Seismic Performance Assessment of Bridges in Montreal Using Incremental Dynamic Analysis

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SUMMARY:
The seismic performance of continuous 4-span bridges with different configurations has been evaluated using incremental dynamic analysis (IDA). The bridges were designed using the 2010 Canadian Highway Bridge code (CHBDC) and the bridges were evaluated using the design spectrum from the 2010 NBCC provisions. The IDA was performed using 39 pairs of ground motions. In addition the analyses were carried out using 2D and 3D models. The probability of exceeding different damage states were predicted at different hazard levels and fragility curves were developed for different damage states. The results demonstrate that the probabilities of exceeding different damage states are reasonably low for the corresponding seismic hazard in Montreal. The results also indicate that the use of 2D and 3D models for IDA may result in different assessments for the single-column bent bridges studied and therefore care should be taken when treating the IDA results.

Keywords: Incremental Dynamic Analysis (IDA), Bridges, Three- dimensional analysis, Fragility curves

1. INTRODUCTION

Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell, 2002) is an analysis method which can be used for more detailed seismic performance predictions of structures subjected to different seismic excitation levels. It involves numerous inelastic time history analyses performed using a set of ground motion records, each scaled (up or down), to study different seismic intensity levels. IDA provides valuable information regarding possible structural responses, required for the probabilistic seismic performance assessment of structures and seismic risk analysis (e.g., development of fragility curves and prediction of the annual rate of collapse, etc).

The IDA procedure has been adopted by some guidelines including the ATC-63 provisions (ATC-63, 2008) to determine the seismic performance, collapse capacity and fragility of buildings. Similar concepts can be used for the seismic assessment of bridges. However, for the case of bridges, bridge-specific experimental and analytical research are required to establish the important aspects of analysis, modelling and the criteria for structural assessment. The assessment criteria needed for bridges may be somewhat different than those for buildings. Specifically, "collapse" may need to be related to the functionality of bridges. As a result in this research in addition to collapse, the seismic performance of the bridges have been studied at different damage states, including yielding, serviceability (strain limits for cracking and spalling) and rebar buckling which can be used to determine the state of functionality of bridges. While there are experimental studies for collapse evaluation and failure modes in buildings, there is lack of research for collapse assessment of bridges. Therefore the assumptions made in this research may be limited to current state of the art and can be improved once such information becomes available.

2. BRIDGE PROPERTIES AND MODELLING ASSUMPTIONS

The structures studied include a regular and an irregular 4-span bridge supported by single columns, as shown in Fig. 2.1. The bridge structures are assumed to be unrestrained against transverse movement at the abutments. It is assumed that the bridges are either designed for free transverse movements at the ends or the bridge shear keys at the abutments are expected to fail at high seismic intensity levels. All column diameters are 2.0 m, the span length is 50 m, and the superstructure consists of a box girder with a uniform dead load of 200 kN/m. For the case of the regular bridges, all column heights are 7 m, while for the irregular bridge the column heights are 7, 14 and 21 m, respectively. The bridges were designed based on the 2010 CHBDC provisions (CSA, 2010) except that the design spectrum given in the 2010 NBCC (NRCC, 2010) was adopted. The design spectra in the 2010 NBCC corresponds to 2% probability of exceedance in 50 years and these spectra are expected to be used in the next edition of the CHBDC. The bridges were designed for Montreal and soil class C (i.e., V_{S30}=555 m/sec) with an importance factor, I of 1.0. However the minimum reinforcement ratio was controlling in all cases, when either a force modification factor of R=3 or R=5 was used in design, and therefore all bridge columns in this study contain the minimum longitudinal reinforcement ratio of 0.8% based on the 2010 CHBDC. For the transverse reinforcement the confinement reinforcement ratio of 1.2% was controlling.

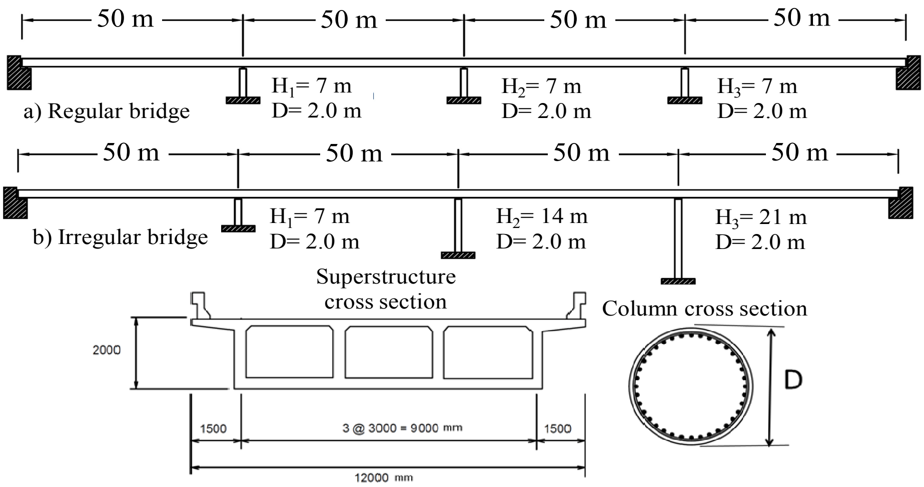

Fig. 2.1. Bridge properties

For the regular bridge the first mode is a torsional mode with T= 1.43 sec (with effective modal mass ratio close to zero). The fundamental period in the transverse direction is 1.30 sec (with 60% contribution) and the period in the longitudinal direction is 0.92 sec. In the transverse direction a third mode with T=0.75 sec (with about 40% contribution) is also important. For the three-dimensional analyses the geometric mean period of 1.09 sec was used to compute the spectral acceleration of the records for the regular bridge. For the irregular bridge the first mode is a torsional mode with T= 4.75 sec (with effective modal mass ratio of about 40%). The fundamental period in the transverse direction is 1.34 sec (with 50% contribution) and the period in the longitudinal direction is 1.49 sec. In the transverse direction a third mode with T=1.08 sec (with about 10% contribution) can also be important. For the three-dimensional analyses the geometric mean period of 1.41 sec was used to compute the spectral acceleration of the records for the irregular bridge. It should be noted that after performing the IDA, the Intensity Measure (IM) values can be computed at any desired period using the elastic response spectra of the records and therefore there is no need to perform the IDA again for different periods. In this study only the results at the fundamental period of the structures (which was found the most critical case) are presented. Similarly the evaluations can be carried out at the other periods. The periods are computed using the effective stiffness of the columns (i.e., $K_e = M_y/\theta_y$ where M_y and θ_y are the moment and rotation at yield, respectively).

The most important factors in structural modelling for IDA are the rotation capacity, θ_{cap}, and the post-capping rotation capacity, θ_{pc}. These parameters are used to define a component backbone curve, as shown in Fig. 2.2. The modified Takeda hysteresis model (Otani, 1981) was used in this study to model the behavior of the RC columns using Ruaumoko software (Carr, 2009). This model has two main parameters, alpha and beta, which control the unloading and the reloading stiffness, respectively. The values of alpha=0.3 and beta=0.3 were adopted to comply with both the recommended values in practice (Priestley et al., 2007) and the median values from the tests (more details are available in Tehrani et al., (2012)). The structural modelling considered in this study is similar to that used by Priestley et al., (Priestley et al., 2007), except that the backbone curve shown in Fig. 2.2, including the post-peak response was used. More details concerning the modelling of the bridges are available in Tehrani et al. (2012) and Tehrani and Mitchell (2012a).

For the 3D analyses the yield interaction surface developed by Tseng and Penzien (1973) was used in the Ruaumoko program (Carr, 2009). The interaction factors for the flexural term (i.e., a and b factors in the original model by Tseng and Penzien (1973)) were considered as 2 for the circular bridge columns. The use of the beam elements in modelling (i.e., only interaction of moments) also resulted in similar predictions with about 10% to 15% differences on average. The small differences are due to the fact that the effects of axial forces were not significant in the bridges considered in this study. The effects of axial forces on the seismic response are expected to be more important for the case of bridges with multi-column bents or bridges with shorter span lengths. In the analyses as the moment strength or yield strength of column degrades, the stiffness also degrades accordingly such that the yield displacements remain constant and the definition of ductility remains consistent (Carr, 2009).

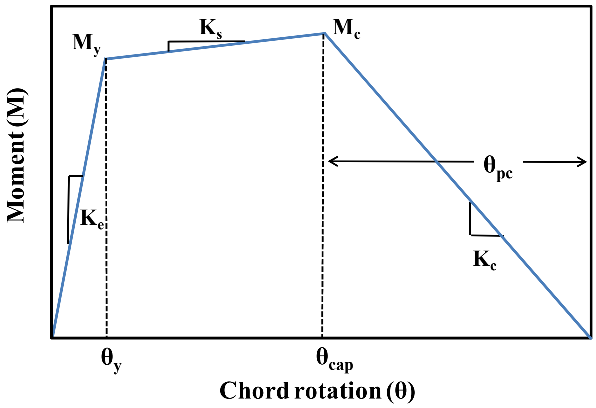

Fig. 2.2. Backbone curve parameters (Adapted from ATC-63 (2008))

3. DAMAGE STATES AND PERFORMANCE INDICATORS

Several damage states were considered in the seismic evaluation of the bridges under study including yielding, serviceability, bar-buckling and collapse. The yielding and serviceability damage states for the bridge columns were predicted using the method described by Priestley et al., (2007). A study by Berry and Eberhard (2007) provides some empirical equations to estimate the engineering demand parameters including drift ratio, plastic rotation, and strain in the longitudinal bars for circular bridge columns based on the properties of typical columns. In this research, the ultimate strain in the steel bars corresponding to bar buckling from the experimental equations by Berry and Eberhard (2007) is used to compute the ultimate curvature of the columns and the corresponding drift and ductility is defined as the point at which strength degradation begins (i.e., θ_{cap} in Fig. 2.2). The drift ratio at the bar-buckling damage state is very similar to that obtained for the "Life Safety" limit state based on the Priestley et al. (2007) recommendations (more details available in Tehrani and Mitchell (2012a)). Very few test results are available to calibrate the post-capping stiffness of the columns. In the ATC-63 provisions an equation is developed to estimate θ_{pc}. A conservative upper limit of 0.1 is recommended for the post-capping chord rotation of columns which is controlling for most columns designed using modern seismic codes. This conservative limit of 0.1 was controlling for the bridge columns and was adopted in this study due to lack of research on this subject. This limit is deemed to be more conservative for bridge columns with spiral transverse reinforcement.

Table 3.1. Deformations at damage states for the bridge columns with D=2.0 m and: a) H=7 m; b) H=14 m

a)

Damage state	Drift	Curvature ductility	Disp. ductility
Yielding	0.54%	1	1
Serviceability	1.05%	4.1	2.0
Bar buckling	4.33%	23.1	8.1

b)

Damage state	Drift	Curvature ductility	Disp. ductility
Yielding	1.01%	1	1
Serviceability	1.95%	4.1	1.9
Bar buckling	7.22%	23.1	7.1

The bridges under study were designed and detailed to meet the code requirements for ductile response, including capacity design concepts and adequate support lengths at the abutments. The ductile columns contain code-compliant spiral reinforcement to confine the concrete, to avoid shear failure and to control buckling of the vertical reinforcing bars. For this continuous bridge, with all other failure modes avoided, the flexural response governs the response of the bridge and sidesway collapse is the governing collapse mechanism. The collapse prediction is based on dynamic instability of the structure (e.g., Vamvatsikos and Cornell, 2002.). The drift and ductility capacities at different damage states are presented in Table 3.1 for the bridge columns studied.

4. ABUTMENT MODELLING

The simplified abutment model developed by Aviram et al. (2008) was used to model the influence of the abutments on the seismic response of the bridges in the longitudinal direction. The longitudinal response in the simplified model is a function of the system response including the gap, the abutment backwall and the soil backfill material. In the simplified model used the effects of the bearing pads on the responses are ignored. After gap closure, the superstructure bears directly on the abutment backwall and mobilizes the full passive backfill pressure. Two rigid elements were connected to the abutment ends to model the superstructure width as recommended by Aviram et al. (2008). More details of modelling the abutments in this study and the influence of the abutments on the longitudinal response are available in Tehrani and Mitchell (2012b).

5. RECORD SELECTION CRITERIA AND SPECTRAL SHAPE EFFECTS

For crustal earthquakes, the “basic far-field” records used by (Haselton and Deierlein, 2007) were used in this study. This set includes 39 crustal records from the PEER-NGA database; while a smaller subset of these records (i.e., 22 records) were used in the ATC-63 provisions for seismic performance assessments. Some minimum limits on event magnitude, peak ground acceleration (PGA) and peak ground velocity (PGV) were imposed in the record selection to make sure that the selected records are strong enough that may cause structural collapse. The criteria imposed for record selection by Haselton and Deierlein (2007) are summarized in Table 5.1.

Table 5.1. Criteria used for the record selection

Magnitude	Distance (km)	V_{S30} (m/sec)	PGA (g)	PGV (cm/sec)	Maximim number of records from each event	Database
≥ 6.5	≥ 10	≥180	≥ 0.2	≥ 15	6	PEER-NGA

The spectral shape of the records and the epsilon values at the fundamental period of the structure are some important aspects that should be considered in record selection for structural analysis (espsilon is defined as the number of logarithmic standard deviations of a target ground motion from a median ground motion prediction equation (GMPE) for a given magnitude, M, and distance, R). The epsilon value of the record at the fundamental period of the structure is a proxy measure of the spectral shape (Baker and Cornell, 2006a) and the selected records for the structural analyses should have epsilon values similar to the mean epsilon value obtained from the seismic hazard deaggregation at the site of interest. The predictions of the epsilon values of the records using the AB06 GMPE (Atkinson and Boore, 2006) for eastern North America and the deaggregation of the seismic hazard for Montreal indicated that the mean epsilon values of the records considered (in the range of periods of the bridge structures studied) are similar to the mean epsilon values obtained from the seismic hazard deaggregation. Therefore no modification of the results for the spectral shape effects was necessary.

6. SEISMIC HAZARD IN MONTREAL

In this study, an updated seismic hazard model developed by Atkinson and Goda (2011) will be used to take advantage of new seismic information and seismological models. The uniform hazard spectra at different hazard levels (i.e., 40%, 10% and 2% probability of exceedance in 50 years) obtained using the updated model along with the 2010 NBCC design spectra are shown in Fig. 6.1 for Montreal. It should be noted that the 2010 NBCC spectrum uses the median values of spectral accelerations, while the updated models use the mean values. The mean values of Sa (T) from the updated model at 2% probability of exceedance in 50 years are smaller at shorter periods and a little higher at longer periods, compared to those from the 2010 NBCC spectrum.

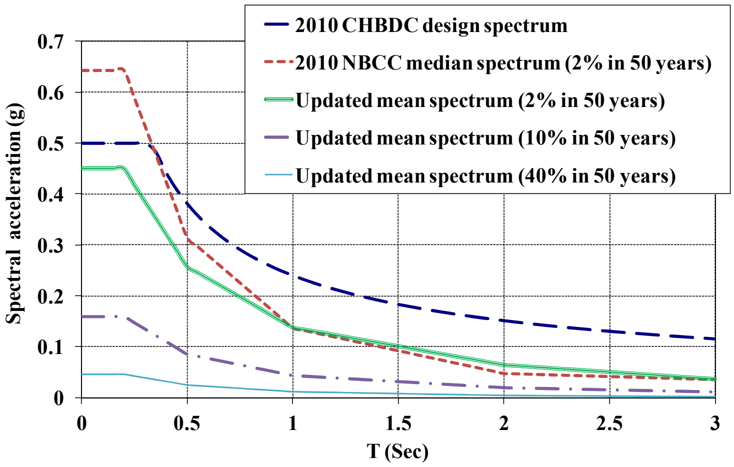

Fig.6.1. Uniform hazard spectra (UHS) at different hazard levels for Montreal and site class C

7. INCREMENTAL DYNAMIC ANALYSIS USING 3D MODELS

To develop the IDA curves the spectral acceleration at the fundamental period of the bridges is used as the intensity measure (IM) and the maximum drift ratio of the critical columns is used as the damage measure (DM). The preferred choice of an IM in the case where the two axes of the structure have different fundamental periods has not been extensively examined. In the absence of further research, the geometric mean of the pair of ground motions, $Sa_{g.m.}$, at the geometric mean of the two periods is used as the IM parameter as recommended by Baker and Cornell (2006b). The DM parameter shown in the IDA plots are the maximum drift ratios obtained in the orthogonal directions combined using the SRSS rule.

The thirty-nine pairs of records were applied twice to the model, once with the ground motion records oriented along one principal direction, and then again with the records rotated 90 degrees. This resulted in seventy-eight IDA curves. The use of pairs of ground motions in IDA using 3D models for the multi degree of freedom bridges with single column bents studied resulted in "waving" behaviour with some points of structural resurrection (i.e., the structure resists seismic forces at higher IM values after a collapse at a lower IM value, see Vamvatsikos and Cornell (2002) for the more details). This is because the timing, details and frequency contents of the ground motions has a significant effect on the seismic response and when the records are applied in pairs the complexity of the seismic response increases even more. Therefore the IDA analyses were performed using very small increments of IM values to determine the maximum seismic demands at each intensity level. The increments were considered as 5% of the spectral acceleration at the maximum considered earthquake level (i.e, 2% in 50 years) (e.g., 0.0065 (g) at T=1.09 sec). The IDA analyses were continued up to twice the intensity level at which the first global dynamic instability (collapse) was observed, due to possible structural resurrection. The lack of structural redundancy in the single-column bridges studied might be one of the reasons for exhibiting such behaviour, because the failure of one bridge column could result in collapse.

The IDA results were summarized using the IM given DM (i.e., IM|DM) and DM given IM (i.e., DM|IM) percentiles. For the IM|DM percentiles the lowest spectral acceleration at which the global dynamic instability occurs is determined as the collapse capacity for each record and subsequently the percentiles are determined. For the analyses with complex IDA curves (i.e., due to waiving and resurrection behaviour) the use of the IM|DM percentiles can be conservative. For the DM|IM approach the IDA percentiles are determined based on the percentiles of the drift ratios at each intensity level, IM. The DM|IM approach for such cases is believed to provide more reasonable predictions of the IDA percentiles. It should be noted that where the IDA curves do not exhibit significant waving and structural resurrection behaviours (e.g., in a typical 2D analysis or for other class of structures) the line connecting the x% percentiles of DM|IM is the same as the one connecting the (100−x)% percentiles of IM|DM (e.g., see Vamvatsikos and Cornell (2002)). The percentile curves shown in Figs. 7.1 are determined based on the DM|IM approach. In Tables 7.1 and 7.2 more details of the IDA results determined for different damage states are presented. The results indicate that the IM|DM percentiles underestimate the overall capacity of the structure especially at higher damage states such as bar-buckling and collapse.

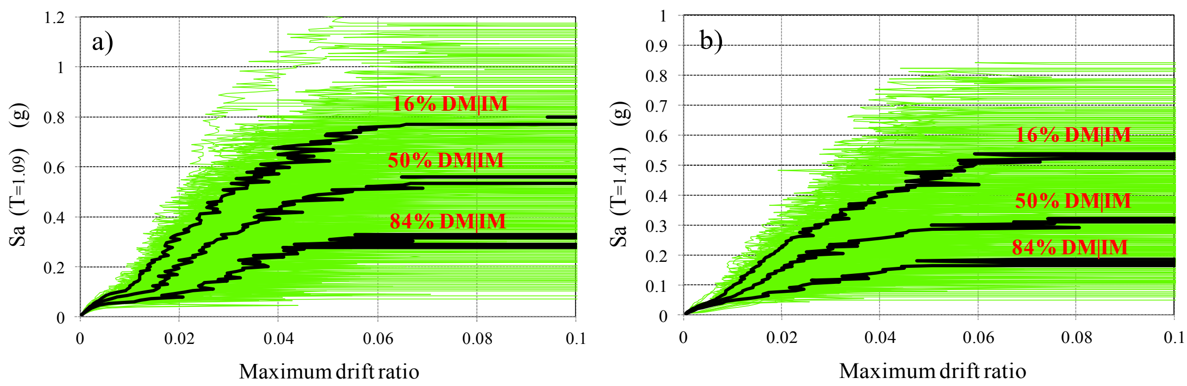

Fig. 7.1. IDA results developed for the case of : a) regular bridge with H_1= H_2= H_3= 7 m and D= 2 m ; b) irregular bridge with H_1=7 m, H_2= 14 m, H_3=21 m and D= 2 m

Researchers have expressed their concerns about the scaling of the pairs of ground motion and the choice of a scale factor for the orthogonal component of a ground motion when a selected component has been scaled by a specified factor (e.g., Baker and Cornell 2006b). In the ATC-63 provisions (ATC-63, 2008) concern is expressed about the results from the 3D analysis being conservatively biased. Because ground motions records are applied in pairs in three-dimensional nonlinear dynamic analyses, the resulting behaviour from each ground motion component is coupled. It has been found that the median collapse capacity resulting from three dimensional analyses is on average about 20% less than the median resulting from two-dimensional analyses for the case of buildings (ATC-63, 2008). Based on the ATC-63 provisions the application and scaling of pairs of ground motion records in IDA for three-dimensional analyses may introduce a conservative bias that is not present in two-dimensional analyses. More research is required to investigate the differences between the 2D and the 3D analyses for performing the IDA.

Fragility curves can be developed using the IDA results to predict the conditional probability that a certain limit-state is exceeded (i.e., probability of failure) at a given IM value. Assuming that the IDA data is lognormally distributed, it is possible to develop the fragility curves at collapse (or any other limit-states) by computing only the median collapse capacity and logarithmic standard deviation of the IDA results at collapse (or any other limit-state). The fragility curves can then be analytically computed using Eq. 7.1:

$$P(failure \mid S_a = x) = \Phi\left(\frac{\ln(x) - \ln(Sa^{C}_{50\%})}{\beta_{TOT}}\right) \tag{7.1}$$

where Φ(.) is the cumulative normal distribution function, $S_{a\ 50\%}^{C}$ is the median capacity determined from IDA, and β_{TOT} is the total uncertainty taken as $\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2}$. Where β_{RTR}, β_{DR}, β_{TD}, and β_{MDL} are uncertainty due to record-to-record variability, design requirements, test data and structural modelling, respectively (ATC-63, 2008). The record-to-record variability can be obtained from the IDA results at different damage states as reported in Tables 7.1 and 7.2. Where no detailed information is available, the other sources of uncertainty may be estimated by judgement based on the quality ratings. For example in ATC-63 provisions (ATC-63, 2008) the quality ratings of "Superior", "Good", "Fair" and "Poor" are translated into quantitative values of uncertainty of 0.20, 0.30, 0.45 and 0.65, respectively. Although there is little research on this subject, the quality of test data and modelling were assumed as "good" and the quality of the design requirements are considered to be "superior" in this study to estimate the total uncertainty.

Table 7.1. IDA results and estimated probability of exceeding different damage states for the regular bridge

	IM|DM percentiles (g)				DM|IM percentiles (g)					Probability of failure at different hazard levels		
Damage state	50%	16%	84%	St Dev	50%	84%	16%	St Dev	β_{TOT}	40% in 50 years	10% in 50 years	2% in 50 years
Yielding	**0.065**	0.050	0.083	0.24	**0.065**	0.050	0.085	0.26	0.54	0.07%	18.99%	90.19%
Serviceability	**0.080**	0.058	0.099	0.27	**0.087**	0.057	0.126	0.39	0.61	0.05%	10.65%	74.47%
Bar-buckling	**0.188**	0.121	0.347	0.50	**0.417**	0.250	0.618	0.45	0.65	0.00%	0.02%	3.75%
Collapse	**0.347**	0.201	0.513	0.50	**0.534**	0.277	0.772	0.51	0.70	0.00%	0.01%	2.14%

Table 7.2. IDA results and estimated probability of exceeding different damage states for the irregular bridge

	IM|DM percentiles (g)				DM|IM percentiles (g)					Probability of failure at different hazard levels		
Damage state	50%	16%	84%	St Dev	50%	84%	16%	St Dev	β_{TOT}	40% in 50 years	10% in 50 years	2% in 50 years
Yielding	**0.037**	0.032	0.044	0.28	**0.038**	0.032	0.044	0.17	0.50	0.25%	39.87%	98.21%
Serviceability	**0.060**	0.048	0.079	0.30	**0.065**	0.049	0.102	0.36	0.59	0.05%	12.82%	80.04%
Bar-buckling	**0.160**	0.105	0.255	0.44	**0.262**	0.153	0.413	0.50	0.68	0.00%	0.12%	9.52%
Collapse	**0.219**	0.120	0.316	0.48	**0.311**	0.166	0.522	0.57	0.74	0.00%	0.13%	7.52%

The IDA results presented in Tables 7.1 and 7.2 can be used along with Eq. 7.1 to develop the fragility curves at different damage states, as shown in Fig. 7.2. In the horizontal axis of the fragility curves in Fig. 7.2 the spectral acceleration at the fundamental period, Sa (T), is normalized by the spectral acceleration at the MCE level, Sa_{MCE}. Since the structures have different periods, the fragility curves for the regular and irregular bridge can be better compared when the acceleration is normalized. For example when Sa(T) is twice the spectral acceleration at the MCE level, the probability of collapse is around 15% and 30% for the regular and irregular bridge, respectively. This demonstrates that the irregularities in column heights can increase the probability of collapse and therefore the probability of collapse can be very different for different bridge configurations. Nevertheless the probability of collapse at the MCE level (i.e., 2% in 50 years), given in Tables 7.1 and 7.2, is reasonably low (e.g., less than 10%) for both bridges. The probability of exceeding different damage states for the regular and irregular bridges also indicates that the bridge irregularity has more significant effects on the higher damage states (e.g., collapse and bar-buckling).

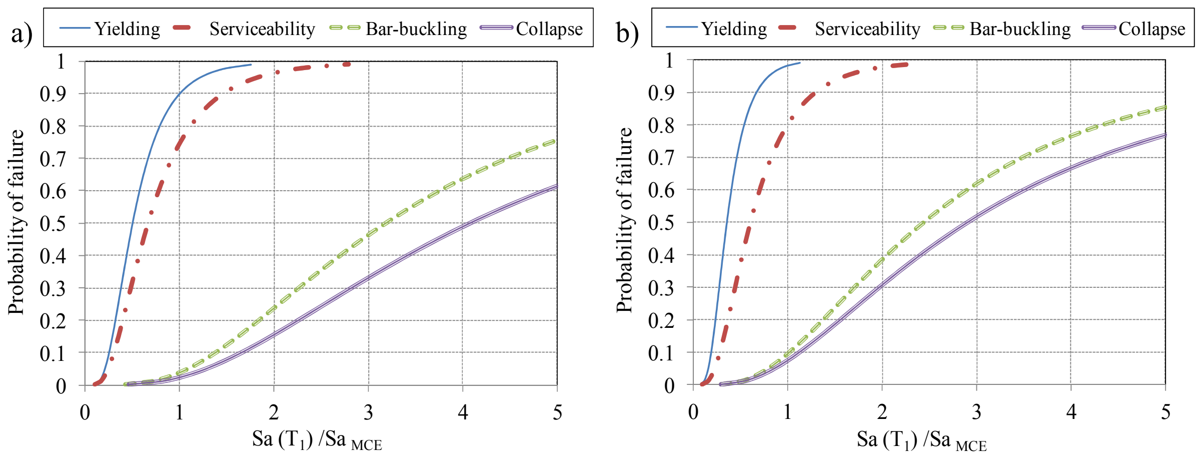

Fig. 7.2. Fragility curves for different damage states for: a) the regular bridge; b) the irregular bridge

8. INCREMENTAL DYNAMIC ANALYSIS USING 2D MODELS

To compare the results obtained from the 2D and 3 D analyses the 3D structural models were also subjected to the horizontal component of the ground motions in the transverse and longitudinal directions independently. The first forty-four horizontal components of the records were chosen for this case to perform the IDA. These records are the same as the far-field record set used in ATC-63 provisions (ATC-63, 2008). The IDA curves for the regular and irregular bridges in the transverse and longitudinal directions are presented in Figs. 8.1 and 8.2 and the IDA results are summarized in Tables 8.1 and 8.2. For the case of the 2D analysis, the x% percentiles of DM|IM (as shown in the IDA graphs) were the same as the (100−x)% percentiles of IM|DM (as presented in the tables).

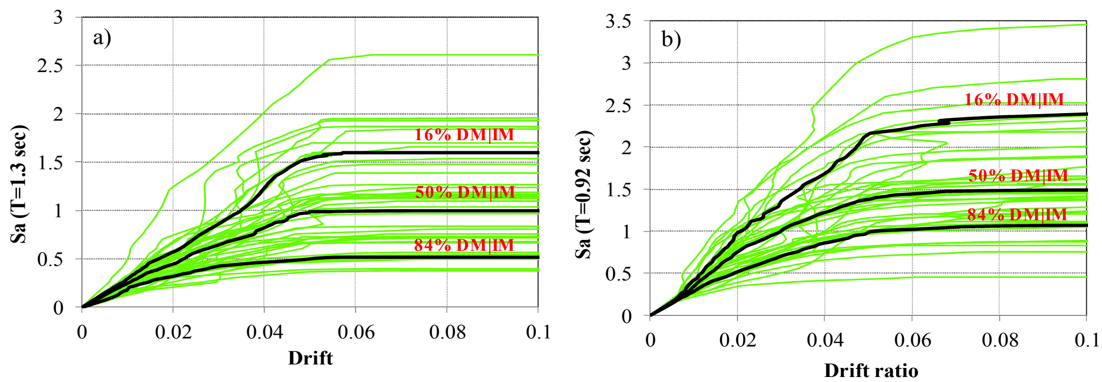

Fig. 8.1. IDA curves for the regular bridge for: a) transverse direction; b) longitudinal direction

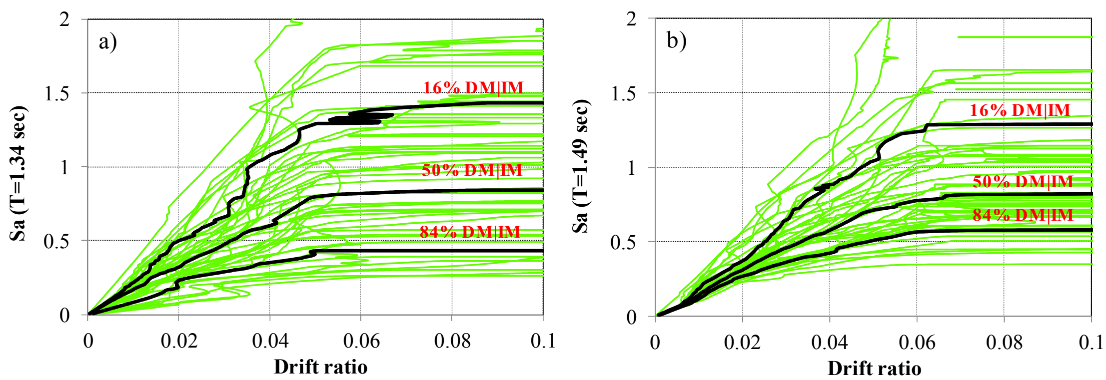

Fig. 8.2. IDA curves for the irregular bridge for: a) transverse direction; b) longitudinal direction

The probability of exceeding different damage states are also presented in the tables for the controlling case. A comparison of the results obtained from the 3D analysis (e.g., Tables 7.1 and 7.2) with those obtained from the 2D analysis (e.g., Tables 8.1 and 8.2) demonstrates large differences (i.e., in order of 50% difference in collapse capacity). The probabilities of collapse at MCE level are close to zero using 2D analysis as presented in Tables 8.1 and 8.2.

Table 8.1. IDA results and estimated probability of exceeding different damage states for the regular bridge

	Percentiles (g) for the transverse direction (T= 1.3 sec)				Percentiles (g) for the longitudinal direction (T= 0.92 sec)					Probability of failure at different hazard levels (for the controlling case)		
Damage state	50%	16%	84%	St Dev	50%	16%	84%	St Dev	β_{TOT}	40% in 50 years	10% in 50 years	2% in 50 years
Yielding	**0.122**	0.07	0.153	0.30	**0.149**	0133	0.162	0.10	0.48	0.00%	1.09%	54.07%
Serviceability	**0.271**	0.216	0.328	0.26	**0.373**	0.306	0.44	0.22	0.54	0.00%	0.01%	5.52%
Bar-buckling	**0.93**	0.48	1.45	0.45	**1.32**	0.93	1.92	0.39	0.65	0.00%	0.00%	0.07%
Collapse	**1.01**	0.54	1.62	0.47	**1.52**	1.11	2.44	0.43	0.66	0.00%	0.00%	0.05%

Table 8.2. IDA results and estimated probability of exceeding different damage states for the irregular bridge

	Percentiles (g) for the transverse direction (T= 1.34 sec)				Percentiles (g) for the longitudinal direction (T= 1.49 sec)					Probability of failure at different hazard levels (for the controlling case)		
Damage state	50%	16%	84%	St Dev	50%	16%	84%	St Dev	β_{TOT}	40% in 50 years	10% in 50 years	2% in 50 years
Yielding	**0.102**	0.091	0.113	0.20	**0.061**	0.055	0.063	0.13	0.49	0.00%	8.42%	85.05%
Serviceability	**0.202**	0.158	0.239	0.28	**0.159**	0.131	0.206	0.22	0.52	0.00%	0.08%	19.12%
Bar-buckling	**0.761**	0.382	1.287	0.53	**0.674**	0.489	0.979	0.37	0.71	0.00%	0.00%	0.38%
Collapse	**0.86**	0.49	1.44	0.53	**0.82**	0.58	1.30	0.45	0.71	0.00%	0.00%	0.20%

The bidirectional effects are expected to be somewhat larger for the bridges in this study (e.g., compared to buildings or multi-column bent bridges) that only have a few columns, which are shared by the moment resisting frames in the two perpendicular directions. To include the bidirectional effects in design, when a 3D analysis is performed using pairs of records, typically the seismic forces in one direction are combined with 30% of the seismic forces in the perpendicular direction (and vice versa). Therefore a 2D nonlinear analysis for a bridge that has been designed with the 100%-30% rule may give a false sense of the capacity and ductility demands, since only one component of the ground motion is applied. For the bridges studied in this research the minimum reinforcement ratio was controlling for all cases (even when the 100% - 30% rule was used in design). As a result, a part of the differences between the 2D and 3D analyses in this study is due to the equal strengths of columns in both cases. It should be noted that the results of the 3D analyses are also dependent and sensitive to the assumptions made in the computer software used for analysis regarding the interaction of the strength and stiffness in the two directions as the yield forces change (especially when strength degradation is also considered). Larger P-Δ effects in the 3D analyses (especially at higher seismic intensities) may also be involved. More research is needed on this subject. However, according to Priestley et al., (2007) the current state of the art is to consider the ductility effects independently in the orthogonal directions for straight bridges.

9. CONCLUSIONS

The seismic performance of a regular and an irregular bridge were studied using incremental dynamic analysis. The probability of exceeding different damage states at different hazard levels was also predicted. IDA was performed using 2D and 3D modelling and the results were compared. The results indicated that the use of 3D models to perform the IDA resulted in much lower predictions of the structural capacity. Possible issues which can justify the differences were described. The use of pair of ground motions in IDA resulted in complex IDA curves, so that the IM|DM and IM|DM percentiles resulted in different predictions. While the application and scaling of pairs of ground motion records in IDA for 3D analyses may introduce a conservative bias, the use of 2D analyses may also somewhat overestimate the capacity of the structure. The results from the 3D analyses were found somewhat sensitive to the methods used in the analysis software regarding the interaction of the stiffness and strength. Therefore care should be taken when treating the IDA results using 2D and 3D analyses. More research is needed on this subject. The comparisons of the fragility curves and the IDA results obtained for the case of the regular and irregular bridges studied indicated that the probability of collapse for the irregular bridge was much larger than that of the regular bridge. Nevertheless, the seismic performances of both bridges were satisfactory and the probability of exceeding different damage states including collapse was reasonably low.

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