大韓民国における建築物の地震被害低減に関する国際共同研究

課題番号：１２６５０５７１

平成12年度～平成13年度科学研究費補助金
（基盤研究C(2)）研究成果報告書

平成14年3月

研究代表者　　西山　峰広
（京都大学工学研究科助教授）

はじめに

過去2世紀にわたる韓国での地震活動度は大変低いものであった。そのため、韓国では長い間、この自然災害は国民から忘れられていた。このため、建物と橋梁の設計においても地震荷重は20世紀後半まで考慮されていなかった。朝鮮戦争後、すべてのものは破壊され、韓国政府の緊急の施策は人々に住まいを与えることであった。したがって、韓国国民が自国における地震活動度に十分気付いていたとしても、耐震設計に予算を割く余裕はなかった。

1978年にSeoulの南約200kmに位置するHong-Sungでリヒタースケール5.0の地震が起こった。この地震は、韓国民衆に地震被害を思い出させる契機となった。マスメディアは、競って韓国における地震活動度を取り上げ、起こり得る地震被害を低減するための行動をとるよう訴えた。2年後、Tangshanが出版されたとき、地震被害低減のための努力がなされていない韓国では同様の大惨事が起こるであろうことを、技術者たちが警告した。1985年のメキシコ地震は、韓国において何らかの反応を起こす結果となった最初の警鐘であった。メキシコ地震以後韓国においては、20階よりも高層のすべてのビルは、アメリカの設計基準であるUBCに基づき、それに規定されているZone 2で設計されなければならなくなった。この設計レベルは、韓国の地震活動度に適合するものと信じられていた。1986年、韓国建築学会は、建築構造物の耐震設計基準に関する研究プロジェクトを立ち上げた。

以上のような背景に基づき、本研究では、韓国における地震被害低減と耐震設計基規準整備のために、韓国における地震活動度、現行の設計基規準、現行設計法で設計された建築構造物の性能を調査した。

研究組織

研究代表者： 西山　峰広 （京都大学工学研究科助教授）
研究分担者： 渡邉　史夫 （京都大学工学研究科教授）
研究分担者： 河野　進 （京都大学工学研究科助手）
研究分担者： 吹田　啓一郎 （京都大学防災研究所助教授）

研究協力者： Lee, Dong-Guen
Sungkyunkwan University, Department of Architectural Engineering, Professor

研究協力者： Lee, Chul-Ho
Kyongnam University, Department of Architectural Engineering, Associate Professor

研究協力者： Kim, Jinkoo
Sungkyunkwan University, Department of Architectural Engineering, Associate Professor

研究協力者： Lee, Han-Seon
Korea University, Department of Architectural Engineering, Professor

研究協力者： Sang Whan Han
Hanyang University, Department of Architectural Engineerig, Professor

交付決定額（配分額）　　　　　　　　　　　　　　　　　　（金額単位：千円）

	直接経費	間接経費	合計
平成 12 年度	1,500	0	1,500
平成 13 年度	1,000	0	1,000
総計	2,500	0	2,500

1. 研究目的

大韓民国（以下韓国と略す）では、過去２世紀の間、地震活動度は大変低かった。そのため、20世紀後半まで地震荷重は、建築物や橋の設計において無視されていた。1978年にSeoulの南約200kmに位置するHong-Sungで起こった地震は韓国国民に地震被害の恐ろしさを改めて思い起こさせる契機となった。1985年にメキシコ地震が起こるに及んで、緊急に耐震設計を導入することとなった。すなわち、20階建て以上のすべてのマンションは、アメリカのUBCに規定されているZone2の耐震設計にしたがって設計されることとなった。1995年兵庫県南部地震は、その人的および物的被害の甚大さにより、韓国での建築物の耐震性を見直そうとする気運を盛り上げた。また、1999年9月に起こった台湾での地震はこの気運をさらに加速することとなった。

韓国は、地震国ではないように考えられがちであるが、実際にはそうではない。1400年から1800年の間に、改正メリカリ震度階で７よりも大きな地震の回数は、152回を数える。1801年から現在までは、その数は大きく減少しているが、計測機器の発達もあり、今世紀に入り、増加する傾向を見せている。このような状況のもと、1988年には最初の耐震設計基準が施行された。その後、10年以上が過ぎ、現在基準の見直し作業が行われている。

韓国の地震は日本ほど大きくはないが、地震被害を低減するためには耐震設計基準の見直しが重要である。また、ほとんどの建物は、1988年に耐震設計基準が施行される以前に建設されており、その補修・補強は急務である。さらには、高層鉄筋コンクリート建築物は「Box System」と呼ばれる構造形式により建設されている。これは、施工が容易であるため、広く普及しているが、地震に対して非常に脆弱であり、新しい構造システムの開発が期待されている。

以上のような背景に基づき、本研究では、韓国における建築物の耐震設計基準の整備に寄与するために、韓国での過去に起こった地震とその被害調査、諸基規準調査、韓国側研究者との情報交換・議論を通して、その適切な構造システムの開発、耐震設計法などを提案する。

2. 研究成果

2．1 韓国における過去の地震およびそれによる被害

韓国行政自治府国立防災研究所で行われた「韓半島(朝鮮半島)の地震災害度作成のための歴史被害地震の評価及び総合整理」研究報告書によると、各世紀における地震の数は、表－１のようになっている。この表に記されている地震は、19世紀までは歴史文献「三国史記」「高麗史」「朝鮮王朝實録」「武者」「承政院日記」から地震に関する記述を拾い、表－２のような基準によりその地震動の強さを決定している。また、20世紀の地震については、計器による観測に基づいている。

歴史書による最大の地震は、1643年7月24日に北緯35.5度、東経129.4度を震央として起きたことが「朝鮮王朝實録」により推定されている。この時の改正メリカリ震度階

表－1　各世紀における地震数

世紀	≦Ⅳ	Ⅴ	Ⅵ	Ⅶ	Ⅷ	Ⅸ	Ⅹ	小計(≧Ⅴ)
1	7	1	0	0	0	4	0	12(5)
2	9	1	0	0	0	1	0	11(2)
3	7	3	0	0	0	0	0	10(3)
4	4	0	0	0	0	3	0	7(3)
5	5	0	0	0	1	1	0	7(2)
6	4	0	0	0	0	2	0	6(2)
7	13	0	0	0	0	1	0	14(1)
8	20	1	0	0	1	2	0	24(4)
9	8	1	0	0	0	0	0	9(1)
10	8	1	0	0	0	0	0	9(1)
11	17	1	7	4	0	2	0	31(14)
12	17	1	0	0	0	1	0	19(2)
13	30	8	0	2	0	2	0	42(12)
14	66	12	0	1	0	2	0	81(15)
15	115	57	51	15	4	1	0	243(128)
16	343	229	102	38	5	4	0	721(378)
17	157	94	54	40	9	4	1	359(202)
18	109	49	27	12	3	7	0	207(98)
19	58	1	0	0	0	1	0	60(2)
20	709	24	10	4	5	2	0	754(45)
計	1706	484	251	116	28	40	1	2626(920)

表2 (a)　人間および動物関連の記述

記述類型	評価根拠	MM 震度	根拠番号
● 民家が倒れ組み敷かれて死んだ人がいる。 ● 土が揺れ、割れて家屋が陥没し死んだ人がいる。	震度評価参考事項1に記述した事故を検討してみると、当時の家屋はRichter(1956)が分類した構造物の中D級にあたるものであり、このような構造物の部分的な崩壊は震度Ⅷ以上から現れ始め、震度Ⅸで完全に破壊する。従って、震度Ⅸを与える。	Ⅸ	A-1
● 死んだ人が多かった。 ● 馬に乗った人が馬と一緒に死んだ。	死んだ直接的な原因に対する説明はないが、家屋の破損、重たいものの落下または転倒、土砂崩れなどが原因と考えられる。このようなことは震度Ⅷ以上から現れ始まるから震度Ⅷを与える。	Ⅷ	A-2
● 牛馬が立つことができないくらいの揺れがあった。 ● 人が立つことができない。 ● 人が倒れた。	震度Ⅵでは立って歩くことが困難であり、震度Ⅶになると立つことすら困難である。従って震度Ⅶを与える。	Ⅶ	A-3
● 鳥や畜類らが驚いて逃げた。 ● 多くの人々が驚いて建物の外に出た。	多数の人々が驚き、不安感で外に飛び立つことが震度Ⅵであらわれ、震度Ⅵを与える。	Ⅵ	A-4

表2 (b)　構造物との関連記述

記述類型	評価根拠	MM 震度	根拠番号
● 城門が自ずと倒れた。 ● 家屋が倒れた。	当時の構造物をRichter(1956)が分類したC-D級と見なすとこのような構造物の破損〜崩壊は震度Ⅸから起こるから震度Ⅸを与える。	Ⅸ	B-1
● 塀が倒れた。 ● 家屋や塀が揺れて倒れた	木造建物の土壁の崩壊は震度Ⅷから主に発生するから震度Ⅷを与える。	Ⅷ	B-2
● 屋根の瓦が落ちた。	高層建物、塔などの周囲装飾物が落ちる現象と同じに見て震度Ⅶを与える。	Ⅶ	B-3
● 屋根が傾いた。 ● 瓦がぶっつけられる音がした。	瓦が動くくらいの振動で被害が出てない場合はJMA震度Ⅵ程度に相当するから震度Ⅵを与える。	Ⅵ	B-4
● 地上のすべての家屋が揺れた。 ● 家屋が動いた。	建物全体が揺れる程度の振動は震度Ⅴであらわれるから震度Ⅴを与える。	Ⅴ	B-5
● 家が少し動いた。 ● 窓が揺れた。	窓が揺れる程度の振動は震度Ⅳであらわれるから震度Ⅳを与える。	Ⅳ	B-6

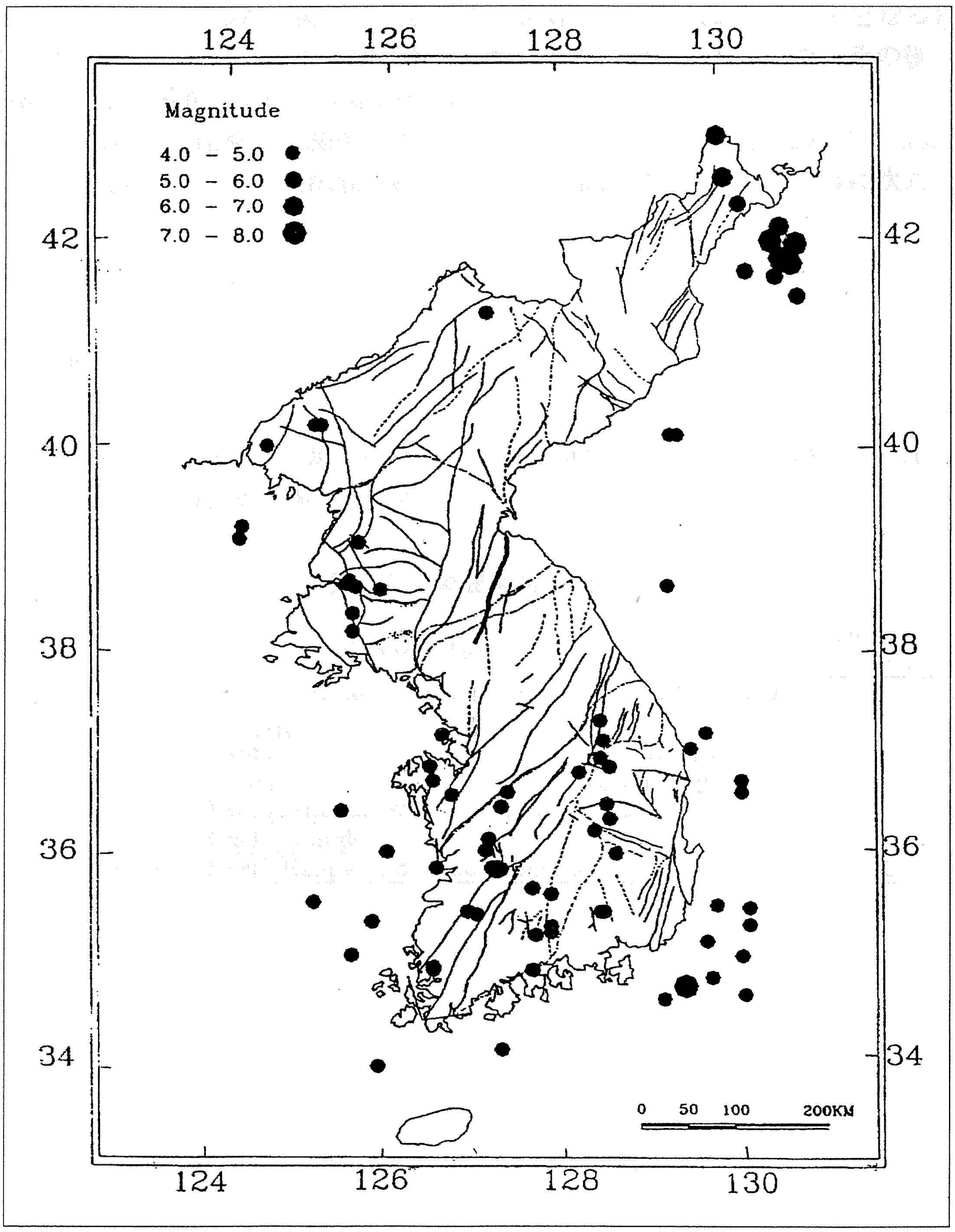
表2(c)　指標及び自然物の変化との関連記述

記述類型	評価根拠	MM 震度	根拠番号
● 地面が裂けて泉が湧いた。 ● 畑が裂けてそこに水が出た。 ● 道が裂けた。 ● 地面が多く破裂された。	震度Ⅷで急斜面やぬれた土に亀裂が生じ、震度Ⅸで大きな亀裂、土の噴出、地震による泉などが見られる。このようなことで震度Ⅸを与える。	Ⅸ	C-1
● 岩が倒れた。 ● 木が折れて倒れた。	凍った雪が割れるのは震度8であらわれて、堅固な石垣に亀裂ができることに相当する。木が折れることは震度Ⅷで見られる現象であり、岩が倒れることは記念塔や重たい家具の転倒と同じと見ると震度Ⅷに相当する。従って、震度Ⅷを与える。	Ⅷ	C-2
● 土が陥没され、泉ができた。 ● 河川の土が裂けた。 ● 池の水が泥水になった。	上のC-1と同じ現象であるが、その程度が相対的に小さい小規模の現象であるため震度Ⅶを与える。	Ⅶ	C-3
● 土が大きく揺れた。 ● 峰が揺れた。	ただ揺れだけがあったことであり、被害に関する記述がないため震度Ⅳを与える。	Ⅳ	C-4
● 雷のような大きな音がした。家屋が動いた。 ● 山が倒れそうな音がした。 ● 大きな鼓のような音がした。	大きな音がしたのは重量の大きいものが通過するころと同じ現象と見て震度Ⅳを与える。	Ⅳ	C-5
● 土が揺れた。 ● 小さな雷の音がした。	上の C-5 に比べて相対的に小さい地震と見られ震度Ⅲを与える。	Ⅲ	C-6

はXとされている。

図－1は、朝鮮半島で1905年から1993年までに起きた、マグニチュード4.3以上の地震の震央を記載したものである。

図－1　朝鮮半島で起きたマグニチュード4.3以上の地震（1905年～1993年）の震央



２．２　韓国における耐震設計基準の調査

韓国では、過去２世紀の間、地震活動度は大変低かった。そのため、20 世紀後半まで地震荷重は、建築物や橋の設計において無視されていた。1978 年に Hong-Sung で起こった地震は韓国国民に地震被害の恐ろしさを改めて思い起こさせる契機となった。1985 年にメキシコ地震が起こるに及んで、緊急に耐震設計を導入することとなった。すなわち、20階建て以上のすべてのマンションは、アメリカのUBCに規定されているZone2の耐震設計にしたがって設計されることとなった。1995 年兵庫県南部地震は、その人的および物的被害の甚大さにより、韓国での建築物の耐震性を見直そうとする気運を盛り上げた。また、1999 年 9 月に起こった台湾での地震はこの気運をさらに加速することとなった。

このような状況のもと、1988 年には最初の耐震設計基準が施行された。その後、10 年以上が過ぎ、現在基準の見直し作業が行われている。

したがって、現在の耐震設計基準はアメリカの主に太平洋岸で利用されているUBCに基づいている。付録１に、その英訳版を添付している。設計用ベースシアー Vは次式で与えられる。

$$V = \frac{AIC}{R}W$$

ここで、A: 地震地域係数、I ; 重要度係数、C: 動的係数、R: 応答修正係数、W: 建物重量

地震地域係数 A と重要度係数 I は、表－３および４のように与えられている。

表－３　地震地域係数 A

Seismic Zone	Relevant Region		Seismic Zone Factor (A)
I	City	Seoul, Inchon, Taejon, Pusan, Taeku, Ulsan, Kwangju	0.11
	Province	Kyonggi-do, South Kangwon-do, Chungchcongnam-do, Kyongsangbuk-do, Kyongsangnam-do, Chollabuk-do, North Chollanam-do	
II	Province	North Kangwon-do, South Chollanam-do, Chcju-do	0.07

表－4　重要度係数Ｉ

Grade	S	1	2
Function and Size of Building	facilities for storage and treatment of hazardous materials with the total floor area not less than 1000m², hospitals, broadcasting stations, power plants, telegraph-telephone offices, public service facilities and facilities for the old and the weak, apartment buildings with the total number of stories above the ground level not less than 15.	arenas, sporting facilities, transportation facilities, exhibition and convention facilities, and stores with the total floor area not less than 5000m², lodging facilities, officetels, dormitories and apartment buildings with the number of stories not less than 5.	For all other buildings
Importance Factor	1.5	1.2	1.0

７階建のアパートを対象にして、韓国と日本における設計用地震荷重を比較すると以下のようになる。まず、条件として、韓国ではソウル、日本では東京に建設するとする。また、構造形式は、ＲＣ造柱梁靭性骨組とする。地盤は、ソウルでは中程度の堅さ、東京では２種地盤とする。

韓国の場合、

$A = 0.11$

$I = 1.2$

$S = 1.5$

$T = 0.0731 h_n^{3/4} = 0.717(s)$

$$C = \frac{1.5}{1.2\sqrt{0.717}} = 1.476$$

$V = 0.039W$　（ＲＣ造柱梁靭性骨組の場合）

$V = 0.056W$　（ＲＣ造柱梁普通骨組の場合）

日本の場合、

$T = 0.02h = 0.42(s)$

$R_t = 1.0$

$Z = 1.0$

$C_o = 0.2$　（１次設計用）　$C_o = 0.3$　（２次設計用）

$D_s = 0.3$

$$V = ZR_tC_oW = 0.2W$$ （１次設計用）

$$V = D_sZR_tC_oW = 0.3W$$ （２次設計用）

同じ終局強度設計となる韓国の0.039Wと日本の0.3Wを比較すると、日本の設計用荷重は韓国の7.7倍となる。

２．３　韓国の耐震設計法とISO3010との比較

現行の韓国における耐震設計基準は1988年に制定された。現在、建物に対する耐震設計は義務となっている。朝鮮半島の地震活動度は、中程度と低レベルの中間にある。韓国の耐震設計基準は基本的にアメリカのUBCとATC3-06に基づいて制定されている。この韓国の耐震設計法をISO3010に規定されている設計法と比較した「" Investigation on Relationship between Seismic Design Method in Korean Seismic Provisions and ISO" by Sang Whan Han」。考慮したのは、地震強度、重要度係数、地震危険地域係数、強度低減係数およびスペクトルである。さらには、設計用地震力の建物高さ方向の分布と、せん断設計に関しても比較した。

その結果、ふたつの基準の最も大きな違いは、考慮している限界状態であることがわかった。ISO3010では、使用限界と終局限界という２つの限界状態を明確に規定しているのに対して、現行の韓国耐震設計基準は、終局限界状態しか考慮していない。しかしながら、韓国の耐震設計基準は、現在改訂中であり、その完了後には、使用限界と終局限界の２つの限界状態を考慮することになる。

２．４　現行耐震設計法で設計された建物の性能

従来の大韓民国における耐震設計法では、要求性能目標は１段階のみであった。1997年に建設交通省と韓国地震工学会は、性能評価型耐震設計を行うための新しい指針を策定した。この指針では、機能保持と破壊防止という２つの性能基準を設定し、対応する地震動を提案している。本研究「" Analysis of a Steel Framed Structure Using Capacity Spectrum Method" by Jinkoo Kim」では、ある鉄骨骨組を、１段階の設定目標しかない従来の耐震設計法を用いて試設計し、崩壊防止性能目標に対応する大地震を受けたときの非線形挙動をCapacity Spectrum Methodを用いて解析した。最大層間変形を新指針に規定されている性能規範と比較することにより、従来の設計法を用いて設計された本試設計建物が、新しく提案された性能に基づく設計指針により推奨されている性能規範を満足するかどうか確認した。また、同時に時刻歴解析も行い、Capacity Spectrum Methodにより得られた結果と比較した。

その結果、本試設計建物が、従来の、設計性能目標が１段階の設計法で設計されたにもかかわらず、崩壊防止性能目標に対応する地震レベルである、再現期間2400年の地震動に対しても、安全性が確保されていることが示された。さらに、Capacity Spectrum Methodにより得られた結果は、時刻歴解析により得られた結果とよく一致することが示された。

３．まとめ

現在、韓国の耐震設計基準は改定作業中である。本改訂完了後には、使用状態と終局状態という２つの限界状態を考慮することになる。使用限界状態と終局限界状態の２つの限界状態に対して設計を行うことは、現在、ほとんどの国の耐震設計基準に取り込まれている。日本においても、許容応力度等計算法では、許容応力度による１次設計が、使用限界状態設計に対応し、保有耐力による２次設計が、終局限界状態に対応している。また、新たに策定された限界耐力計算では、損傷限界と安全限界という２つの限界状態が規定されている。

本研究では、現行基準で設計された鉄骨造建物の耐震性能を解析的に検討した結果、本試設計建物が、従来の、設計性能目標が１段階の設計法で設計されたにもかかわらず、崩壊防止性能目標に対応する地震レベルである、再現期間2400年の地震動に対しても、安全性が確保されていることが示された。韓国の地震活動度は、日本に比べてかなり低いため、建物の設計は、地震力ではなく、風や積載荷重によって決定されている場合もある。風荷重や積載荷重の見積もりは、さほど大きくはずれることはない。このため、設計用荷重と建物強度が拮抗するような設計が行われることがある。このような場合、偶発的に設計用荷重よりも大きな荷重が作用すると、建物が崩壊する場合もあり得る。このような事故は、地震活動度が低い地域において、建物が突然崩壊するという現象となって表れることがある。

一方、地震のように不明確な荷重は、設計用荷重と実際に地震が起こって建物に作用する荷重とは大きく異なることがある。そのような場合においても、人命だけは救われるような設計がなされるべきである。

韓国の地震活動度は日本と比べて低いものの、地震がないわけではない。現在、多方面で精力的な研究が研究が積み重ねられているので、将来、優れた耐震設計基準が制定されるものと信じている。

添付資料：

1. 現行韓国耐震設計基準（英文）

2. " Investigation on Relationship between Seismic Design Method in Korean Seismic Provisions and ISO" by Sang Whan Han

3. " Analysis of a Steel Framed Structure Using Capacity Spectrum Method" by Jinkoo Kim

現行韓国耐震設計基準（英文）

1. General

1.1 Scope

This chapter prescribes design earthquake load. Structures shall be designed with adequate strength to withstand the base shear, lateral seismic force, story shear, torsional moment, and overturning moment induced by earthquake loads and consideration shall be given to the story drift and building separation.

1.2 Definitions of Terms and Notations

(a) Terms

① **BASE** is the level at which the earthquake motions are considered to be imparted to the structure.

② **BASE SHEAR** is the total design lateral force or shear at the base of a structure.

③ **SHEAR WALL** is a wall designed to resist lateral forces parallel to the plane of the wall.

④ **BEARING WALL SYSTEM** is a structural system that vertical and lateral loads are resisted by shear walls

⑤ **MOMENT RESISTING FRAME SYSTEM** is a structural system that vertical and lateral loads are resisted by frames composed of beams and columns.

⑥ **DUAL SYSTEM** is a ductile moment-resisting frame, taking more than 25% of the lateral loads, combined with shear walls or braced frames.

⑦ **BUILDING FRAME SYSTEM** is essentially a space frame that provides support for vertical loads, combined with the shear walls and braces resisting lateral loads.

⑧ **DUCTILE MOMENT RESISTING FRAME** is a moment resisting frame specially detailed to provide ductile behavior and comply with the requirements given in Section 10 or Section 11.

⑨ **ORDINARY MOMENT RESISTING FRAME** is a frame not meeting the requirements in Sections 10 or Section 11 for ductile behavior.

⑩ **BRACED FRAME** is an essentially vertical truss system to resist lateral forces.

⑪ **LATERAL SEISMIC FORCE** is a lateral force distributed from the base shear to each floor .

⑫ **STORY DRIFT** is the lateral displacement of one level relative to the level above or below.

⑬ **NONSTRUCTURAL ELEMENTS** are the components which are not part of the structural resisting system.

(b) Symbols

A : Seismic zone factor

A_e : Cross-sectional area, in square meters, of a shear wall parallel to the seismic load direction in the first story

b_c : Column flange width

C : Dynamic coefficient

C_p : Dynamic coefficient for nonstructural components

d_b : Beam depth

d_c : Column depth

D_e : The length of a shear wall, in meters, parallel to the seismic load direction in the first story

f_a : Axial compressive stress in a column

F_i : Lateral seismic force applied to level i

F_p : Lateral seismic forces on a part of the structure.

F_x : Lateral seismic force applied to level x

F_y : Specified minimun yield strength

F_{yb} : Yield strength of a beam

F_{yc} : Yield strength of a column

g : Acceleration due to gravity

h_i, h_x : Height, in meters, above the base to level i, and x, respectively.

h_n : Height, in meters, above the base to the top floor of the building.

I : Importance factor

M_{pz} : Sum of the beam moments framing into the column flanges at the connection when the shear strength of the panel zone under consideration reaches the value specified in Section 11.2 (3).

R : Response modification factor

S : Site coefficient for soil characteristics

T : Fundamental period, in seconds, of vibration of the structure

t : Total thickness of panel zone in joints.

t_{cf} : Thickness of column flange

V : Design base shear

V_x : The design story shear in story x

W : The total seismic dead weight defined in Section 2.7

W_i, W_x : The portion of W located at or assigned to level i or x

W_p : The total weight of a nonstructural element.

Z_b : Plastic section modulus of a beam

Z_c : Plastic section modulus of a column

δ_x : Horizontal displacement at level x relative to the base due to applied lateral forces

δ_{xe} : Horizontal displacement at level x relative to the base obtained from the elastic analysis

2. Design Base Shear

2.1 Determination of design base shear

$$V = \left(\frac{AIC}{R}\right) W \tag{1}$$

V : design base shear
I : importance factor
R : response modification factor
A : seismic zone factor
C : dynamic coefficient
W : total seismic dead load

2.2 Seismic zone factor

The seismic zone factor in Section 2.1 shall be determined from the following table in accordance with the seismic zone.

Table 1. Seismic zone factor

Seismic Zone	Relevant Region		Seismic Zone Factor(A)
I	City	Seoul, Inchon, Taejon, Pusan , Taeku, Ulsan, Kwangju	0.11
	Province	Kyonggi-do, South Kangwon-do*, Chungcheongnam-do, Kyongsangbuk-do, Kyongsangnam-do, Chollabuk-do, North Chollanam-do**	
II	Province	North Kangwon-do***, South Chollanam-do****, Cheju-do	0.07

*South Kangwon-Do : Youngwol, Chongseon, Samcheok, Kangnung, Tonghae, Wonju, Taebaek
**North Chollanam-Do : Jangseong, Tamyang, Kokseong, Kulye, Janghung, Poseong, Yochon, Hwasun, Kwangyang, Naju, Yochon, Yosu, Sunchon
***North Kangwon-Do : Hongchon, Chulwon, Hwachon, Hoingseong, Pyongchang, Yanggu, Inje, Koseong, Yangyang, Chunchon, Sokcho
****South Chollanam-Do : Puan, Sinan, Wando, Youngkwang, Jindo, Haenam, Youngam, Kangjin, Kohung, Hampyong, Mokpo

2.3 Importance factor

The importance factor in Section 2.1 shall be determined from the following table in accordance with the function, size of the building and the site location.

Table 2. Importance factor

Grade	S	1	2
Function and Size of Building	facilities for storage and treatment of hazardous materials with the total floor area not less than 1000 m^2, hospitals, broadcasting stations, power plants, telegraph-telephone offices, public service facilities and facilities for the old and the weak, apartment buildings with the total number of stories above the ground level not less than 15.	arenas, sporting facilities, transportation facilities, exhibition and convention facilities, and stores with the total floor area not less than 5000 m^2, lodging facilities, officetels, dormitories and apartment buildings with the number of stories not less than 5.	For all other buildings
Importance Factor	1.5	1.2	1.0

2.4 Dynamic coefficient

The dynamic coefficient in Section 2.1 shall be determined from the following:

(a) The dynamic coefficient, C, shall be determined from the following formula, but need not exceed 1.75.

$$C = \frac{S}{1.2\sqrt{T}} \tag{2}$$

T : fundamental period of vibration, in seconds, of the structure
S : site coefficient for soil characteristics

(b) The fundamental period of vibration of the structure, T, in Item (a) may be approximated from the following formulae (3), (4), and (5) or may be calculated from the eigenvalue analysis. However, the value of T from the eigenvalue analysis shall not exceed 1.2 times the value obtained from the formulae (3), (4), and (5).

$$T = 0.0853(h_n)^{3/4} \quad \text{(steel moment resisting frames)} \tag{3}$$

$$T = 0.0731(h_n)^{3/4} \quad \text{(reinforced concrete moment resisting frames)} \tag{4}$$

$$T = 0.0488(h_n)^{3/4} \quad \text{(all other buildings)} \tag{5}$$

h_n : height, in meter, above the base to the top floor of the building
Alternatively, the value of T for buildings with shear walls may be determined from the formula (6).

$$T = 0.0743(h_n)^{3/4}/\sqrt{A_c} \qquad (6)$$

$$A_c = \Sigma A_e[0.2 + (D_e/h_n)^2]$$

where the value of D_e/h_n shall not exceed 0.9 .

A_e : Cross-sectional area, in square meters, of a shear wall parallel to the seismic load direction in the first story

D_e : the length, in meter, of a shear wall in the first story in the direction parallel to the applied forces.

2.5 Site coefficient for soil characteristics

The site coefficient, S, in section 2.1, shall be determined in accordance with the following criteria.

(a) The site coefficient, S, shall be determined according to the soil type from the following table.

Table 3. Site coefficient for soil characteristics

Type	S_1	S_2	S_3	S_4
Site Coefficient	1.0	1.2	1.5	2.0

(b) Soil profile type shall be selected in accordance with Table 4 and the classification of the soil shall be determined in accordance with Table 5.

Table 4. Soil profiles

TYPE	DESCRIPTION
S_1	A soil profile with either: (a) A rock-like material characterized by a shear-wave velocity not less than 700m/s, or (b) Medium-dense to dense or medium-stiff to stiff soil conditions, where soil depth is less than 60m
S_2	A soil profile with predominantly medium-dense to dense or medium-stiff to stiff soil conditions, where the soil depth not less than 60m
S_3	A soil profile containing more than 6m of soft to medium-stiff clay but not more than 12m of soft clay
S_4	A soil profile containing more than 12m of soft clay characterized by a shear wave velocity less than 150m/s

Table 5. Classification of soil condition[1,2]

Soil Condition		SPT N-index	Relative Density(%)	Undrained Shear Strength (kgf/cm^2)	Shear Wave Velocity (m/sec)
Rock					> 700
very dense or hard	sand	> 40	75 - 100		
	clay	> 24		> 3.0	
dense or stiff	sand	30 - 40	65 -75		
	clay	16 - 24		2.0 - 3.0	
compact or loose	sand	< 30	< 65		
	clay	< 16		< 2.0	

1. When the soil profile consists of more than 2 types, the larger site coefficient should be applied. In locations where the soil condition are not clear, a qualified professional geologist certified by the Government can determine the soil condition.
2. Even with the pile foundations lying on the rock bed, site condition shall be determined neglecting the effect of piles.

2.6 Structural systems

The response modification factor used in Section 2.1 shall be determined from the table 6 in accordance with the corresponding structural system.

Ductile Moment Resisting Frame or Dual System shall satisfy the detailing requirements of Section 10 or Section 11. If not, it shall be classified to Ordinary Moment Resisting Frame. If a building has combinations of different structural systems, the value of R used in the design of any story shall be less than or equal to the value of R used in the given direction for the story above. However, this requirement needs not be applied to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure. In the above table, boundary elements shall be considered to resist axial force and moment, while shear walls resist only shear force. And boundary elements shall satisfy the minimum requirement of columns.

2.7 Total seismic dead weight

The total seismic dead weight in formula (1) of Section 2.1 shall be taken equal to the total weight of the structure and applicable portions of other components including partitions and permanent equipments. In storage and warehouse occupancies, 25 percent of the floor live load shall be applicable.

Table 6. The response modification factors

<table>
<tr><th>Structural System</th><th colspan="2">Vertical Seismic Resistance System</th><th>Response Modification Factor(R)</th></tr>
<tr><td>Bearing Wall System: A structural system with bearing walls providing support for all, or major portions of, the vetical loads. Seismic force resistance is provided by shear walls or braced frames.</td><td colspan="2">Reinforced Concrete Shear Wall</td><td>3.0</td></tr>
<tr><td rowspan="4">Moment Resisting Frame System: A structural system with an essentially complete Space Frame providing support for vetical loads. Seismic force resistance is provided by Ductile or Ordinary Moment Resisting Frames</td><td rowspan="2">Ductile Moment Resisting Frame</td><td>Reinforced Concrete</td><td>5.0</td></tr>
<tr><td>Steel</td><td>6.0</td></tr>
<tr><td rowspan="2">Ordinary Moment Resisting Frame</td><td>Reinforced Concrete</td><td>3.5</td></tr>
<tr><td>Steel</td><td>4.5</td></tr>
<tr><td rowspan="3">Dual System: A Ductile Moment Resisting Frame shall be provided which shall be capable of resisting at least 25% of the prescribed seismic forces. Total seismic force resistance is provided by the combination of the Ductile Moment Reisting Frame and shear walls or braced frame in proportion to their relative rigidities.</td><td colspan="2">Reinforced Concrete Shear Wall and Steel Ductile Moment Resisting Frame</td><td>6.0</td></tr>
<tr><td colspan="2">Reinforced Concrete Shear Wall and Reinforced Concrete Ductile Moment Resisting Frame</td><td>5.5</td></tr>
<tr><td colspan="2">Steel Braced Frame and Ductile Moment Resisting Frame</td><td>4.5</td></tr>
<tr><td rowspan="2">Building Frame System: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.</td><td colspan="2">Reinforced Concrete Shear Wall</td><td>4.0</td></tr>
<tr><td colspan="2">Braced Frame</td><td>3.5</td></tr>
<tr><td colspan="3">Vertical Cantilever Type Structural System: (elevated water tank, control tower, observatory, silos, chimneys, steel tower etc.)</td><td>1.5</td></tr>
<tr><td colspan="3">Undefined System</td><td>3.0</td></tr>
</table>

3. Lateral Seismic Force

The lateral seismic force, F_x, induced at any level, shall be determined in accordance with the following formula :

$$F_x = \left(\frac{W_x\, h_x^{\;k}}{\sum_{i=1}^{n} W_i\, h_i^{\;k}} \right) V \qquad (7)$$

where $k=1.0$ for $T<1.0$ sec

$k=1.5$ for $1.0<T<2.0$ sec

$k=2.0$ for $T>2.0$ sec

F_x : lateral seismic force at level x

W_i, W_x : weight at level i, x

h_i, h_x : the height from the base to level i, x

V : base shear

4. Story Shear

The story shear in any story is obtained from the following formula:

$$V_x = \sum_{i=x}^{n} F_i \tag{8}$$

where V_x : shear in story x

F_i : lateral seismic force at level i

5. Horizontal Torsional Moment

The horizontal torsional moment shall be determined from the torsional moment resulting from the eccentricity between the centers of mass and stiffness plus an accidental torsional moment. In this case the accidental torsional moment shall be obtained by multiplying the lateral seismic force to the assumed accidental eccentricity corresponding to 5% of the plan dimension perpendicular to the direction of the load.

6. Story Drift

Story drift shall be less than 1.5% of story height, and story drift at each story is computed by the following formula:

$$\delta_x = R\delta_{xe} \tag{9}$$

where δ_x : story drift in story x

R : response modification factor

δ_{xe} : story drift in story x from elastic analysis

7. Building Separations

All structures shall be separated from adjoining structures. Separation shall allow for the distance obtained from the following formula:

$$d_T = \sqrt{d_1^2 + d_2^2} \tag{10}$$

where, d_T : minimum distance between adjoining structures

d_1, d_2 : drift of a roof obtained from each adjoining structure in accordance with the formula in Section 6

8. Nonstructural Element

8.1 General.

Permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 8.2. Floor- or roof-mounted equipment weighing less than 180kg are excluded.

8.2 Lateral seismic force

The lateral seismic force, F_p, shall be determined from the following formula:

$$F_P = A\,I\,C_P\,W_P \tag{11}$$

where, A : seismic zone factor

I : importance factor

C_p : dynamic coefficient for nonstructural element

W_p : total weight of nonstructural element

8.3 The seismic zone factor shall be determined from the Table 1 in Section 2.2.

8.4 The importance factor shall be determined from the Table 2 in Section 2.2.

8.5 The dynamic coefficient C_p is for the rigidly connected equipment, and the corresponding values are as follows:

(a) Chimneys, trussed towers and tanks on legs, sign and billboards,
exterior and interior ornamentations and appendages 2.00

(b) All others 0.75

8.6 Forces determined from the Section 8.2 shall be distributed in proportion to the mass distribution of the nonstructural elements.

9. Irregular Structures

9.1 Irregular structures, featured by vertical irregularities and/or plan irregularities, have significant physical discontinuities in configuration or in their lateral-force-resisting systems.

9.2 Structures having any of the features listed in Table 7 shall be designated as having a vertical irregularity. Where no story drift under design lateral forces is greater than 1.3 times the story drift of the story above, the structure may be deemed not to have the structural irregularities of Type 1 or 2 in Table 7.

9.3 Structures having any of the features listed in Table 8 shall be designated as having a plan irregularity.

9.4 Structures classified as Grade 2 in any seismic zone need to be evaluated only for vertical irregularities of Type 5 (Table 7) and plan irregularities of Type 1 (Table 8).

Table 7. Vertical structural irregularities

Irregularity Type	Definition
1. Stiffness irregularity	Stiffness irregularity shall be considered to exist where the lateral stiffness of a soft story is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above..
2. Weight irregularity	Weight irregularity shall be considered to exist where the effective weight of any story is more than 150% of the effective weight of an adjacent story. A roof that is lighter than the floor below need not be considered.
3. Geometric irregularity	Geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130% of that in an adjacent story. Where the sum of projections of a penthouse is not greater than 1/8 times the plan area of the building structure, any penthouse not higher than 12m need not be considered.
4. In-plane discontinuity in lateral-force-resisting element	In-plane discontinuity shall be considered to exist where an in-plane offset of the lateral-load-resisting elements is greater than the length of those elements.
5. Discontinuity in capacity	Discontinuity in capacity shall be considered to exist where the strength of a weak story is less than 80% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

Table 8. Plan Structural Irregularities

Irregularity Type	Definition
1. Torsional irregularity	Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.
2. Re-entrant corners	Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15% of the plan dimension of the structure in the given direction.
3. Diaphragm discontinuity	Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50% from one story to the next.
4. Out-of-plane offsets	Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.
5. Nonparallel systems	The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.

10. Ductile Moment Resisting Reinforced Concrete Frame

Design and construction of ductile moment resisting reinforced concrete frames shall conform to the requirements in the Appendix A of "Building Code Requirements for Reinforced Concrete using Strength Design Method" of Architectural Institute of Korea.

11. Ductile Moment Resisting Steel Frame

11.1 General

Design and construction of ductile moment resisting steel frames shall conform to the requirements of the general code using the allowable stress design method and to the requirements of this section.

11.2 Beam-column connections

(a) The beam-column connection shall be adequate to develop the lesser of the strength of the girder in flexure and the moment corresponding to development of the panel zone shear strength.

(b) The beam-column connection may be considered to be adequate to develop the flexural strength of the girder if it conforms to the following:

1. The flanges have full-penetration butt welds to the columns
2. The girder web-to-column connection shall be capable of resisting the girder shear determined for the combination of gravity loads and the seismic shear forces which result from compliance with Item (a). This connection strength need not exceed that required to develop gravity loads plus response modification factor, R, times the girder shears resulting from the prescribed seismic forces.

(c) Connection configurations utilizing welds or high-strength bolts not conforming with Item (b) may be used if they are verified to meet the criteria in Item (a) by test or calculation. Where conformance is shown by calculation, 125 percent of the strength of the connecting elements may be used.

(d) For steel whose specified ultimate strength is less than 1.5 times the specified yield strength, plastic hinges shall not form at locations in which the beam flange area has been reduced, such as for bolt holes. Bolted connections of flange plates of beam-column joints shall have the net-to-gross area ratio equal to or greater than 1.25 times the ratio of yield strength to minimum tensile strength.

(e) Trusses may be used as horizontal members in ductile moment-resisting frame if the sum of the truss seismic force flexural strength exceeds the sum of the column seismic force flexural strength immediately above and below the truss by a factor of at least 1.25. For this determination the strengths of the members shall be reduced by the gravity load effects. In buildings of more than one story, the column axial stress shall not exceed $0.4F_y$ and the ratio of the unbraced column height to the least radius of gyration shall not exceed 60. The connection of the truss chords to the column shall develop the lesser of the following:

1) The strength of the truss chord
2) The chord force necessary to develop 125 percent of the flexural strength of the column

11.3 Beam-column joint restraint

(a) Where it can be shown that the columns remain elastic, the flanges of the columns need be laterally supported only at the level of the girder top flange.

** original

$$\Sigma Z_c(F_{yc}-f_a)/\Sigma Z_b F_{yb} > 1.25 \tag{12}$$

$$\Sigma Z_c(F_{yc}-f_a)/1.25\Sigma M_{pz} > 1.25 \tag{13}$$

where

f_a : the axial compressive stress in a column

F_{yb} : the yield strength of the beam

F_{yc} : the yield strength of the column

Z_b : the plastic section modulus of the beam

Z_c : the plastic section modulus of the column

M_{pz} : the sum of beam moments when panel zone shear strength reaches the value specified in formula (14)

$$V = 0.55\ F_y d_c t\left[1+\frac{3\ b_c t_{cf}^2}{d_b d_c t}\right] \tag{14}$$

where,

F_y : the yield strength

b_c : column flange width

d_b : beam depth

d_c : column depth

t : the total thickness of the joint panel zone including doubler plates

t_{cf} : the thickness of the column flange

2) The flexural strength of the column is at least 1.25 times the moment that corresponds to the panel zone shear strength.

3) Girder flexural strength or panel zone strength will limit column stress under the load combination

to the yield strength of the column.

4) The column will remain elastic under gravity loads plus R times the prescribed seismic forces.

(b) Columns without lateral support transverse to a joint shall conform to the requirements with the column considered as pin ended. The column stress shall be determined from gravity loads plus the lesser of the following:

1) R times the prescribed seismic forces.

2) The forces corresponding to either 125 percent of the girder flexural strength or the shear strength of the panel zone.

The slenderness ratio for such columns shall not exceed 60.

As truss frames the column shall be braced at each truss chord for lateral force equal to one percent of the compression yield strength of the chord.

(c) Abrupt changes in beam flange area are not permitted within possible plastic hinge regions of ductile moment-resistant frames.

12. Dynamic Analysis Procedures

Dynamic analysis procedure shall be used for the structures of Importance Grade S or 1, which belong to one of the following conditions:

1) Regular structures with not less than either 70m in height or twenty one stories.

2) Irregular structures with not less than either 20m in height or six stories.

12.1. The ground motion representation shall, as a minimum, be one having a 10 percent probability of being exceeded in 50 years and may be one of the following.

(a) An elastic design response spectrum constructed in accordance with the equivalent static procedure.

(b) Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories shall approximate the site-specific design spectrum conforming to the equivalent lateral-force procedure.

12.2. A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response.

12.3. An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response shall be used. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal period. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

A. It shall be demonstrated that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

(b) Story forces, story shears, displacements, member forces and base reactions for each mode shall be combined by recognized methods. If necessary, other methods with equivalent accuracy can be used. When three dimensional models are used for analysis, modal interaction effects shall be considered in the process of combining modal maxima.

(c) When the design base shear calculated using dynamic analysis procedure for a given direction is less than that determined in accordance with the equivalent static procedure using the fundamental period scaled by multiplying the period described in Section 2.4(b)by the following factor, it shall be scaled up to the base shear corresponding to the scaled fundamental period. All corresponding response parameters, such as deflections, member forces and moments, shall be increased proportionally.

1) 1.5 times for regular structures
2) 1.2 times for irregular structures

(d) The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 6.5. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent lateral force procedures.

(e) Time-history analysis may be performed using the procedure based on an accepted principles of structural dynamics for analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

(f) Dual system shall be designed to resist at least 25 percent of the design base shear solely by ductile moment resisting frames.

Investigation on Relationship between Seismic Design Method in Korean Seismic Provisions and ISO

Sang Whan Han
Professor, Department of Architectural Engineering, Hanyang University, Seoul, Korea

Abstract

The Korean seismic design provision was developed in 1988. In present seismic design for building is compulsory in Korea. The seismicity of Korean peninsula is between low and moderate. Basically the Korean seismic provision was developed based on Uniform Building Code and ATC 3-06 provision. The objective of this paper is to investigate the relationship between the seismic design method in Korean seismic provision and ISO 3010. This paper classifies the major parameters included in both provisions and compares them. The parameters considered in this study are seismic intensity, importance factor, seismic hazard zone factor, strength reduction factor, and spectral ordinates. Also the vertical distribution of seismic force and shear of two provisions is also investigated and compared. Based on the investigation it is concluded that the most prominent difference between these two provisions is the limit states under consideration. In ISO 3010 two limit states are explicitly considered which are for serviceability and ultimate. In Korean provision one limit state is considered which is ultimate limit state. However, the provision in progress of revision considers two limit states for operation and collapse prevention.

1. Introduction

The current Korean earthquake provision presents only minimum criteria for the protection of life safety in buildings subject to earthquakes. There are, of course, some provisions which allow buildings to resist earthquake without structural or nonstructural damage rather than life safety. In the general seismic design provisions, however, the target limit state is that buildings must resist the possible maximum earthquake without loss of life during its life time.

In order to stipulate the earthquake resistant design procedure in Korea, the current building law, "Building Law Enforcement Ordinance and Detailed Structural Design Regulations," has been developed in 1988. The Korean seismic earthquake provision is based on ATC 3-06 provision and UBC (Uniform Building Code).

The following philosophy described in SEAOC "Blue Book" commentary is also valid to Korean seismic provisions since the provision is based on US seismic provisions.

(1) Minor earthquakes should be resisted without structural or nonstructural damage
(2) Moderate earthquakes should be resisted without significant structural damage but with some nonstructural damage
(3) Severe earthquakes should be resisted without collapse of the structure.

2. Earthquake Hazard of Korea

The historical records of Korean earthquakes indicate that Korea had many seismic activities as shown in Fig. 1. Since recording the earthquake using modern recording instrument in 1970 the Mt. Sokli earthquake-September, 1978, (M= 5.2), Hongsung earthquake-October, 1978, (M= 5.0), and Youngwall earthquake-December, 1996 (M=4.5) have been recorded. Table 1 shows the frequency of the occurrence of earthquake during a certain period of time in Korea. Also Table 2 shows the frequency of earthquake occurrence during 1978-1995 according to magnitude. Fig.2 shows the place and magnitude of earthquake occurred during 1986-1996.

Table 1 Earthquake records in Korea (based on human feeling)

	Date	**Occurrence/period**	**Occurrence/yr.**	**Remarks**
Three kingdoms	AD 2 – 917	99/916	0.1	the first record (AD 2)
Korea dynasty	917 –1392	182/474	0.4	
Chosun dynasty	1392 – 1904	1492/513	3	
Modern	1905 – 1995	456/91	5	the first seismograph installed (1905)

Table 2 Magnitudes and frequencies of recorded earthquakes (1978 – 1995)

Magnitude	**Frequency**	**Occurrence/yr.**	**Remarks**
6.5 < M < 7.0	-	-	possible limit range
6.0 < M	4	0.2	
5.0 < M	19	1.1	
4.0 < M	133	7.4	
3.0 < M	152	8.4	
Sum	308/18 yr.	17.1	

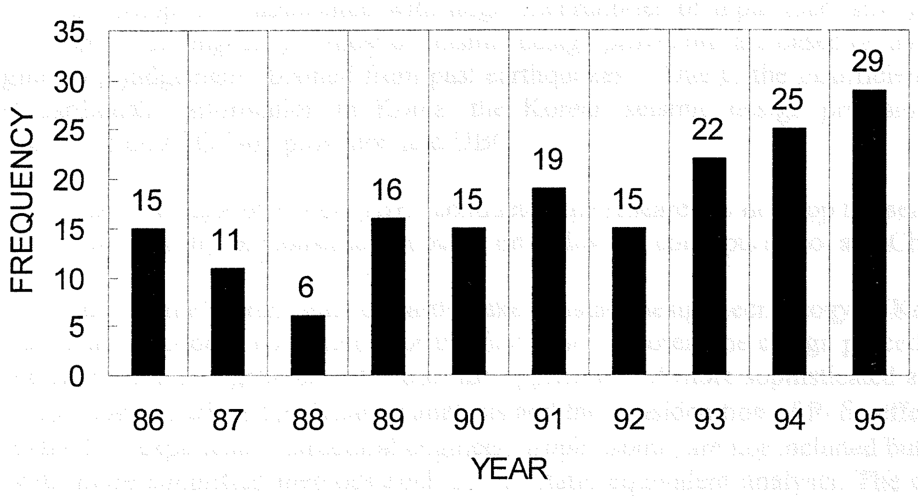


Figure 1　Occurrence of earthquake per year in Korea

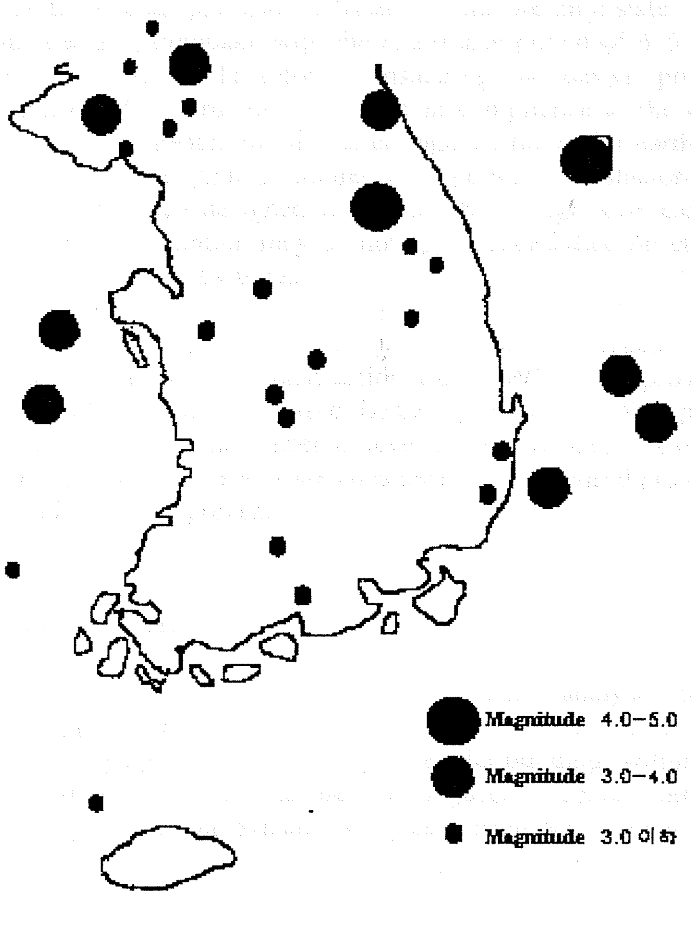


Figure 2　Place and Magnitude of Earthquakes in Korea during 1986-1996

3. Legislation and Revision in the Korean Seismic Design Provision

Because of the complexity associated with large uncertainties of input load, strength, analysis model in seismic building design, most of seismic design provisions are based on the experience and engineering judgement obtained from past earthquakes. Due to the insufficient records of the past earthquake information in Korea, the Korean seismic design provision has been developed based on ATC 3-06 provision and UBC.

The Architectural Institute of Korea (AIK) conducted the research to develop the seismic design provision for the ministry of Construction based on following concepts (Choi and Chung, 1992).

1) Considering the current state of earthquake resistant design technology in Korea and the fact that the code is constituted for the first time in Korea, the design procedures should be simplified to a great extent. Thus, the application of more sophisticated and complex technologies such as the dynamic analysis and the consideration of P- δ effect, etc., that make less-experienced structural engineers troublesome, are not included but substituted with more simplified methods such as the static equivalent analysis. The code can be improved in earthquake resistant design in the future.
2) In order to avoid any serious impact on national economy, the scope of application of the first code is narrowed down to the important buildings such buildings which have 6 floors or more as assembly hall, hospitals, communication centers, etc.

Also it is notify that the Korean provision is based on ultimate limit state. Thus, structures are designed using one design earthquake with the recurrence period of 475 year (seismic design load for ultimate limit state). Therefore, considering the design philosophy behind the provision as mentioned before structures designed in compliance to the current provision are expected not to collapse or threaten the life safety against the major earthquake. Behind this expectation it is also assumed that structures also perform satisfactorily against minor or moderate earthquakes if they are designed against one big design level earthquake for ultimate limit state. However, this assumption may be not appropriate based on current researches and experiences obtained from recent earthquakes.

Because of the obscurity in current seismic design procedure the revision of the provision is in progress by the Korean ministry of Construction from 1997. The provision moves toward adopting the concept of Performance Based Design (PBSD). In this provision two design earthquakes are considered which has different levels. The level depends on limit states and the importance of buildings. Two limit states are considered in the revised provision which are limit states for operation and collapse prevention.

4. Korean Seismic Provision

The Korean seismic code utilizes the static equivalent analysis technique, which is advantageous in computational efforts and economy, and also dynamic analysis. The static equivalent analysis may be applied appropriately to regular buildings within certain height limit, which have similar stiffnesses and masses between adjacent members and floors. The Korean seismic design code urges to use the dynamic analysis in irregular buildings.

4.1. Base Shear (V)

Base shear induced by design level earthquake is determined by the following equation:

$$V = \frac{AISC}{R} W \quad (1)$$

where

V : base shear,
A : seismic zone factor,
I : important factor,
C : dynamic coefficient,
S : site coefficient for soil characteristics,
R : response modification coefficient,
W : effective weight of structure. Each factor is decided by the following concepts.

1) Seismic Zone Factor (A)

The seismic map in Korean provision is divided into three zones as shown in Table 3 and Figure 3. The seismic zone factors in the provision, which are directly related to the maximum effective ground acceleration (EPA), were decided based on the historical and instrumental seismic records.

Table 3 Seismic zone factor

Zone	0	1	2
A	-	0.08	0.12

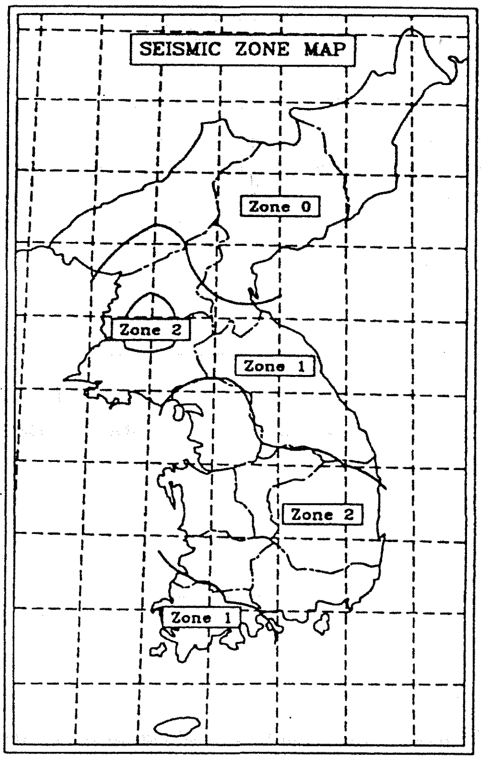


Figure 3 The Seismic zone map in Korea

2) Importance factor (I)

In Korean provision, importance of a building is decided by social and economic value of the building. Different values of important factors for the same importance are assigned to the city planning area and rural area in order to reflect the fact that the same ground shaking may cause more severe damage in the population concentrated urban area. Table 4 shows the values for important factors.

Table 4 Importance factor

Importance	3	2	1
Urban area	1.0	1.2	1.5
Rural area	0.8	1.0	1.2

3) Dynamic Coefficient (C) and Natural period (T)

The natural period may be appropriately calculated based on formulations in ATC3-06 (Table 5). Dynamic coefficient represents the normalized response spectrum which can be calculated by the following equation.

$$C = \frac{1}{1.2\sqrt{T}} < 1.5$$
$$CS \leq 1.75 \tag{2}$$

where T and S denote natural period and soil profile coefficient respectively. The natural period (T) can be calculated by the equation in Table 5 or proper dynamic analysis. However the period obtained by dynamic analysis can not exceed the period calculated by the equation in Table 4 multiplied by 1.2.

Table 5 Natural priods

	Steel moment resisting frame	**RC moment resisting frame**	**All order Building**
Natural period	$0.085h^{3/4}$	$0.060h^{3/4}$	$0.09h/\sqrt{B}$

4) Soil Profile Coefficient (S)

To consider the effects of soil characteristics to earthquake amplification, the site coefficients are classified into three categories as shown in Table 6. Soft soil has larger value of soil factor than hard soil. To prevent the overestimation of design spectrum, an upper bound of CS is limited to 1.75.

Table 6 Soil factor

Soil type	Hard ⟵	⟶	Soft
	S1	S2	S3
S	1.0	1.2	1.5

5) Response Modification Factor (R)

This factor accounts for ductility, damping, redundancy, and overstrength of the structure. The factors in ATC3-06 have adopted in the Korean provision with some modifications.

4.2 Vertical Distribution of Base Shear and Story Shear

The vertical distribution of the base shear calculated from Eq. 1 is obtained by the following equation.

$$F_x = \frac{W_x h_x^k}{\sum_{i=1}^{n} W_i h_i^k} V \tag{3}$$

k=1.0 for T<1.0sec
k=1.5 for 1.0 < T <2.0 sec
k=2.0 for T>2.0sec

where F_x : seismic force at story x
W_i, W_x : weight at story i, x
h_i, h_x : the height from the base to story i, x
V : base Shear

The story shear at the x-th floor is calculated by the following equation.

$$V_x = \sum_{i=x}^{n} F_i \tag{4}$$

4.3 Torsional and Overturning Moment

The torsional responses in building arise from two sources: (1) eccentric torsional moment, and (2) additional torsional moment.
The eccentric torsional moment arising from eccentric distributions of mass and stiffness can be taken into account by determining the distance between the center of mass and stiffness. The

additional accidental torsional moment is also determined by assuming that the mass is displaced from the calculated center of mass in each direction a distance equal to five percent of the building dimension at that level perpendicular to the direction of the force under consideration.

The overturning moments to be resisted are determined using those lateral seismic forces (F_x) which act on levels above the level under consideration as following equation. The reduction factor is applied in order to take into account the fact that the lateral seismic forces under consideration may not be imposed at once.

$$M_x = \rho \sum_{i=x}^{n} F_i (h_i - h_x) \tag{5}$$

where, ρ is equal to 1.0 for top 10 stories, 0.8 below the top 20 stories and is interpolated for top 11-10 stories.

4.4 P- Δ Effects, Story Drift Limitation and Building Separation

Story drift is directly related to the non-structural damages of buildings. Excessive story drift causes the severe P- Δ effect which should be considered in seismic design. In the Korean provision story drift should be less than 1.5% of story height as follows.

$$\delta_x = R\delta_{xe} \leq 0.015 h_{sx}$$

where δ_x : story drift

δ_{xe} : story drift from elastic analysis

Adjacent structures should be separated away from each other by two times the sum of deformation of adjacent structures as follows.

$$d > 2(d_1 + d_2) \tag{6}$$

where d : distance for building separation

d_1, d_2 : building deformation in adjacent buildings

4.5 Dynamic Analysis

For the important structures such as tall buildings (with more than 20 floors), and structures with irregular configuration, a more sophisticated analysis methods such as medal analysis mode superposition method and direct integration method, are implicitly recommended to use in the provision.

5. Comparison between the Seismic Provisions of ISO and Korea

Tables 7.1-7.4 show the relationship and difference between Korean and ISO seismic provisions. Most prominent difference is that two provisions use different limit states. However the revision of the Korean provision is in progress proceeded which will adopts two limit states for operation and collapse prevention.

References

1. The Korean Ministry of Construction, Research on Korean Seismic Design Provision, 1987.12.
2. The Korean Ministry of Construction, Seismic Design Standards, Vol. 1, 2, 1997.
3. The Architectural Institute of Korea (AIK), Loading Standards for Structural Design and Commentary, 1998.
4. Ch.-K. Choi and H.-K. Chung, "Tall Building Design in the Low Seismicity Area: The Case of Korea", Proceeding of 10^{th} World Conference of Earthquake Engineering, Balkerma, Rotterdam, 1992.
5. ICBO, Uniform Building Code, International Conference on Building Officials, Whittier, California, 1988.
6. ATC, Tentative Provisions for the Development of Seismic Regulations for Buildings, Applied Technology Council Report ATC 3-06, 2nd Printing, Palo Alto, California, 1984.
7. SEAOC, Recommended Lateral Force Requirements and Commentary, Seismology Committee, Structural Engineering Association of California, Sacramento, California, 1959, 1988.
8. SEAOC, Performance Based Seismic Engineering of Buildings, Vision 2000 Committee, Structural Engineering Association of California, Sacramento, California, 1995.
9. BSSC, NEHRP Recommended Provisions for Seismic Regulations for New Buildings, Building Seismic Safety Council, National Hazard Reduction Program Report, Washington, D.C., 1994.

Table 7-1 Code Comparison

Items	ISO 3010	KOREAN 1988	KOREAN 1998
Basic Philosophy	1) to prevent human injury 2) to ensure continuity of vital service 3) to minimize damage to property	1) to resist the minor earthquake without structural or nonstructural damage 2) to resist to moderate earthquake without structural damage 3) to resist major earthquake without collapse or threatening on life safety	
Analysis Method	1) Earthquake Static analysis 2) Response Spectrum analysis 3) Time history analysis	1) Equivalent Lateral force procedure 2) Dynamic analysis (not well defined) · Response Spectrum analysis · Time history analysis	1) Operational level - Equivalent lateral procedure - Modal Analysis 2) Collapse Preventive level - Nonlinear dynamic response analysis - Pushover analysis
Limit State	1) Serviceability limit state 2) Ultimate limit state	Ultimate limit state	Operational level Collapse Preventive level
Key Parameter for Seismicity	· Effective Peak Ground Acceleration · Effect Peak Ground Velocity · Peak Ground Acceleration · Peak Ground Velocity · Selection depends on available data	Effective Peak Ground acceleration(EPA) 2 seismic zones – 0.12g – 0.08g	EPA, EPV 2 seismic zones
Soil Condition	Dynamic properties of supporting soil layers of the structure should be investigated	classify three soil conditions (S_1, S_2, S_3)	classify 5 soil conditions S_A, S_B, S_C, S_D, S_E

Table 7-2 Code Comparison

Items	ISO 3010	KOREAN 1988	KOREAN 1998
Equivalent Static Loading	• Design Lateral Force of *i*th level 1) for Sever EQGM $F_{i,u} = \alpha_u \beta \gamma_u \rho \delta \phi_i \sum_{j=1}^{n} G_j$ 2) for Moderate EQGM $F_{i,s} = \alpha_s \beta \gamma_s \rho \phi_i \sum_{j=1}^{n} G_j$ • Design Lateral Seismic Shear Force 1) for Sever EQGM $Q_{i,u} = \alpha_u \beta \gamma_u \rho \delta \psi_i \sum_{j=1}^{n} G_j$ 2) for Moderate EQGM $Q_{i,s} = \alpha_s \beta \gamma_s \rho \psi_i \sum_{j=1}^{n} G_j$	• Seismic Base shear $V = \frac{AICS}{R} W$ • Vertical distribution of Seismic force $F_x = \left(\frac{W_x h_x^k}{\sum_{i=1}^{n} W_i h_i^k}\right) V$	Not described in detail
Importance Load Factor	α_u, α_s Function of 1) target reliability for corresponding limit state 2) representative actions for corresponding limit state 3) coefficients of the variation of seismic action 4) uncertainty associated with action modeling and estimation of the structural performance	I Function of consequence of failure to the human life and society	classify as three categories according to the importance of building 1) Special I: the buildings containing hazardous material, broadcasting station 2) Grade I: the buildings for recovery after earthquake or with large occupancy 3) Grade II: ordinary buildings

Table 7-3 Code Comparison

Items	ISO 3010	KOREAN 1988	KOREAN 1998
	for α_u (4 categories) 1) structures containing large quantities of hazardous material 2) structures closely related to safety of the public 3) structures with large occupancy 4) ordinary structures	(3 categories) 1) structures for recovery of seismic hazard 2) structures with high occupancy [I Urban area II Rural area 3) ordinary structures	
	for α_s - α degree of incapability of the expected use, inconvenience and reparability	None	same as above
Seismic Hazard Zoning Coefficient	maximum value $\beta = 1$ (Exception for extremely high seismic zone)	classified as zone 1 and 2	classified as two zones - zone 1 and 2
Standard Base Shear Coefficient	Representative value of seismic ground motion intensity denoted by horizontal peak acceleration - γ_u : PGA for 500 yr. return period - γ_s : PGA for 20 yr. return period	Effective Peak Acceleration for 500 yr. return period - 0.12g for zone 2 - 0.08g for zone 1	Operational / Collapse proof Special: 200 yr. / 2400 yr. Grade I: 100 yr. / 1000 yr. Grade II: 50 yr. / 500 yr.
Structural Coefficient	δ : - account for ductility, acceptable deformation, hysteric characteristics, overstrength	R (response modification factor): - account for ductility, damping redundancy, overstrength	Not defined yet

Table 7-4 Code Comparison

<table>
<tr><th>Items</th><th>ISO 3010</th><th>KOREAN 1988</th><th>KOREAN 1998</th></tr>
<tr><td>Normalized Design Spectra</td><td>ρ :
- $\rho = 1$ for $T = 0$
- linear interpolation for $0 \le T \le T_c'$
- $\rho = \rho_0$ for $T_c' \le T \le T_c$
- $\rho = \rho_0 (\frac{T_c}{T})^{\eta}$ for $T > T_c$</td><td>C (Dynamic coefficient):
- $C = \frac{1}{1.2\sqrt{T}} \le 1.5$
- $C.S \le 1.75$</td><td>$C = C_a$ for $T = 0$
Linear interpolation for $0 < T < T_s$
$C = 2.5C_a$ for $T_0 < T < T_s$
$C = C_v / T$ for $T > T_s$</td></tr>
<tr><td>Seismic Force Distribution</td><td>ϕ_i : distribution of seismic force in elevation
- $\sum \phi_i = 1$</td><td>$\phi_i = \frac{W_x h_x^k}{(\sum W_i h_i^k)}$
$F_x = \phi_i V$</td><td rowspan="6">Not defined yet</td></tr>
<tr><td>Shear Distribution Coefficient</td><td>φ_i : distribution of seismic shear force in elevation
- $\varphi_i = 1$ at base</td><td>$V_x = \sum_{i=1}^{n} F_i$</td></tr>
<tr><td>Gravity Load</td><td>G_i : gravity load at ith level</td><td>W_x, W_i : gravity load at x, ith level</td></tr>
<tr><td>Seismic Action</td><td>Two horizontal components of ground motions are considered</td><td>Each component of ground motions is considered separately</td></tr>
<tr><td>Torsion</td><td>considered</td><td>considered</td></tr>
<tr><td>Dynamic Analysis Procedure</td><td>Response Spectrum analysis for
- linear or equivalent linear system
- time history analysis for linear or nonlinear system</td><td>Not well described</td></tr>
</table>

Analysis of a Steel Framed Structure Using Capacity Spectrum Method

Kim, Jinkoo

Assistant Professor, Dept. of Architectural Engineering,
Sungkyunkwan University, Suwon, Korea

1. Introduction

Performance-based seismic design provides insight into the actual performance of structures during earthquakes, and requires engineers to design a structure to meet performance objectives for multi-level seismic loads. In this regard the Korea Ministry of Construction and Transportation and The Earthquake Engineering Society of Korea established a new guideline[9] for performance-based seismic design in 1997. In the guideline the seismic coefficients for construction of the design spectrum shown in Fig. 1 were provided for double performance levels, functional and collapse prevention performance levels, and for earthquakes with various recurrence periods as shown in Table 1.

In this study a steel framed structure was designed in accordance with the conventional single-level seismic design code, and was analyzed using capacity spectrum method to evaluate the nonlinear performance of the structure subjected to the strong earthquake load corresponding to the collapse prevention performance objective. The maximum inter-story drift was compared with the performance criteria specified in the new guideline to check whether the building structure designed in accordance with the conventional design code satisfy the performance criteria recommended by the new performance-based seismic design guideline.

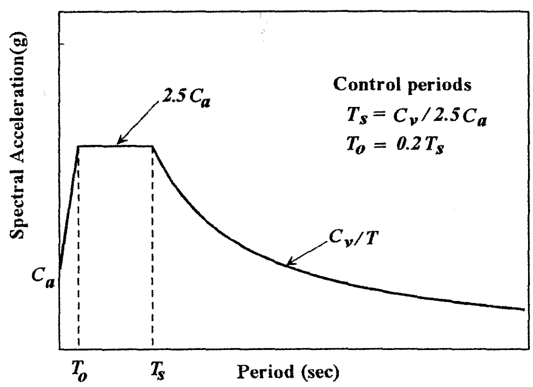


Fig. 1 Design spectra

Table 1(a). Seismic coefficient C_a (zone I)

Performance level	Functional			Collapse prevention		
Recurrence period (years)	50	100	200	500	1000	2400
Importance / Soil type	II	I	Special	II	I	Special
S_A	0.036	0.051	0.066	0.090	0.126	0.180
S_B	0.044	0.063	0.080	0.110	0.154	0.220
S_C	0.052	0.074	0.095	0.130	0.182	0.260
S_D	0.064	0.091	0.117	0.160	0.224	0.320
S_E	0.088	0.125	0.161	0.220	0.308	0.440

Table 1(b). Seismic coefficient C_v (zone I)

Performance level	Functional			Collapse prevention		
Recurrence period (years)	50	100	200	500	1000	2400
Importance / Soil type	II	I	Special	II	I	Special
S_A	0.036	0.051	0.066	0.090	0.126	0.180
S_B	0.044	0.063	0.080	0.110	0.154	0.220
S_C	0.072	0.103	0.131	0.180	0.252	0.360
S_D	0.092	0.131	0.168	0.230	0.322	0.460
S_E	0.148	0.211	0.270	0.370	0.518	0.740

Table 1(c). Seismic coefficient C_a (zone II)

Performance level	Functional			Collapse prevention		
Recurrence period (years)	50	100	200	500	1000	2400
Importance / Soil type	II	I	Special	II	I	Special
S_A	0.02	0.0285	0.0365	0.05	0.07	0.1
S_B	0.028	0.0399	0.0511	0.07	0.098	0.14
S_C	0.032	0.0456	0.0584	0.08	0.112	0.16
S_D	0.044	0.0627	0.0803	0.11	0.154	0.22
S_E	0.068	0.0969	0.1241	0.17	0.238	0.34

Table 1(d). Seismic coefficient C_v (zone II)

Performance level	Functional			Collapse prevention		
Recurrence period (years)	50	100	200	500	1000	2400
Importance / Soil type	II	I	Special	II	I	Special
S_A	0.02	0.0285	0.0365	0.05	0.07	0.1
S_B	0.028	0.0399	0.0511	0.07	0.098	0.14
S_C	0.044	0.0627	0.0803	0.11	0.154	0.22
S_D	0.064	0.0912	0.1168	0.16	0.224	0.32
S_E	0.092	0.1311	0.1679	0.23	0.322	0.46

2. Seismic performance evaluation by capacity spectrum method

For performance-based seismic design the capacity of a structure to resist seismic loads needs to be evaluated first. This can efficiently be carried out by the capacity spectrum method (CSM). It is basically an approximate method with the following significant simplifications : The first is the transformation of the response of a multi-degree-of-freedom (MDOF) structure to that of an equivalent single-degree-of-freedom (SDOF) system. The second is the calculation of the response of the inelastic SDOF system based on information extracted from elastic response spectrum. Nevertheless it is considered as a convenient tool for evaluation of seismic performance of a building structure subjected to a strong earthquake ground motion. The general procedure for the CSM is described in the following sections.

2.1 Formation of a pushover curve

The relationship between the base shear and the top story displacement, which is generally called pushover curve or capacity curve, is obtained by gradually increasing the lateral loads appropriately distributed over the stories. There are many alternatives for the distribution of the lateral load. In this study the capacity curve was obtained by applying three different types of lateral seismic loads:

1) Forces are applied in proportion to the product of story masses and the fundamental mode shape of the elastic model structure :[1]

$$F_i = \frac{m_i \Phi_{i1}}{\sum_{i=1}^{N} m_i \Phi_{i1}} V \tag{1}$$

where F_i is the seismic story force in the i th floor, m_i is the mass of the i th floor, Φ_{i1} is the i th component of the mode shape vector for the fundamental mode, V is the base shear, and N is the number of floors.

2) The effect of the higher modes are considered by means of combination of the story force of all modes using the SRSS method :[4]

$$F_i = \sqrt{\sum_{j=1}^{N}\left(\left[\frac{\sum_{i=1}^{N} m_i \Phi_{ij}}{\sum_{i=1}^{N} m_i \Phi_{ij}^2}\right]\Phi_{ij} Sa_j m_i\right)^2} \tag{2}$$

where Φ_{ij} is the i th component of the j th mode shape vector and S_{aj} is the spectral acceleration corresponding to the j th mode.

3) Equivalent mode shape considering modal participation factors is utilized :[5]

$$F_i = \frac{m_i \overline{\Phi_i}}{\sum_{i=1}^{N} m_i \overline{\Phi_i}} V \tag{3}$$

$$\Gamma_j = \frac{[\sum_{i=1}^{N} m_i \Phi_{ij}]}{[\sum_{i=1}^{N} m_i \Phi_{ij}]} \qquad \overline{\Phi_i} = \sqrt{\sum_{i=1}^{N} (\Phi_{ij}\Gamma_j)^2} \tag{4}$$

The first method can be properly applied to a structure in which the first mode dominates the dynamic motion. The second and third methods have advantage that higher mode effect can be included in the estimation of seismic lateral load.

2.2 Conversion to ADRS spectra

Application of the capacity spectrum technique requires that both the demand spectra and structural capacity curve be plotted in the spectral acceleration vs. spectral displacement domain, which is known as acceleration-displacement response spectra (ADRS). To convert a response spectrum from the standard S_a vs. T format to ADRS format, it is necessary to determine the value of S_d for each point on the curve, S_a and T. This can be done with the equation :

$$S_d = (T^2/4\pi^2) S_a g \tag{5}$$

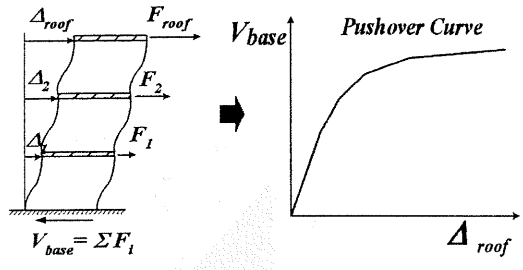
In order to develop the capacity spectrum from the capacity curve, it is necessary to do a point by point conversion to the first mode spectral coordinates. Any point V, Δ_R on the capacity curve is converted to the corresponding point S_a and S_d on the capacity spectrum using the equations :

$$S_a = \frac{V}{M_1^*} \qquad S_d = \frac{\Delta_R}{\Gamma_1 \Phi_{R1}} \tag{6}$$

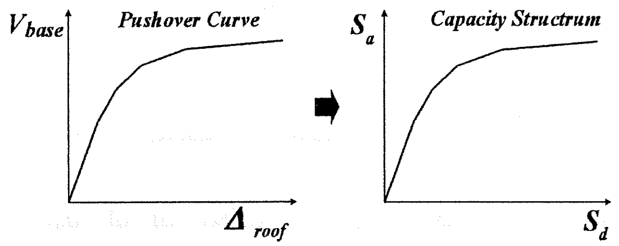
where Γ_1 is the modal participation factor for the first natural mode of the structure and ϕ_{R1} is the roof level amplitude of the first mode. The mathematical expressions for the modal mass coefficient M_1^* is as follows :

$$M_1^* = \frac{\left(\sum_{j=1}^{N} m_j \phi_{j1}\right)^2}{\sum_{j=1}^{N} m_j \phi_{j1}^2} \tag{7}$$

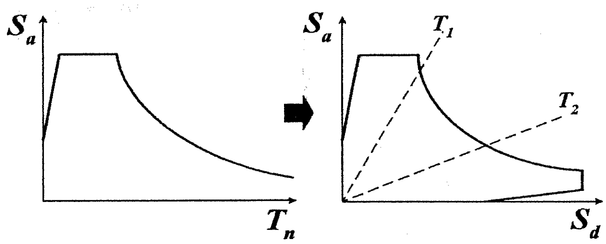
The general process of obtaining structural responses using CSM is described in Fig. 2.



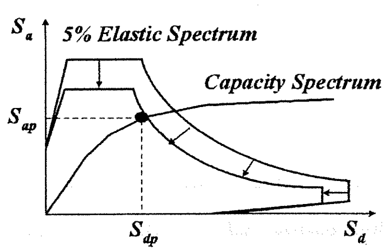
(a) Formation of capacity curve



(b) Transformation of capacity curve to capacity spectrum



(c) Transformation of response spectrum to ADRS format



(d) Evaluation of performance point

Fig. 2 Process of seismic capacity evaluation using capacity spectrum method

2.3 Estimation of equivalent viscous damping ratio

A bilinear representation of the capacity spectrum, as described in Fig. 3, is needed to estimate the effective damping and appropriate reduction of spectral demand. ATC-40 recommends that the area under the original capacity curve and the equivalent bilinear curve be equal so that the energy associated with each curve is the same. When the bilinear system undergoes inelastic action at displacement S_{dp} with corresponding acceleration equal to S_{ap}, the effective period, T_{eff}, is determined from the secant (or effective) stiffness at maximum displacement

$$T_{eff} = 2\pi\sqrt{\frac{S_{dp}}{S_{ap}}} \tag{8}$$

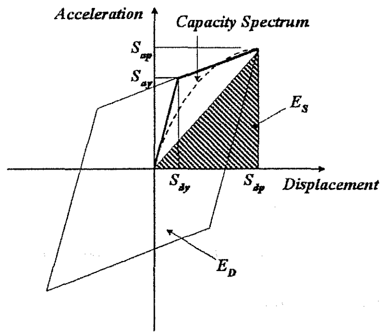


Fig. 3 Estimation of equivalent damping ratio

The equivalent visçous damping ratio for the yielding structure is determined from the energy dissipated by the hysteretic behavior, E_D, which is the area enclosed by the hysteresis loop at maximum displacement, and the stored potential energy corresponding to the area of the shaded triangle, E_S :[(1)(2)(6)]

$$\beta_{eq} = \frac{1}{4\pi}\frac{E_D}{E_S} = \frac{2(S_{ay}S_{dp} - S_{dy}S_{ap})}{\pi S_{ap}S_{dp}} \tag{9}$$

If the inherent viscous damping of the structure is assumed to be β, then the effective damping ratio of the system can be obtained as[(1)(2)(6)]

$$\beta_{eff} = \beta + \kappa\,\beta_{eq} \tag{10}$$

where κ is called the efficiency factor or damping modification factor which is equal to the actual area enclosed by the hysteresis loop divided by the loop area of the corresponding perfect bilinear hysteretic system. The factor is selected on the basis of experience with particular structural systems. In this study the factor is taken to be 1.0 because perfect bilinear system is assumed in the analysis.

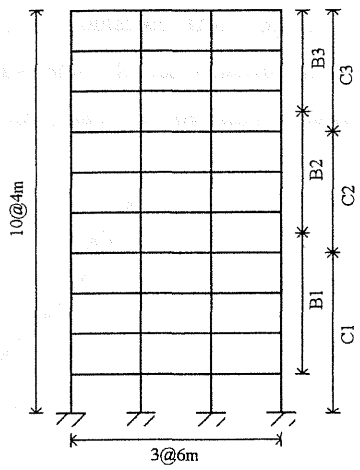


Fig. 4 10-story model structure

3. Application to a framed structure

3.1 Model structure

The model structure for analysis, shown in Fig. 4, is a 10-story steel framed structure designed to resist gravity and seismic loads. For gravity load uniform dead load of 540 kgf/cm^2 and live load of 250 kgf/cm^2 were applied throughout the stories. The yield stress of the structural steel used is 2.4 $tonf/cm^2$ and 3.3 $tonf/cm^2$ for beams and columns, respectively. The structural analysis and design was carried out using the program MIDAS GEN.[8] The size of each selected structural member is tabulated in Table 2, and the dynamic modal characteristics are shown in Table 3.

Table 2 Sectional properties of the model structure

Columns (Beams)	Columns		Beams
	Interior columns	Exterior columns	
C1(B1)	H 400×400×13×21	H 344×348×10×16	H 400×200×8×13
C2(B2)	H 350×350×12×19	H 300×300×10×15	H 396×199×7×11
C3(B3)	H 344×354×16×16	H 298×299× 9×14	H 350×175×7×11

Table 3 Dynamic characteristics of the model structure

Mode	1	2	3	4	5
Period (sec)	.1.41	0.49	0.28	0.19	0.14
Period ratio (T_1/T_m)	1.00	2.87	4.98	7.49	10.14
Modal participation factor	1.34	0.51	0.30	0.22	-0.16
Effective mass (%)	77.54	11.68	4.28	2.23	1.53

Fig. 5 shows the mode shapes obtained for the three methods for lateral load distribution mentioned above, where it can be found that the mode shape obtained from SRSS and the equivalent mode shape is almost identical. However Fig. 6 indicates that the story forces resulting from the SRSS method are higher than those from other methods in the lower stories but smaller in the higher stories.

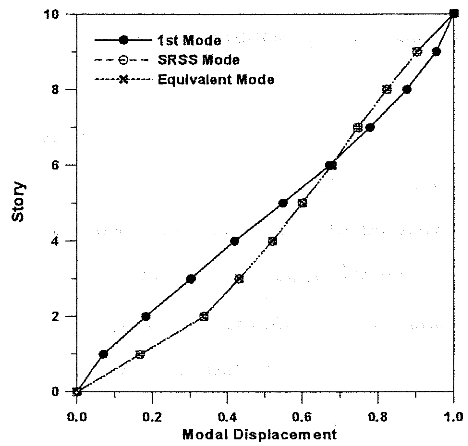


Fig. 5 Mode shapes

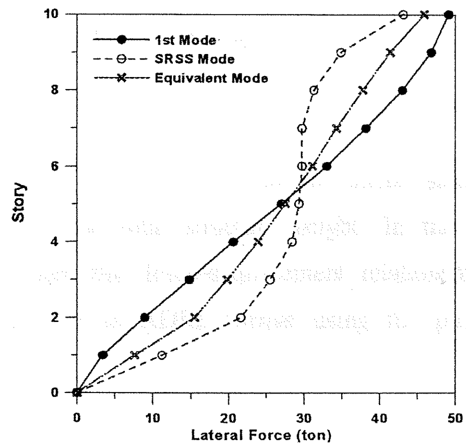


Fig. 6 Seismic story forces

3.2 Design spectrum and earthquake time history record

Site-specific elastic design response spectrum was constructed as shown in Fig. 7 in accordance with Fig. 1 with the values C_a and C_v as suggested in the *Seismic Design Guidelines II*[9] for earthquakes with recurrence period of 2,400 years (collapse prevention performance level for special structure) and soil type of S_E. Based on the design spectrum time history record shown in Fig. 8 was generated using the program SIMQKE[10] for comparison of the static procedure with the nonlinear dynamic time history analysis. The response spectrum constructed from the time history record generated from the design spectrum are plotted in Fig. 7 along with the design spectrum.

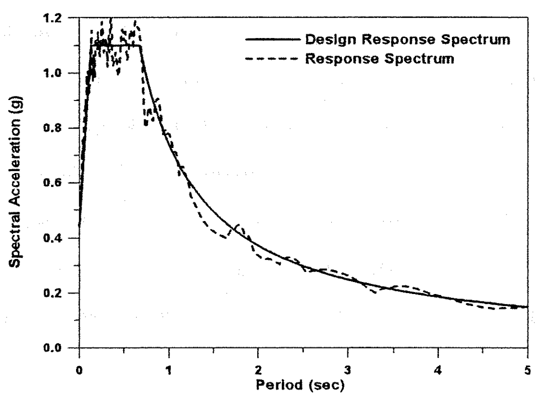


Fig. 7 Response spectrum of artificial ground motion

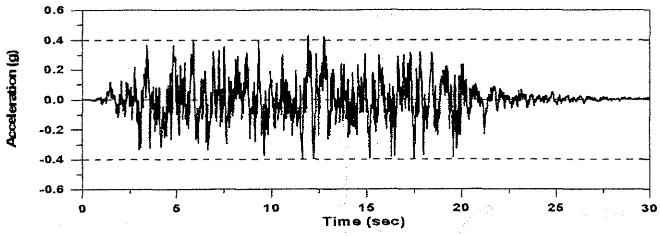


Fig. 8 Artificial ground excitation generated from the design spectrum

3.3 Capacity spectrum

The base shear - top story displacement relationship shown in Fig. 9 was obtained with the lateral static load gradually increased until the top story displacement reached 4% of the total structure height. In this study, pushover analysis was carried out using DRAIN-2D+[(11)] program. Then this force-displacement relationship was transformed into the capacity spectrum of an equivalent SDOF system in ADRS format using the procedure mentioned previously. This is plotted in Fig. 10.

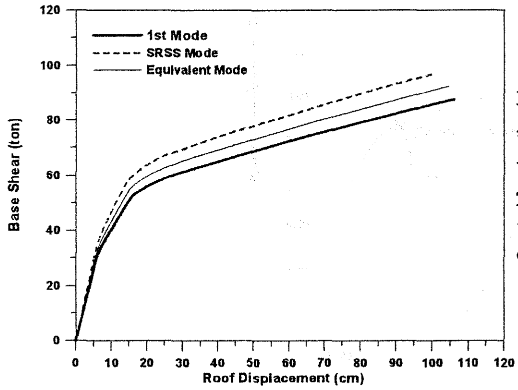


Fig. 9 Pushover curves

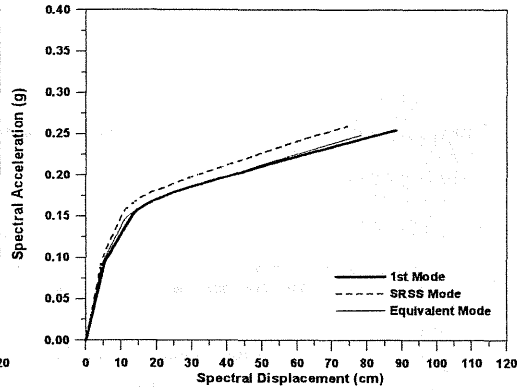


Fig. 10 Capacity spectra

3.4 Estimation of performance point

To estimate the performance points the capacity spectrum and the demand spectra with various damping ratios are plotted simultaneously in ADRS format. Fig. 11 and Table 4 present the process of estimation of the performance points for each lateral story force distribution method.

Table 5 shows the displacement and acceleration obtained for the equivalent SDOF system and the displacement and base shear for the original MDOF structure induced from the results of the SDOF system using Eq. 6. The results from the three different lateral story forces were also compared in the table, where it can be seen that the top story displacement obtained from the lateral load by SRSS combination of the mode shapes is the lowest while the base shear is the largest. The opposite is true for the lateral loads constructed from the fundamental mode shape of the structure. The results from the equivalent mode are placed in the middle.

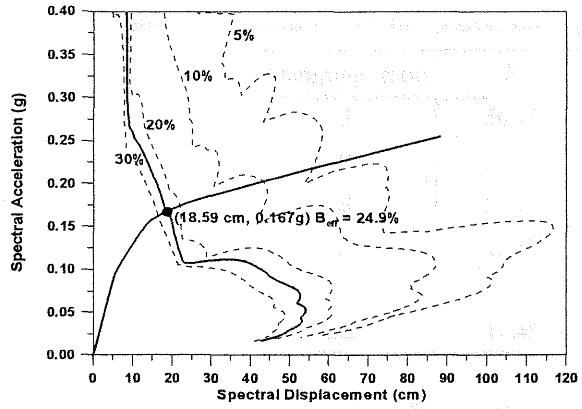


Fig. 11(a) Estimation of performance point (first mode)

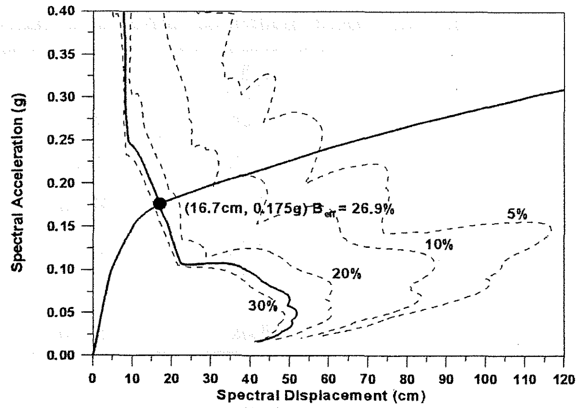


Fig. 11(b) Estimation of performance point (SRSS mode combination)

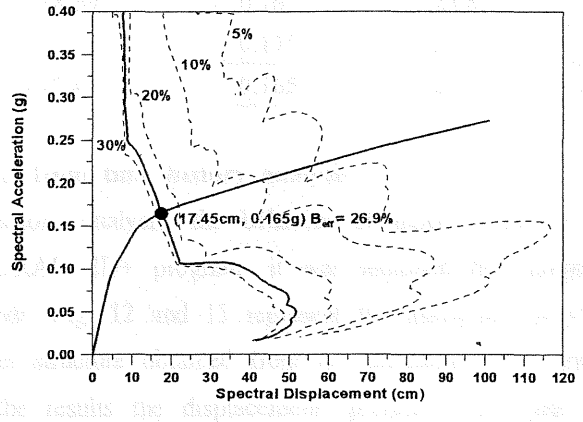


Fig. 11(c) Estimation of performance point (equivalent mode)

Table 4(a) Evaluation of the performance points considering the 1st mode (unit : cm)

Damping ratio	S_{di}	S_{ai}	β_{eff}
5	55.80	0.217	37.4
10	43.25	0.202	37.0
20	21.90	0.174	28.6
30	16.30	0.162	21.3
:	:	:	:
24.9	**18.59**	**0.167**	**24.9**

Table 4(b) Evaluation of the performance points considering SRSS mode combination (unit : cm)

Damping ratio	S_{di}	S_{ai}	β_{eff}
5	57.45	0.238	37.3
10	33.80	0.203	37.2
20	21.05	0.183	31.8
30	15.62	0.173	25.0
:	:	:	:
26.9	**16.72**	**0.175**	**26.9**

Table 4(c) Evaluation of the performance points considering equivalent mode (unit : cm)

Damping ratio	S_{di}	S_{ai}	β_{eff}
5	56.15	0.221	36.8
10	43.25	0.202	37.6
20	21.91	0.174	31.3
30	16.32	0.162	25.6
:	:	:	:
26.9	**17.45**	**0.165**	**26.9**

Table 5 Structural responses for each method

Method	Performance points		MDOF system		Effective damping (β_{eff})
	Displacement (cm)	Acceleration (g)	Displacement (cm)	Base shear (tonf)	
1st mode	18.59	0.167	24.85	56.70	24.9
SRSS combination	16.72	0.175	23.19	66.07	26.9
Equivalent mode	17.45	0.165	24.23	62.29	26.9

3.5 Comparison with the results from time history analysis

To perform nonlinear time history analysis, the behavior of model structure was simulated by using the Beam-Column element of the DRAIN-2D+ program. It was assumed that hinge rotation takes place at the point-plastic hinge at element ends. Fig. 12 and 13 represent the maximum inter-story drift and the maximum story displacements of the model structure obtained from the nonlinear static analysis and the nonlinear time history analysis. According to the results the displacements predicted from the SRSS mode combination for lateral load forms upper bound in lower stories, and forms lower bound in upper stories compared with those predicted by the other two forms of static lateral loads. This can be expected considering the shape of the static lateral load. It also can be noticed that the maximum story displacements predicted by CSM with three types of lateral load distribution are lower than those computed by time history analysis. However for inter-story displacements, although the results from the static nonlinear analysis are smaller than those from dynamic analysis in the lower stories, the opposite is true in the upper stories. The inter-story drifts shown in Fig. 12 indicate that the structure satisfies the *collapse prevention* limit state prescribed in the reference[9], which is the maximum inter-story drift of 2.5% of the story height.

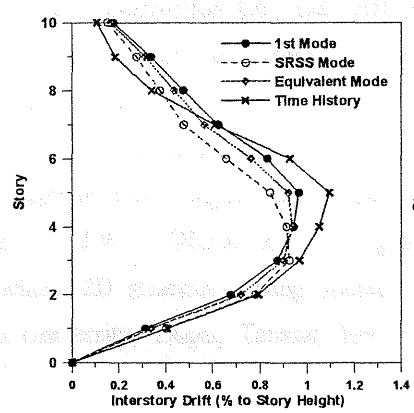


Fig. 12 Maximum interstory drift

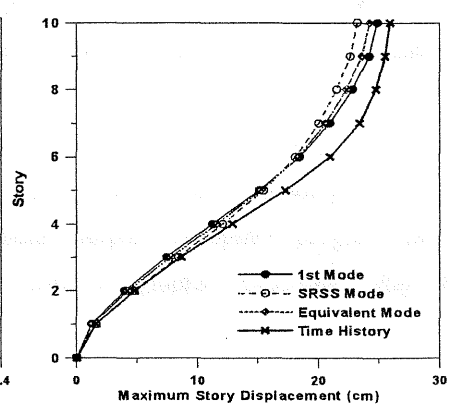


Fig. 13 Maximum story displacements

4. Conclusion

In this study seismic performance of a multi-story steel frame designed in accordence with current seismic code was evaluated for the earthquake load corresponding to the collapse prevention performance objective. The capacity spectrum method with three different lateral load patterns was applied to evaluate the nonlinear performance of the model structure, and the results were compared with the time history analysis result. It was found that the model structure, although designed for the conventional single level earthquake load, retains enough margin for safety for the earthquake load of 2,400 year return period (earthquake level for collapse prevention performance objective). It was also found that for the given model structure the results from the capacity spectrum method generally coinside well with those obtained from the time histroy analysis.

References

1. Applied Technology Council, "Seismic evaluation and retrofit of concrete buildings," *ATC-40*, Applied Technology Council, Redwood City, California, 1996.
2. Federal Emergency Management Agency, "NEHRP Guidelines for the seismic rehabilitation of buildings," *FEMA-273*, Federal Emergency Management Agency, Washington, D.C.,1997.
3. Kim, J. K., Choi, H. H., "Calculation of optimum damping ratio of viscous dampers using capacity spectrum method," *Journal of the Earthquake Engineering Society of Korea,* To appear in the February issue, 2001.
4. Freeman, S. A., Sasaki, K. and Paret, T., "Multi-mode pushover procedure(MMP)-A method to identify the effects of higher modes in a pushover analysis," *Proceedings of the 6th National Conference on Earthquake Engineering,* EERI, Seattle, WA, 1998.
5. Valless, R. E., Reinhorn, A. M., Kunnath, S. K., Li, C. and Madan, A., "IDARC 2D version 4.0 : A computer program for the inelastic damage analysis of buildings," , *Technical Report* NCEER-96-0010, National Center for Earthquake Engineering Research, State university of New York at Buffalo, 1996.
6. Tsopelas, P., Constantinou, M. C., Kircher, C. A. and Whittaker, A. S., "Evaluation of simplified method of analysis for yielding structures," *Technical Report* NCEER-97-0012, National Center for Earthquake Engineering Research, State university of New York at Buffalo, 1997.
7. Chopra, A. K., *Dynamics of structures*, Prentice Hall, 1995.
8. POSCO Engineering & Construction Co., Ltd., *MIDAS-GEN,* Vol. I ~Vol. IV, 1999.
9. Earthquake Engineering Society of Korea, "Seismic Design Guidelines II," Ministry of Construction & Transportation, Seoul, Korea, 1997.
10. Vanmarcke, E. H. and Gasparini, D. A., *A program for artificial motion generation, User's manual and documentation*, Dept. of Civil Engineering, Massachusetts Institute of Technology, 1976.
11. Tsai, K.C. and Li, J.W., " DRAIN 2D +, A general purpose computer program for static and dynamic analyses of inelastic 2D structures supplemented with a graphic processor," *Report* No. CEER/R86-07, National Taiwan University, Taipei, Taiwan, 1997.