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SEISMIC RETROFIT OF REINFORCED CONCRETE BRIDGES BY USING STRUCTURAL STEEL ELEMENTS

E. Namlı[1] , D.H.Yıldız.[2], M.Erinçer.[3], E.Özer.[4]

ABSTRACT

Within the scope of a Project commissioned by İstanbul Metropolitan Municipality (İBB), it has been aimed by Emay International Engineering and Consulting Inc. to evaluate the seismic load carrying capacity of 14 critical bridges and if so required to perform seismic retrofit design so as to ensure a sufficient level of seismic safety. In this context, seismic retrofit design methodology has been developed so as to ensure a required seismic safety level for such bridges which were found to be seismically inadequate as a result of the previous evaluation stage. In this paper, the evaluation and retrofit work implemented on Üsküdar Haydarpaşa Overpass Bridge is presented. In the process of seismic retrofit design work, it is envisaged to strengthen the existing weak reinforced concrete beam–column connections by using steel gusset plate elements and high strength bolts. Furthermore, the strengthened joints have been checked in accordance with the capacity design principles.

[1] MSCE, Emay International Engineering and Consultancy Inc., İstanbul
[2] MSCE, Emay International Engineering and Consultancy Inc., İstanbul
[3] MSCE, Emay International Engineering and Consultancy Inc., İstanbul
[4]Professor, Dept. of Civil Engineering, Istanbul Technical University, İstanbul

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ABSTRACT

Within the scope of a Project commissioned by İstanbul Metropolitan Municipality (İBB), it has been aimed by Emay International Engineering and Consulting Inc. to evaluate the seismic load carrying capacity of 14 critical bridges and if so required to perform seismic retrofit design so as to ensure a sufficient level of seismic safety. In this context, seismic retrofit design methodology has been developed so as to ensure a required seismic safety level for such bridges which were found to be seismically inadequate as a result of the previous evaluation stage. In this paper, the evaluation and retrofit work implemented on Üsküdar Haydarpaşa Overpass Bridge is presented. In the process of seismic retrofit design work, it is envisaged to strengthen the existing weak reinforced concrete beam–column connections by using steel gusset plate elements and high strength bolts. Furthermore, the strengthened joints have been checked in accordance with the capacity design principles.

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 within a very short period of time 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 bridge, constitute one of the vital systems which should be given due attention in the process of taking the required anti seismic devices.

Within the scope of the Project implemented by İstanbul Metropolitan Municipality (İBB), certain 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 requiring to be strengthened.

Üsküdar Haydarpaşa Overpass Bridge carries the traffic flow coming from Üsküdar direction towards Haydarpaşa direction. The bridge was built in 1957; the length and width of the structure is 40,4m and 20m respectively. The reinforced concrete bridge structure consists of 7 identical frames. The structure has a monolithic (fixed) type of beam-column connection. The bridge has 3 spans, the central (main) span being 25,00m. There are 2 identical cantilever

[2] MSCE, Emay International Engineering and Consultancy Inc., İstanbul
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wide spans. The main girders are 0,45m wide and has a variable depth between 1,33-2,70m. The deck of beams is 22cm thick. The columns have a variable cross-section with a height of 5,62m. There is a hinged connection between the columns and the isolated footings, allowing free rotation. Figure 1.1 shows the general view of the bridge. The longitudinal section of the bridge structure with general dimensions is given in Figure 1.2.

Upon an examination of the existing structural drawings, the following information was obtained. The reinforcement of the beam: at the support section 16ϕ32 top, 2ϕ32 bottom; at midspan section (main span) 16ϕ32 bottom, 2ϕ32 top. At the columns the vertical reinforcement of 10ϕ32 at face overlooking the side (cantilever) span and 5ϕ32 overlooking the main span.

Figure 1.1. General view of the bridge

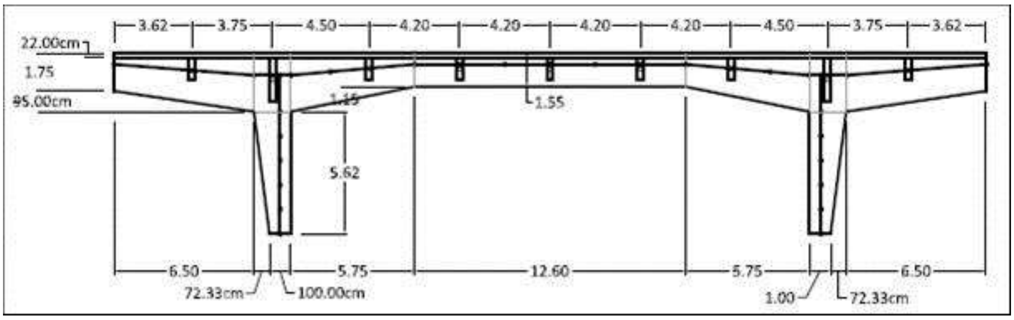

Figure 1.2. The longitidunal section of the structure

2. Preliminary Investigations and Tests

The data pertaining to the existing condition of the bridge have been compiled in accordance with the studies, measurements conducted and the results of the material behaviour tests obtained during the preliminary investigation stage. Such studies consist of 3 dimensional mapping/in situ measurement studies, in situ inspection and measurements and detailed damage investigation, material test (carbonation,etc) and concrete core specimen compression tests and other relevant sources.

In this bridge a total of 8 concrete core specimens of 64mm diameter and 77mm length were extracted from the footings, columns and cap beams and subjected to compression strength test in a material laboratory, as a result of which compressive strength values were obtained. The characteristic compressive strength value obtained for (15×15)cm. Cube specimens was 37,47Mpa, with a mean strength value of 43,8 Mpa. The corresponding values obtained for the (15×30)cm cylinders are 31,22 Mpa and 36,5 Mpa respectively. The concrete cover was stripped off on certain points, thus allowing for the direct measurement of rebar diameter (by using a caliper gage) and type. As a result of visual evaluation at the site and an examination of existing drawings, the rebar class (type) was determined to be S220 (yield strength 220 Mpa). The concrete cover thickness was measured to be 35-40mm.

In the course of the study, 2 soil boring tests were conducted with geotechnical purposes on a planned orientation. As a result of soil boring tests it was found that the footings test on a sandstone layer. The soil parameters for sandstone unit has been deduced from the DLH Seismic Design Guidelines, General Directorate for the Construction of Railways, Harbours and Airports of (RHA) Ministry of Transportation, as shown below in Table 2.1 as follows:

Table 2.1. Soil Parameters

Seismic zone	Zone 1.				
Geological unit	Sandstone				
Soil class	B				
Spectrum values	Ss<0.25	Ss=0.50	Ss=0.75	Ss=1	Ss>1.25
	1.0	1.0	1.0	1.0	1.0

3.General Principles of Seismic Evaluation

The DLH Seismic Design Code (2008) has been taken as a basis in the evaluation of bridges. Accordingly, following the classification of bridges as “special”, “ordinary” or “simple”, by using the seismic levels D1, D2 and D3 analyses were carried out so as to make the required evaluations.

Special bridges: Bridges located on strategic highway sections and critical bridges expected to be used immediately after an earthquake.

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

Simple bridges: Single span bridges with span length less than 10m and bridges located on small areas with effective ground acceleration less than 0,1g.

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 seperate seismic levels were taken into account, namely D1, D2 and D3 levels (DLH Code 2008 1.2.1). D1 seismic level has the lowest intensity with the highest probability of occurence, whereas D3 seismic level has the highest intensity with the lowest probability of occurence. The return periods of D1, D2, and D3 seismic hazard levels correspond to return periods of 72 years, 475 years and 2475 years, respectively.

Table 3.1. 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
Probability of excedence within 50 years	50%	10%	2%
Return period	72 years	475 years	2475 years

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

The Ss and S_1 values derived from the coordinates are multiplied by adjustment coefficients in accordance with DLH Code 2008 Table 1.1 and Table 1.2 by using the soil class and spectral acceleration values. The seismic spectrum has been established from the equations given in the DLH Code. The general form of this spectrum is shown in Figure 3.1.

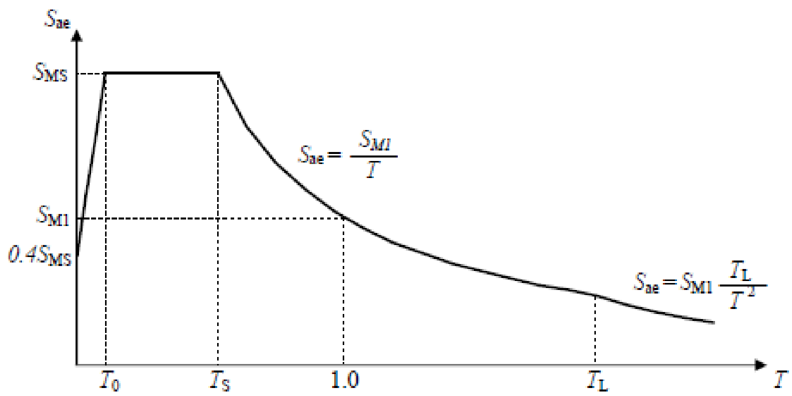

Figure 3.1. Seismic design response spectrum [DLH 1.2.2.1 & 1.2.2.2]

3.1. Methods of Seismic 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 action. 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 structurally not 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 3.2 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 3.2. 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

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 ratio (C/D). The capacity/demand 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:

$$r = \frac{C - D_g}{D_{EQ}} \tag{3.1}$$

where, r is the capacity/demand 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/demand 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 behaviour. In the strength based evaluation method (SBE) for bridge piers, the effect of nonlinear behavior on energy dissipation which might be observed at minimum damage (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. The seismic load reduction factor (R) was determined by virtue of horizontal load carrying system characteristics. However, in the case of shear force evaluation, the seismic load reduction factor (R) is taken as equal to 1, in other words, a nonlinear ductile behavior is not applicable in this case. For bridges with different types of piers, the bridge load bearing system performance coefficient is calculated from the weighted average of the piers concerned.

The linear seismic analysis was carried out by applying seismic response spectrum in both the principal bridge axes.

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 the members is defined as frame members concentrated in plastic hinge formation points, whereas for the wall (plate) members it is defined by sections of nonlinear layers. Plastic hinge formation point is a concept widely used in nonlinear analysis and is frequently named as plastic hinge. The plastic behaviour 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 representing the start of hinging and bending moments M_1 and M_2 at both perpendicular axes. This relationship can be observed 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 occur are determined to be at the ends of members expected to have plastic hinges taken to be at the midpoint of the calculated plastic hinge length. The plastic hinge lengths were calculated in accordance with FWHA 7.8.1.1 and DLH 2008 3.2.4.1.1. The detailed flowchart for the seismic evaluation of bridges is detailed on Figure 3.2.

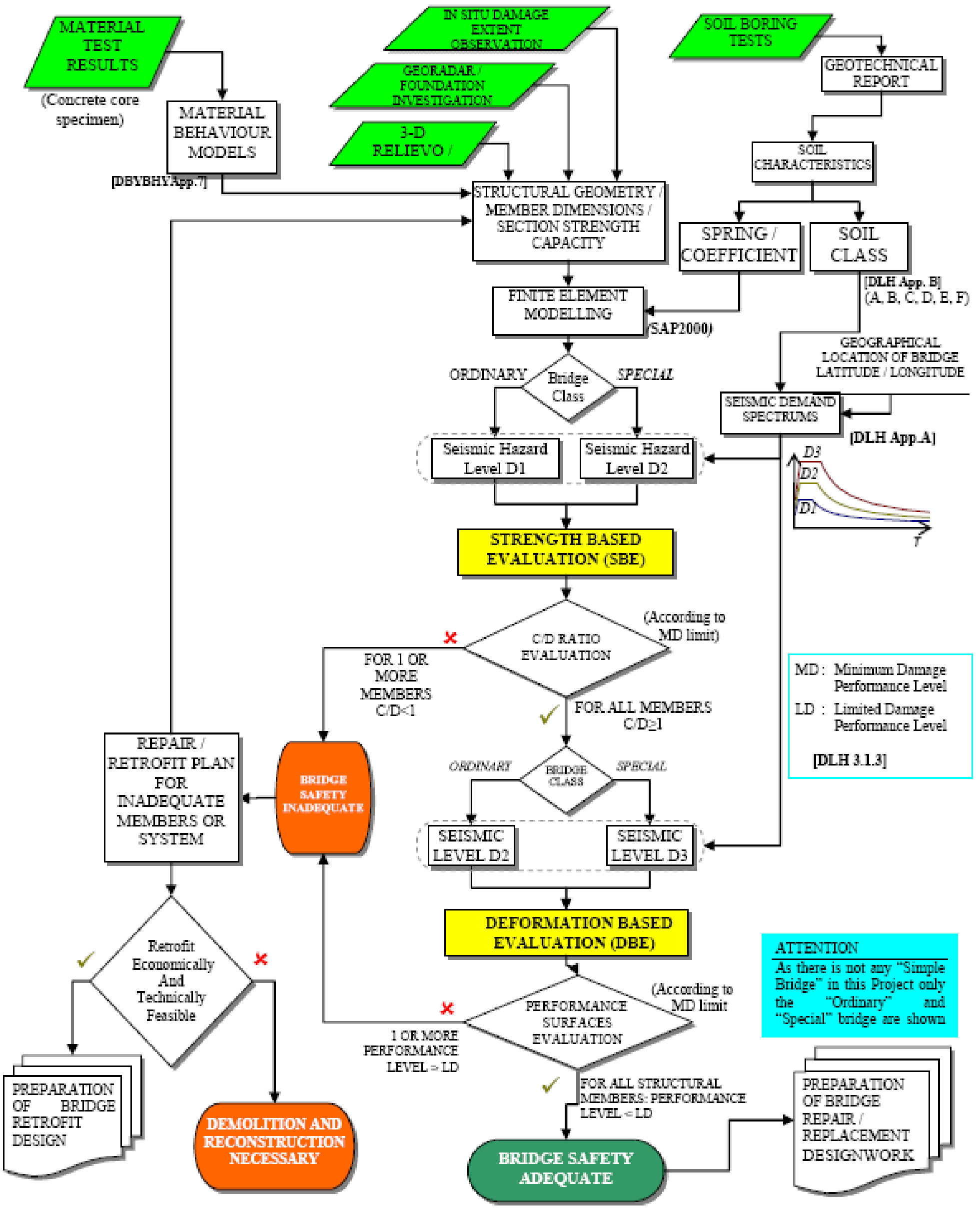

Figure 3.2. Flowchart relating to seismic performance evaluation of bridges

3.2. Analysis Methods

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. A minimum of 7 sets of seismic records were employed which represent design earthquake spectrum for the time history analysis.

The unit deformation limits for the respective performance limit can be taken from Table 3.4 given below (DLH 2008, Table 3.4). The incremental pushover analysis method details are shown in Figure 3.3.

Table 3.4. 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

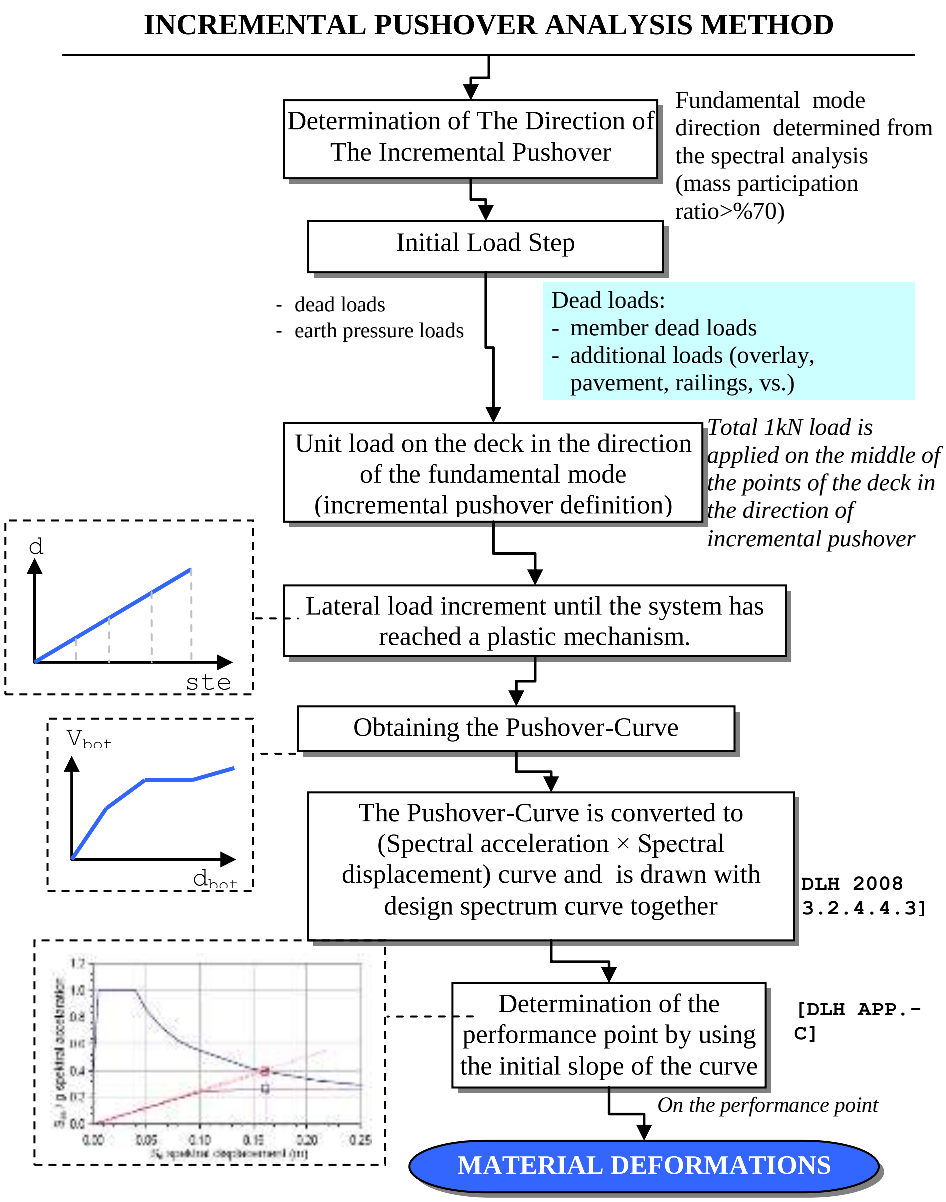

Figure 3.3. Flowchart of the incremental pushover analysis

4. Seismic Evaluation of Bridge Structure

4.1. Structural Model

The structural modelling has been done by using finite element method with SAP 2000 program. Two seperate modelling was used fort the existing seismic capacity of the bridge, namely, one for the SBE and the other for the DBE. For the retrofitted bridge however, these models have been revised to suit for the new condition.

For the SBE analysis a 3-dimensional model has been established with frame members such as main beams, diaphragms and columns. The points around the rigid panel region at the column and beam connection have been defined by ascribing them body constraints. The slab has been represented by defining the main beam cross-sections as a ‘T’ beam. In the diaphragms, the effective slab width was used. In exterior beams the additional weight resulting from the excessive cantilever slab span length has been taken into account by revising section properties. Where main beams and diaphragms do not intersect in the vertical direction the end points are connected by rigid links. Where main beams and diaphragms do not intersect in the vertical direction, the end points are connected by rigid links. In the vertical members the cracked section properties were defined by effective flexural rigidity; in beams however, the cracked section rigidity is taken as 50% of the gross section rigidity. In the existing bridge model fixed bearings in vertical and horizontal direction were adopted at column bases. The end of cantilever beams however are free.

For the DBE analysis the structural system was considered with frame members in a 3- dimensional model. Depending on type of analysis the sections exhibit linear or nonlinear behaviour.

The concrete behaviour (stres-strain) model has been established in accordance with Mander Model as defined in DBYBHY-Appendix 7B. The confined and unconfined concrete behaviour has been determined by this model.

4.2. Evaluation of Results

As a result of detailed investigation of the bridge conducted, the results of SBE are depicted on Table 4.1.

Table 4.1. Evaluation results with respect to strength based analysis

Member evaluated			r (demand/capacity)	Result
Flexural check	Pier columns	Bridge longitudinal direction	1.713	≥ 1.00 adequate
		Bridge transverse direction	0.78	< 1.00 inadequate
Shear check	Pier columns	Bridge longitudinal direction	0.80	< 1.00 inadequate
		Bridge transverse direction	1.60	≥ 1.00 adequate
Isolated footing check	Pier footing	Bridge longitudinal direction	0.56	< 1.00 inadequate
		Bridge transverse direction	32.10	≥ 1.00 adequate
Deck displacement check		Bridge longitudinal direction	1.42	≥ 1.00 adequate
		Bridge transverse direction	0.52	< 1.00 inadequate

Consequently, it has been established that the structural members of the bridge do not possess the required safety in terms of the seismic levels adopted, and that in the transverse direction there is an inadequacy in terms of shear capacity. Furthermore, the soil stresses under the isolated footings exceed the allowable value in the longitudinal direction. The deck displacement did not satisfy the prescribed limit requirements in the transverse direction. The main beams also have inadequate flexural capacity in the longitudinal direction. During the incremental pushover analysis, the system has immediately reached the plastic mechanism before reaching the system performance level point so there is no need to evalute the bridge according to deformation based analysis for the existing system. As a consequence of such SBE and DBE evaluation studies, it has been decided to strengthen pier columns, isolated footing and column-beam connection zone.

5. Repair and Strengthening Measures

As a consequence of SBE and DBE evaluations carried out on the existing bridge it has been decided to strengthen the R.C. pier columns by concrete jacketing. It has been decided further to make the existing notationable column-footing base fixed and to enlarge the footing in the longitudinal direction. The SBE and DBE models were revised to fulfill the retrofitted situations and the evaluations are repeated and DBE evaluation results are depicted on Table 5.1. The application of 25cm jacketing to pier columns and the enlarged footing by anchoring additional rebars to existing concrete and additional concreting proved to be satisfactory. The longitudinal bending reinforcement placed in the jacket at the lower end of columns have been inserted into the footing concrete by the required anchor length, thereby resulting in the fixation of column-footing connection. The strengthened main beams were reevaluated and it has been decided to bolt steel plates to beam concrete both at support and midspan sections.

Table 5.1. Evaluation results with respect to deformation based analysis for the retrofitted bridge

MEMBERS EVALUATED	DEFORMATION BASED EVALUATION	ANALYSIS	LD	RESULT
Pier columns	Concrete	0.00102	0.020	Adequate
	Reinforcement	0.00311	0.040	Adequate
Beams	Concrete	0.00112	0.020	Adequate
	Reinforcement	0.00594	0.040	Adequate

Figure 5.1. shows the strengthening of existing columns and footing by concrete jacketing, and Figure 5.2. shows the strengthening of existing column-beam connections and of beams by additional steel plates.

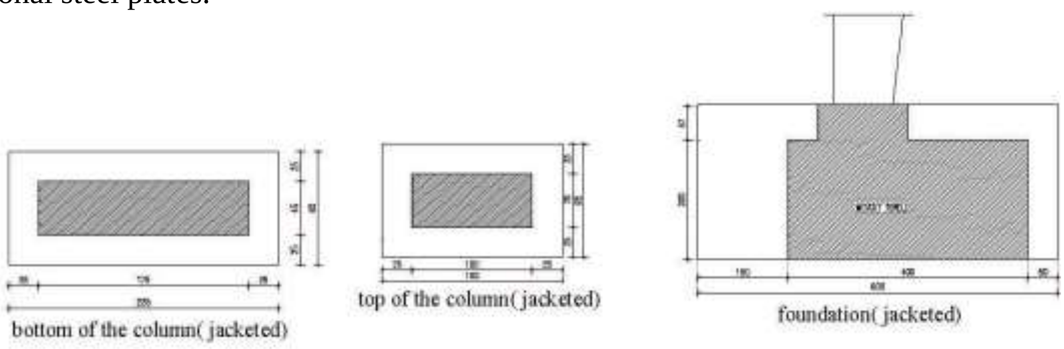

Figure 5.1. Strengthening of existing columns and footing by concrete jacketing

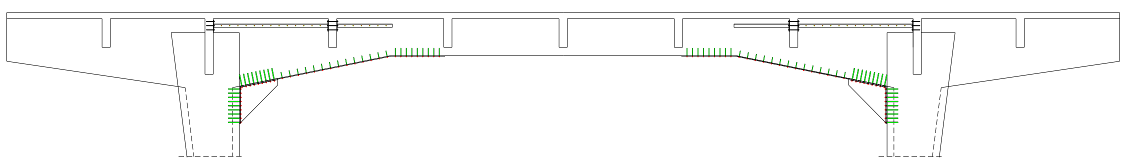

Figure 5.2. General view of strengthening of existing column-beam connections and of beams by additional steel plates.

The strengthened connection has been checked in accordance with capacity design principle and it has been proved that the strength capacity of the connection satisfactorily meet the demand of the internal forces due to the increased total effect. Figure 5.3. shows the strengthening of existing colum-beam connections and of beams by additional steel plates.

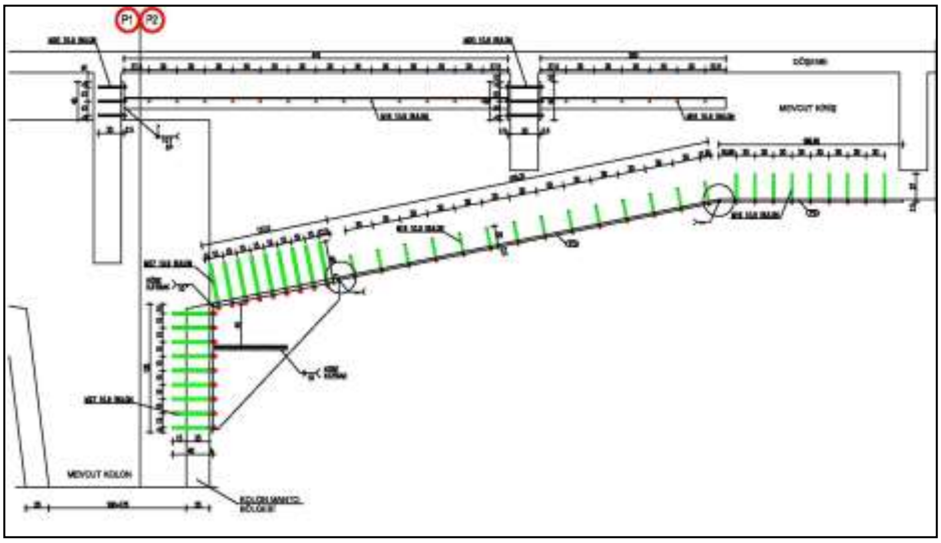

Figure 5.3. Strengthening of existing column-beam connections and of beams by additional steel plates.

6. Conclusion

In this paper, with reference to a particular project commissioned by İstanbul Metropolitan Municipality (İBB) which involved the inspection, evaluation and seismic retrofit design of various existing bridges, the seismic performance of bridges have been studied and the employed calculations methods 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 and on the other hand in the case of deformation based evaluation pushover analysis. 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. In conclusion, considering the high seismicity of our country such evaluations and studies should be generalized in order to minimize the unenviable results of the earthquake effects.

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