MODAL PUSHOVER ANALYSIS OF INNER CONTAINMENT STRUCTURE

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ABSTRACT

The containment structure is the most important civil engineering structure of a nuclear power plant (NPP), housing the reactor and major safety related components. Seismic safety of the containment structure is of utmost importance. Seismic re-evaluation of older generation nuclear power plants is carried out to ensure its seismic safety. In seismic re-evaluation, the reserve capacity of the structures and systems owing to its inelastic behavior is also considered. Nonlinear response history analysis (NLRHA) is the ultimate tool for reliable estimation of seismic demand for inelastic systems. However, the method is computationally expensive and overly complex for general use. Modal pushover analysis (MPA), which retains the conceptual simplicity of a conventional pushover analysis, has been examined in the present study as a tool to estimate the seismic demand of inner containment structure of NPP. The response of the IC structure, modelled as a 2-D stick, has been studied for different types of ground motions. Efficiency of MPA is determined by comparing the responses estimated by MPA with that of NLRHA of an MDOF model. It is observed that the estimations tend to be more biased when the non linear behaviour of the structure becomes predominant.

INTRODUCTION

The containment structure is the most important civil engineering structure of a nuclear power plant (NPP), housing the reactor and major safety related components. Seismic safety of the containment structure is of utmost concern. The current generation of nuclear power plant is engineered to withstand earthquakes having a probability of exceedence of less than 1 in 10000 years. In order to ensure the seismic safety of older generation nuclear power plants, seismic re-evaluation is carried out. The seismic re-evaluation of a NPP is prompted by one or more of the following:

1. Evidence of a greater seismic hazard at the site than expected before
2. Regulatory requirements, such as periodic safety reviews, to ensure that the plant has adequate safety margins for seismic loads
3. Lack of anti-seismic design or poor anti-seismic design
4. New technical findings, such as vulnerability of some structures or equipment subjected to real earthquakes

A seismic re-evaluation programme evaluates the current capability of the plant to withstand the maximum potential seismic effect at the site and to identify any necessary upgrades. Current NPP design criteria and comprehensive seismic design procedures, as applied to the design of new facilities are applied in the seismic evaluation programme. However unlike new designs, the reserve capacity of the structure owing to its inelastic behaviour is also considered during a re-evaluation programme. Nonlinear response history analysis (NLRHA) is possibly the most accurate solution available for reliably estimating the inelastic seismic demand and cumulative damage in all structural systems. However, NLHRA is computationally expensive and overly complex for general use. Nonlinear static procedures, commonly termed as pushover analysis methods were developed to overcome this issue of NLRHA.

A static pushover analysis provides reasonably good estimates of seismic demands on structures, whose inelastic behaviour is well distributed along the height of the structure and the seismic response is predominantly governed by the fundamental mode of vibration. However, the method has its limitations: inability to account for contribution from higher modes and not including the redistribution of forces due to yielding. To overcome these limitations of conventional pushover analysis, a number of modified pushover analysis methods evolved. One set of method involves the use of different loading patterns and thereby accounting for higher mode effects [1, 2, 3] and the other involves the use of adaptive load distributions to account for redistribution of forces over time [4, 5]. The first set of methods uses multiple equivalent single degree of freedom (ESDOF) systems to estimate the seismic response of the structure, as opposed to a single ESDOF system used in the conventional pushover analysis. One popular method falling in the first category is the modal pushover analysis (MPA) developed by Chopra & Goel [1].

In the present paper, application of MPA to estimate the seismic demands on an inner containment (IC) structure of an NPP is attempted. The response of the IC structure, modelled as a 2-D stick, is studied for different types of ground motions. Seismic demands based on MPA are compared with those based on NLRHA of MDOF model to understand the efficiency of MPA in estimating the behaviour of the structure.

MODAL PUSHOVER ANALYSIS

Modal pushover analysis (MPA) overcomes the limitations of conventional pushover analysis, with regard to higher mode contributions, without losing the conceptual simplicity and computational attractiveness of the conventional pushover analysis. MPA as a tool for seismic evaluation of engineering structures has grown since its demonstration by Chopra and Goel in 2002 [1]. The method has been successfully applied to evaluate seismic demands on symmetrical framed structures [1, 6], asymmetrical framed structures [7], continuous systems such as bridges [8], chimneys [9] etc. MPA has also been used for the evaluation of energy demands on structures [10] and for the estimation of Park-Ang damage index [11].

Modal pushover analysis is based on three principal assumptions:

1. Coupling among modal coordinates arising from the yielding of the system can be neglected.
2. The peak response of the inelastic multi degree of freedom (MDOF) systems associated with each modal force distribution can be determined from pushover analysis
3. The total response can be determined by combining the peak modal responses by standard modal combination rules.

Mathematical concepts of MPA

The governing equation of a multi degree of freedom (MDOF) system having N degree of freedom, subject to an earthquake ground motion $\ddot{u}_g(t)$ can be written as

$$\mathbf{m}\ddot{\mathbf{u}} + \mathbf{c}\dot{\mathbf{u}} + \mathbf{f_s} = -\mathbf{m}\iota\ddot{u}_g(t) \tag{1}$$

Here, $\mathbf{u}$ is the vector of lateral displacements relative to the ground; $\mathbf{m}$ and $\mathbf{c}$ are the mass and the damping matrices of the system and ι is the influence vector. $\mathbf{f_s}$ is the resistance force vector which varies with time. For inelastic systems, the lateral forces are nonlinear functions of the displacement history:

$$\mathbf{m}\ddot{u} + \mathbf{c}\dot{u} + \mathbf{f_s}(u, \mathrm{sign}\dot{u}) = -\mathbf{m}\iota\ddot{u}_g(t) \tag{2}$$

Equation 2 represents a set of coupled equations and direct solution of these is what is carried out in an NLRHA of MDOF system. To carryout MPA, Equation 2 is transformed to the modal coordinates of the initial linear elastic system. Each structural element of the elastic system is defined to have the same stiffness as the initial stiffness of the structural element of the inelastic system; and both systems have the same mass and damping properties. Therefore, the dynamic properties of the corresponding linear system are the same as those of the inelastic system undergoing small oscillations. As given in [1], the displacements of the inelastic system can be expanded in terms of the modal vibration properties as

$$\mathbf{u}(\mathbf{t}) = \sum_{n=1}^{N} \phi_{\mathbf{n}} q_n(t) \tag{3}$$

where $\phi_{\mathbf{n}}$ and $q_n(t)$ are the n^{th} mode generalized displacement vector and the n^{th} modal coordinate respectively. Using Equation 3 and pre-multiplying it by $\phi_{\mathbf{n}}^{\mathbf{T}}$, Equation 2 can be rewritten as

$$\ddot{q}_n + 2\zeta_n\omega_n\dot{q} + \frac{F_{sn}}{M_n} = -\Gamma_n\ddot{u}_g(t) \tag{4}$$

where

$$\Gamma_n = \frac{L_n}{M_n} \quad L_n = \phi_{\mathbf{n}}^{\mathbf{T}} \mathbf{m} \iota \quad M_n = \phi_{\mathbf{n}}^{\mathbf{T}} \mathbf{m} \phi_{\mathbf{n}} \tag{5}$$

The solution for $q_n(t)$ can be determined by relating it $D_n(t)$, the displacement coordinate obtained from the equation of motion for the SDOF system having the properties of the n^{th} mode and subjected to the ground motion $\ddot{u}_g(t)$:

$$\ddot{D}_n + 2\zeta_n \omega_n \dot{D} + \omega_n^2 D_n(t) = \ddot{u}_g(t) \tag{6}$$

From Equations 4 and 6, we get

$$q_n(t) = -\Gamma_n D_n(t) \tag{7}$$

and

$$\mathbf{u_n}(\mathbf{t}) = \Gamma_n \phi_{\mathbf{n}} D_n(t) \tag{8}$$

The preliminary step in MPA for inelastic MDOF systems is to determine the peak responses for the inelastic SDOF systems given by Equation 8. This requires the knowledge of inelastic properties of the N number of SDOF systems. This requires a relation between lateral forces $\mathbf{f_s}$ and lateral displacements $\mathbf{u}$. This is obtained by carrying out a pushover analysis of the structure using the lateral force distributed over the building height in accordance to $\mathbf{s_n}$:

$$\mathbf{s_n} = \mathbf{m} \phi_{\mathbf{n}} \tag{9}$$

The pushover curve will provide the plot of base shear V_{bn} versus the top node displacement u_{rn}. This curve needs to be converted to a relation between F_{sn}/L_n and D_n. This is done using the following relations:

$$F_{sn} = \frac{V_{bn}}{\Gamma_n} \quad D_n = \frac{u_{rn}}{\Gamma_n \phi_{rn}} \tag{10}$$

The yield values of F_{sn}/L_n and D_n are given by

$$\frac{F_{sny}}{L_n} = \frac{V_{bny}}{M_n^*} \quad D_{ny} = \frac{u_{rny}}{\Gamma_n \phi rn} \tag{11}$$

Here $M_n^* = L_n \Gamma_n$ is the effective modal mass for the n^{th} mode. The two variables F_{sny}/L_n and D_{ny} are related by

$$\frac{F_{sny}}{L_n} = \omega_n^2 D_{ny} \tag{12}$$

Equation 12 implies that the initial slope of the second bilinear curve between D_n and F_{sn}/L_n is ω_n^2. From F_{sny}/L_n and D_{ny} based on Equation 11, the elastic vibration period of the n^{th} mode inelastic SDOF system can be computed from

$$T_n = 2\pi \sqrt{\frac{L_n D_{ny}}{F_{sny}}} \tag{13}$$

Using the properties of n^{th} mode inelastic SDOF system, the peak response can be evaluated by solving Equation 6 and the resulting $D_n(t)$ can be substituted in Equation 8 to get the target roof displacement for that mode. The seismic demand quantities of interest can be determined from the pushover curve of the particular mode corresponding to a roof displacement of u_{rn}. This is repeated for all modes of interest. The total demand is determined following any acceptable modal combination rules.

MPA methodology

Modal pushover analysis of the primary containment structure is carried out in the following steps:

1. From the results of eigenvalue analysis, for the n^{th} mode, the invariant force distribution $\mathbf{s_n}$ is determined. Using this force distribution the plot of base-shear versus top node displacement for the structure is obtained from a pushover analysis.

2. The pushover curve is idealized to a bilinear curve. The nonlinear force deformation behaviour for an ESDOF system is developed using Equation 11 - 13 and this curve.

3. The maximum drift D_n of the ESDOF system is evaluated from an NLRHA in the specific acceleration record.

4. floor displacements in the structure are evaluated using the relation $u_n = \Gamma_n \phi_n D_n$. The roof displacement is given by $u_{rn} = \Gamma_n \phi_{rn} D_n$.

5. Other response quantities of interest such as bending moment, shear force and plastic rotations are extracted from the pushover database values at roof displacement u_{rn}.

6. Steps 1-5 are repeated for other modes of vibration.

7. The total response is determined by combining the modal responses by the square root of sum of squares (SRSS) rule.

CASE STUDY

Estimation of seismic response for the inner containment (IC) structure of a pressurized heavy water based Indian NPP using MPA is studied. The results are compared with NLRHA of the MDOF model to determine the bias in the MPA.

Description of structure

The IC structure considered for this study consists of a prestressed concrete cylindrical wall capped by a segmental prestressed concrete dome through a massive ring beam. The containment shell is supported on a circular raft. The typical containment structure considered for the study is depicted in Figure 1. The containment structure responds to seismic excitation like a cantilever beam with a circular cross-section. The segmental dome along with the ring beam acts to stiffen the circular cross section and also adds to the mass of the system. The potential failure mode of the containment structure subjected to seismic loading considered in the study is the flexural failure resulting from concrete crushing in the compression zone of the cylindrical shell.

Mathematical model

The inner containment structure is idealized as a system of lumped masses at elevations of mass concentrations, connected by 2-dimensional beam elements having equivalent geometric properties. The structure is assumed to be fixed at the top of the raft foundation. The earthquake excitation is constrained along a single horizontal direction only. The stick model of a containment structure so developed is also shown in Figure 1. Nonlinearity of the system is modelled in the program Drain-2DX as a function of the axial force-bending moment interaction diagrams and post-yield modulus of elasticity. Seismic response of the containment structure is evaluated using MPA (and NLRHA, for comparison).

Ground motions considered

With the objective of having a comprehensive evaluation of the applicability of MPA to estimate the seismic response of a containment structure, an ensemble of ground motions with different characteristics have been considered in the present study. Details of ground motions considered and their characteristics are presented in Figure 2. The ground motions in this figure are scaled to a PGA of 1.0g, 1.5g and 2.0g for the purpose of evaluation and comparison. These very high values of PGA are considered only to check the effectiveness of MPA when significant nonlinear behaviour is expected.

Analysis

Table 1 describes the dynamic characteristics of the IC structure. Based on the modal analysis results, the first three modes, that together contribute to more than 90% mass participation, are considered for this MPA-based study. The invariant force distributions for MPA, scaled to a base shear value of unity for each mode, are illustrated in Figure - 3. Pushover curves for the three selected modes are provided in Figure 4. The characteristics of the bilinear pushover curves along with the ESDOF system properties evaluated using Equations 11 - 13 are tabulated

Table 1: Modal properties

Mode	**Period** (s)	**Frequency** (Hz)	**Modal participation factor**	**Mass participation** (%)
1	1.79E-01	5.59	1.37	70.77
2	4.84E-02	20.65	0.55	14.23
3	2.67E-02	37.40	0.35	8.34
4	2.01E-02	49.66	0.29	2.26
5	1.61E-02	62.25	0.19	0.21
6	1.29E-02	77.50	0.13	0.51
7	1.15E-02	87.24	0.10	0.46

Table 2: Properties of the ESDOF system for different modes of the structure

Modes	K_n (N/m)	αK_n (N/m)	V_{bny} (N)	u_{rny} (m)	M_n^* (kg)	Γ_n	ω_n (rad/s)
Mode-1	8.82E+09	8.09E+05	1.28E+08	1.45E-02	9.79E+06	1.37E+00	35.15
Mode-2	6.02E+10	1.54E+06	2.96E+08	4.91E-03	1.97E+06	5.50E-01	129.72
Mode-3	1.88E+11	7.12E+06	6.18E+08	3.29E-03	1.15E+06	3.46E-01	237.26

in Table 2. From the peak response of the ESDOF system for each ground motion excitation, the top node (dome crown) displacement is evaluated using Equation 8. The maximum displacement values of the top node for different ground motions considered in the study are tabulated in Table 3. The response quantities of interest, viz. shear force, bending moment and storey displacements are extracted from the pushover curves for the computed value of top node displacement. NLRHA of the structure modelled as a MDOF system for different ground motions is carried out in DRAIN-2DX, and the response quantities of interest are evaluated. The bias in the response of the structure computed from MPA for the seismic excitation is evaluated using

$$\text{Bias} = \frac{\text{Response from NLRHA}}{\text{Response from MPA}} \tag{14}$$

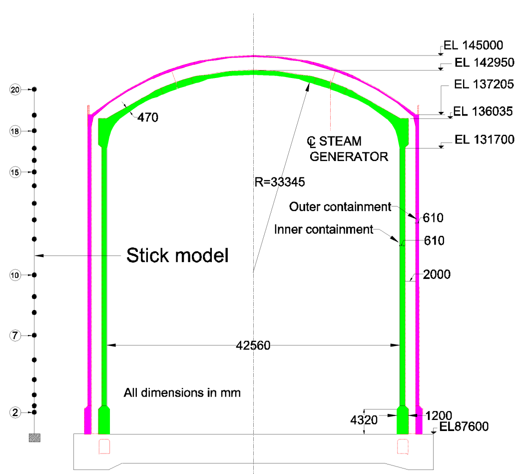

Fig. 1: Containment structure and stick model

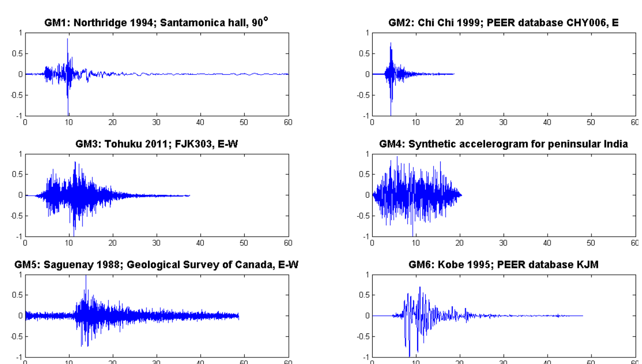

Fig. 2: Ground motions considered for study (All 1g PGA)

Results and discussion

Each of the six ground motions are scaled to PGA 1g, 1.5g and 2g as a part of the study. Except in case of GM3, PGA of 2g results in nonlinear behaviour for the first mode. GM1, GM4, GM5 and GM6 provide nonlinear

Table 3: Maximum top node displacement (u_{rn}, in m) for the considered ground motion

	PGA	GM1	GM2	GM3	GM4	GM5	GM6
	1g	2.38E-02	9.50E-03	5.54E-03	3.00E-02	2.02E-02	1.47E-02
Mode-1	1.5g	3.68E-02	1.42E-02	8.30E-03	6.21E-02	3.17E-02	3.18E-02
	2g	5.93E-02	2.18E-02	1.11E-02	1.56E-01	4.28E-02	1.43E-01
	1g	3.69E-04	6.66E-04	5.83E-04	5.94E-04	5.39E-04	3.25E-04
Mode-2	1.5g	5.50E-04	9.96E-04	8.75E-04	8.91E-04	8.09E-04	4.84E-04
	2g	7.32E-04	1.33E-03	1.17E-03	1.19E-03	1.08E-03	6.49E-04
	1g	6.57E-05	7.96E-05	1.28E-04	6.57E-05	7.27E-05	6.23E-05
Mode-3	1.5g	1.00E-04	1.21E-04	1.90E-04	9.69E-05	1.07E-04	9.00E-05
	2g	1.31E-04	1.59E-04	2.56E-04	1.28E-04	1.45E-04	1.21E-04

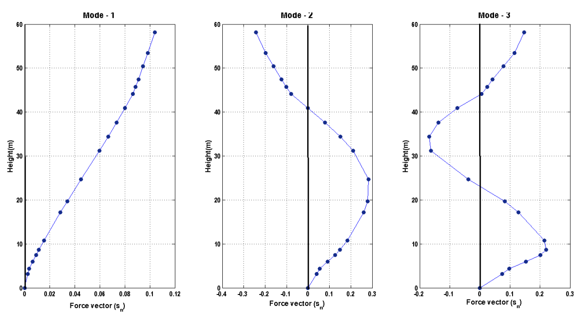

Fig. 3: Invariant force distributions for MPA

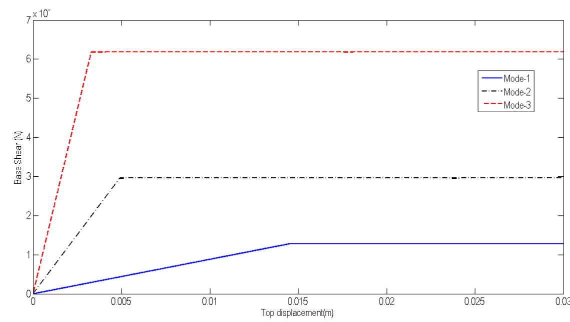

Fig. 4: Pushover curves for the three modes of vibration

behaviour for all the three PGA values in the first mode. A summary of bias statistics for MPA for various ground motions and PGA values for displacement and shear force is tabulated in Table 4.

The bias in displacement, plastic rotation, shear force and bending moment are plotted along the height through Figures 5 - 8. It is observed that the bias increases as the nonlinear behaviour of the structure becomes dominant, i.e. with increased scale factor. In case of GM3, where the structure is always in linear zone, the bias is relatively lower and almost the same across all PGA values.

In general MPA is found to underestimate the shear force and bending moment demands when compared to NLRHA. MPA also underestimates the displacement and plastic rotation demands. The underestimation of bending moment and shear force demands increases with the increase in PGA values. However, similar trend is not observed for the estimation of displacement and plastic rotation demands.

CONCLUSIONS

Modal pushover analysis, as a method to determine seismic demand on primary containment structure is examined in the present study, subjected to various ground motions. The results of MPA were compared with NLRHA.

Table 4: Bias statistics for various PGA values considering all ground motions

	Shear force			**Bending moment**			**Displacement**			**Plastic Rotation**		
PGA →	1g	1.5g	2g	1g	1.5g	2g	1g	1.5g	2g	1g	1.5g	2g
Mean	1.15	1.27	1.36	1.09	1.17	1.24	1.07	1.13	1.16	0.99	1.03	1.07
Std. Dev	0.14	0.20	0.26	0.13	0.16	0.18	0.15	0.25	0.35	0.09	0.15	0.22
Maximum	1.50	1.82	2.15	1.48	1.69	1.64	1.49	1.80	2.13	1.18	1.33	1.49
Minimum	0.79	0.97	0.97	0.79	0.96	0.96	0.88	0.87	0.66	0.81	0.79	0.63
COV	0.02	0.04	0.07	0.02	0.03	0.03	0.02	0.06	0.12	0.01	0.02	0.05

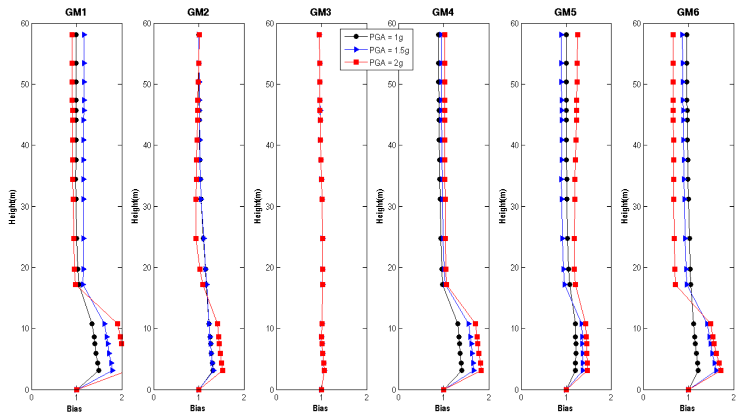

Fig. 5: Bias in displacement

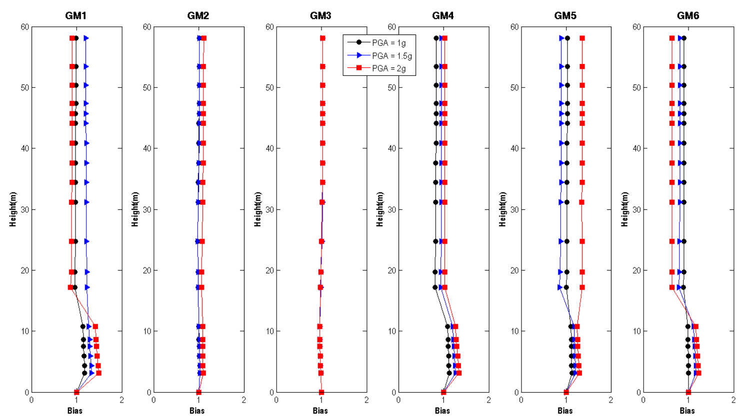

Fig. 6: Bias in plastic rotation

1. The comparison indicates that the error in MPA increases with the increase in nonlinearity of the structure. For systems in linear zone, including even slightly nonlinear behaviour, the error is relatively less. As the general range of operation of the considered structure is in this range, MPA can be considered as a viable alternative to NLRHA.

2. The bias is observed to increase towards the base of the structure compared to the top. The mean bias and standard deviation for all the considered parameters increase with an increase in PGA values.

3. Estimation of seismic demand by MPA is computationally less demanding compared to NLRHA and conceptually simple.

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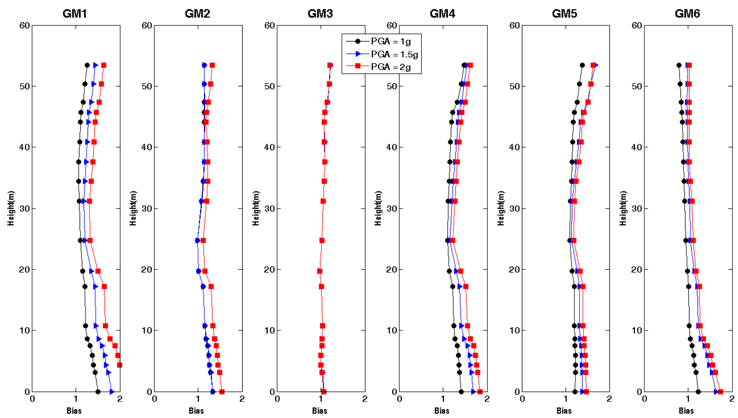

Fig. 7: Bias in shear force

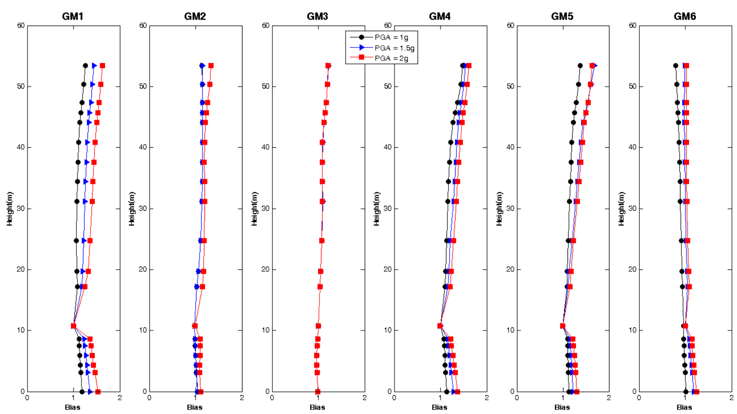

Fig. 8: Bias in bending moment

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