# On a Method of Evaluation of Anti-Earthquake Design Code of Industrial Facilities

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Summary

This paper deals with a method of evaluating a newly drafted anti-earthquake design code for industrial facilities such as petro-chemical industries, oil refinaries, equipment and pipings for fire protection in high-rized buildings and so on. At first, various types of structural design against extreme loads like a destructive earthquake are discussed based on the development of structural design. Then, evaluation method of "margin" on each code is discussed using "Design Ampleness Index", and the values of the index on some examples are examined. Also the concept of "Open form code" is discussed in relation to abstraction, or concreteness of regulatory statement expression.

## 1. Introduction

As the author described in the previous paper[1], the effort to establish the codes for anti-earthquake design of various kinds of industrial facilities has been continued in Japan and also in various countries. It started from the drafting work for nuclear power plants, and now to various kinds of conventional equipment for ordinary industrial facilities and buildings.

According to the developments of such codes, we felt some necessity of the standard practice to build up the regulatory code for this purpose. The author discussed this in the previous paper[1]. The key blocks of design code are as follows:

| | |
|---|---|
| Defining *Factor of Importance* | Estimating the Mechanism of Failure |
| Defining *Design Basis Earthquake* | Analyzing the Vibration Characteristics |
| Finding their Mode of Failure | Defining the *Allowable Stress and Limit* |
| Supplementing the Design by General Design Criteria | |

The evaluation of adequacy of each code should be made based on the key blocks above mentioned.

i) Does it contain all key blocks of procedure?
ii) Does it have well balanced blocks to described the design policy?
iii) Is the degree of its abstraction level balanced with the engineering level and the related codes?
iv) Does it have a well-organized structure and does not have any conflicting part with the related codes and itself?

Three blocks written by *italic letter* in the above seven blocks are adjusted by the political balance of the anti-earthquake designs of related facilities to ensure the safety of the surrounding communities of the plant under destructive earthquake conditions. Unless their balances are well in the political sense, it might give another type of economical impact to the communities.

Even if we employ the higher design basis earthquake, the higher allowable stress can mitigate its effect. The level of the design basis earthquake should be compared with the probable maximum earthquake in the site, and be discussed with the safety factor of their structural elements. In the chapter 4, the author will survey on these relation of several typical codes by introducing the "Design Ampleness Index".

Most of anti-earthquake design codes, including the Building Code of Japan and others established before introducing the practice of dynamic response analysis to the design, have a "closed form", that is, all design procedures are defined strictly in them. However, some of recent codes describe only on the design policy, the fundamental theory, key procedures and parameters, and some general design criteria. We may call such codes as a "open form" code against a "closed form" code. The degree of *open* is also related to the degree of the abstraction, and can be evaluated the concept of "regulatory tree" defined as the twin form of the fault tree which has a damage caused by an earthquake as its top event.

## 2. Structural Design to Extreme Loads

Structural design to natural loads has been developed through the experience of failures. At present, we call such problems as "Extreme Load Design Problem"[2], and are dealing with man-induced critical external loads as well as natural loads. According to Timochenko's book[3], there were a discussion and an experiment followed by it on dynamic load of railway bridge in the nineteen century. Since the beginning of 1800s, their specification and design had been changed, and many new topics such as fatigue, dynamic load effect and so on were discussed. However, unfortunately his book does not refer to the "factor of safety". Such developments were made mainly in Europe. On the other hand, the design of pressure vessels was developed in the United States for steam boots in their rivers as we can see in the development of the ASME pressure vessel code. It employed the idea of the "factor of safety". We can describe the history of structural design and the relation to extreme loads in detail, however, the author does not want to spend the room of this paper for them. Therefore, the directly introduces several design methods based on the history of their development.

### 2.1 Various Types of Extreme Design

Design methods under consideration of extreme load effect can be mentioned as follows:

- Proportional Design
- Equivalent Load-Strength Design
- Safety Factor Design
- Weighted Load-Coefficient Design
- Design with High Accuracy Analysis
- Endurance Limit Design

Although their namings have not established as general ones, the author wants to do so for the convenience of the following discussion.

As we know, extreme load has stochastic nature, and strength of structure has also such nature more or less as shown in Fig. 1. In the shaded area, its failure occures. Two values, $R$ and $L$, express the resistance of structure and external load on the structure respectively, and $R_m$ and $L_m$ are the representative values of each ensémble. Then the factor of safety is defined as

$$s = L_m / R_m \tag{1}$$

It is very well known story, however, there are many questions; What shapes are their distributions? How to represent their strength in $R_m$? How to combine various load effects? How to decide their structural damping coefficient? How to evaluate the effect of imperfection on their buckling criteria? There have been many subjects since the origination of the idea. Although some of the questions, mentioned-above, involve very modern theoretical subjects, structural engineers have overcome many accidents based on their knowledges of year by year.

Galilei originated "strength of material" as a field of science in the seventeen century.[3] Girard presented an idea of a variable $M/Z$, we call this parameter as "bending stress" in the eighteen century. Also Parent discussed the stress-distribution in a thick cantilever beam subjected by a bending load in the same period. These studies were done mainly for the design of bridges. So the design of complicated shape structure like ship-vessels were mainly done according to the experience of design engineers like a design of bridges before the seventeen centuries. Even at the end of the nineteen century, the design of ship vessels was still using such procedure. Also the design of shell of oil storages was walking on the same way. If the figures of its specification was fixed, such as the total capacity, the variety of oil, soil condition and so on, then its hight, diameter, wall thickness, bottom plate configuration and other dimensions were decided by using rather simple formula or design curve plates. We call such design procedure as a "proportional design". This approach has been used until now partially for some basic relations. Of course, it is based on the concept of allowable strength of materials, however, their details are decided mainly by the experiences obtained through past troubles, that is, unexpected failure: collapse, cracking and buckling. These design relations are updating based on recent informations. The API code can be understood as one of the typical examples of such design procedure. And designers and the regulatory authority in Japan had been followed this, at least, until the experience of Niigata earthquake-1964. The fact, occured after the event, shows the difficulty on combining "proportional design" with an extreme load problem, unless we experience such events often, and unless we have chances to modify our design procedure through the experiences.

### 2.2 Equivalent Load-Strength Design

Most of anti-earthquakes design codes for building structures started from "seismic coefficient" methods. We employed this method in the very early stage of anti-earthquake design development, that is, immediately after Kwanto Earthquake-1923. The maximum ground acceleration in Tokyo was estimated 0.3 G from the record of the ground motion records in displacement. According to the fact that the elastic limit of ordinary concrete structures was one third of their ultimate strength (fig. 2), they employed 0.1 G for the seismic coefficient for the design. After the years, the quality of concrete was much improved, and its allowable stress was doubled. However, the people related this problem did not want to reduce structural rigidity and ductility, then the seismic coefficient was also doubled as 0.2 G. This story was not recorded exactly, therefore, some ten years later, most of structural engineers believe the value of 0.2 G is comming from the maximum ground acceleration of Kwanto Earthquake-1923 in Tokyo. We should notice that this method ignored also the amplification factor of structures, or was made under the assumption of applying to very rigid structures. The method assumes some level of input ground acceleration and limit of induced stress of structural element independently to the maximum ground acceleration ever experienced, the strength of materials which are concerning the dynamic response of the structure and so on. So, we call this method as "Equivalent Load-Strength Design".

### 2.3 Safety Factor Design

As clearly we know, the structural design is based on the relation of the minimum strength of the structure to the maximum expected load. So, if the load is simple, only the point of discussion remains on their ratios. And "Equivalent Load-Strength Design" is one of the cases where both are apparent for design. However, if the loading condition under earthquakes consists of several types of loading, that is, seismic induced load, internal pressure load, thermally induced load and so on, the allowable stress criteria should be based on its actual strength with some adequate margin. The stress induced by the internal pressure load is very exact one, and there is no uncertainty like natural hazard type extreme loads. The allowable stress under seismic condition should be considered by being based on the allowable stress to internal pressure. However, the stress induced by seismic load continues in very short duration compare to that induced by normal loads. Then we can cut their safety margin under consideration on the probability of the event. Such idea was employed as "short range allowable stress" in Japan, and also the allowable stress in Emergency Condition of ASME Section III. In another word, it is "safety-factor-reduction". We usually overcome the extreme load problem by this method, and can bridge the allowable stress criteria to the structural reliability problem..

### 2.4 Weighted Load-Coefficient Design

"Weighted Load-Coefficient Design" method is introduced to overcome the difficulty brought by the different probability of occurrences of two extreme events. According to the probability of occurrence and the margin which we want, we put a load-coefficient independently for each type of load. These coefficients can be defined by the structural reliability theory, however, we can define them by the experience. For the design of a bridge, this design method has been employed. Recently, it was also employed in the design of over-head crane guarder. In general, it is rather seldom to apply this method to anti-earthquake design.

### 2.5 Meaning of High Accuracy Design Analysis

We employ the Finite Element Method to analyze the stress distribution of the structure. In some cases of anti-earthquake design, sizes of meshes exceeded ten thousands. The author doubts its effectiveness, because the input force sequence is only assumed one. In a future earthquake, we will be not able to expect the same time history as we assumed, then the very high accuracy of calculation may be no meaning. Of course, the detail stress analysis to know the fine stress distribution or stress concentration factor is very useful for the design, especially to estimate their modes of failure. The fact, which the author wants to say, is that the high accuracy of stress analysis does not overcome the uncertainty of input loads, and still we need some concept of "safety factor" or "margin".

## 3. Design and Margin Evaluation

The author refered to the word of "margin" very frequently. However "the design based on safety factor" and "estimation of the margin" are different ones. The proposed Japanese New Building Code provides two steps, "design" and "margin evaluation". Basically the building code in Japan is "Equivalent Load-Strength Design" already described. The idea of the new code is that the first step is the *design* based on this, and the second step is the *margin evaluation* against the higher level earthquake input, maybe the probable maximum one.

How do we evaluate the margin of structural safety? The structural engineers in Japan solved this problem, and are applying them as actual practice for buildings. However, it is very difficult to evaluate those of equipment, vessels, pipings and other active components appeared in industrial facilities, such as chemical engineering plants, nuclear power plants. The author discussed this problem in an occasion of the 5th International Conference on Structural Mechanics in Reactor Technology[4] in 1979. Although procedure is fundamentally clear, we need a lot of practical data, especially on materials. And the author wants to mention that the reliability of structures can be led by the "Design Ampleness Index" described in next chapter.

## 4. Evaluation of Code Design Ampleness

### 4.1 Definition of Design Ampleness

To evaluate anti-earthquake code, there are several points. Its design approach was described in the chapter 3 is one of the points. Which categoly does it belong to? This fundamental question can be converted into a problem of the ratio of "design basis seismic coefficient" to the "probable maximum ground acceleration" in the concerning area. For example, the design basis seismic coefficient of spherical storage tanks in last several years was 0.3 G in Japan. In Southern Kwanto Area including Keihin Area (Southern part of Tokyo, Kawasaki and Yokohama) we should expect 0.5 ∿ 0.6 G earthquake according to Kobayashi[5] and Kawasumi's estimation. Their eigenperiods are 0.4 sec in average, and amplification factor in soft soil area, the categoly IV in the Building Code, is approximately 2.2. Totally it becomes 1.0 ∿ 1.2 G, then the reduced factor of *Design Basis Seismic Coefficient* $\eta$, that is

$$\eta = \frac{\text{design basis seismic coefficient}}{\text{probable maximum response acceleration}} \tag{2}$$

is counted as 0.3 ∿ 0.25. At present, some spherical storage tanks, which are categolized in Class I according to the regulatory guide of Kanagawa-pref.[6], are designed 0.6 G. Then this value is almost twice compare to those in other areas. The proposed anti-earthquake regulatory code for high-pressure facilities requires 0.66 G for spherical tanks in such categoly based on simplified dynamic design, then the reduction factor $\eta = 0.379$.

Another important factor of evaluating such codes is *stress allowance* factor $\xi$, that is,

$$\xi = \frac{\text{seismic allowable limit}}{\text{elastic limit of material}} \tag{3}$$

We recommend the elastic limit or 150% of the allowable stress for normal condition in the regulatory guide of Kanagawa-pref. as the allowable stress for the earthquake condition and the author refered to this in the previous paper[1]. In Emergency Condition defined by ASME Code 120% of the allowable stresses are those for equipment and pipings of nuclear power plants under seismic condition expect 2.25 Sm for (PL+Pb) of pipings.

Then $(\eta/\xi)$ is an index of the code for expressing its ampleness, then we may say "Design Ampleness Index". In the case of 0.6 G seismic coefficient design of spherical tanks by the regulatory guide of Kanagawa-pref., this value is 1.74/1.00=1.74. The building code originally intended to set this value to be $(\eta/\xi)=1/3$ based on the collapsing condition of concrete structures. In the case of nuclear power plants, the design basis earthquake $S_I$ *should be said to be the maximum probable ground acceleration, and the* amplification factor, or the response factor, is estimated as the worst case of response. Therefore, we can say $\eta=1$, and then at least $(\eta/\xi)=1/1.20=0.833$ for bending of piping, and $(\eta/\xi)=1/0.667\times1.2=1.25$ for general membrane stress. However, the expected maximum ground acceleration may be higher than this (Table 1, #12).

### 4.2 Design Ampleness Index Table and Various Codes

In Table 1, several typical examples of such figures are shown. The data for this table was prepared by the working group for the proposal of the anti-earthquake code of fire-protection equipment and pipings in buildings by the National Agency of Fire Protection. Through the table we can observe factors $\beta_1$, $\beta_2$, $\beta_3$ and $K$. These notations were employed in the equation (1) of the previous paper.

In the column #1 of the Table 1, the values, which we will be expected in a future earthquake in some areas, that is, the area of Olive-view Hospital in San Fernando earthquake-1971, Odawara in Kwanto earthquake-1923, are shown. The earthquake, which is assumed by the author here, may be MMI= X∿XII. The figures in [] under the first line are the overall amplification factor from free-field surface to equipment. And values in () with ### are the some worst cases caused by pseudo-resonance of a soil-supporting-structure-equipment system. The values of $\xi$ are tabulated according to several cases of allowable stresses as shown in the table. The columns #11 and #12 are the examples proposed by the author, and #12B is the assumed behavior of the plant under a future destructive earthquake condition estimated from the expected ground acceleration (marked with *$# in the Table 1) prepared by the earthquake prediction group.

The value of "design ampleness index" may be 0.5 or more for ordinary mild steel structure. The limit of the value depends on the ratio of collapsing stress to elastic limit. The value of 0.3 should be the lowest limit. And even though some codes have the lower values, this fact can be said that the code could not guarantee the complete safety of related structures. On the other hand, the author already mentioned on the uniformity of the value is also the condition of the well-designed code. In this point of view, ATC-3, #5 and #6 are well-designed. The author's proposal of #11 is not so good, because he employed the response spectra which have the tendency of the lower values in the lower frequency range, and which have the lower and flattened peaks in general. As the author has been described, this index has a lot of information to evaluate the various types of codes and regulatory statements.

## 5. Code Form and its Expression

Regulatory code or requirement will be analyzed by a quite different way compare to design method. The author mentioned on a classification of such regulatory codes in the other paper[7]. He defined two types of regulatory codes, that is, "closed form regulatory code" and "open form regulatory code". The degree of open form may be described in an index of concreteness or abstraction. He introduced an idea of "regulatory tree", which is twin of a fault tree. The turget of the regulation is the top event of the fault tree. To prevent an occurrence of top event, the regulatory statements, introduced from the regulatory tree are effective, and this tree is useful to evaluate the index.

Recent codes sometimes have very huge structure and are related with many codes in the neighbour fields and sub-systems. To avoid confliction to them and also statements within itself, we should examine its logical structure. This can be done by using LISP. This technique was described in the reference (7) written by Dr. Tsutsumi and the author.

## 6. Acknowledgement

This research has been made through the author's activity in the code committees for various items. He would like to express his gratitude to the committee members. Especially the Table 1 is the result of the survey made by the working group on the code of "anti-earthquake design of equipment and pipings for the fire-protection in high-rized building, operated by the National Agency of Fire Protection.

## Reference

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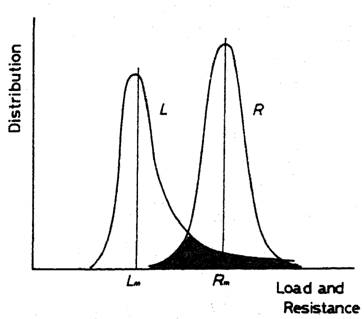


Fig. 1 Schematic Drawing on the Relation of Resistance and Load

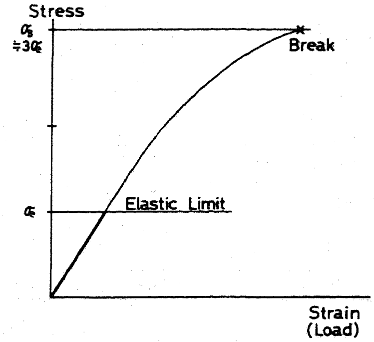


Fig. 2 Schematic Explanation of Resistance and Load Criteria of RC Structure

| CODE NUMBER | #1 | #2 | #3 | #4 | #5 |
|---|---|---|---|---|---|
| CODE AND NATION | TURGET VALUE FOR EVALUATION: ASSUMED BY AUTHOR | EMERGENCY DIESEL EQUIPMENT (GUIDELINE) PROPOSED IN AUG. '78 -JAPAN- | EMERGENCY POWER UNIT (MODIFICATION CRITERIA): BUILDING SAFETY CENTER -JAPAN- | EMERGENCY AND FIRE PROTECTION EQUIPMENT (REPORT): TOKYO MET. GOVERNMENT, PROPOSED IN MARCH '79 -JAPAN- | TENTATIVE PROVISION, SEISMIC REGULATION FOR BUILDING; NON-STRUCTURAL PART: PROPOSED BY ATC IN JUNE, '78 -U.S.A.- |
| SEISMIC COEFFICIENT RESALTANT :$c$ | 0.84~17.5(~42.0)### [ 1.2~25(~60) ] | 0.3~2.0 | 0.4~1.2 | 0.3~2.0 | 0.36~1.8 |
| MAX. GROUND ACC. AT ASSUMED BASED ROCK :$x_0$ | ----- | 0.15*,# | 0.10*,# | 0.15# | 0.20*,#(0.3) |
| MAX. GROUND ACC. AT FREE-FIELD SURFACE :$x$ | 0.7 | 0.3* | 0.2* | 0.3 | 0.4(0.6) |
| REDUCTION FACTOR FOR SITE LOCATION :$\gamma$ | 1.0 | 1.0~0.8 | UNIF. | UNIF. | 1.0~0.125 |
| REDUCTION FACTOR FOR IMPORTANCE :$\beta_1$ | ----- | ----- | ----- | 1.0/0.6*** | 0.5~1.5 |
| RESPONSE FACTOR OF SUPPORTING BUILDING :$\beta_{2b}$ | 1~5 | 1.00~3.33 | 1.00~3.00# | 1.00~3.33/ 1.00~3.00***,# | (0.6~3.0)** |
| RESPONSE FACTOR OF EQUIPMENT AND PIPINGS:$\beta_{2e}$ | 1.2~12 | 1.00/2.00 | 2.00$ | 1.00/2.00 | |
| SOIL AMPLIFICATION FACTOR :$\beta_3$ | ----- | ----- | ----- | ----- | ----- |
| $\eta$: REDUCED FACTOR OF DBSC | | 0.357~0.114 | 0.476~0.069 | 0.357~0.114 | (0.514~0.514) |
| DESIGN AMPLENESS INDEX ($\eta/\xi$) | | | | | [ 0.7~3.5 ] |
| N.C. OF ASME SECTION VIII TYPE($\xi$=0.500) | | 0.714~0.228 | 0.952~0.138 | 0.714~0.228 | (1.028~1.028) |
| N.C. OF ASME SECTION III TYPE ($\xi$=0.667) | | 0.535~0.171 | 0.714~0.103 | 0.535~0.171 | (0.771~0.771) |
| ELASTIC LIMIT ($\xi$=1.000) | | 0.351~0.114 | 0.476~0.069 | 0.357~0.114 | (0.514~0.514) |
| E.C. OF SEC. III TYPE; SHORT TERM 120% ALLOWANCE ($\xi$=1.200) | | 0.298~0.095 | 0.397~0.058 | 0.298~0.095 | (0.429~0.429) |
| VERTICAL GROUND MOTION CONSIDERATION | ----- | 1/2 | ----- | 1/2~2/3 | ----- |

* : Estimated value from another value appeared in the concerning code expression.

# : Estimated value from the assumption that soil amplification factor is two.

| CODE NUMBER | #6 | #7 | #8 | #9 | #10 | #11 | #12A | #12B |
|---|---|---|---|---|---|---|---|---|
| CODE AND NATION | TENTATIVE PROVISION SEISMIC REGULATION FOR BUILDING; ELECTRICAL & MECHANICAL EQUIPMENT: PROPOSED BY ATC IN JUNE, '78 -U.S.A.- | ARMY, NAVY & AF FACILITY (SPECIFICATION): TRI SERVICES IN '73 -U.S.A.- | UNIFORM BUILDING CODE: '76 -U.S.A.- | GSA EARTHQUAKE REGISTANCE OF BUILDINGS (GUIDELINE): PUBLIC BUILDING SERVICES '76 -U.S.A.- | NZA 4203 BUILDING CODE: '76 -NEW ZEALAND- | CONVENTIONAL PLANTS: PROPOSED BY THE AUTHOR | NUCLEAR POWER PLANT DESIGN; BWR 500MW IN SOFT-ROCK AREA: ASSUMPED BY THE AUTHOR | ASSUMED BEHAVIOR UNDER A DESTRUCTIVE EARTHQUAKE IN THE SITE |
| SEISMIC COEFFICIENT RESALTANT :$c$ | 0.36~7.2 | 12.5≥ | 0.24~8.44 | 0.15~9.00 (3.00) | 0.9~7.79 (~12.47) | (0.0378)#$ ~0.126~9.45 | 6.0 | 18.0 |
| MAX. GROUND ACC. AT ASSUMED BASED ROCK :$x_0$ | 0.2*,#(0.3) | 0.5~1.0# | 0.5~0.75*,# | 0.075*,# | 0.10*,#(0.3) | 0.15 | [ 0.226 ]## | [ 0.552 ] |
| MAX. GROUND ACC. AT FREE-FIELD SURFACE :$x$ | 0.4(0.6) | 1.0~2.0 | 1.0~1.5* | 0.15* | 0.20*(0.6) | 0.21~0.315 | 0.3 | 0.5*$# |
| REDUCTION FACTOR FOR SITE LOCATION :$\gamma$ | 1.0~0.125 | 1.0~0 | 1.0~0.19 | 1.0~0.083 | 1.0~0.66 | 1.0~0.4 | | |
| REDUCTION FACTOR FOR IMPORTANCE :$\beta_1$ | 0.5~1.5 | ----- | 1.0~1.5 | ----- | 1.0~1.6/1.0~3.0 | 0.5~1.0 | 1.0 | 1.0 |
| RESPONSE FACTOR OF SUPPORTING BUILDING :$\beta_{2b}$ | (0.6~3.0)** ×(1.0~2.0) | 0.10~0.25 | (0.8~2.5)** ×(1.0~1.5) | 1.0/1.5 | 1.5~3.0 | 0.3$$~1.0 ~3.0$ | 1.8 | 3 |
| RESPONSE FACTOR OF EQUIPMENT AND PIPINGS:$\beta_{2e}$ | 1.0/2.0 | 25≥ | 0.2~1.0 | 1~40** | 1~4.33* | 1.2~10$ | 12 | 10 |
| SOIL AMPLIFICATION FACTOR :$\beta_3$ | ----- | ----- | ----- | ----- | ----- | 1.4~2.1 | ----- | |
| $\eta$: REDUCED FACTOR OF DBSC | 0.429~0.411 | 0.714≥ | 0.286~0.482 | 0.179~0.514 | 1.071~0.445 | 0.150~0.540 | 0.333 | |
| DESIGN AMPLENESS INDEX ($\eta/\xi$) | | | | (0.171) | | | | |
| N.C. OF ASME SECTION VIII TYPE($\xi$=0.500) | 0.837~0.822 | 1.428≥ | 0.571~0.964 | 0.357~1.028 | 2.142~0.890 | 0.300~1.080 | 0.667 | |
| N.C. OF ASME SECTION III TYPE ($\xi$=0.667) | 0.643~0.617 | 1.071≥ | 0.428~0.723 | 0.268~0.771 | 1.606~0.667 | 0.225~0.810 | 0.500 | |
| ELASTIC LIMIT ($\xi$=1.000) | 0.429~0.411 | 0.714≥ | 0.286~0.482 | 0.179~0.514 | 1.071~0.445 | 0.150~0.540 | 0.333 | |
| E.C. OF SEC. III TYPE; SHORT TERM 120% ALLOWANCE ($\xi$=1.200) | 0.357~0.343 | 0.595≥ | 0.238~0.402 | 0.149~0.428 | 0.893~0.371 | 0.125~0.450 | 0.278 | |
| VERTICAL GROUND MOTION CONSIDERATION | ----- | ----- | ----- | 2/3 | ----- | 1/2~2/3 | 1/2 | |

Table 1 Seismic Coefficients and Design Ampleness Indices of Various Codes