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STRENGTH AND DEFORMATION BASED SEISMIC PERFORMANCE EVALUATION OF EXISTING BRIDGES

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ABSTRACT

In Turkey, which has a high seismic risk, it is required to design bridge structural systems so as to ensure a sufficient level of safety, as similar to other types of structural systems. In addition, it is of paramount importance to evaluate the seismic safety of existing bridge structural systems. In this context, currently it is often required to resort to deformation based performance evaluation methods in addition to the conventional strength based performance methods.

In this paper, with reference to a particular project commissioned by İstanbul Metropolitan Municipality (İBB) which involved the inspection, evaluation and seismic retrofit of various existing bridges, the seismic performance of bridges have been studied and the employed calculation methods explained. In this study, principles of "DLH Seismic Design Guidelines, General Directorate for Construction for Railways, Harbours and Airports of (RHA) Ministry of Transportation (2008)" have been taken into consideration. As can be followed from the simplified flow chart depicted in Figure 2.1, initially the existing bridge site inspections (cores, soil boring and structural in place measurements, etc) were carried out, followed by the establishment of structural calculation models.

In the process which involved strength based evaluation, response spectrum method, and on the other hand in the case of deformation based evaluation pushover analysis and nonlinear time history analysis have been performed. In the case of nonlinear analysis frame and finite element models have been used. By taking into account the particular bridge class and the expected performance level corresponding to this class, the seismic performance of bridges studied have been evaluated and those structures which exhibited insufficient level of seismic safety have been retrofitted.

1. INTRODUCTION

Istanbul is located in a zone with a high degree of seismicity and at the same time a high density of population. The provision of the transportation system needed for the search, rescue and evaluation activities and the transportation of vital material, equipment and supplies is of paramount importance considering the case of a major earthquake. It thus follows that the whole transportation system and its critical sections such as bridges constitute one of the vital systems which should be given due attention in the process of taking the required anti seismic measures.

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Within the scope of the Project implemented by Istanbul Metropolitan Municipality (İBB) some bridges in the Istanbul metropolitan area have been inspected and evaluated by Emay International Engineering and Consultancy Inc., and it has been aimed to provide a sufficient degree of structural safety by preparing detailed and final seismic retrofit design for some of those bridges which required to be strengthened.

Besides the ground motion effect, the significance of the bridge with respect to transportation and structural risk are factors which determine the importance of a bridge from the point of an earthquake. The importance of a bridge with respect to transportation in an urban area is determined by the importance and density of traffic on the particular highway, whether or not the road alignment is of emergency line and whether or not there are alternative alignments. In addition, whether or not the bridge structure is large, old and/or damaged is another factor which contributes to structural risk; such circumstances do exist in the bridges inspected and those for which seismic retrofit design has been prepared. Bridges in general proved to have insufficient degree of structural seismic safety due to the fact that existing specifications and codes used in the design of existing bridges include low seismic coefficients and do not attribute the necessary importance to details which would enhance ductility (lap lengths, confinement and other details) and that they are based upon the principles of linear elastic (working stress) design method rather than the principles of performance based method. In addition, unsatisfactory supervision of material, construction quality and workmanship, the use of low standard material class, vehicular impact damage and insufficiency due to general wear or external effects are structural weaknesses observed in the bridges inspected.

In the process of evaluation and seismic retrofit of bridges, the seismic technical code with regard to the DLH Seismic Design Guidelines, General Directorate for Construction for Railways Harbours and Airports of (RHA) Ministry of Transportation (2008) which include the strength and deformation based calculation approaches has been used. Since all projects in this Project were classified as “special bridge”, multi mode spectral analysis has been carried out and minimum damage (MD) performance level was checked for D2 seismic level by using the strength based calculation method. In addition, nonlinear methods (such as push-over or time history analysis) were used for D3 seismic level and limited damage (LD) performance level was checked by using the deformation based evaluation method.

2. PRINCIPLES OF BRIDGE SEISMIC EVALUATION AND CALCULATION METHODS EMPLOYED

2.1. PRINCIPLES OF INITIAL EVALUATION

Initial evaluation reports which include the result of studies concerning the geometrical properties of bridges, structural systems, soil conditions and extent of damage accompanied by relevant recommendations have been prepared and submitted. The material test results (concrete core specimen and/or steel material specimen), soil boring results, foundation base investigation by georadar, three-dimensional mapping and damage inspection have all been used as an input for the subsequent detailed analysis conducted so as to evaluate the structure.

2.2. PRINCIPLES OF DETAILED EVALUATION

In accordance with the information and input data acquired during the initial evaluation stage, an analytical model of the bridge has been established and by adopting the DLH seismic technical code, the load carrying capacities of bridges under seismic loading have been checked. In accordance with the results of the detailed examination, the seismic safety of the structures has been checked by evaluation, in terms of both strength and deformation. For those structures observed to have adequate degree of seismic strength, drawings for bridge repair were prepared and for others observed to be insufficient, various alternative retrofit methods were considered. By using such alternatives trial checks were repeated until the structure proved to be seismically safe.

In cases for bridges where such retrofit measures technically included great difficulties and/or proved to be uneconomical , the replacement of the bridge have been proposed as an alternative in a comparative study. Flow chart for the evaluation of bridges under seismic loading is given in Figure 2-1.

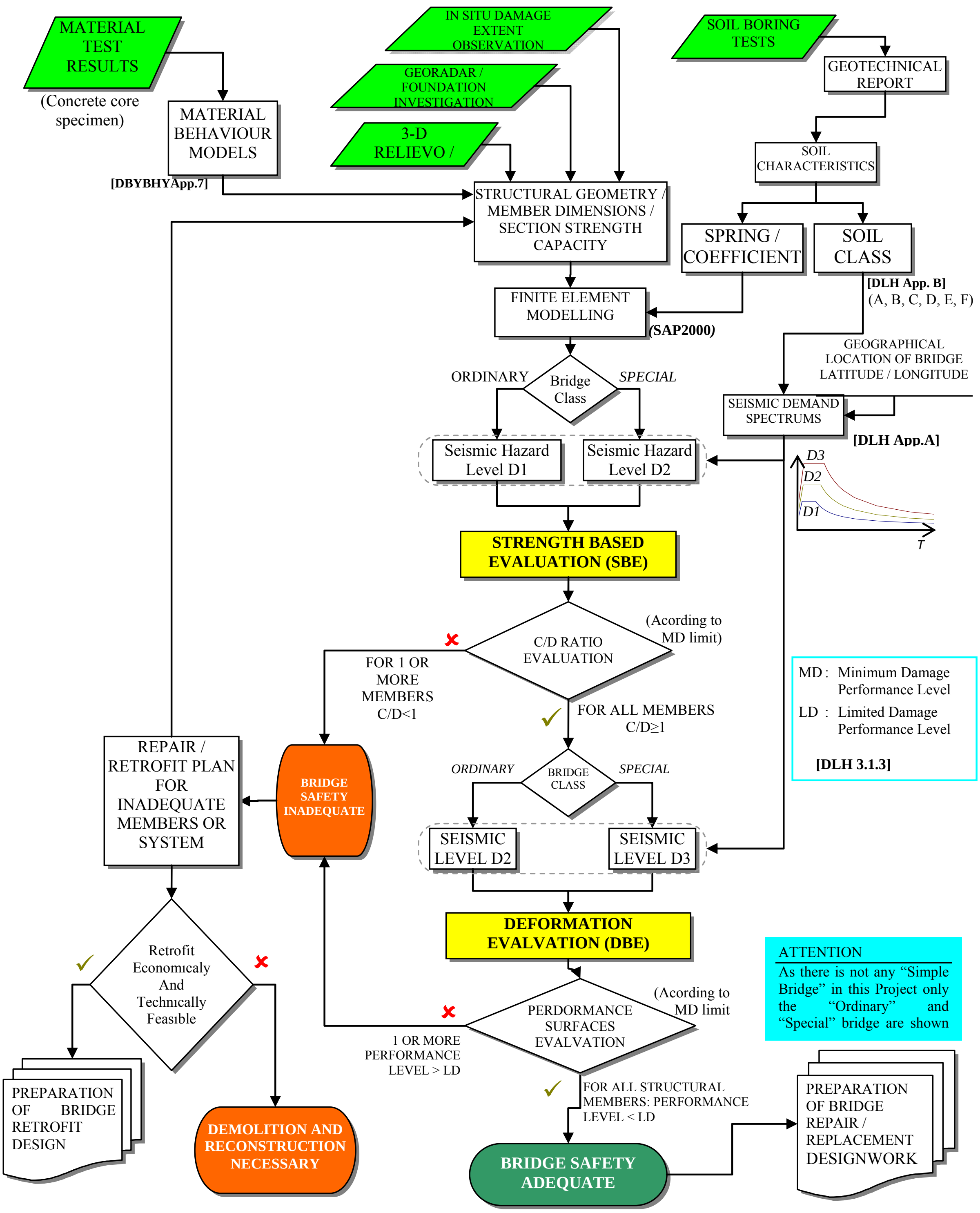


Figure 2.1. Flowchart relating to seismic performance evaluation of bridges

2.2.1. BRIDGE CLASSIFICATION

The DLH 2008 Code has been adopted for the evaluation of existing bridges. Hence, following the classification of each bridge as “special bridge”, “ordinary bridge” or “simple bridge” the evaluation of the bridges was completed by using the DLH 2008 earthquake levels (such as D1, D2 or D3). In the code, such bridge classes are defined as follows:

Special Bridges:
- Bridges located on strategic highway sections
- Critical bridges expected to be used immediately after an earthquake

Ordinary bridges:
- Bridges which are neither “special” nor “simple”

Simple bridges:
- Single span bridges which are not “special” with span length less than 10m
- Bridges which are not “special” located on areas with effective ground acceleration less than 0.1g

2.2.2. SEISMIC PERFORMANCE SPECTRUM

The seismic performance spectrum has been prepared by the method depicted in DLH 2008 Code Appendix A by using the bridge coordinates and adjusted in accordance with the soil class concerned. In the performance based bridge calculations 3 separate seismic levels were taken into account, namely D1, D2 and D3 levels (DLH 2008 1.2.1). D1 seismic level has the lowest intensity but the highest probability of occurrence, whereas D3 seismic level has the highest intensity with the lowest probability of occurrence. The return periods of D1, D2 and D3 earthquakes are 72 years, 475 years and 2475 years, respectively.

Table 2.1. Seismic Hazard Levels (DLH 2008,1.2.1.)

Seismic hazard level	D1	D2	D3
DLH code article	1.2.1.1	1.2.1.2	1.2.1.3
Excedence probability within 50 years	50%	10%	2%
Return period	72 years	475 years	2475 years

The spectral acceleration values depicted in DLH 2008 Appendix A have been given for both S_s (short period spectral acceleration) and S_1 (spectral acceleration for 1 second period), soil class B.

The S_s and S_1 values derived from coordinates are multiplied by adjustment coefficients in accordance with DLH 2008 Code Table 1.1 and Table 1.2 by using the soil class and spectral acceleration values. The seismic spectrum has been established from equations given in DLH Code. The general form of this spectrum is shown in Figure 2.2.

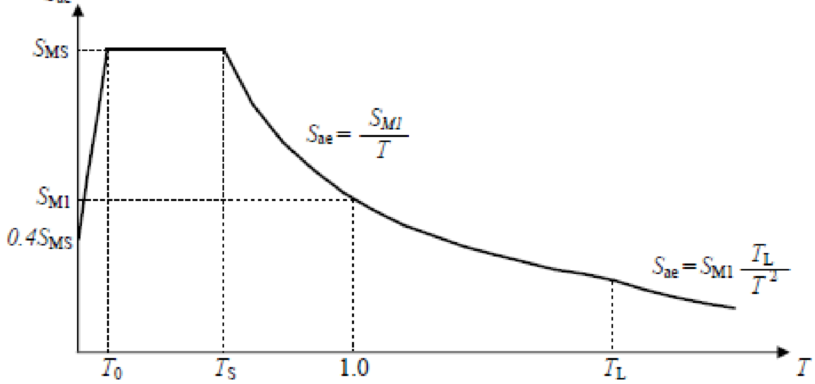


Figure 2.2. Seismic design response spectrum [DLH 1.2.2.1 & 1.2.2.2]

2.3. METHODS OF DETAILED EVALUATION

Two methods were used in the seismic evaluation of existing bridges: Strength based evaluation (SBE) and deformation based evaluation (DBE) (DLH Code 2008, 3.1.5). The seismic safety level of the structure was determined by checking the seismic performance of bridge and bridge members derived from analyses performed previously against performance limits given the DLH Code. Minimal damage (MD) and limited damage (LD) performance level limits were used in the evaluation. Minimal damage performance level in such that either no damage at all is caused or very limited amount of damage occurs due to seismic actions on the structure. In this case either traffic flow continues uninterrupted or problems which might arise can easily be removed within a few days (DLH Code 2008 3.1.3.1). A limited (or controlled) damage performance level (LD) however, is defined as a damage level where damages occurring due to seismic actions are permitted, provided that such damages are not structurally very serious and can be repaired. In this case, it is reasonable to expect short period interruptions in bridge operations lasting a few days or weeks. (DLH Code 2008, 3.1.3.2).

Table 2.2 shows the required evaluation method and the seismic performance level to be used for a given bridge class (DLH Code 2008, Table 3.1 and 3.2)

Table 2.2. DLH Code Evaluation Methods with Respect to Bridge Class [DLH Table 3.1 & 3.2]

	Essential bridges		Ordinary bridges		Simple bridges
Seismic hazard level	D2	D3	D1	D2	D2
Evaluation method	SBE	DBE	SBE	DBE	SBE
Performance level	MD	LD	MD	LD	LD

2.3.1. STRENGTH BASED EVALUATION

In the strength based evaluation method (SBE), starting with the linear elastic behaviour of the structure, seismic effects are determined and these effects are evaluated by capacity/demand (C/D) ratio. The capacity/effect ratio method is used in accordance with the definitions made in FHWA Seismic Retrofit Manual 2006 section 5.4 method C.

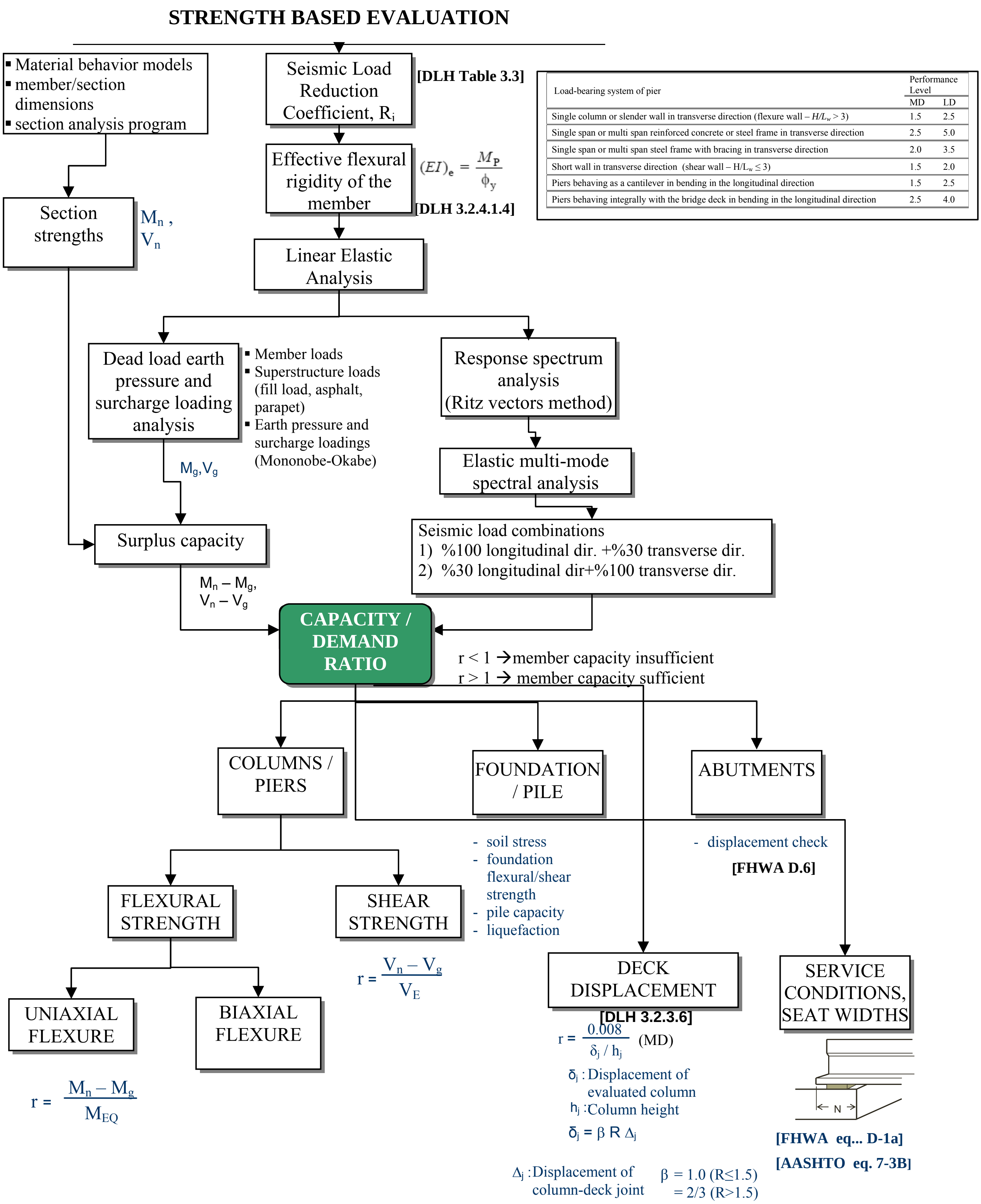
The surplus capacity is to be taken into account as follows:

$$r = \frac{C - D_g}{D_{EQ}} \tag{2.1}$$

where, r is the capacity ratio, C is the section capacity, D_g is the effect of vertical loading (except that due to seismic loading), D_{EQ} is the effect due to seismic loading.

For all bridge members, the capacity is considered adequate where the capacity/effect ratio(r) is greater than 1, is considered inadequate where this ratio is less than 1.

In the strength based evaluation process, the structural analysis was performed by linear- elastic analysis method by a mathematical model developed from the finite element program SAP2000. The seismic effects thus obtained were divided by seismic load reduction factor, R_a in order to take into account nonlinear structural behavior in flexure. (MD) level, has been taken into account by reducing the seismic forces by a certain amount depending on the type and ductility of the respective structural member. In case of flexure the failure mode is of ductile nature, whereas the shear failure is of brittle nature. Hence in the case of shear force evaluation it would not be safe to reduce the seismic forces by reduction factor (R) greater than 1. Therefore, R factor is taken as equal to 1 in the case of shear check. For bridges with different types of piers, the bridge load bearing system performance coefficient is calculated from the weighted average of the piers concerned a flowchart for strength based evaluation is shown in Figure 2.3.



Load-bearing system of pier	Performance Level	
	MD	LD
Single column or slender wall in transverse direction (flexure wall – $H/L_w > 3$)	1.5	2.5
Single span or multi span reinforced concrete or steel frame in transverse direction	2.5	5.0
Single span or multi span steel frame with bracing in transverse direction	2.0	3.5
Short wall in transverse direction (shear wall – $H/L_w \leq 3$)	1.5	2.0
Piers behaving as a cantilever in bending in the longitudinal direction	1.5	2.5
Piers behaving integrally with the bridge deck in bending in the longitudinal direction	2.5	4.0

Figure 2.3. Flowchart for strength based evaluation

2.3.1.1. COLUMN/WALL LOAD CARRYING CAPACITY DUE TO FLEXURE AND SHEAR

In columns and walls, capacity/demand ratio in two dimensional flexure accompanied by a vertical load is determined from a two dimensional bending moment interaction diagram. The bending moment effects in the diagram in both directions due to vertical loads are reduced in order to deduce the required reduced capacity. The capacity/demand ratio can be found by dividing the reduced flexural capacity ("surplus load") by the resultant of seismic moments. This procedure is explained in Figure 2.4.

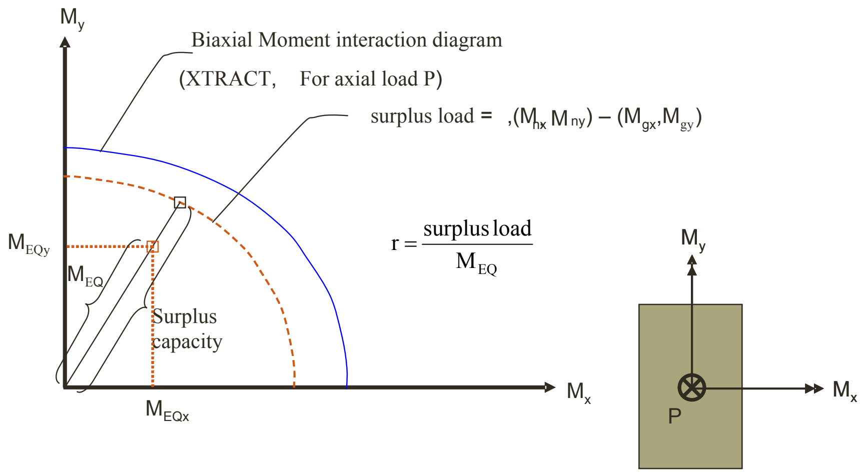


Figure 2.4. Calculation of column capacity/demand ratio

The capacity/demand ratio of bridge columns in shear is determined from the column surplus shear capacity of shear forces derived from the analysis.

2.3.1.2. COLUMN/WALL RELATIVE DISPLACEMENT LIMIT

For columns, relative displacement capacity/demand ratio is evaluated, in addition. The displacement limits at the column deck joint zone is given in the DLH 2008 Code (DLH Code 2008,3.2.3.6).

$$\delta_j = \beta R \Delta_J \tag{2.2}$$

where, δ_j is the displacement which the evaluation be based upon, Δ_j is the pier-deck joint displacement, R is the behavior performance coefficient (R will be taken as equal to 1 provided that the displacements calculated from the analysis model are not reduced, and coefficient β=1 for R=1.5, otherwise β=2/3).

The displacement subject to evaluation so calculated were divided by column heights in order to obtain relative displacements (δ_j / h_j). The relative displacement limits are given as 0.008 for minimal damage and 0.015 for limited (controlled) damage situation (DLH 2008, 3.2.3.6).

The displacement capacity/demand ratio(r) was found as follows;

$$r = \text{limited relative displacement/calculated relative displacement} \tag{2.3}$$

2.3.1.3. SEISMIC LOAD REDUCTION COEFFICIENT

In the course of the evaluation of bridge piers and abutments by the strength based evaluation method (SBE), the influence of possible nonlinear behavior which may result in energy damping is taken into account by reducing the seismic forces by certain amounts depending on the type and ductility of the structural member.

For bridges with different types of piers, the bridge load bearing system performance coefficient is calculated from the weighted average of the piers concerned. These coefficients are given in the preceding Table 2.3 [DLH 2008, 3.2.3.2.1].

Where $R \leq 1.5$, the seismic load reduction factor is taken as R_a (T) = R. For cases where $R > 1.5$ however, R_a (T) is determined based on the natural vibration period T as $1.5 + (R - 1.5) \times (T/T_S)$ [DLH 3.2.3.2.2], where T is the first natural vibration period of the bridge load bearing system and T_s is the spectrum corner period.

Table 2.3. Pier/ Abutment Behaviour Coefficients [DLH 2008, Table 3.3]

Load-bearing system of pier	Performance Level	
	MD	LD
Single column or slender wall in transverse direction (flexure wall – $H/L_w > 3$)	1.5	2.5
Single span or multi span reinforced concrete or steel frame in transverse direction	2.5	5.0
Single span or multi span steel frame with bracing in transverse direction	2.0	3.5
Short wall in transverse direction (shear wall – $H/L_w \leq 3$)	1.5	2.0
Piers behaving as a cantilever in bending in the longitudinal direction	1.5	2.5
Piers behaving integrally with the bridge deck in bending in the longitudinal direction	2.5	4.0

2.3.2. DEFORMATION BASED EVALUATION

The deformation based evaluation method, although much more complicated and tedious than the linear- elastic calculation method, is regarded as more realistic since it takes into account the nonlinear behaviour arising when the structural members and their connections approach, reach or exceed their strength capacity. In this method, materials and connections are defined by nonlinear stress-deformation relationships and the relevant calculations are performed in accordance with the structural displacements. This approach is also known as “performance based evaluation”. The calculated values of material deformation in the members derived from seismic analysis are compared against performance limits. The performance limits to be used were taken from the appropriate sections of DLH code. It is envisaged to use limited damage (LD) performance limit for the deformation based evaluation method.

The nonlinear behavior of frame members is defined by plastic hinges in which nonlinear deformations are concentrated, whereas for the wall (plate) members it is defined by sections of nonlinear layers. The plastic behavior which is expected to occur at the end sections of frame members where the maximum forces may be observed is determined by the section yield surface at the particular point. The yield surface defines the normal force P and bending moments M_1 , M_2 at both perpendicular axes which represent the start of hinging and. This relationship can be obtained by various methods including section moment-curvature analysis or section fiber analysis. In the two-dimensional analysis, the yield surface changes into yield curve, related to normal force P and bending moment M. In this study the yield curve and hence the plastic hinge definition has been automatically defined in accordance with FEMA356 hinges defined in SAP2000 program and section characteristics. The sections where plastic hinges would develop are considered to be at the ends of members. The plastic hinge lengths were calculated in accordance with FWHA 7.8.1.1 and DLH 2008 3.2.4.1.1.

Column/wall plastic hinge length:

$$L_p = 0.08H + 0.0022 f_{yk} d_b \geq 0.044 f_{yk} d_b \qquad (2.4)$$

Beam and pile plastic hinge length:

$$L_p = 0.5H \qquad (2.5)$$

where, H (mm) is the pier height, f_{yk} (Mpa) is the characteristic yield stress of the reinforcing bar, d_b (mm) $_{is}$ the diameter of bar and h (mm) is the cross-section dimension in the direction considered.

2.3.2.1. ANALYSIS METHODS FOR DEFORMATION EVALUATION

Incremental pushover analysis can be performed for deformation evaluation, provided that the first (prevalent) vibration mode effective mass participation ratio is greater than 70 percent (DLH Code 2008 4.4.4). In cases where this condition is not satisfied or for complicated and/or curved bridges requiring a more comprehensive study, a nonlinear time history analysis method has been employed. The incremental pushover analysis method has been used in calculations in accordance with DLH 2008 Code Section 3.2.4. A minimum of 7 sets of seismic records were employed which represent design earthquake spectrum for the time history analysis. The average values of internal forces, displacements and deformation effect, derived from the time history analysis result based on the 7 earthquake records were used in the evaluation.

2.3.2.2. THE EVALUATION OF MEMBER DEFORMATIONS DUE TO SEISMICLOAD

The unit deformation limits for the respective performance limit can be taken from Table 2.4 given below (DLH 2008, Table 3.4).

The unit deformations given in the table have been transformed into the equivalent curvature and/or rotation values by using the section moment-curvature (M-K) analysis.

Table 2.4. Unit Deformation Limits Defined for Pier Plastic Sections [DLH 2008 Table 3.4]

Unit deformation	Performance level	
	MD	LD
Unit deformation of concrete in compression, ε_c	0,004	0,020
unit deformation of reinforcing steel, ε_s	0,010	0,040

$$LD = 0.004H + 0.016(\rho_s / \rho_{sm}) \le 0.02 \qquad (2.6)$$

where, ρ_s: transverse reinforcement ratio of the column, ρ_{sm}: minimum transverse reinforcement ratio. Unit deformation of concrete in compression e_c for the limited damage performance level (LD) is taken as a variable ratio between 0.004 and 0.02 relating to the transverse reinforcement ratio.

Deformation based evaluation flow chart is depicted in Figure 2.5. Relating Üsküdar Haydarpaşa Bridge, Figure 2.6 shows the general view of the bridge; Figure 2.7 and 2.8 gives an impression of the initial in situ works, and the summary of the detailed evaluation results depicted in Figure 2.9.

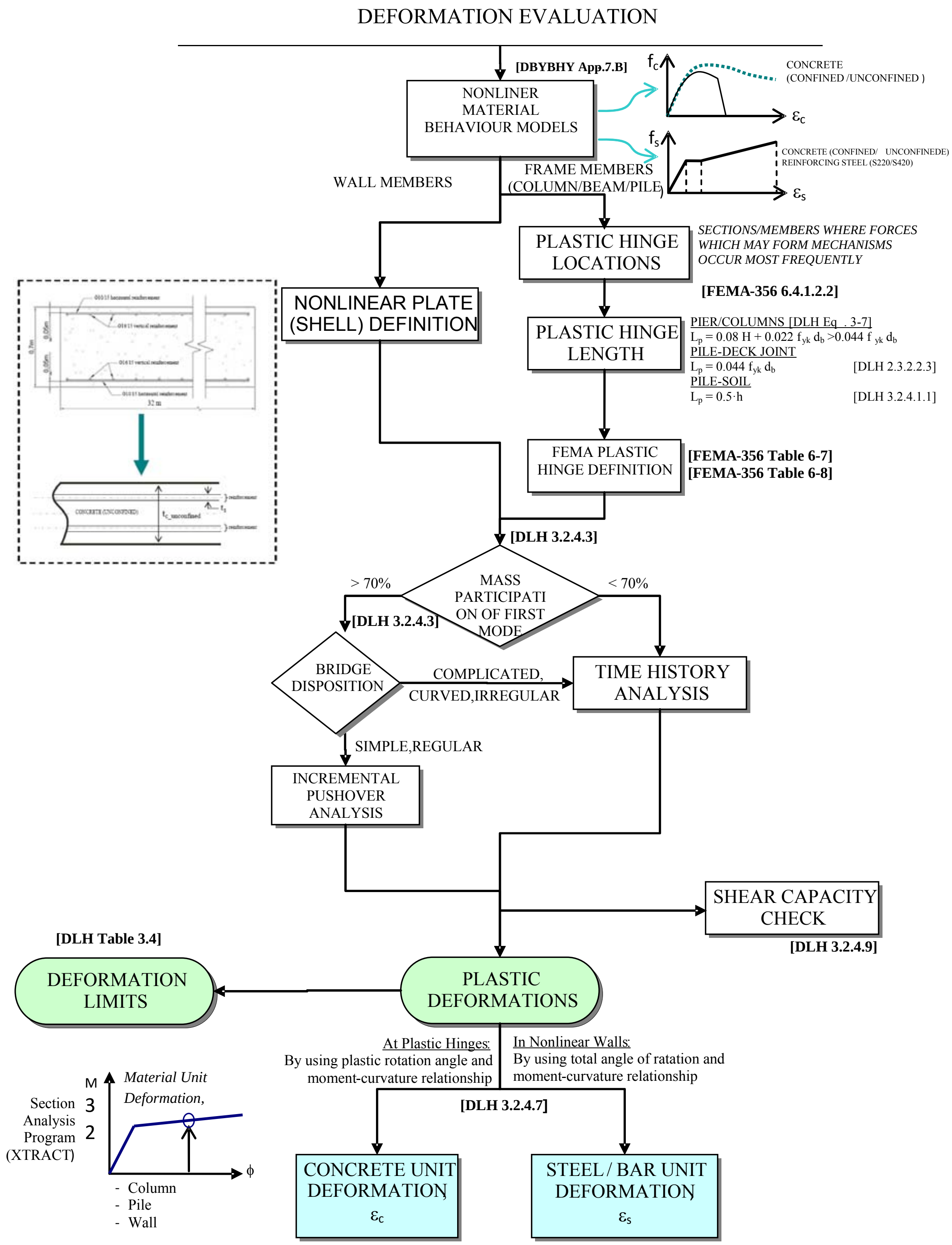


Figure 2.5. Flowchart for deformation based evaluation

Figure 2.6. General view of Üsküdar-Haydarpaşa Overpass Bridge

Figure 2.7. Extraction of concrete core specimens from the pier for material quality test (Üsküdar-Haydarpaşa Overpass Bridge)

Figure 2.8. Measurement of existing steel rebar diameters at piers. (Üsküdar-Haydarpaşa Overpass Bridge)

- STRENGTH BASED EVALUATION

  Linear Elastic Spectral Analysis
  - Flexural moment
    - *Pier column bottom end: INSUFFICIENT*
    - *Pier column top end: INSUFFICIENT*
  - SHEAR FORCE
    - *Pier column bottom end: INSUFFICIENT*
    - *Pier column top end: INSUFFICIENT*
  - DECK DISPLACEMENT
    - *INSUFFICIENT*
  - FOUNDATION CHECKS
    - *SUFFICIENT*
- DEFORMATION BASED ANALYSIS

  NONLINEAR ANALYSIS
  - MATERIAL DEFORMATION LIMITS(COLUMNS)
    - *concrete: INSUFFICIENT*
    - *reinforcement: INSUFFICIENT*

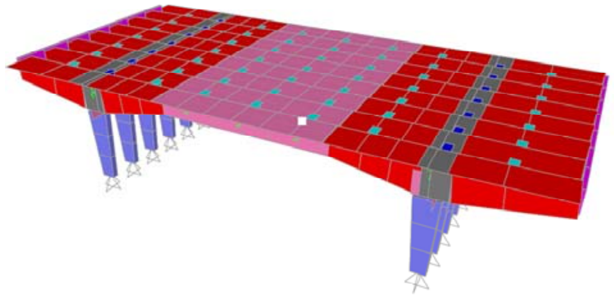


Figure 2.9. SAP2000 mathematical model and detailed evaluation results (Üsküdar-Haydarpaşa Overpass Bridge)

CONCLUSION

In this paper, with reference to a particular project commissioned by İstanbul Metropolitan Municipality (İBB) which involved the inspection, evaluation and the repair and/or seismic retrofit design of various existing bridges, the seismic performance of bridges have been studied and the calculations methods employed are explained. In this study, principles of “DLH Seismic Design Guidelines, General Directorate for Construction for Railways Harbours and Airports of (RHA) Ministry of Transportation (2008)” have been taken into consideration. Within the scope of this Project, the studies involved strength based evaluation, response spectrum method. On the other hand, in the case of deformation based evaluation, pushover analysis and nonlinear time history analysis have been performed. In the case of nonlinear analysis, frame and finite element models have been used. By taking into account the particular bridge class and the expected performance level corresponding to this class, the seismic performance of bridges studied have been evaluated and those structures which exhibited insufficient level of seismic safety have been retrofitted. It is expected by the authors that, considering the high seismicity of our country such evaluations and studies should be generalized in order to minimize the undesirable results of the earthquake effects.

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