear Force Q:

Determined by column capital moment for columns:

(1) The total plasticity moment for column M_4:

$$M_1 = Z_p \times \sigma y = 558 \times 1.1 \times 2.4 = 1473 \text{ t cm}$$

M_1 = 1478 t cm

Zp : Plastic cross-section coefficient

σy: Yield strength

σy is 1.1-times nominal

(2) Moment, M_5, determining elastic side buckling for column:

$$\lambda = \frac{\ell b}{i} = \frac{190}{4.68} = 40.6$$

$$\lambda' = \frac{2}{3}\lambda = 27.1$$

$$fb > 1.6 \text{ t/cm}^2$$

2/3 of the actual thinness-length ratio.

Accordingly, elastic side buckling does not occur.

(3) the column capital moment, M_6, determined by the shear resistance of the column base

Anchor bolt 2-22 Φ d = 2.2cm

Base plate thickness: t = 1.6 cm

$$t > 1.4 d$$

$$n_e = 2$$

$$M_6 = (0.4 p + 0.75 n_e \, {}_tA \sigma_B) h$$

$$= 0.75 \times 2 \times 3.801 \times 4.1 \times 1.1 \times 397$$

$$= 10208 \text{ t cm}$$

$$(p = 0)$$

fA: Anchor bolt's axial cross-sectional area

σB: tensile strength

σB is 1.1 times the nominal value

Mb must be sufficiently large even when p = 0

M_6 = 10208 t cm

Moment of column capital determined by beam:

(1) Beam's total plastic moment M_2

$${}_bM_2 = 868 \times 1.1 \times 2.4 = 2291 \text{ t cm}$$

(2) Beam's elastic side buckling determining moment M_2:

$$\lambda = \frac{\ell b}{i} = \frac{450}{4.58} = 98.3$$

$$\lambda' = \frac{2}{3}\lambda = 65.5$$

$$fb > 1.6 \text{ t/cm}^2$$

D41

Accordingly, there is no elastic side buckling.

(3) Considerations on load distribution

Considering the effects of load distribution on M_2.

$$0.065 \times 4.5 = 0.293 \ t/m$$

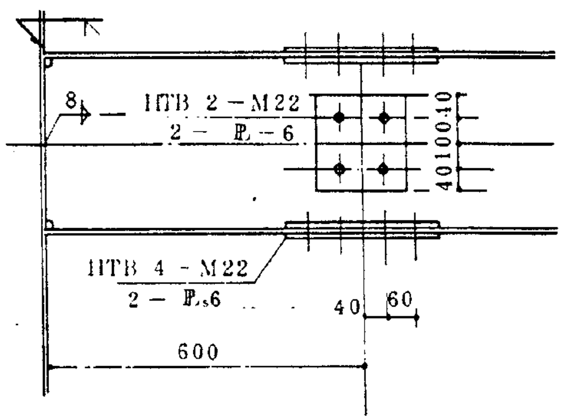
$$M_w = \frac{\omega \ell^2}{8} = \frac{0.00293 \times 1800^2}{8} = 1187 \ t/m$$

$$\xi = \frac{M_w}{{}_0M_2} = \frac{1187}{2291} = 0.52 > 0.5$$

$$M_2 = 2291 \times 2\left(\sqrt{2 \times 0.52} - 0.52\right) = 2290 \ t\,cm$$

The material end moment determined by the breakage of the beam junction.

(1) Beam junction breakage determining moment, M_4.



$$P_{Bf} = (17.5 - 2 \times 2.4) \times 1.1 \times 4.1 \times 1.1 = 63.0 \ t$$ Flange breakage

$$0.75 \times 4 \times 2 \times 3.90 \times 10 \times 1.1 = 250.8 \ t$$ Fastener breakage

from the above:

$$4 \times 4.0 \times 1.1 \times 4.1 \times 1.1 = 79.4 \ t$$ bridge opening breakage

$$P_{Bf} = 63.0 \ t$$

$$M_B = 0.8 \times \frac{63.0}{17.5 \times 1.1 \times 2.4 \times 1.1} \times 2291$$ Breakage of splice

$$= 2272 \ t\,cm$$

$$Q_B = \frac{1}{\sqrt{3}} \times (18.0 - 2 \times 2.4) \times 0.6 \times 2 \times 4.1 \times 1.1$$

$$= 41.2 \ t$$ breakage of fastener

$$0.75 \times 2 \times 2 \times 3.80 \times 10 \times 1.1 = 125.4 \ t$$

from the above

$$Q_B = 41.2 \ t$$

$$\ell = 1800 \ cm$$

$$\ell = 60 \ m$$

D42

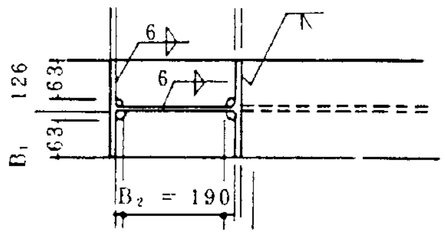
$$\frac{\ell}{\ell - 2\ell_0} M_B = 2434\ \mathrm{t\,cm}$$

$$Q_t\ \ell - \frac{(41.2 - 2.6)}{2} \times 1800 = 34740\ \mathrm{t\,cm}$$

$$M_4 = \frac{\ell}{\ell - 2\ell_0} M_B = 2434\ \mathrm{t\,cm}$$

Accordingly

M_4 = 2434 t cm



$B_1 = 17.5 - 0.7 - 1.6 \times 2 = 12.6\ cm$

$B_2 = 24.4 - 1.1 \times 2 - 1.6 \times 2 = 19.0\ cm$

$S_1 = S_2 = 0.8\ cm$

$T_s = 1.1\ cm$

$N_{t_1} = 0.7 \times [1.1 + 2.5(1.1 + 1.6)] \times 4.1 \times 1.1 = 24.8\ \mathrm{t}$

$N_{t_2} = 2 \times 12.6 \times 0.7 \times 0.8 \times \frac{4.1 \times 1.1}{\sqrt{3}} = 36.7\ \mathrm{t}$

$12.6 \times 1.1 \times 4.1 \times 1.1 = 62.5\ \mathrm{t}$

$4 \times 19.0 \times 0.7 \times 0.8 \times \frac{4.1 \times 1.1}{\sqrt{3}} = 111\ \mathrm{t}$

$2 \times 19.0 \times 1.1 \times \frac{4.1 \times 1.1}{\sqrt{3}} = 109\ \mathrm{t}$

$\therefore N_{t_2} = 36.7\ \mathrm{t}$

Accordingly:

$N_t = N_{t_1} + N_{t_2} = 24.8 + 36.7 = 61.5\ \mathrm{t}$

$N_{c_1} = 0.7 \ [1.1 + 1\ (1.1 + 1.6)] \times 1.2 \times 2.4$
$\times 1.1 = 8.4\ \mathrm{t}$

$N_{c_2} = 1.2 \times 12.6 \times 1.1 \times 2.4 \times 1.1 = 43.9\ \mathrm{t}$

$4 \times 19.0 \times 0.7 \times 0.8 \times \frac{4.1 \times 1.1}{\sqrt{3}} = 111\ \mathrm{t}$

$2 \times 19.0 \times 1.1 \times \frac{4.1 \times 1.1}{\sqrt{3}} = 109\ \mathrm{t}$

$N_{c_2} = 43.9\ \mathrm{t}$

Theretore

Accordingly

$N_c = N_{c_1} + N_{c_2} = 8.4 + 43.9 = 52.3\ \mathrm{t}$

N_T と N_c を比較して

$P_B = 52.3\ \mathrm{t}$

PB = 52.3 t

$h_w = 35.0 - 2 \times (1.1 + 1.6) = 29.6\ cm$

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$$P_{BW} = \frac{1}{\sqrt{3}} \times (29.6 - 2 \times 0.8) \times 0.7 \times 0.8 \times$$

$$4.1 \times 1.1 = 40.8\ t$$

$$M_5 = 48.7 \times 35.0 + \frac{1}{4} \times 40.8 \times 29.6 = 2006\ t\,cm$$

M_5 = 2006 t cm

Total plastic moment of panel:

Panel's effective volume

$$V_P = 33.9 \times 23.3 \times 0.7 = 552.9\ cm^3$$

$$M_3 = 0.77 \times 552.9 \times 2.4 \times 1.1 = 1124\ t\,cm$$

M_3 = 1124 t cm

M_1 thru M_6 comparison reveals that the moment on the column capital, Mr, is determined by the panel's total plastic moment M_3 at the beam-column junction:

$$M_r = M_3 = 1124\ t\,cm$$

Mr = 1124 t cm

Yield Shear Force Q:

$$Q = \Sigma M_r / h = \frac{1124 \times 2 \times 9}{397} = 50.96\ t$$

Q - 51.0t

2.4 Yield shear forc coefficient determination:

$$\alpha = Q / W = 50.96 / 59.4 = 0.858$$

alpha = 0.858

2.5 Calculation of V index:

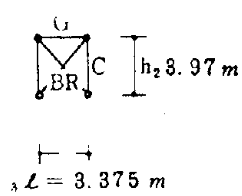
$$V = 330\ \alpha\ T = 330\ 0.858 \times 0.66 = 187$$

V = 187

3 Computation of V index in cross-beam direction:

3.0 Standard Computation:

Construction model:



The same the of building weight computation as in span direction.

W = 59.4 t

W = 59.4t

3.1 Spring constant calculation

BR 22 φ

$_BA$ 3.801 cm^2

$$\cos\theta = \frac{337.5}{\sqrt{337.5^2 + 397^2}} \doteq 0.6477$$

$_B\ell$ = 337.5

Therefore:

$$K = \frac{\Sigma E\ {}_BA \cos^3\theta}{{}_B\ell} = \frac{2100 \times 3.801 \times 0.6477^3}{337.5} \times 4 = 25.7\ t/cm$$

K = 25.7 t/cm

D44

3.2 Individual Perioo T computation:

$$T = 2\pi\sqrt{\frac{W}{gK}} = 2 \times 3.14 \times \sqrt{\frac{59.4}{980 \times 25.7}}$$

= 0.305 seconds

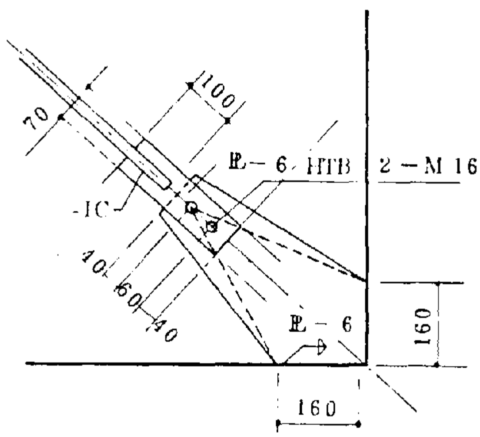
T = 0.305 Seconds

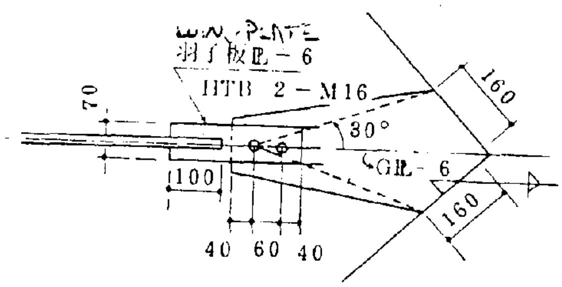
3.3 Computation of Yield Shear Force Q:

Diagonal beam's Yield axial force:

$$_BP_1 = {}_BA\sigma_y = 3.801 \times 2.4 \times 1.1 = 10.0\,t$$

Diagonal beam's yield axial force:





Diagonal beam and wing plate weld breakage:

Weld length (one side) 10 cm

$$_BP_2 = (10 - 0.8) \times 0.4 \times 2 \times \frac{4.1 \times 1.1}{\sqrt{3}} = 19.16\,t$$

Primary Stock Breakage:

$$_BP_2 = 0.75 \times 3.801 \times 4.1 \times 1.1 = 12.86\,t$$

Wing plate breakage:

$$_BP_2 \doteqdot (7.0 - 1.8) \times 0.6 \times 4.1 \times 1.1 = 14.07\,t$$

D45

Fastener Breakage:

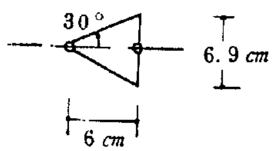
$$BP_1 = 0.75 \times 2 \times 1 \times 2.01 \times 10 \times 1.1 = 33.17\ t$$

Bridge opening breakage:

$$BP_1 = 2 \times 4.0 \times 0.6 \times 4.1 \times 1.1 = 21.65\ t$$

Gusset Plate Breakage

$$BP_2 = (6.9 - 1.8) \times 0.6 \times 4.1 \times 1.1 = 13.80\ t$$



Gusset plate and Column weld breakage

$$BP_2 = 2 \times (16.0 + 16.0) \times 0.4 \times 0.7 \times \frac{4.1 \times 1.1}{\sqrt{3}} = 46.7\ t$$

From the above, BP_2 determines the breakage of the primary stock.

$BP_2 = 12.86$ t

BP_1 and BP_2 compared are:

$BP = BP_1 = 10.0t$ BP = 10.0t

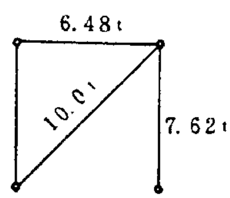
Horizontal component Bg is:

$$Bg = BP\cos\theta = 10.0t \times 0.6477 = 6.477$$

Bg = 6.477t

Accordingly, yield shear force Q is:

$$Q = 4 \times 6.477 = 25.9\ t$$



Q = 25.9t

Considering axial force in the vicinity of diagonal beams:

Column thinness-length ratio:

$$\lambda = \frac{190}{4.18} = 45.5$$

$$fc = 1.42$$

$$P = 1.5 \times 1.42 \times 56.24 = 120\ t > 7.62\ t \text{ O.K}$$

Beam thinness-length ratio:

$$\lambda = \frac{337.5}{2.37} = 142$$

$$fc = 0.475$$

$$P = 1.5 \times 0.475 \times 26.84 = 19.1\ t > 6.48\ t \text{ O.K}$$

D 46

So long as the diagonal beams do not break, beams and columns do not buckle.

The column base's breakage axial force is sufficiently greater than the uplift force upon the foundation of 7.62t.

3.4 Calculation of the Yield Shear Force Coefficient:

$$Q/W = 25.9/59.4 = 0.436$$

3.5 The diagonal beam's accumulative plastic deformity magnification rate $B\eta$:

$$B\eta = 10 + 40 \times \left(\frac{12.86}{10.0} - 1\right) = 21.2$$

3.6 Computation of the V Index:

$$V = 270\,\alpha T\sqrt{0.5 + B\eta}$$
$$= 270 \times 0.436 \times 0.305 \times \sqrt{0.5 + 21.2} = 167$$