SCALING OF EARTHQUAKE GROUND MOTION RECORDS FOR SEISMIC ANALYSIS AND DESIGN OF BRIDGES

Nevine A. Markous[1] Sameh S. F. Mehanny[2] and Mourad M. Bakhoum[3]

ABSTRACT

There are many options to define the seismic input for structural analysis. Natural recordings are considered as the most attractive option. But when insufficient - previously recorded - earthquake accelerograms are available or when they do not belong to the same seismic environment for a particular case study, it is common in practice to select other remotely recorded ground motions – or artificially generated ones - and 'scale' them considering number of criteria to get an unbiased estimation of seismic demand.

The study presented in this paper aims to investigate the efficiency of a proposed scaling scheme for ground records, which is based on matching the code design response spectrum at the fundamental period of the bridge, (Sd (T1)), or minimizing the error between this spectrum and the record-specific response spectrum at different control periods associated with the main vibration modes governing the response of the bridge. Scaling records to match Sd (T1), successfully predicts the response of the bridge (with a relatively limited variation in results from different records) even when using general ground records and not site-specific actual records. The study is accomplished by applying the proposed scaling scheme to some selected continuous bridge systems commonly encountered in Egypt. These bridges feature four equal spans with lengths of 25, 45 and 65 m investigated to represent short, medium and long span bridges, respectively. A set of twenty three international ground motion records were chosen from worldwide available strong motion database to assess the efficiency of the proposed scaling technique.

Keywords: seismic analysis; seismic demand; record intensity measure; record selection; earthquake record scaling.

INTRODUCTION

There are two main questions to define the seismic input for structural analysis which successfully predicts structure response: they are *how to select the records and how to scale them.* International and national seismic codes prescribe general guidelines but do not provide specifics for selecting and scaling earthquake records required for dynamic analysis/ design purposes. It is still the designer's responsibility to find a 'reasonable' way for selecting a set of 'appropriate' earthquake records, a task that may seem apparently easy. Such task is nonetheless difficult since any discrepancy in the computed structural response must be kept reasonably low. This is hence a complex task that cannot be accomplished on a 'trial and error' basis without an understanding of the basic concepts behind selection and scaling of earthquake records for use in dynamic analysis for design purposes. In other words, the current codes framework for ground motion record selection is considered rather simplified compared to the potential impact of the selection process on the dynamic analysis results, so several studies have been published proposing methods for selecting the set of ground motions to be used for analysis.

Ground motion records selection criteria

The selection method may have an effect on the bias resulting from scaling the ground motions. Evangelos I et al. [1] summarized some of these selection methods as follows: record selection based on earthquake magnitude (M) and distance (R); additional record selection criteria (soil

[1]Graduate student, Department of Structural Engineering, Faculty of Engineering, Cairo University, Giza, Egypt , E-mail: venna_marc@yahoo.com

[2]Professor, Department of Structural Engineering, Faculty of Engineering, Cairo University, Giza, Egypt ,E-mail: sameh.mehanny@stanfordalumni.org

[3]Professor, Department of Structural Engineering, Faculty of Engineering, Cairo University, Giza, Egypt, E-mail: bakhoumm@gmail.com

profile, strong motion duration, seismotectonic environment and other geophysical / seismological parameters, acceleration to velocity ratio a/v)); record selection based on spectral matching; and record selection based on ground motion intensity measures, (IM).

Seismic code provisions for selection of real records

National seismic codes prescribe general guidelines but do not provide specifics for selecting the type of earthquake records required for nonlinear dynamic analysis purposes. This is for three main reasons:
(a) time-history analyses are rather recent in engineering practice and expertise developed to date is not considered sufficient; (b) research on this topic is still under development and regulations to include the recent innovations require at least a few years time; and, (c) full agreement has not yet been reached regarding the establishment of commonly accepted selection criteria for earthquake records.

Most contemporary seismic codes, such as Eurocode 8 (EC8-1 and EC8-2), ASCE7- 05,AASHTO, ASCE 4-98 and FEMA regulations FEMA 368 [2-6], as well as various national norms New Zealand Standards (NZS), Italian Code (OPCM) and Greek Seismic Code (EAK) [7-9] describe relatively similar procedures for the simulation of seismic actions to be used as dynamic loading in structures. However some discrepancies between different codes take place as is clear from the following points:

Most frequently, seismic motions can be represented by real, artificial or even simulated records. Some differences between the codes on strong motion representation remain. For example, the New Zealand Standards allow the use of real records only, while Eurocode 8 leaves this choice to the structural engineer.

Some important seismological parameters, such as earthquake magnitude, distance, the seismotectonic environment and the local soil conditions should be reflected in the local seismic scenarios. Some code-based selection strategies require inclusion of additional parameters. For instance, Greek seismic code (EAK) and (ASCE 4-98) specify that duration of the selected accelerograms has to be representative of expected ground motion at the site of interest.

Spectral matching between the design spectrum and the response spectrum of a selected record is required in most codes. However, the period range for spectral matching varies among different codes provisions. For example:
SEAOC [10]: (T1-1s) to (T1+2s)
Eurocode 8: (0.2T1) to (2T1)
ASCE7- 05: (0.2T1) to (1.5T1)
where: T1 is the fundamental period of the structure.

The average elastic spectrum of selected records is though not to underestimate the code spectrum, with a 10% tolerance, in a broad range of periods.

Most of codes do not distinguish between record selection for unidirectional and bi-directional dynamic analysis, except AASHTO which report that where three-component sets of time histories are developed by simple scaling rather than spectrum matching, it is difficult to achieve a comparable aggregate match to the design spectra for each component of motion when using a single scaling factor for each time history set. It is desirable, however, to use a single scaling factor to preserve the relationship between the components. It is however preferred to use a scaling factor to meet the aggregate match for the most critical component with the match somewhat deficient for other components.

The minimum number of records required for structural analysis is three in all cases, the exception being (ASCE 4-98) which specifies that at least one record should be used, unless the structure is sensitive to long-period motion.

For a set of at least seven ground-motions, the structural engineer is allowed to compute the mean structural response. Otherwise, only a maximum response value is computed if three to six recordings are used.

Some codes identify some records as suitable candidates for time history analysis like (ATC-40) [11] that identifies records to be used for time history analysis of buildings at soil site with peak ground acceleration of 0.2g or greater: records at soil site greater than 10 km from sources, and records at soil site near source.

Scaling Method

Several studies have been published proposing methods for identifying earthquake scenarios (including scaling techniques) that can be derived through seismic hazard analysis (SHA), which is linked to the estimation of some key measures of strong ground motion expected to occur during a pre-defined time interval at a given site.

Performance-based design and evaluation methodologies prefer intensity-based methods to scale ground motions over spectral matching techniques that modify the frequency content or phasing of the record to match its response spectrum to the target spectrum. In contrast, intensity-based scaling methods preserve the original non-stationary content and only modify its amplitude. The primary objective of intensity-based scaling methods is to provide scale factors for a small number of ground motion records so that Response History Analysis (RHA) of the structure for these scaled records is accurate, i.e., it provides an accurate estimate of the median or mean value of the Engineering Demand Parameters (EDPs) conditioned on specific intensity measure, and is efficient, i.e., it minimizes the record-to record variations in the EDP. Scaling ground motions to match a target value of Peak Ground Acceleration (PGA) is the earliest approach to the problem, which produces inaccurate estimates with large dispersion in EDP values as per Shome and Cornell [12]. Other scalar Intensity Measures (IMs) such as: effective peak acceleration, Arias intensity and effective peak velocity have also been found to be inaccurate and inefficient as highlighted in Kurama and Farrow [13]. None of the preceding IMs consider any property of the structure to be analyzed.

Including a vibration property of the structure led to improved methods to scale ground motions. For instance, scaling records to a target value of the elastic spectral acceleration, Sa (T1) derived from the code-based design spectrum or from a Probabilistic Seismic Hazard Analysis (PSHA)-based uniform hazard spectrum at the fundamental vibration period of the structure, T1, provides improved results for structures whose response is dominated by their first-mode (Shome et al. [12]).

However, this scaling method becomes less accurate and less efficient for structures responding significantly in their higher vibration modes or far into the inelastic range as pinpointed by Kurama and Farrow [13] and Mehanny [14]. To consider higher mode response, a scalar IM that combines the spectral accelerations Sa (T1) and Sa (T2) at the first two periods in a vector format of the IM has been developed. Although this vector IM improves accuracy, it remains inefficient for near-fault records with a dominant velocity pulse; refer to Baker and Cornell [15].

Among other known scaling techniques, ASCE offers a simple form. For two dimensional analyses of regular structures, ground motions are scaled such that the average value of the 5% - damped elastic response spectra for a set of scaled motions is not less than the design response spectrum over the period range from 0.2T1 to 1.5T1. For structures having plan irregularities or structures without independent orthogonal lateral load resisting systems where three-dimensional analyses need to be performed, ground motions should consist of appropriate horizontal components.

All the preceding scaling methods utilize IMs based on elastic response of the structure, but do not explicitly consider its inelastic response. They lead to scale factors that depend only on the structure period(s), independent of the structural strength. However, such simple and straightforward techniques are deemed suitable and satisfactory for seismic design purposes for bridges investigated in the present paper.

Methodology

Selected bridges and characteristics of the structural models

The selected bridges for the present study and all relevant information concerning their geometric features, critical sections where analysis results are reported, and a summary of considered analysis types, parameters and scaling schemes presented throughout the current investigation are shown in Figs.1, 2a and 2b. All bridges investigated are 4-span continuous bridges. All spans are equal and span lengths of 25,45and 65 m are tried to cover different ranges of short, medium and long span bridges, respectively. Columns are assumed 8 m-high for all studied cases and a concrete box girder deck is adopted.

The criterion for bridge selection is to have a representative sample of commonly used systems in Egypt. Selected bridges have different cross section depth and supporting systems. Selected bridge systems (identified as System (1) and (6)) and structural models are as presented in Bakhoum [16].

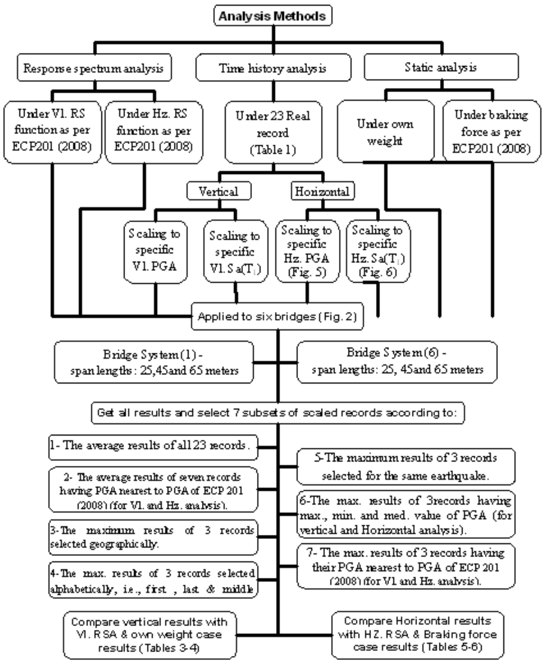


Fig. 1- Flow chart of the proposed investigation

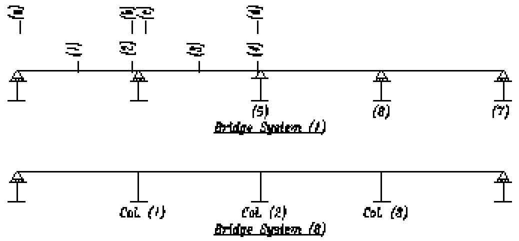


Fig. 2a- The bridge structural systems and critical sections considered in the present study

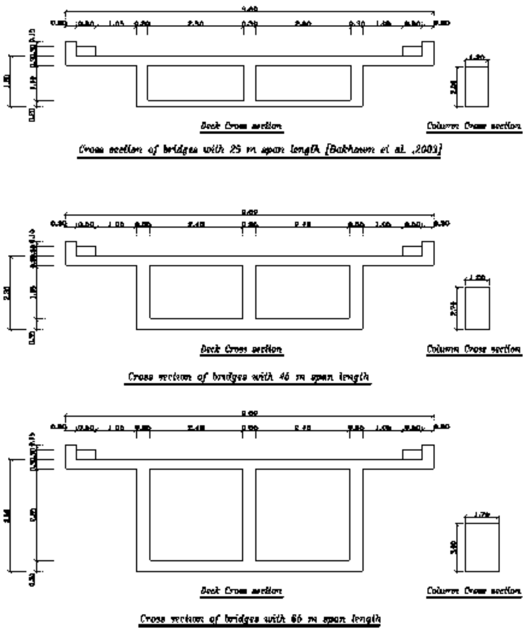


Fig. 2b- Bridges Cross Sections (all shown dimensions are in meters)

It should be also noted that a simple design under conventional code dead and live loads has been performed in order to estimate the overall dimensions of the pre-stressed concrete box girder deck of each of the bridges for different span lengths.

Seismic Design loads

Bridge systems are analyzed using the multi-mode response spectrum and time history analysis methods in presence of the own weight of the bridge. Loads considered are as follows:

140KN/m: for 25 m span length bridges.
170KN/m: for 45 m span length bridges.
215KN/m: for 65 m span length bridges.

For the dynamic structural analysis, the bridge deck is divided into adequate numbers of three-dimensional frame elements.

Section properties

Determination of the moment of inertia and torsional stiffness of the superstructure is based on un-cracked cross sectional properties due to the fact that the deck has a pre-stressed concrete section. On the other hand, for the current study the gross inertia has been similarly considered for columns to avoid assumption of cracking levels corresponding to different anticipated levels of axial load and flexural demands associated with different levels of seismic demands considered herein. Checking the effect of various levels of cracking, and hence different values of cracked inertia to be considered for columns - including code's recommendation - is however the subject of an ongoing research by the authors.

Boundary Conditions

Two bridge structural systems are considered with different types of connections between the bridge deck and the columns as shown in Fig. 2a. However, all column bases are modeled as fixed support thus ignoring soil-structure interaction. In case where pot-bearings are used, column top is released for shear and moment in major and minor directions, and for shear wherever applicable. Axial, i.e., vertical direction is always restrained.

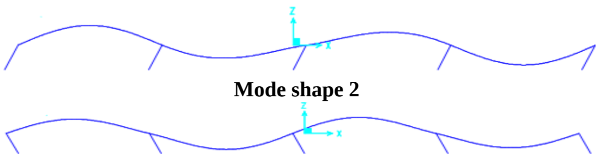
Dynamic characteristics

The commercial structural analysis software, SAP2000-Version 11, is used to perform modal analyses, considering a large number of mode shapes to fulfill code requirement that at least 90 percent of the structure mass (Participating Mass

Ratios ≥ 0.9) is included in the calculation of response using modal superposition for each principal direction. The values of maximum modal mass participation factors for all bridges, their corresponding period and mode number for longitudinal and vertical directions are presented in Table 1.

For longitudinal direction of all selected bridges - except for bridge system (6) bridge with span-length 45 meters - almost the total considered mass is mobilized through the first mode; i.e., these bridges act as a single degree of freedom system in this direction.

To the contrary, for System (6) bridge with span-length 45 meters, the span is long so that the girder is fairly flexible and its flexure deformation comes into play; 97% of total mass is mobilized by 2 modes (second & third modes) in the longitudinal direction(refer to Fig. 3 and Table 1). The structure thus acts as two-degrees of freedom system.



Mode shape 3

Fig. 3- Mode shape 2&3 of Bridge System (6), 45 meters span-length

Table 1 Selected ground motion records

Eq.	Record Sequence Number as PEER website	Earthquake Name	Year	Earthquake Magnitude	Vertical-PGA (m/s^2)	Horizontal-PGA (m/s^2)
1	825	Cape Mendocino - United States	1992	7.01	7.3929	14.6886
2	558	Chalfant Valley-02 -United States	1986	6.19	3.1484	3.9212
3	1181	Chi-Chi, Taiwan	1999	7.62	0.4004	0.9722
4	415	Coalinga-05 - United States	1983	5.77	3.8616	10.6211
5	-	Elcentro	1940	7.00	2.0601	2.0993
6	821	Erzican, Turkey	1992	6.69	2.4305	4.8610
7	125	Friuli, Italy-01	1976	6.50	2.6252	3.4465
8	132	Friuli, Italy-02	1976	5.91	0.9342	2.5497
9	1144	Gulf of Aqaba	1995	7.20	1.0701	0.9483
10	1994	Gulf of California	2001	5.70	0.0388	0.1693
11	181	Imperial Valley-06 -United States	1979	6.53	16.2359	4.0268
12	1106	Kobe, Japan	1995	6.90	3.3624	8.0572
13	879	Landers - United States	1992	7.28	8.0292	7.7416
14	796	Loma Prieta -United States	1989	6.93	0.5708	0.9757
15	451	Morgan Hill -United States	1984	6.19	3.8045	6.9735
16	529	N. Palm Springs - United States	1986	6.06	4.2646	5.8278
17	495	Nahanni, Canada	1985	6.76	20.4684	9.5925
18	1084	Northridge 1 Sylmar - Converter	1994	6.69	5.7510	12.6073
19	1051	Northridge 2 - Pacomia	1994	6.69	12.0579	6.0083
20	1087	Northridge 3 - Tarzana	1994	6.69	10.2820	9.7150
21	77	San Fernando- United States	1971	6.61	6.8527	12.0262
22	592	Whittier Narrows -01 -United States	1987	5.99	2.2508	2.9391
23	2053	Yorba Linda - United States	2002	4.27	0.1083	0.5042

On the other hand, System (6) bridge with the span-length of 65 meters thus with a longer deck has an increased deck cross section inertia. The girder is thus still rigid relative to columns and 87 % of the total mass is mobilized also by a single mode (mode 7) in the longitudinal direction. The deformation in this mode is mainly column shear deformation with a very small contribution from girder flexure deformation. Therefore, the structure still approximately acts as a single degree of freedom system. It is to be noted that all bridge Systems (1) and (6) considered herein with span-lengths 25,45and 65 meters have nearly the same model behavior in vertical direction. Value of maximum modal mass participation factor in this direction is ranging from 0.586 to 0.649 and about 90% of total mass considered is mobilized in the vertical direction by typically more than three modes. The number of modes contributing to mobilize 90% of the bridge mass in each direction for these six bridges is shown in Table 1.

Recorded and reported internal forces and displacements

The locations where the internal forces and displacements are considered in this study are as follows:

For vertical seismic analysis purposes:

For Systems (1) and (6), four critical sections for bending moments and shear forces in deck (at columns and mid-spans), three sections for axial force in columns, and two sections at mid spans for vertical displacement are considered as shown in Fig. 2a.

For horizontal seismic analysis purposes:

For System (1) only a single section is considered for monitoring the bending moment and horizontal reaction in the middle column. Further-more, the deck displacement in the longitudinal direction of the bridge is also reported. For System (6) three critical sections are selected (at the bottom of monolithic columns) to monitor the bending moment under horizontal seismic demand. Horizontal (longitudinal) displacement is also monitored for the deck at the top of the columns.

Analysis methods considered in the present research

Dynamic and static analyses are conducted for the selected bridges. Dynamic analysis is carried out under vertical and horizontal acceleration time history of scaled records (as explained below), as well as under vertical and horizontal elastic acceleration response spectra as per ECP201 [17] requirements, which are in line with European standard EC8-2 [2] requirements. Following parameters are considered: soil type (B), design ground acceleration (ag)= 0.15 g m/s^2, importance factor = 1.3, damping correction factor (η) =1.0, Response Spectrum type (2) - which is in line with type (1) of EC8-2-for horizontal direction and type (1) for vertical direction.

Static analysis is also performed herein under (1) the effect of own weight to be compared to results under vertical acceleration; and (2) the effect of braking force as per ECP201 [17] to be compared to results under horizontal acceleration.

Ground motion records

For Time History Analysis (THA) in the present study, denoted as method (d) in ECP201 [17], a set of ground records compatible with the site seismicity shall be chosen. Due to lack of measured records in Egypt, a set of twenty three international ground motion records named Set#2 in Markous [18] is therefore however chosen from a public strong motion database websites. This set of records is selected from the website of The Pacific Earthquake Engineering Research Strong Motion Database (PEER) [19].

Main characteristics of the selected records are given in Table 2. Among data given are: the moment magnitude M and the peak ground acceleration, PGA, in both the horizontal and vertical direction. Then, the hazard is identified in terms of the code design response spectrum at the site of the bridge with a given probability of exceedance (or a given recurrence period) of 10% in 50 years as per ECP201 [17] to which the chosen records are scaled up and/or down as will be explained in the following section to represent the specific hazard at the site. Finally, the bridge located in that site is subjected to this suite of scaled records and a series of time history analyses is carried out.

Ground motions selection methods

To investigate the efficiency of a proposed scheme for scaling of ground records - addressing both vertical and horizontal components of the record - for seismic analysis of bridges, and to prove its superiority to another commonly used method (scaling to certain Peak Ground Acceleration, PG A), different bins of records considered in the present research are selected in different manners (arbitrary or based on the record's PGA value) as follows:

Selection method (1): Consider all twenty three records and get the average of associated results.

Selection method (2): Consider a sub-set of seven records (No# 3, 5, 6, 8, 9, 14 & 22) selected according to Vertical PGA nearest to (0.9*ag * S = 0.9*0.15*g*1.2=1.59 m/s^2), for analysis in vertical direction, and get the average of associated results, and a sub-set of seven records (No# 3, 5, 8, 9, 14, 22 & 23) with Horizontal PGA nearest to (0.15g*S =1.76 m/s^2), for analysis in horizontal direction, and get the average of associated results.

Selection method (3): Consider a sub-set of three records (No# 11, 14 & 22) selected geographically, i.e., located close to each other, and get the maximum of associated results.

Selection method (4): Consider a sub-set of three records (No#1, 12 & 23) selected alphabetically, i.e., first, last and middle records alphabetically ordered, and get the maximum of associated results.

Selection method (5): Consider a sub-set of three records (No# 18, 19 & 20) selected from the same earthquake, and get the maximum of associated results.

Selection method (6): Consider a sub-set of three records (No# 10, 17 & 20) selected according to Vertical PGA(records having maximum, minimum and median value of vertical PGA), for analysis in vertical direction, and get the maximum of associated results; and three records (No# 4, 6 & 10) selected according to Horizontal PGA (records having maximum, minimum and median value of horizontal PGA), for analysis in horizontal direction, and get the maximum of associated results.

Selection method (7): Consider a sub-set of three records (No# 5, 8& 9) having their Vertical PGA nearest to (0.9*ag*S= 0.9*0.15*g*1.2=1.59 m/s^2), for analysis in vertical direction, and get the maximum of associated results; and three records (No# 14, 5 & 8) having their Horizontal PGA nearest to ag *S (0.15g*S = 1.76 m/s^2), for analysis in horizontal direction, and get the maximum of associated results.

Table 2- Parameters used for calculating "α" scale factor for vertical and longitudinal direction analysis

	Vertical direction					Longitudinal direction			
	Effective Mode (Z)	(Ti)	Modal mass participation factor (Ti)	λi	Sd (Ti)	Effective Mode (X)	(Ti)	Modal mass participation factor (Ti)	Sd (T_i)
Bridge System(1) span-length 25m	6	0.18626	0.11935	0.128	4.035	2	1.03724	0.9999999	2.767
	9	0.11637	0.64876	0.695	5.165				
	13	0.05467	0.01115	0.012	5.165				
	19	0.02822	0.15354	0.165	4.132				
Bridge System(6) span-length 25m	6	0.17603	0.14647	0.156	4.188	3	0.33217	0.98058	5.739
	8	0.11533	0.62237	0.661	5.165				
	13	0.05383	0.01041	0.011	5.165				
	19	0.02821	0.16165	0.172	3.443				
Bridge System(1) span-length 45m	7	0.41666	0.11719	0.129	1.781	3	1.39072	0.99999	2.063
	9	0.25051	0.61929	0.680	3.026				
	13	0.11747	0.01099	0.012	5.165				
	20	0.05740	0.01994	0.022	5.165				
	24	0.05246	0.14299	0.157	5.165				
Bridge System(6) span-length 45m	5	0.39563	0.13910	0.155	1.858	3	0.46939	0.54094	5.739
	9	0.24840	0.59818	0.664	3.099				
	20	0.05705	0.02331	0.026	5.165	4	0.40819	0.43566	5.739
	24	0.05239	0.13966	0.155	5.165				
Bridge System(1) span-length 65m	7	0.56291	0.11672	0.122	1.318	3	1.09261	0.99993	2.626
	9	0.33579	0.61222	0.639	2.246				
	15	0.15741	0.01078	0.011	4.812				
	23	0.07608	0.01748	0.018	5.165				
	26	0.06769	0.13609	0.142	5.165				
	39	0.03267	0.06429	0.067	4.132				
Bridge System(6) span-length 65m	4	0.52830	0.14380	0.155	5.165	8	0.35056	0.86945	5.739
	9	0.33224	0.58607	0.632	2.246				
	26	0.06759	0.13293	0.143	5.165				
	39	0.03267	0.06427	0.069	4.132				

Details of the proposed scaling methods

There are currently many methods of ground motion selection and scaling, but little guidance is available to engineers concerning which methods are appropriate for their specific applications. Herein this research we try to study scatter in structure responses due to seismic input with two different scaling methods: scaling to PGA versus scaling to Sa (T1)(i.e., Sd (T1) as per ECP201 [17] terminology), see Informative appendix (A), refer to Figs.4-6, considering vertical and horizontal components as defined in ECP201.The target values of PGA equal to 1.722 m/s2 and 2.296 m/s2 for vertical and horizontal directions, respectively, as shown in Fig. 5, and the target values of Sd (T1) are given in Table1. When scaling ground-motion records to Sa (T1) (or, Sd (T1) as per ECP201 terminology) we mean to increase or decrease each of the ground-motion records by a constant factor so that the spectral acceleration at a given frequency and damping is equal to the target spectral acceleration, i.e., the code design value (refer to Fig. 6 and Table 1). In this process, the spectral shape, relative phases, and duration of the ground motion should remain unchanged. The advantage of scaling of records (demonstrating magnitude, M, and distance, R, conditional independence of response given spectral acceleration) is that when we are given a target ground motion intensity we need not be overly concerned with what is the M and R of the ground motion records that we use for structural analysis; but we should pay attention that the mean value of all scale factors applied to all records in a given suite to match a given preset IM should have the value of approximately one. Biases from scaled-up and scaled-down records would then offset each other, resulting in unbiased median response, Baker [20].

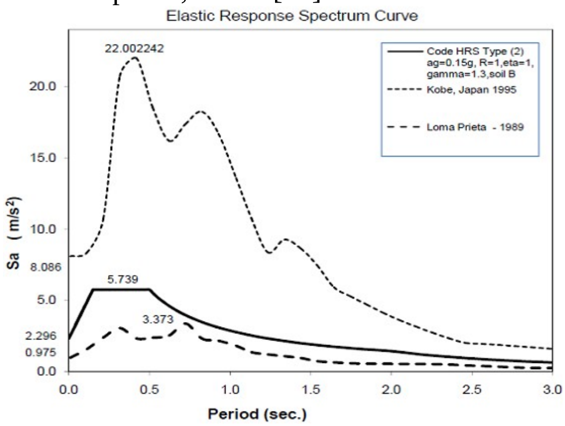


Fig.4-Horizontal Response Spectrum curve of Real Record of Kobe, Japan 1995 , Loma Prieta – 1999 & Code HRS Type (2)

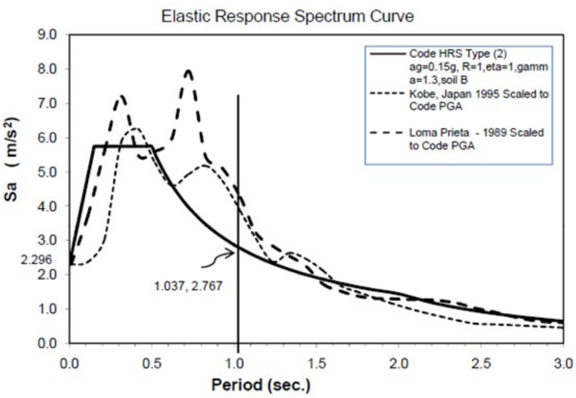


Fig.5- Horizontal Response Spectrum curve of Real Record of Kobe, Japan 1995, Loma Prieta -1999 Scaled to PGA of HRS - Curve as per ECP201 (2008)

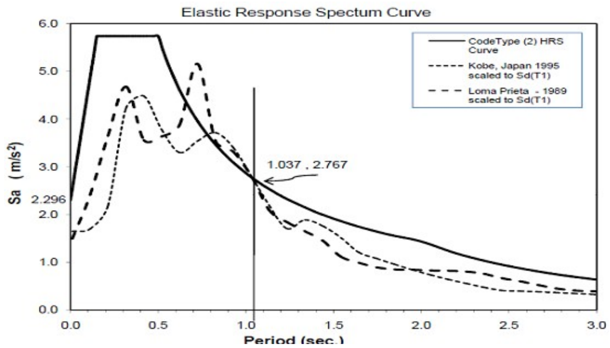


Fig.6- Horizontal Response Spectrum curve of Real Record of Kobe, Japan 1995, Loma Prieta - 1999 Scaled to Sa(T_0) - as per ECP201 (2008)- of Bridge System(1) ,span-length 25 meters as an example.

For multi-mode-controlled vibrations of bridges a weighted least square method can be used to find an appropriate scaling factor, “α”, to minimize the error between the code design response spectrum and the record-specific response spectrum at different relevant control periods associated with the main vibration modes governing the response of the bridge“α” is determined as per Eq(1) according to Mehanny et al., [21]:

$$\alpha = \frac{\sum_{i=l}^{n} \lambda_i \, S_i \, S_{di}}{\sum_{i=l}^{n} \lambda_i \, S_i^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where:

S_i=S (Ti): The record spectral acceleration at period i.

S_{di}= S_d (Ti): The target spectral acceleration at the same period i.

λ_i: assigned weight factor for each of the control periods; it is computed as the ratio between corresponding modal masses mobilized by the modes of vibration represented by the control periods.

Results and Discussion

Time history analysis has been carried out on various selected bridges under the scaled records with the two proposed scaling methods as previously mentioned. The current investigation covers both deformation and force demands. Response Spectrum Analysis (RSA) as per ECP 201 [17] has been performed in order to compare obtained bridge seismic response with that resulting from results from (THA) using the two scaling methods to determine which method is more sufficient and efficient in representing site seismic hazard. Static analysis under own weight and code braking force has been also carried out to serve as references for comparison purposes while investigating various seismic response; results under own weight are used in association with seismic analysis under vertical acceleration, while results under braking forces have been used in conjunction with seismic analysis under horizontal acceleration in the longitudinal direction of various investigated bridges.

All analysis results for different bridges and for different records selection methods are presented in table format as a percentage of static analysis results in each direction (refer to Tables3-6).

Table 3- Results of Time history analysis under vertical component of records scaled to PGA and Multi-modal response spectrum analysis under ECP201 Vertical RS (as a percentage of own weight case results).

		Bending Moment in Girder				Reactions in Columns			Vertical Displ.		Shear in Girder			
Bridges	**Selec. Method**	**Sec. 1 %**	**Sec. 2 %**	**Sec.3 %**	**Sec.4 %**	**Sec.5 %**	**Sec.6 %**	**Sec.7 %**	**Sec.1 %**	**Sec.3 %**	**Sec.a %**	**Sec.b %**	**Sec.c %**	**Sec.d %**
Sys. (1) – L = 25 m	1	30	31	71	62	44	26	24	29	78	25	23	36	44
	2	28	30	65	57	40	24	23	26	72	24	22	33	41
	3	39	56	146	127	90	48	34	34	158	34	33	72	91
	4	42	38	83	72	51	32	32	41	90	34	31	45	56
	5	45	49	116	101	71	41	34	44	127	35	33	57	73
	6	45	35	67	58	41	28	34	44	74	35	33	35	42
	7	36	42	84	73	52	35	30	31	92	30	26	43	52
	VRS	36	38	89	77	55	33	29	35	96	29	27	44	55
Sys. (1) - L=45 m	1	20	21	50	44	30	17	16	19	60	18	15	25	32
	2	19	19	47	41	28	15	15	18	57	16	13	24	30
	3	35	33	89	78	54	28	26	36	106	28	24	44	56
	4	49	40	117	103	70	34	36	51	141	36	31	53	69
	5	37	44	103	90	62	36	31	31	123	31	24	48	61
	6	37	44	103	90	62	36	31	33	123	31	24	48	61
	7	29	31	72	64	43	25	24	27	88	23	19	34	45
	VRS	19	22	52	45	33	20	17	17	60	17	15	27	33
Sys. (6) - L=65 m	1	16	18	42	37	26	15	13	15	51	15	13	23	27
	2	19	20	45	40	27	16	15	18	54	18	15	24	29
	3	38	35	74	65	45	29	29	37	89	30	28	0	0
	4	25	32	94	84	56	27	20	22	119	22	20	46	61
	5	16	24	68	60	41	21	14	14	83	15	12	33	43
	6	30	32	65	58	39	26	23	31	80	23	20	34	40
	7	21	20	40	35	25	16	17	19	47	17	13	23	26
	VRS	14	16	39	34	26	15	14	12	45	13	12	21	25
Sys. (6) - L=25 m	1	31	32	69	59	42	26	27	30	72	25	24	36	44
	2	28	29	64	55	40	24	22	27	68	23	22	34	41
	3	39	53	145	124	90	46	33	34	151	34	32	74	90
	4	43	40	78	66	48	31	32	44	80	35	33	45	53
	5	44	51	112	96	69	42	47	42	119	37	35	57	72
	6	44	40	63	54	39	30	34	42	67	37	35	35	40
	7	31	40	85	72	52	33	26	26	90	26	25	44	52
	VRS	38	41	85	73	54	32	30	37	89	30	27	45	54
Sys. (6) - L=45 m	1	23	23	51	44	31	18	18	20	56	19	16	27	32
	2	24	24	53	46	32	19	19	21	57	21	17	28	34
	3	39	30	88	76	53	27	29	40	100	30	27	45	55
	4	53	44	110	95	67	36	40	53	125	40	35	54	65
	5	36	45	101	87	61	37	30	28	113	0	0	0	0
	6	36	45	101	87	61	37	30	27	113	30	24	50	60
	7	32	35	67	59	41	27	26	25	75	31	24	42	46
	VRS	19	24	51	43	33	20	17	18	56	17	15	28	33
Sys. (6) -L=65 m	1	17	18	41	35	25	14	13	16	47	13	11	21	25
	2	20	20	44	38	27	16	15	19	49	15	13	23	27
	3	38	35	71	61	43	28	29	37	80	30	25	37	43
	4	24	30	90	78	54	26	20	25	107	20	17	43	54
	5	17	20	68	59	41	18	14	16	79	14	11	33	41
	6	39	32	60	51	36	25	29	40	68	29	24	33	36
	7	22	23	41	35	25	18	18	20	45	18	15	23	25
	VRS	14	18	37	32	25	15	13	13	42	13	12	21	25

Table 3 gives summary of results for bridges subjected to the vertical component of records scaled to PGA code value. It shows that percentage of seismic responses at different sections varies from selection method to the other. For example, the percentage of bending moment at section (1) – bridge System (1) with 25 m span-length - varies from 30 to 45 %, and at section (4) it varies from 72 to 127 %, this is because of the fact that by scaling the records to a particular value of peak ground acceleration, PGA, we somehow slightly reduce the variability in records' accelerations by making them having the same value at a certain point (namely, at T = 0), while values of the spectral acceleration at the effective periods (which mainly affect seismic response) are however still largely variable from record to record and so from selection method to the other.

Table 4 gives summary of results under the effect of the vertical component of records scaled to Sa (T1) - with, “α”, scale factor. Comparing these results with those presented in Table 3 we find that the variation in percentage of force and deformation responses decreases in Table 4, and all percentages for different selection methods are nearby those of code response spectrum analysis results. Such observation shows that this scheme of scaling is minimizing the error between the code design response spectrum and the record-specific response spectrum at different control periods associated with the main vibration modes governing the response of the bridge.

Table 4 - Results of Time history analysis under vertical component of records scaled to Sa (T_1)-, and Multi-modal response spectrum analysis under ECP201 Vertical RS. (as a percentage of own weight case results).

		Bending Moment in Girder				Reactions in Columns			Vertical Displ.		Shear in Girder			
Bridges	Selec. Method	Sec. 1 %	Sec.2 %	Sec.3 %	Sec.4 %	Sec.5 %	Sec.6 %	Sec.7 %	Sec.1 %	Sec.3 %	Sec.a %	Sec.b %	Sec.c %	Sec.d %
Sys. (1) – L = 25 m	1	40	40	89	77	54	33	32	38	97	33	31	45	56
	2	41	42	90	79	55	34	33	39	100	35	32	46	57
	3	61	49	108	95	65	39	47	59	122	52	50	48	66
	4	51	40	90	79	56	33	39	50	99	41	37	46	56
	5	57	44	100	87	61	36	43	56	109	45	43	49	63
	6	57	46	92	81	56	37	43	56	105	45	43	45	55
	7	38	44	103	90	63	37	31	33	114	32	28	48	64
	VRS	36	38	89	77	55	33	29	35	96	29	27	44	55
Sys. (1) - L=45 m	1	20	21	48	42	29	17	16	19	58	18	15	25	31
	2	23	23	58	51	35	19	18	21	70	20	16	29	37
	3	29	23	60	53	36	19	21	29	74	23	20	30	41
	4	36	24	61	54	37	19	27	37	73	26	22	28	36
	5	29	26	64	57	38	21	22	29	81	23	20	31	43
	6	25	26	59	52	36	21	19	26	71	20	17	28	35
	7	24	26	61	53	37	21	20	22	73	23	16	29	37
	VRS	19	22	52	45	33	20	17	17	60	17	15	27	33
Sys. (6) - L=65 m	1	15	17	39	35	24	14	12	14	48	15	13	21	26
	2	18	19	43	38	26	15	14	17	52	17	14	23	28
	3	22	20	49	43	29	16	17	21	58	19	16	25	30
	4	19	21	55	49	33	17	16	17	69	17	16	27	36
	5	14	19	50	45	30	16	11	13	62	12	12	25	32
	6	19	21	49	43	30	17	15	20	59	15	13	22	26
	7	21	20	41	35	25	17	17	19	49	20	16	25	28
	VRS	14	16	39	34	26	15	14	12	45	13	12	21	25
Sys. (6) - L=25 m	1	40	41	84	72	52	32	34	38	88	32	30	45	53
	2	41	41	87	74	53	33	32	39	91	33	31	46	55
	3	53	50	99	84	60	37	41	51	105	46	45	49	61
	4	48	43	83	71	50	33	37	46	89	39	36	46	53
	5	57	52	96	82	59	38	44	55	101	48	45	49	61
	6	57	52	81	69	50	38	44	55	86	48	45	44	50
	7	35	41	101	87	62	34	27	34	108	29	27	49	63
	VRS	38	41	85	73	54	32	30	37	89	30	27	45	54
Sys. (6) - L=45 m	1	22	23	49	42	30	18	18	19	53	20	16	26	31
	2	24	25	55	48	33	20	19	21	59	21	18	30	35
	3	29	21	60	52	36	18	21	29	68	23	20	31	38
	4	37	29	58	50	35	20	28	37	65	28	25	28	34
	5	31	26	58	51	35	21	23	31	71	23	20	30	38
	6	22	26	58	50	35	21	17	21	65	19	16	30	36
	7	26	29	60	52	36	23	21	21	63	26	18	30	36
	VRS	19	24	51	43	33	20	17	18	56	17	15	28	33
Sys. (6) -L=65 m	1	18	19	43	37	26	15	14	17	49	14	12	22	26
	2	21	21	48	41	29	17	16	19	54	16	14	25	29
	3	27	25	50	43	30	20	21	26	56	21	18	26	30
	4	22	25	60	52	36	20	17	22	71	17	14	29	36
	5	17	22	57	49	34	18	14	17	66	14	12	28	34
	6	30	25	50	43	31	20	22	31	56	22	19	27	31
	7	24	26	45	38	27	19	19	22	50	19	16	25	27
	VRS	14	18	37	32	25	15	13	13	42	13	12	21	25

Referring to Tables 3 and 4 one could note the following:

Variation in deformation response percentage is close to variation in force response percentage at the same section for all bridges and for different selection methods.

Most of results of bridge System (1) with different span lengths are similar to those of bridge System (6) with different span lengths. This is because of the fact that the difference between System (1) and System (6) is only the intermediate column-deck connections. For System (1), top of intermediate columns is released for shear and moment in major and minor directions, and only axial direction is restrained; while in System (6) all intermediate columns are monolithic. For both systems, the axial degree of freedom with respect to the column is restrained which has major effect on vertical motion of the bridge, and hence the behavior of the bridge in the vertical direction is almost the same for the two systems.

Percentage of response results from RSA ranges from 27 to 89% for bridges with 25 meters span length, from 15 to 52% for bridges with 45 meters span length and from 12 to 45% for bridges with 65 meters span length. Ratio of vertical seismic responses relative to own weight results decreases when span length increases.

The same findings can be extracted for horizontal direction analysis. Tables 5 and 6 give summary of results for bridges subjected to horizontal component of records scaled to PGA and Sa (T1) code values, respectively. The following maybe noted:

Percentage of horizontal seismic response with respect to braking force results at different sections varies from one selection method to the other, when scaling to PGA. For example, the percentage of seismic bending moment relative to moment under code braking force at section (1) of bridge System (1) with 25 m span length varies from 505 to 991%, whereas in Table 6 (i.e., when instead scaling to Sa (T1)) this percentage is constant for all selection methods as well as for horizontal RSA and is equal to 628%. That is due to scaling method strategy similar to what has been observed as mentioned above in vertical direction analysis, in addition to the main characteristics of the dynamic behavior of these bridge systems in the longitudinal direction. All case study bridges - except bridge System (6) with 45 meters span length - are first mode dominant structures in longitudinal direction. They have one effective mode which mobilizes about the total considered bridge seismic mass in the longitudinal direction; and scaling to Sa (T1) is performed to match the value of the spectral acceleration corresponding to this effective mode (first fundamental mode). Therefore, variation in response percentage (in either forces or displacements) is almost zero.

Table 5- Results of (THA) under horizontal component of records scaled to PGA and Multi-modal response spectrum analysis under ECP201 (2008) Horizontal.-RS. (as a percentage of braking force case results).

		B.M in Mid. Column	HZ. Reaction in Mid. Column	Hz. Displacement at Mid. Column top			Bending Moment			Horizontal Displacement at columns top		
Bridge	**Selec. Method**	**Sec.1 %**	**Sec.1 %**	**Sec.2 %**	**Bridge**	**Selec. Method**	**Col. 1 %**	**Col. 2 %**	**Col. 3 %**	**Col. 1 %**	**Col. 2 %**	**Col. 3 %**
Sys. (1) - L=25 m	1	505	505	505	Sys. (6) - L=25 m	1	1105	1133	1105	1123	1123	1123
	2	497	497	497		2	1223	1253	1223	1243	1243	1243
	3	965	965	965		3	1471	1495	1471	1488	1488	1488
	4	439	439	439		4	1473	1502	1473	1493	1492	1493
	5	644	644	644		5	1606	1632	1606	1628	1628	1628
	6	991	991	991		6	2002	2048	2002	2032	2032	2032
	7	965	965	965		7	1471	1495	1471	1488	1488	1488
	HRS	627	627	627		HRS	1272	1305	1272	1308	1299	1288
Sys. (1) - L=45 m	1	542	542	542	Sys. (6) - L=45 m	1	1412	1391	1412	1410	1410	1410
	2	535	535	535		2	1779	1784	1779	1799	1799	1799
	3	891	891	891		3	1754	1705	1754	1747	1749	1747
	4	964	964	964		4	1948	2024	1948	1993	1990	1993
	5	791	791	791		5	2518	2577	2518	2556	2555	2556
	6	1231	1231	1231		6	1622	1712	1622	1694	1691	1694
	7	891	891	891		7	2054	2154	2054	2121	2118	2121
	HRS	761	761	761		HRS	1714	1765	1714	1742	1740	1742
Sys. (1) - L=65 m	1	1140	1140	1140	Sys. (6) - L=65 m	1	2509	2390	2509	2409	2416	2409
	2	1073	1073	1073		2	2913	2745	2913	2791	2801	2791
	3	1969	1969	1969		3	3324	3207	3324	3250	3256	3250
	4	1873	1873	1873		4	3679	3405	3679	3449	3467	3449
	5	1864	1864	1864		5	3773	3609	3773	3571	3586	3571
	6	2566	2566	2566		6	4319	4118	4319	4137	4144	4137
	7	1969	1969	1969		7	3324	3207	3324	3250	3256	3250
	HRS	1439	1439	1439		HRS	2804	2687	2804	2681	2686	2681

Table 6- Results of (THA) under horizontal component of records scaled to Sa(T1) and Multi-modal response spectrum analysis under ECP201-(2008) Horizontal RS.(as a percentage of braking force case results).

		B.M in Mid. Column	HZ. Reaction in Mid. Column	Hz. Displacement at Mid. Column top			Bending Moment			Horizontal Displacement at columns top		
Bridge	Selec. Method	Sec.1 %	Sec.1 %	Sec.2 %	Bridge	Selec. Method	Col. 1 %	Col. 2 %	Col. 3 %	Col. 1 %	Col. 2 %	Col. 3 %
Sys. (1) - L=25 m	1	628	628	628	Sys. (6) - L=25 m	1	1278	1309	1278	1298	1297	1298
	2	628	628	628		2	1286	1315	1286	1305	1305	1305
	3	628	628	628		3	1285	1313	1285	1300	1299	1300
	4	630	630	630		4	1296	1343	1296	1315	1315	1315
	5	631	631	631		5	1295	1316	1295	1313	1313	1313
	6	628	628	628		6	1281	1318	1281	1295	1295	1295
	7	628	628	628		7	1302	1331	1302	1321	1320	1321
	HRS	627	627	627		HRS	1272	1305	1272	1308	1299	1288
Sys. (1) - L=45 m	1	762	762	762	Sys. (6) - L=45 m	1	1819	1771	1819	1804	1806	1804
	2	762	762	762		2	1781	1761	1781	1787	1788	1787
	3	762	762	762		3	2032	1780	2032	1880	1890	1880
	4	765	765	765		4	1901	1886	1901	1892	1894	1892
	5	763	763	763		5	1803	1803	1803	1814	1814	1814
	6	761	761	761		6	2006	1932	2006	1979	1981	1979
	7	764	764	764		7	2092	1899	2092	2001	2006	2001
	HRS	761	761	761		HRS	1714	1765	1714	1742	1740	1742
Sys. (1) - L=65 m	1	1442	1442	1442	Sys. (6) - L=65 m	1	2885	2763	2885	2781	2788	2781
	2	1442	1442	1442		2	2896	2727	2896	2775	2786	2775
	3	1440	1440	1440		3	2929	2826	2929	2864	2870	2864
	4	1446	1446	1446		4	3076	2892	3076	2944	2944	2944
	5	1447	1447	1447		5	2996	2960	2996	2960	2958	2960
	6	1440	1440	1440		6	2984	2877	2984	2900	2910	2900
	7	1447	1447	1447		7	2993	2879	2993	2922	2930	2922
	HRS	1439	1439	1439		HRS	2804	2687	2804	2681	2686	2681

Bridge System (6) with 45 meters span length is a two-degree of freedom structure, hence scaling factor "α" is used. The variation in this bridge case is however still small.

Results of bridge System (1) are different from those of bridge System (6). This is because of the difference in the intermediate column-deck connections which has a major effect on the longitudinal motion of the bridge. Hence, the dynamic behavior of the bridge in the longitudinal direction is different for the two systems.

Percentage of response from horizontal longitudinal RSA relative to braking force extremely increases when span length increases. It also increases when the number of monolithic connection increases.

Low dispersion observed when scaling using Sa (T1) encourages us to use a fewer number of records to guarantee a good degree of certainty in results. Therefore, Sa (T1) intensity measure is more sufficient and efficient than PGA intensity measure.

One should however iterate that average value of scaling factors for a given set of records should be almost equal to one to avoid bias in bridge responses resulting from the scaling technique especially for multi-mode-controlled bridges.

Conclusions

The main findings and general conclusions from this research can be summarized in the following points:

1- Record-to-record high variability causes the spectral acceleration values corresponding to effective periods (i.e., associated with modes of vibration with maximum modal mass participation factors) to significantly vary from one record to another. When scaling these records to a certain value of peak ground acceleration, PGA, the dispersion in the response decreases, because we reduce the variability of records accelerations by forcing them to have the same value at a certain period (namely, at T=0) but other values of ground acceleration corresponding to the effective periods of the structure are still widely variable.

2- To achieve more reduction in dispersion we should scale the records not to a certain value of

PGA but to a certain value of spectral acceleration associated with the effective (fundamental) mode, Sa (T1) for first mode dominant structures, or use a weighted scaling factor, "α", involving the spectral accelerations at various control periods contributing to the vibration response of the bridge for multi-mode-controlled bridges.

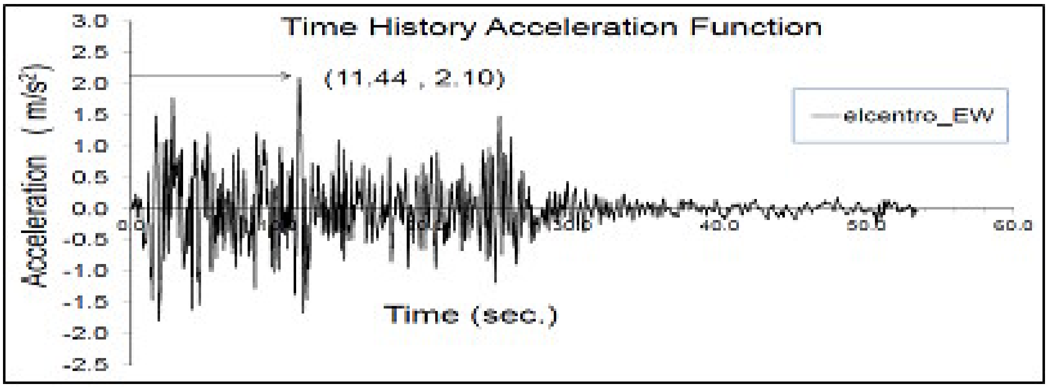
3- Average value of various scaling factors applied to any bin of selected records should be as close as possible to 1.0in order to avoid bias in bridge responses resulting from scaling techniques, especially for multi-mode-controlled bridges. Such conclusion is in line with previous conclusion drawn by Baker [20].

Further effort, looking at different bridge systems, other ground records, and conducting non-linear time history analysis should be spent in order to investigate whether the same conclusions can be generalized or not.

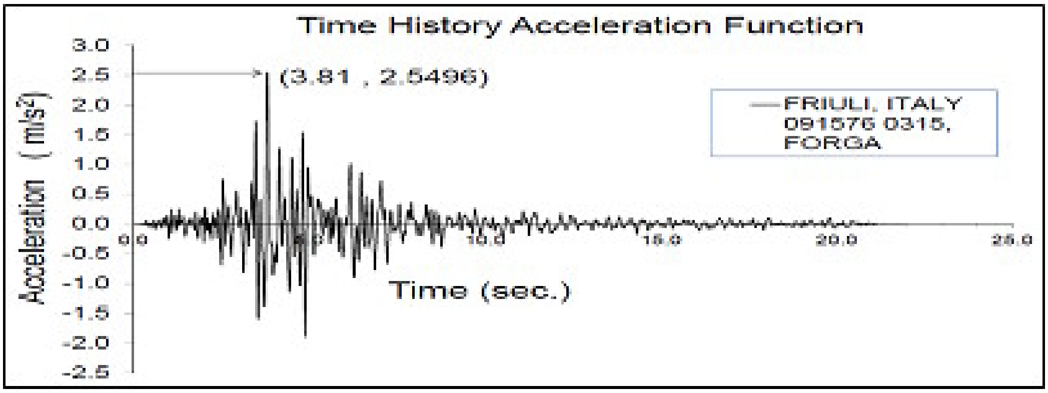
Informative appendix (A): Steps for getting scale factors of scaling methods (horizontal component of real record chosen in selected subset # 7 applied to bridge System (1) – 25 meters span-length as an example)

Step (1): Get the real time history of the selected earthquake

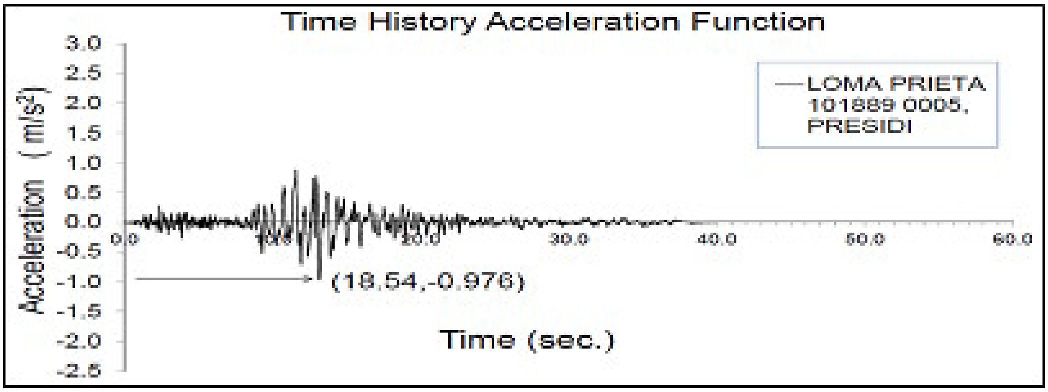
Earthquake No. 5 (Elcentro)



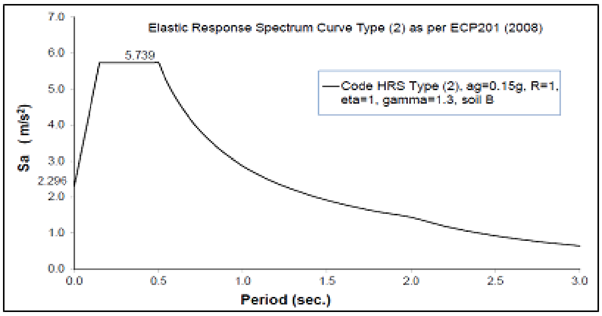
Earthquake No. 8 (Fruili)



Earthquake No. 14 (Loma Prieta)



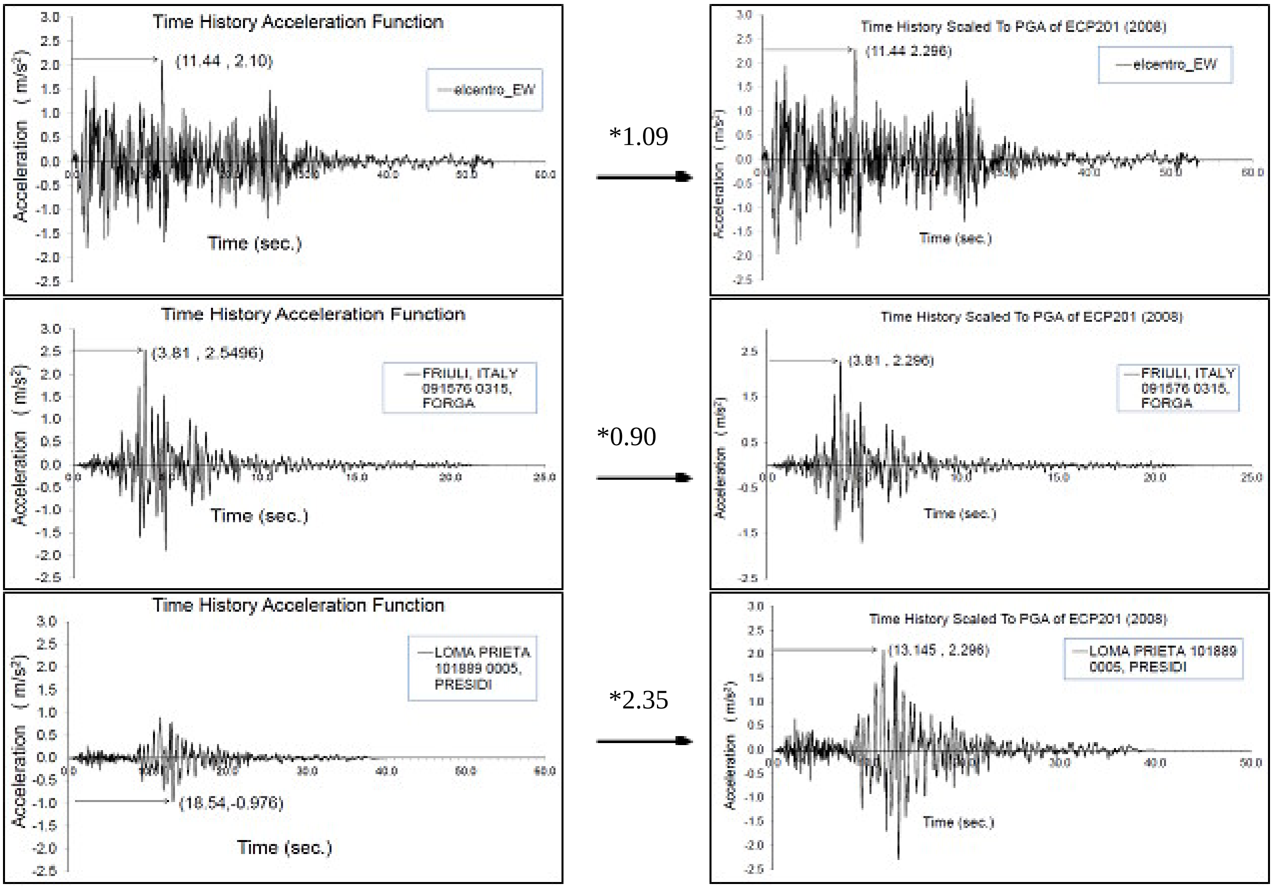
<u>Step (2):</u> Get the PGA code value as per ECP201 (2008) Response spectra curve Type (2)



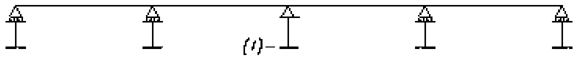
<u>Step (3):</u> Scale the real records to PGA code value

PGA $_{code}$= 2.296 m/s^2

Earthquake	PGA (m/s^2)	Scale factor (PGA $_{code}$ /PGA)
(5) Elcentro	2.099	1.09
(8) Friuli, Italy-02	2.550	0.90
(14) Loma Prieta	0.976	2.35



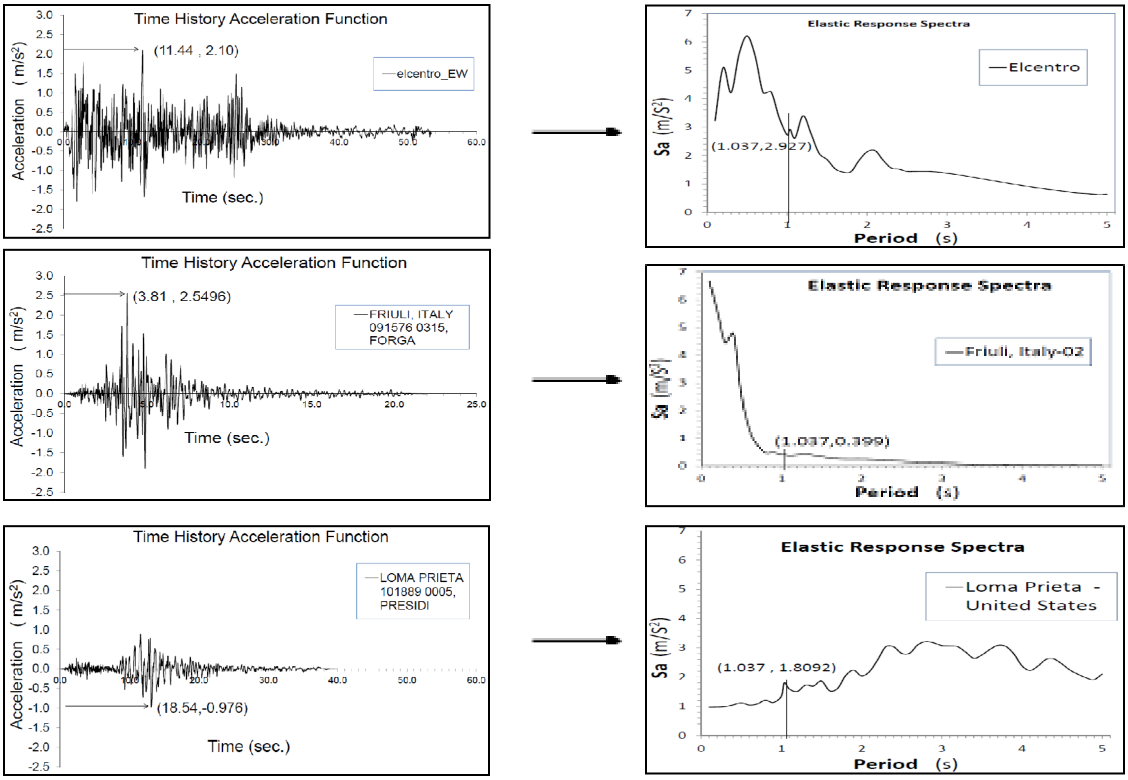
<u>Step (4):</u> Apply these scaled time history functions to Bridge system (1) - 25 meters span-length and get bending moment at section (1) results from time history analysis of each scaled record then take the maximum value (as recommende by ECP201 (2008)



Earthquake	Bending Moment in Middle Column - Sec.(1) (tm)
(5) Elcentro	3657
(8) Friuli, Italy-02	410
(14) Loma Prieta -United States	4865
Maximum	4865
Breaking Force	504
Hz. Response Spectrum (HRS)	3159
% of maximum to breaking force	965

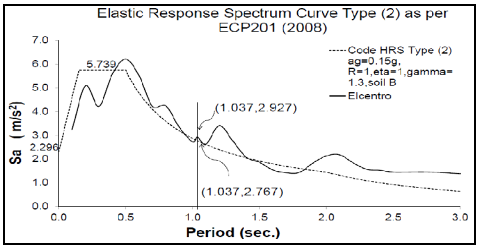
<u>Step (5):</u> Get the effective period of the bridge (which has modal mass participation ratio equals at least 0.90), and get Sa (T_1) corresponding to the effective period of the bridge from code response spectra curve, [T_1 = 1.037 s].

<u>Step (6):</u> Get the response spectra curve of real records using a generic program and get Sa (T_1) corresponding to the effective period of the bridge for each record.

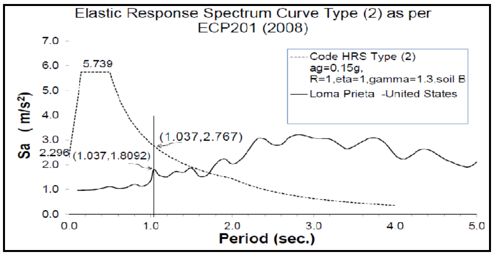


<u>Step (7):</u> Get scale factor for each record

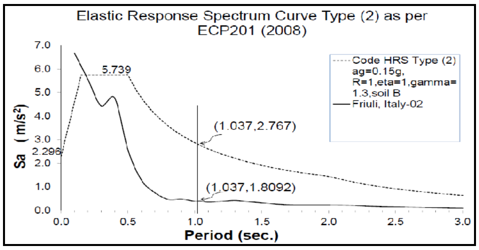
Earthquake No. 5 (Elcentro)



Earthquake No. 14 (LomaPrieta)



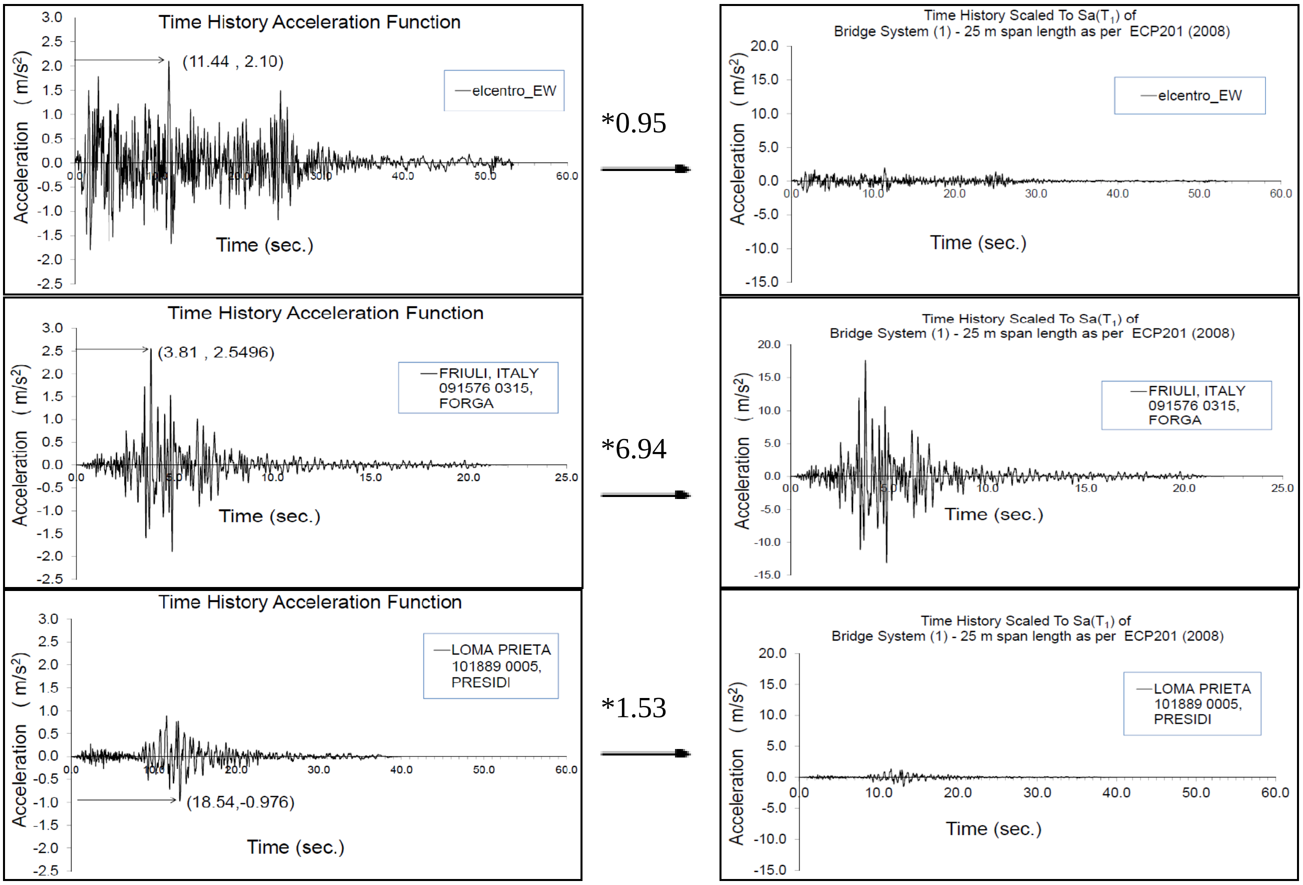
Earthquake No. 8 (Fruili)



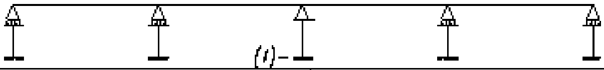
$Sa(T_1)_{code} = 2.767\ m/s^2$

Earthquake	Sa (T_1) (m/s^2)	Scale factor ($Sa(T_1)_{code}$ / Sa (T_1))
(5) Elcentro	2.9273	0.95
(8)Friuli, Italy-02	0.3988	6.94
(14) Loma Prieta	1.8092	1.53

Step (8): Scale the real records to Sa (T_1) code value.



Step (9): Apply these scaled time history functions to Bridge system (1)- 25 meters span-length and get bending moment at section (1) results from time history analysis of each scaled record then take the maximum value (as recommended by ECP201 (2008).



Earthquake	**Bending Moment in Middle Column - Sec.(1) (tm)**
(5) Elcentro	3161
(8) Friuli, Italy-02	3162
(14) Loma Prieta -United States	3163
Maximum	3163
Breaking Force	504
Hz. Response Spectrum (HRS)	3159
% of maximum to breaking force	628

References

[1] Evangelos I. Katsanos, Anastasios G. Sextos, George D. Manolis. Selection of Earthquake Ground Motion Records: A State-of-the-arte View from a Structural Engineering Perspective. Soil Dynamics and Earthquake Engineering. 2010; 30: 157-169.

[2] Eurocode 8: "Design of Structures for Earthquake Resistance". Part 1: "General Rules, Seismic Actions and Rules for Buildings", Final Draft prEN 1998, European Committee for Standardization, Brussels. EC8-1 (109) CEN, Part 2: "bridges" EC8-2; 2003

[3] American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures. ASCE07-05; 2006.

[4] American Association of State Highway and Transportation Officials (AASHTO). Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 5th ed., Washington, D.C 20001; 2010.

[5] American Society of Civil Engineers. Seismic Analysis of Safety-related Nuclear Structures and Commentary. ASCE Standard No. 004-98; 2000.

[6] Building Seismic Safety Council for the Federal Emergency Management Agency. FEMA 368. Recommended Provisions for Seismic Regulations for New Buildings and Other Structures. Washington, DC, 2001; 2001

[7] New Zealand Code and supplement Standards. NZS 1170.5.Structural Design Actions, Part5: earthquake actions. NewZealand. Wellington; 2004.

[8] Ordinanza del Presidente del Consiglio dei Ministri (OPCM) n. 3274. Norme Tecniche per il Progetto, la Valutazione e l'Adeguamento Sismico Degli edifici. Gazzetta Ufficiale Della Repubblica Italiana;2003.

[9] Hellenic Antiseismic Code (EAK).Ministry of Public Works, Athens; 2000.

[10] Recommended Lateral Force Requirements and Commentary, Seismology Committee, Structure Engineers Association in California,(SEAOC); 1996 Sixth Edition and 1999 Seventh Edition.

[11] Seismic Evaluation and Retrofit of Concrete Buildings, (ATC-40). California Seismic Safty Commission SSC 96-01; 1996.

[12] Shome N, Cornell C, Bazzurro P, Carballo J. Earthquake, Records, and Nonlinear Responses. Earthquake Spectra. (1998); Vol. 14 (3):469 –500.

[13] Y. C. Kurama and K. T. Farrow. Ground Motion Scaling Methods for Different Site Conditions and Structure Characteristics. Earthquake Engineering and Structural Dynamics. 2003; 32: 2425-2450.

[14] Sameh S.F. Mehanny. A Broad-Range Power-Law form Scalar-Based Seismic Intensity Measure. Engineering Structures. 2009; 31: 1354-1368

[15] Baker J.W., Cornell C.A. Correlation of Response Spectral Values for Multi- Component Ground Motions. Bulletin of the Seismological Society of America. 2006; Vol. 96 (1): 215-227.

[16] Mourad M. Bakhoum. Effect of Vertical Acceleration on the Seismic Design of Bridges. Helwan University Scientific Journal. Egypt, April 2003.

[17] The Egyptian Code for Loads on Structures. (ECP201). Egypt; 2008.

[18] Nevine A. Markous. Scaling of Earthquake Ground Motion Records for Seismic Analysis and Design of Bridges. [M.sc. Thesis]. Cairo University; 2011.

[19] The Pacific Earthquake Engineering Research Center Ground Motion Database.(PEER)[Internet] 2010. Available from: http://peer.berkeley.edu/peer_ground_motion_database.

[20] Jack W. Baker. Measuring Bias in Structural Response Caused by Ground Motion Scaling. Proceedings of the 8th Pacific Conference on Earthquake Engineering, 2007 December 5-7, Paper No 056.