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Seismic Evaluation and Upgrading Design of Overhead Roads between Reactor Buildings of WWER-1000MW type NPP

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ABSTRACT

This paper presents results obtained during the study of overhead roads between Reactor Building (RB) of WWER-1000MW NPP and possible measures for their seismic upgrade. The main objective of this project is to evaluate the behavior of overhead roads under site-specific seismic loading and to determine whether this structure satisfies current international safety regulations, followed by development of upgrading concepts.

Overhead roads are precast RC structure, which can be divided to separate substructures. They comprise of pedestrian gallery and pipeline box, connecting reactor buildings with auxiliary building. They are mounted at approximately 10m above ground level. The overhead roads are evaluated for Review Level Earthquake (RLE) as seismic category II structures. As seismic input motion is RLE, free field response spectra anchored to 0.2g PGA are used with 0.5 scaling factor. Soil-Structure Interaction effects are taken into account through equivalent soil springs with frequency adjusted stiffnesses.

In order to meet the objective of the project a technical design specification is developed for conformance with International, US and Bulgarian standards and codes, taking into account site specific conditions. The general approach is consistent with up-to-date practice for evaluation and upgrade of nuclear power plant facilities. The separate steps comprising the overall fulfillment of project’s major objectives may be summarized as follows: study of all available data for initial design and as built conditions, creation of 3-D detailed finite element models for as-built structure, determination of dynamic characteristics, evaluation of adequacy of initial design under new seismic loading (calculation of D/C ratios for structural members and connections, evaluation of embedment lengths for embedded parts and rebars, defomation evaluation, stability checks), development of upgrading concepts for enhancement, verification of capability of upgraded structure to resist relevant design load combinations, calculation of all important characteristics for the upgraded structure, including derivation of response spectra for specific parts of the structure, where equipment is attached.

KEY WORDS: overhead roads, upgrading concepts, precast RC, RLE, free field response spectra, Soil-Structure Interaction, seismic upgrade, floor response spectra

INTRODUCTION

Overhead Roads are situated between Reactor Buildings and Auxiliary Building. They consist of separate segments divided by expansion joints. There are six segments. Segment 1 and Segment 6 as well as Segment 2 and Segment 5 are identical.

The structure of Overhead Roads consists of a reinforced concrete (RC) box (technological trestle), its superstructure enclosing a pedestrian gallery and their RC frame supports. It is analysed for vertical and lateral loads using a three-dimensional model developed with SAP2000 structural analysis programme. Schematic layout of the Overhead Roads between Reactor Buildings 5&6 and Auxiliary Building is presented on Fig 1.

STRUCTURE AND MODEL DESCRIPTION

Typical structural section of the Overhead Roads is 12m long and is repeated along the whole structure with some modifications as shown on the original drawings. The typical structural section is supported at each end. Each footing supports a main “Inverted U-shaped” frame (Main Frame). Above this base frame the RC box (technological trestle) is mounted. Another “Inverted U-shaped” frame (Upper Frame) enclosing the pedestrian gallery is mounted above with spacing of 6.0m. Typical cross section of the structure is shown on Fig. 2. The Upper Frames, located above Main Frame structure are directly supported on Main Frame columns, whereas Upper Frames at midspan (Main Frame spacing is 12.0m) are supported on RC box. That is why upper frames are subdivided in two basic substructures:

Upper Frame Type A - supported directly on Main Frame (UFA)

Upper Frame Type B - supported on RC box (UFB).

All elements (footings, columns, girders, box segments, roof and cladding panels) are precast concrete members with connections as specified on original drawings. Figures 3, 4, 5, 6, show models of separate segments. In the evaluation stages, an estimate of mutual displacements was performed.

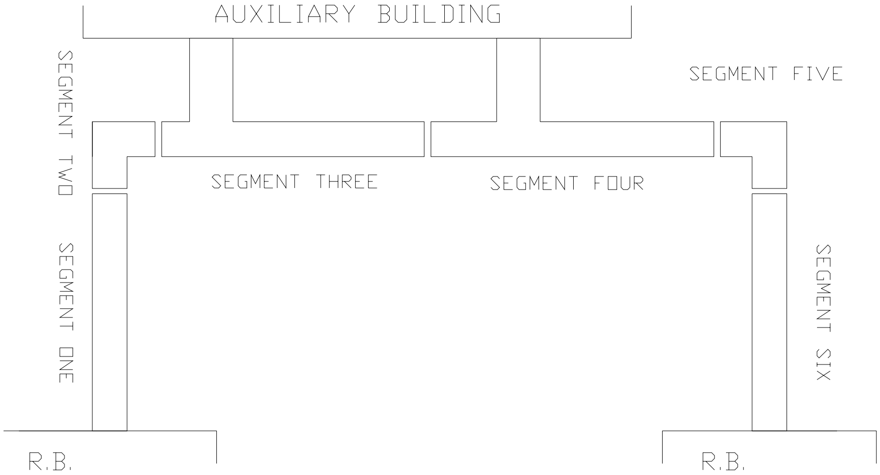

Fig.1

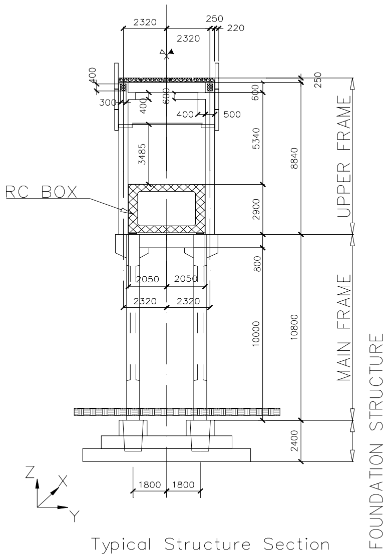

Fig.2

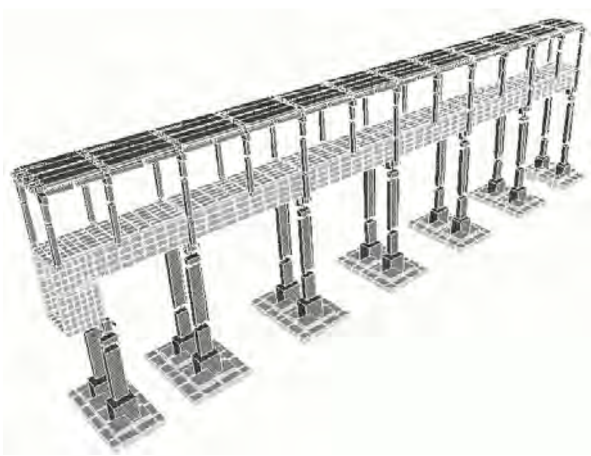

Fig.3 Model of Segment 1

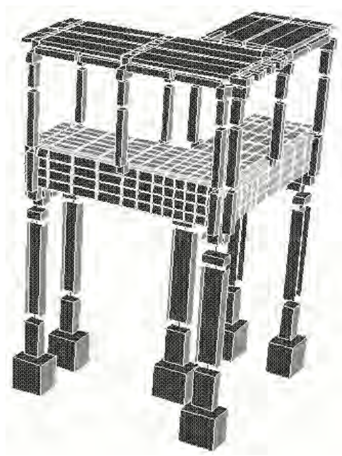

Fig.4 Model of Segment 2

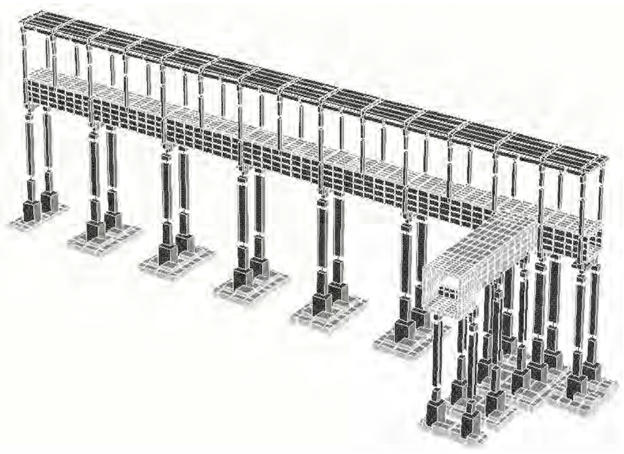

Fig.5 Model of Segment 3

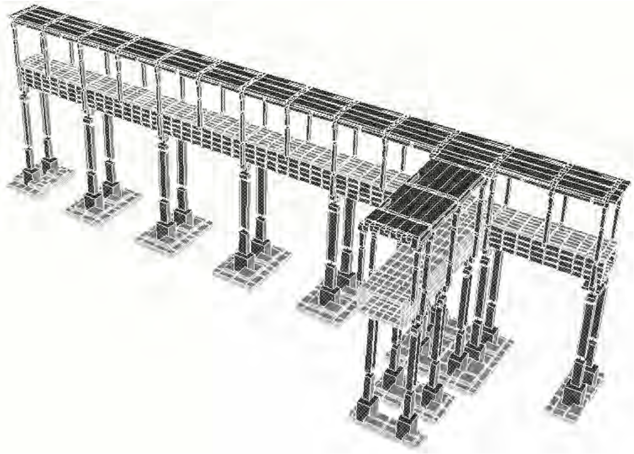

Fig.6 Model of Segment 4

Typical footing is modeled with planar shell elements using appropriate thickness. Soil-structure interaction is accounted through nonlinear elements with frequency independent damping. Rotational springs are determined and included in the model before the evaluation of the stress-strain distribution in the structure is performed during seismic response analyses. At support nodes are applied mass moments of inertia, corresponding to the inertial characteristics of the foundations. Footing caps are modeled with frame elements rigidly connected to the footing shell elements and with rigid zone offset corresponding to the footing thickness. The rigid end factor is taken as unity.

Typical Main Frame columns have two types of sections along their height - a rectangular section 0.7x1.2m at both ends, and in the middle part (from 2.57m to 9.37m) - a double tee section.

Reinforced concrete box typical element is 12m long. It is a precast concrete one piece rigid member. Its side walls are 0.5m thick whereas bottom and top plates are 0.4m thick respectively. The weight of this box without finishes and special protective covers is ~15t/m'. It is supported on bearings, which can transfer only shear.

All structural members are present in the model with their real geometry and material characteristics and that is why they are included with a selfweight multiplier equal to unity for both mass and load cases. The other important masses and loads respectively are from three main sources:

- Pipelines and their steel supporting structures.
- Facade panels, enclosing the pedestrian road.
- Finishes from roof, suspended ceiling, inside and outside Reinforced Concrete Box (RCB).

Approximate total mass of Segment 1 is 3500t, of Segment 2 - 810t, Segment 3 – 5500t, and of Segment 4 – 5300t.

Preliminary model variations

In order to provide optimal mesh size without loosing accuracy, nor with unnecessary high mesh density, four preliminary structural models for the typical reinforced concrete box spanning at 12m were prepared.

They all have the inherent properties of the structure:

- 3D model comprising of shell elements for the box horizontal and vertical walls with hinge supports at four ends as a one span beam;
- upper frame structure supported at RC box's midspan.

These models are created with relevant cross sections, rigidities, masses and loads adequately generalized. The different meshes were chosen based upon previous experience and engineering judgement and are shown in Table 1 below. Table 2 shows normalized periods for the different models.

Table 1 Preliminary mesh variations

Direction	number of elements			
	model 1	model 2	model 3	model 4
Transverse	6	6	4	4
Verical	4	4	4	4
Longitudinal	8	12	8	12
Signature	6x4x8	6x4x12	4x4x8	4x4x12

Fig.7 Preliminary 6x4x12 model – undeformed shape

Table 2 Normalized periods for the first nine modes of the four models

model	6x4x8	6x4x12	4x4x8	4x4x12
mode	period,(s)	period,(s)	period,(s)	period,(s)
1	1.0000	1.0000	1.0158	1.0159
2	1.0000	1.0005	1.0357	1.0369
3	1.0000	1.0155	1.1000	1.1300
4	1.0000	1.0115	1.0700	1.1000
5	1.0000	1.0135	1.1300	1.2200
6	1.0000	1.0162	1.0500	1.4100
7	1.0000	1.0232	1.0300	1.5500
8	1.0000	1.0140	1.0100	1.4200
9	1.0000	1.0189	1.0500	1.4200

The normalized to model 1 reactions in one specific support from vertical load (LOAD 1) and from seismic load (SPEC 1) were examined. They are shown in Table 3.

Table 3 Normalized reactions in a specific node for the four models

model	6x4x8		6x4x12		4x4x8		4x4x12	
Direction	LOAD 1	SPEC 1	LOAD 1	SPEC 1	LOAD 1	SPEC 1	LOAD 1	SPEC 1
Longitudinal	1.00	1.00	1.00	1.00	1.00	1.11	1.00	1.07
Transverse	1.00	1.00	1.00	1.01	1.00	1.03	1.00	1.02
Verical	1.00	1.00	0.99	1.00	1.04	1.08	1.03	1.06

The observation of mode shapes shows that they are similar. A difference occurs nevertheless, concerning the RC box masses and rigidities participation in the dynamic responses of the overall structure. These differences grow with the mode number growth. The first two modes include mainly Upper Frame mass excitation. That is why there is a growing relative difference between the periods especially in higher modes between models with 6 and 4 transverse elemets. At the same time there is a neglectable difference between 6x4x8 and 6x4x12 models.

Based on the implemented model investigation above and on the result obtained, the model with 6x4x8 mesh was chosen as a basic model for the corresponding structure of pedestrian and technological trestle. The comparison of the mode periods and reactions at specific node shows that enough accuracy is gained for that model. The model created represents sufficiently the real physical behavior and the deformation distribution in the structure, under static and seismic loads.

For footings also were performed different comparative analyses for various models. On Figures 8,9, 10, 11 are shown some of them. In the modeling of Segments foundation mesh from Model 2 (Fig. 9) is used.

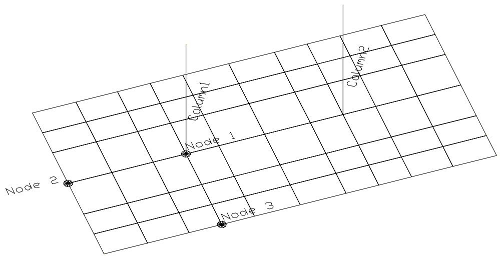

Fig.8 Foundation mesh (Model 1)

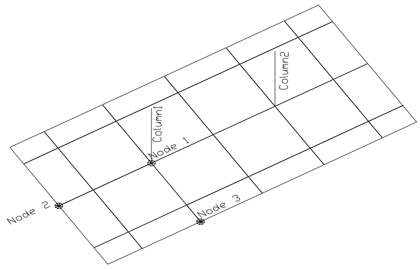

Fig.9 Foundation mesh (Model 2)

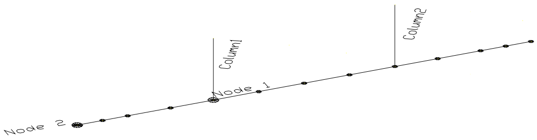

Fig.10 Foundation mesh (Model 3)

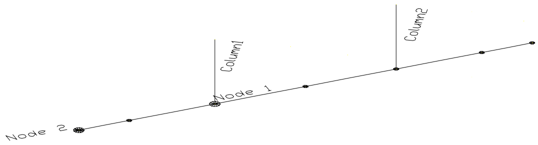

Fig.11 Foundation mesh (Model 4)

DYNAMIC CHARACTERISTICS OF THE MODELS

The dynamic analysis of Segments shows fundamental frequencies in the range of 0.9Hz-1.0Hz, which is low for this type of structure. The direction of the first mode is as expected longitudinally along the gallery. Models with upper bound soil stiffness are quite close to fully restrained structure. All models of different segments have geometrically similar first modes. Frequencies and mass participation ratios for Segment 1 are presented in Table 4.

This analysis marks high relative displacements (especially between UF and RCB at top of RCB), which is result of structural irregularity, specifically stiffness irregularity and irregular mass distribution.

For determination of response of equipment mounted at overhead roads instructure floor response spectra were generated following requirements of Regulatory Guide 1.122 of NRC [7].“Development of floor design response spectra for seismic design of floor supported equipment or components”. These locations are at three elevations in height and in plan as is shown on Fig.12.

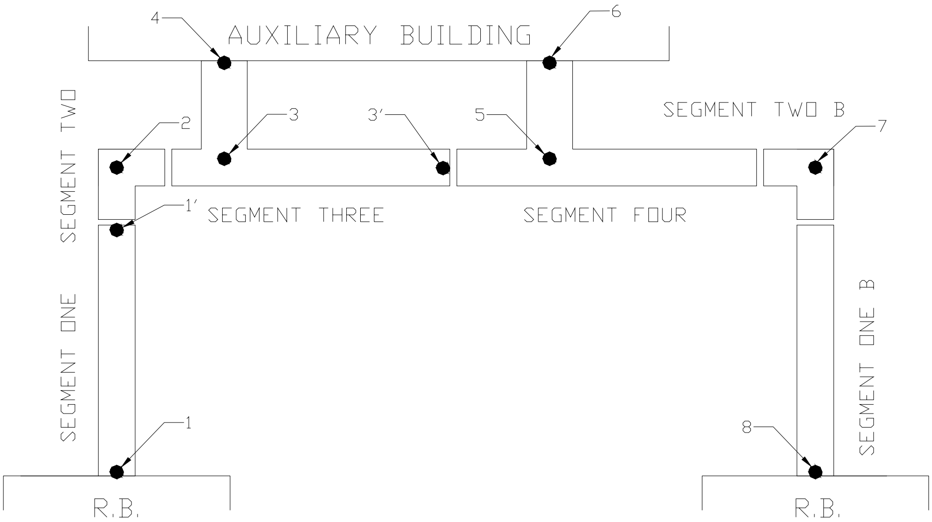

Fig.12 Locations at which response spectra are generated

Table 4 Dynamic characteristics – frequencies and mass participation ratios

MODE	PERIOD	FREQUENCY	INDIVIDUAL MODE(PERCENT)			CUMULATIVE SUM(PERCENT)		
#	s	Hz	UX	UY	UZ	UX	UY	UZ
1	1.071	0.93	57.73	0.00	0.00	57.73	0.00	0.00
2	0.983	1.02	0.00	50.39	0.00	57.73	50.39	0.00
3	0.804	1.24	0.00	6.73	0.00	57.73	57.12	0.00
4	0.514	1.94	0.00	1.19	0.00	57.73	58.31	0.00
5	0.446	2.24	4.39	0.00	0.00	62.12	58.31	0.00
6	0.353	2.83	0.00	3.05	0.00	62.12	61.36	0.00
19	0.092	10.92	0.00	0.00	2.91	62.17	62.48	9.28
20	0.091	11.00	0.00	0.00	41.95	62.17	62.48	51.23
21	0.090	11.15	0.00	0.00	13.31	62.17	62.48	64.55
22	0.087	11.54	0.00	0.00	0.27	62.17	62.48	64.82
23	0.085	11.71	0.00	0.00	1.27	62.17	62.48	66.09
28	0.082	12.14	0.00	0.00	10.55	62.24	62.49	76.72
35	0.077	12.92	0.01	0.00	1.06	62.28	62.65	78.16
39	0.072	13.96	0.00	0.00	5.14	62.28	62.67	83.43
56	0.058	17.16	0.29	0.00	0.15	62.61	62.68	83.74
58	0.056	17.74	0.13	0.00	0.61	62.73	62.69	84.34
62	0.053	18.75	0.02	0.00	0.20	62.75	62.71	84.54
64	0.053	18.97	0.01	0.00	0.35	62.76	62.71	84.91
68	0.051	19.74	4.43	0.00	0.04	67.21	62.71	84.94
69	0.050	19.89	11.62	0.00	0.00	78.84	62.71	84.94
70	0.049	20.26	1.17	0.00	0.02	80.01	62.71	84.96
74	0.049	20.40	0.00	15.28	0.00	80.01	78.98	84.96
77	0.048	20.71	2.12	0.00	0.17	82.14	79.02	85.14
80	0.047	21.16	0.00	3.60	0.00	82.31	82.62	85.14
96	0.045	22.22	0.45	0.00	0.18	82.80	82.64	85.33
97	0.045	22.27	2.96	0.00	0.04	85.77	82.64	85.37
…………………………………………………………..								
151	0.037	27.16	0.00	4.87	0.00	86.72	92.88	89.23
152	0.037	27.18	1.21	0.01	0.01	87.93	92.89	89.24
153	0.037	27.22	4.99	0.00	0.00	92.92	92.89	89.24
…………………………………………………………..								
191	0.032	31.44	0.00	0.00	0.00	95.67	95.77	89.87
192	0.032	31.45	0.07	0.00	0.13	95.74	95.77	90.00

Load Cases

Dead load includes the following: selfweight of all structural elements and weight of finishes.

Live load is located on top of RCB, which is the corridor floor of the pedestrian gallery. Its value is 4.0 kN/m^2. The participation of live load in axial forces is less than 5% compared to dead load.

Snow design load value is 1 kN/m^2 for the NPP site. It may occur only over roof of pedestrian gallery and over RCB at diverting to Auxiliary building (Segment 3).

Earthquake load is applied as described in [6].

Load combinations

The seismic category II structures are analysed for their capability to withstand a RLE scaled with 0.5 factor.

For category II steel structures, the load combinations and applicable criteria can be summarized as follows, according to ACI 349-89 Code, see also [1]:

$$DL + LL + S + 0.5E_{RLE}/F\mu$$

Where:
DL - Dead Load
LL - Live Load
S - Snow Load
RLE - Review Level Earthquake
Fμ - inelastic energy absorption factor; Fμ=1 due to non-ductile details and design solutions.

Demand to capacity ratios are calculated for all important sections and connections. Below are given the demand to capacity ratios for Main Frames of Segment 1.

Table 5 Demand to capacity values for Main Frames – Segment 1

Segment	Axis	Columns	Beams	Beam to column connection
1	2	3	4	5
1	1	0.991	1.171	1.90
	2	0.895	0.849	2.08
	3	0.915	1.241	2.97
	4	0.966	1.554	3.35
	5	1.005	1.731	3.71
	6	0.973	1.849	3.89
	7	0.953	1.427	3.39

It is obvious that the weakest link are connections. This is generally the problem with precast RC structures, especially if special attention is not paid to connection details and to sustain reversal loading. Most of connection details possess either insufficient capacity, either detailing leading to brittle failure mode or both of them.

CONCLUSIONS

Analyses, which were performed, showed that the structures of the overhead roads do not have the capacity to withstand prescribed loads. This is mostly due to the increased seismic loads, developed after reevaluation of the site seismic hazard.

Detailing is the weakest point of the precast concrete structures. At the time of design of overhead roads the ductile detailing and construction was not applied. Footings are rigid bodies and are capable of resisting design loads. Demand to capacitites of main frame columns do not exceed 1.10. More problematic are the main frame beams. They have high strength deficiency and will require significant strengthening. This strengthening will consist of local and global measures. Longitudinal direction of structure is weaker than transverse. Main frame columns are so oriented that they have to resist transverse loading with their weak axis. The weakest connection of main frame is the beam-column connection. Demand-to-capacities of upper frame are similar to that of main frame, except that upper frame type B, as described, is supported on steel corbel. This corbel is welded to steel embeds in reinforced concrete box and thus has very poor dynamic behaviour. Reinforced concrete box does not have problems with satisfying strength requirements, since the radioactive protection requirements prevail.

This analysis demonstrated the importance of correct soil-structure interaction modeling, which has significant influence on final results. Seismic upgrading of the structure is proven as result of this study. Also it is going to be a bias for further upgrading of the structure. The upgrading solutions include implementation of new structural members, and use of high tech materials for strengthening of joints.

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