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DETERMINISTIC AND PROBABILISTIC ASSESSMENT FOR SEISMIC PERFORMANCE OF EXTERIOR BEAM-COLUMN JOINTS WITH HEADED BARS

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ABSTRACT

Section 12.6 provisions of ACI 318-08 detail the development of headed and mechanically anchored deformed bars, and ACI 352R-02 reports design recommendations for headed bars in reinforced concrete beam-column joints. However, both ACI 318-08 and 352R-02 have been developed based on quite limited experimental data. Given this concern, both were re-evaluated using an extensive database encompassing most available test data for reinforced concrete exterior beam-column joints with headed bars subjected to cyclic loading, including some data relatively new. Using the database, deterministic assessment for joint seismic performance was first conducted, then existing design guidelines were verified or new guidelines were proposed. Subsequently, probabilistic assessment was conducted to reveal the influence of design parameters on joint behavior and performance.

Introduction

The use of headed bars is becoming more popular since they provide a solution to the constructional problem associated with steel congestion, particularly in reinforced concrete beam-column joints. Relevant provisions and limitations have been provided in the 2008 edition of ACI 318 (§12.6.1 and 12.6.2). The limitations or restrictions include bar strength, bar and head size, clear cover and bar spacing, and concrete weight. Prior to this, design guidelines for headed bars in beam-column connections were incorporated into the 2002 edition of the ACI 352 report based on both monotonic and cyclic tests. This ACI-ASCE Committee 352 report recommends the development length for headed bars along with some other details such as the location of heads and the amount of head-restraining reinforcement for preventing the prying action of headed bars placed near a free surface of concrete.

In 2000’s, significant amounts of experimental investigation have also been carried out to determine the suitability of each parameter restriction and limitation as imposed by the ACI documents (Kang 2008a, 2008b, Kang et al. 2009). However, no study has yet been done to investigate the coupled effect or quantification of these design parameters which probably

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influence joint response. Furthermore, no previous study has provided a probabilistic estimation of satisfactory seismic joint performance over unsatisfactory performance given a set of design parameters as specified by the code recommendations.

To bridge this gap, a thorough data analysis was conducted using a variety of statistical and empirical techniques, as well as a comparison with results from the laboratory tests. Also, given the widespread use of headed bars in both lateral-force-resisting and non-participating systems, a more rigorous and comprehensive investigation on this subject is urgently needed. Hence, Joint ACI-ASCE Committee 352 recommendations on headed bars in beam-column joints are currently under revision by the Task Group within the committee.

This research study investigates and quantifies the coupled effect of design parameters that might influence seismic behavior of exterior beam-column joints with headed bars based upon an accumulated extensive dataset of experimental investigations on the seismic performance of those joints with headed bars. The dataset consists of 63 interstory exterior joint tests performed by researchers around the world (Kang 2008a, 2008b, Kang et al. 2009). A probabilistic estimate has also been provided of satisfactory over unsatisfactory performance of the joints with headed bars given a set of design parameters that might influence joint responses.

Overview of Assessment

An extensive dataset spanning a wide range of design parameters for reinforced concrete interstory exterior beam-column joints (exterior joints hereafter) with headed bars has been assembled. Only the interstory exterior joints are considered in this study, as the data regarding other types of joints are lacking (e.g., roof level interior and knee joints). All the specimens were subjected to quasi-static reversed cycling loading to simulate seismic forces. This dataset of 63 investigations of exterior joints with headed bars includes classifications of satisfactory and unsatisfactory seismic joint performance based on the following performance indices: 1) the ratio of measured peak moment to nominal moment capacity; 2) drift ratio at the point of 20% drop from the peak lateral load; 3) ratio of strain in the headed bar at the joint-member interface to yield strain; and 4) joint shear distortion during about 3.0% drift cycles. Joint behavior was assumed unsatisfactory if the ratio of peak to nominal moments was less than 1.0 and no bar yielding was monitored by strain gauges. If the specimen exhibited more than 20% reduction in strength until 3.5% drift, and exceeded 1.2% of joint shear distortion until 3.5% drift cycles, the joint was also considered to have exhibited unsatisfactory seismic performance. More details are available in the papers by Kang (2008a, 2008b) and Kang et al. (2009).

The contribution of bond slips to the drift ratio could not be small, but it would not affect the overall effectiveness of the head anchorage. The unsatisfactory behavior resulted mainly from substantial joint shear distortion, along with moderate bond deterioration within the joint. Even after bond deterioration, head bearing resistance was maintained with a relatively small loss. Some degree of pinching (bond slips) is common for reinforced concrete beam-column joints that are part of Moment Frames when subjected to cyclic loading, and it is tolerable for joints that satisfy ACI 374.1-05 seismic performance criteria including pinching indices.

In this paper, the dataset has first been intuitively and empirically assessed to obtain joint performance trends and forensic evidence of the observed behavior. Various factors that impact on seismic joint performance are investigated. Subsequently, binomial logistic regression methodology has been developed to obtain a probabilistic estimate of satisfactory over unsatisfactory performance for exterior joints with headed bars subject to seismic loading, given a set of design parameters. The probabilistic methodology also quantifies the effect of each design parameter in determining the performance of the joint.

Deterministic Assessment for Joint Seismic Performance

In this section, some representative results are depicted and empirically assessed. The overall trends are deterministically discussed in connection with ACI 318-08 and 352R-02.

Figure 1(a) depicts the provided development length for headed bars used in the investigated satisfactory specimens (‘o’ marks), and unsatisfactory specimens (‘x’ marks) that were affected by improper bond development, compared with the values required by ACI 318-08 and 352R-02. Note that these marks are consistently used throughout the paper. The ACI 318-08 equation resulted in conservative estimations for the specimens that exhibited satisfactory seismic performance, whereas most unsatisfactory specimens are located on the left side of the ACI 352 line, indicating that the ACI 352 equation corresponds quite well with the data. Figure 1(a) also demonstrates that unsatisfactory specimens that were affected by improper bond development did not satisfy either ACI 318-08 provisions or 352R-02 recommendations for development length, indicating that the ACI 318 or 352 provides the designer a proper tool to rule out these bond-slip failures.

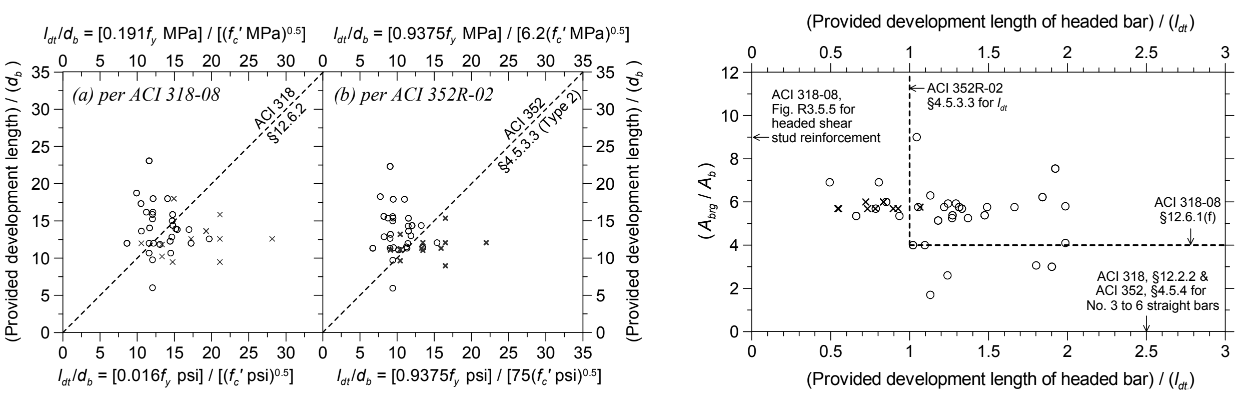

Figure 1. (a) Dataset in terms of development length; and (b) head size vs. development length.

In fact, both the development length and head size determine the anchorage capacity of a headed bar. After considerable bond deterioration (at about 2.5 to 4% drift), anchorage relies in large part on the head bearing acting against the concrete. Therefore, the head size should be large enough to ensure that no pullout (due to local crushing) eventually occurs at the face of the head during this stage. However, the larger head size does not necessarily warrant a shorter required development length to ensure adequate bond behavior at low-to-moderate drift levels (up to 2.5%) (see ‘x’ marks in Fig. 1(b)). Based on the deterministic investigation (Kang et al. 2009), a minimum head bearing area (A_{brg}) of 4 times the bar area (A_b) is feasible for headed bars terminating in beam-column joints, provided that the development length of the bar complies with ACI 352R-02. Perhaps, a size of ($A_{brg}/A_b = 3$) will even be allowed for the seismic design of beam-column joints (Kang et al. 2010). Note that ACI 318-08, Ch. 12 does not consider seismic loading; thus, the findings are of value in the updating ACI 352R-02 and ACI 318-08, Ch. 21.

ACI 318-08, §12.6.1(f) specifies that the minimum clear spacing between headed bars should be $4d_b$; quite large as compared to the conventional practice, where d_b is the bar diameter. The ACI 352R-02 recommendations do not provide guidelines for clear spacing between headed bars in a layer; therefore, the clear bar spacing specified for conventional reinforcing bars would also be used for headed bars as per ACI 318-08 §7.6.1 and §12.2.2, where the bond capacity is known to be affected by the clear bar spacing, when less than $2d_b$. For clear spacing not less than $2d_b$, bond may not be a serious issue for any type of bars (hooked, headed or straight). The clear bar spacing between headed bars may affect the concrete breakout capacity "near the head."

For the satisfactory specimens (Fig. 2(a)), there were neither apparent anchorage splitting cracks, nor side-face blowout failure, nor concrete breakout failure, and no data providing evidence that bearing or pullout failure occurred. Further, it is shown that the small clear bar spacing did not adversely affect the drift ratio measured at a drop to 80% of the peak lateral load, which is considered as one of the seismic performance indicators in this study. Therefore, it is concluded that there was no influence of the clear bar spacing, if not less than $2d_b$, on the lateral force resistance of the tested beam-column joints. Based on these observations of the available experimental database, a recommended limit of $2d_b$ is proposed to be used for the design of beam-column joints in lieu of the current limit of $4d_b$.

ACI 318-08, §7.7 specifies the minimum clear cover for bar protection against extreme weather and/or fire. Following §7.7.1(c) and R7.7, the clear cover to the outermost part of the "head" (c_{ch}) should not be less than 1.5 in. Concurrently, for the purpose of preventing side-face blowout, ACI 318-08 §12.6.1(e) sets a lower limit for the clear cover to the headed "bar" (c_{cb}) as $2d_b$. Both requirements of §12.6.1(e) and §7.7.1(c) are in general not difficult to meet for headed bars anchored within an exterior beam-column joint, if adequate clear cover is also provided for the joint transverse reinforcement based on §7.7. ACI 352R-02 does not provide explicit recommendations for minimum clear cover to the "head." Rather ACI 352R-02 specifies the minimum amount of restraining reinforcement engaging the headed bar just before the head which is needed to produce the strength of $0.25A_sf_y$ for a nonseismic (Type 1) joint or $0.5A_sf_y$ for a seismic (Type 2) joint, where A_s is the headed bar area near the free surface and f_y is the specified bar yield stress.

Figure 2(b) depicts the tested range of side clear cover to the head (c_{ch}) for the satisfactory specimens, along with comparisons to §7.7.1, §12.6.1(e), and Eq. (D-17) of ACI 318-08 (see Kang 2008a and Kang et al. 2009 for more details). In five of the satisfactory specimens, c_{ch} was smaller than the values given by §12.6.1(e), as shown in Fig. 2(b). For all the interstory exterior joints including those five, no horizontal head-restraining reinforcement was provided, as it was not a common practice. However, side-face blowout or spalling of the side clear cover was not observed in any of these joints. The absence of side-face blowout failures was also supported by strain data measured in joint hoops. The hoop strains were below 2,500 μs until the drift exceeded 3%, indicating that the satisfactory behavior was attributed in part to good lateral confinement of the joint core near the head, which was located within the core.

Furthermore, the unsatisfactory joints did not experience side-face blowout, nor did the joints with closely spaced beam bars adjacent to free vertical faces of the joint. Based on the results showing that side-face blowout is not a concern, the requirement of §12.6.1(e) of ACI 318-08 can also be applied for headed bars terminating in beam-column joints. Furthermore, a design recommendation is proposed such that horizontal head-restraining reinforcement is not required for headed beam bars adjacent to a free "vertical" face of an interstory joint, provided that the requirement of §12.6.1(e) is met, and that the lateral confinement is supplied by closed joint hoops and by at least a beam member covering at least 3/4 of the column width.

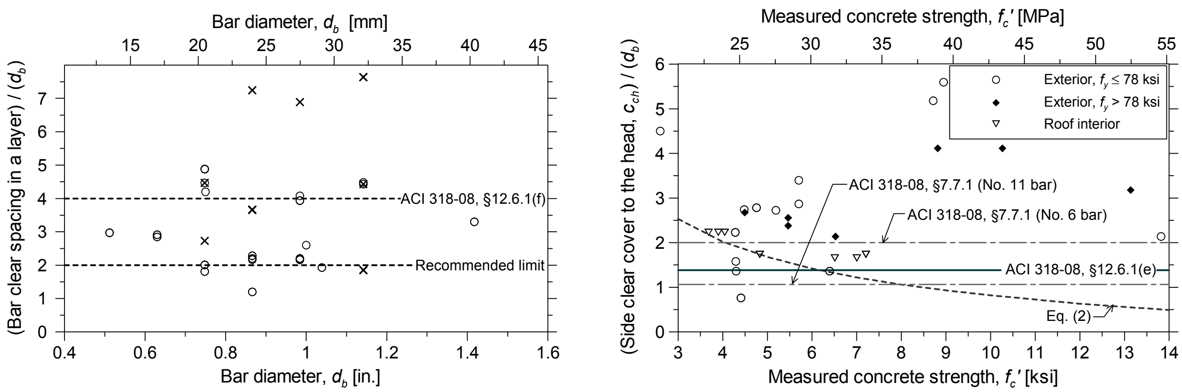

Figure 2. (a) Dataset in terms of bar clear spacing; and (b) side clear concrete cover to the head.

More information regarding empirical and deterministic assessment can be found elsewhere (Kang 2008a, 2008b, Kang et al. 2009). In the following section, detailed information of probabilistic assessment that has been conducted for the given dataset is provided.

Probabilistic Assessment for Joint Seismic Performance

Experimental observations provide a qualitative measure of the impact of various design parameters on joint response. For the current study, a statistical model linking quantitative design parameters and qualitative joint response is desired. Linear and/or nonlinear regression is one possible approach for developing such a model; however, this is not ideal because it requires assigning a quantitative measure to the qualitative joint response parameter. On the other hand, logistic regression is ideally suited for developing this type of model. This method allows for quantification of the conditional probability of a qualitative measure based on quantitative data. For the current study, logistic regression was used to develop a relationship between the headed bar-reinforced beam-column joint performance under seismic loading (a qualitative measure) and a set of independent design parameters (quantitative measures). In comparison with linear/nonlinear regression, the logistic regression model has less stringent requirements, since it does not assume linearity (or nonlinearity) of relationship between the independent variables and the dependent variable nor require normally distributed variables. Both the linear and logistic models employ the regression relationship as:

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \beta_k X_k \tag{1}$$

However, while an ordinary linear regression model defines a quantitative relationship between covariates or continuous variables (X_i) and a continuous dependent variable (Y), a logistic regression model defines a relationship between continuous quantitative variables or covariates (X_i) and the likelihood of occurrence of a discrete qualitative event (Y). For a linear regression model, model parameters (β_i) in Eq. 1 are computed to minimize an error function, typically the sum of the square of the difference between the measured and computed (Eq. 1) response parameter (Y). In a logistic regression model, the discrete nature of the variable (Y) precludes this approach. Thus, the method of maximum likelihood, which provides a means of choosing an asymptotically efficient estimator for a set of parameters, is typically used to compute logistic regression parameters (β_i). A logistic model may be used to predict the likelihood of multiple discrete outcomes. For the current study, only two outcomes of satisfactory and unsatisfactory performance of exterior joints with headed bars subject to cyclic loading were considered. Thus, a binomial logit model was developed for the study.

Development of binomial logit model

In this study, the dependent variable (Y) in Eq. 1 represents the likelihood of observing a satisfactory joint performance versus an unsatisfactory joint performance. To facilitate presentation of the calculations, the case of satisfactory performance is referred to as *Event 1* and the case of unsatisfactory performance is referred to as *Event 0*. It should be noted that the values 0 and 1 do not have any physical significance. Additionally, only event values of 0 and 1 are acceptable and event values between 0 and 1 are meaningless.

In Eq. 1, the likelihood of observing a discrete event of satisfactory performance is defined by the log of the odds ratio for that event. The odds ratio for *Event 1* is the ratio of the probability of occurrence of *Event 1* ($P_{E=1}$) to the probability of occurrence of *Event 0* ($P_{E=1}$) as:

$$Y = \log\left(\frac{P_{E=1}}{1-P_{E=1}}\right) = \log\left(\frac{P_{E=1}}{P_{E=0}}\right) = \beta_0 + \sum_{k=1}^{K} \beta_k X_k \tag{2}$$

where β_i's are logistic regression parameters, X_i's are the covariates or joint design parameters and K is the total number (= 8) of design parameters considered.

Binomial logit analysis results

The design parameters that were considered to affect the performance of the joint were taken as follows: 1) the ratio of provided development length to required development length as per ACI 352 (labeled as *A*); 2) ratio of head thickness to bar diameter (labeled as *B*); 3) ratio of net bearing area to bar area (labeled as *C*); 4) ratio of joint shear at probable beam moment to joint shear capacity (labeled as *D*); 5) specified yield strength of the headed bar (labeled as *E*); 6) joint transverse reinforcing ratio in the direction of lateral loading (labeled as *F*); 7) side cover for the headed bar to bar diameter (labeled as *G*); and 8) ratio of column axial load to compressive strength of the concrete (labeled as *H*).

As described earlier, the regression parameters of β_i in Eq. 2 were obtained using the method of maximum likelihood and a logistic regression model for exterior beam-column joints with headed bars subjected to seismic loading was developed. For each of the independent variable in the models, Table 1 shows the computed regression parameters (β_i). The sign of a regression parameter indicates whether an increase in the associated design parameter increases or decreases the likelihood of the satisfactory performance of exterior joints with headed bars. A positive regression parameter indicates that increasing the associated design parameter increases the likelihood of a satisfactory performance. Similarly, a negative regression parameter indicates that increasing the associated design parameter increases the likelihood of satisfactory performance. Based on the signs of the parameters in Table 1, an increase in $\underline{A}$, $\underline{B}$, $\underline{C}$ and $\underline{D}$ would result in an increase in the likelihood of satisfactory performance of the joint response. An increase in other parameters such as $\underline{E}$, $\underline{F}$, $\underline{G}$ or $\underline{H}$ would result in increased likelihood of unsatisfactory joint performance.

Table 1. Estimated regression parameter (β) and influence factor for each design parameter.

Covariate	$\underline{A}$	$\underline{B}$	$\underline{C}$	$\underline{D}$	$\underline{E}$	$\underline{F}$	$\underline{G}$	$\underline{H}$	Constant
Estimated β	5.22	1.89	1.03	19.88	-0.25	-650	-0.29	-85.3	-1.46
Influence factor	6.01	1.1	5.77	16.56	-24.5	-2.97	-0.95	-2.68	

An increase in the ratio of provided development length to required development length ($\underline{A}$) typically means an improved bond condition of the specimen. Therefore, an improved bond capacity would qualitatively mean better performance of the joint. Figure 1(a) graphically supports the idea that the provided development length directly affects the joint performance. An increase in ratio of the thickness of the anchored head to the diameter of the reinforcing bar ($\underline{B}$) qualitatively means an increase in resistance of the head against deformation which would also equate to better performance of the joint. No comprehensive research on the head thickness has been carried out, nor are standards on head thickness available in ACI 318 codes and ASTM specifications. Based on a prior experimental work by Kang et al. (2010), the head thickness of at least $1d_b$ was considered reasonable.

If the head size ($\underline{C}$) is increased, a better bearing is achieved which qualitatively also results in better performance of the joint. A comprehensive review of the database showed that both the development length and head size determine the anchorage capacity of a headed bar, and that the head size should be large enough to ensure no pullout failure (e.g., $A_{brg} \geq 3A_b$). If the joint shear demand ($\underline{D}$) is decreased, a better performance can also be achieved. The results obtained from the statistical analysis are in direct agreement with what has been observed experimentally (e.g., Chun et al. 2007). Less joint shear deformation was monitored for the connection subject to a smaller joint shear, but with the same or comparable other conditions.

On the other hand, an increase in yield stress of the longitudinal bar ($\underline{E}$) results in an increase in the elastic stiffness of the connection along with exhibition of a rather brittle response and reduced ductility. The reduced ductility due to the use of high strength bars ($f_y > 60$ ksi) results in unsatisfactory performance of the joint. All of the specimens with very high strength steel ($f_y \geq 120$ ksi; see Kang et al. 2009) exhibited unsatisfactory seismic joint performance. Further, only 3 specimens with $f_y \geq 100$ ksi (see Kang et al. 2009) showed satisfactory seismic performance. Therefore, the feasibility of the applications of very high strength steel in beam-column joints is questionable, and further scientific research on this topic is highly needed.

Contrary to the common belief, it was observed that increasing the area of the transverse reinforcement within the joint (F) results in unsatisfactory performance. On a relative note, it has been noted that for joints subjected to seismic loading, transverse reinforcement within the joint does not have any significant influence in failure mechanism of beam-column joints under low axial loads (e.g., $P \leq 0.12A_g f'_c$). All of the specimens in the database were subjected to axial loads not greater than $0.12A_g f'_c$, where A_g is the column gross cross-sectional area and f'_c is the specified concrete strength. Based on a review of the database, a ratio of ($\rho_h/\rho_h^{ACI,2}$) even at the level of about 0.3 appeared not to pose a serious joint shear distress problem under low axial loads, where $\rho_h = (A_{sh}/s_h h'')$, A_{sh} is the area of joint transverse reinforcement in principal direction within hoop spacing (s_h), s_h is the joint hoop spacing, and h'' is the joint core width.

It was observed that increasing the side cover (G) to the headed bar beyond the minimum value results in an increased probability of unsatisfactory performance. Typically the larger side cover is, the less vulnerable the joint is to side-face blowout failure. The review of the database indicates that side-face blowout is not a concern for headed bars anchored in beam-column joints with sufficient side cover, and that the side cover ($2d_b$) requirement of ACI 318-08 §12.6.1(e) is appropriate to prevent side-face blowout. Only 2 of 63 specimens in the database had side cover to the headed bar less than $2d_b$; thus, an adverse effect of increased side cover appears to have no significant physical meaning.

Increasing the column axial ratio (H) also results in an increase in probability of unsatisfactory performance; however, as mentioned earlier, the level of the applied axial load was quite low for all specimens ($P \leq 0.12A_g f'_c$). Thus, no information regarding the effect of high axial loads is available from the database.

The magnitude of a regression parameter multiplied with the mean of its corresponding design variables (referred to as "Influence factor" column in Table 1) indicates the relative importance of the design variables in determining connection failure initiation response. It should be noted that the sign of the influence factor is similar to the sign of the regression parameter and presents similar. Based on results obtained in Table 1, the yield strength of the headed reinforcing bar is the most influential parameter, a decrease of which would result in higher satisfactory performance, whereas the least influential parameters are the side cover and head size. Again, this signals that there is a great and urgent need to carry out research regarding high-strength headed bars in beam-column joints subjected to inelastic deformation reversals.

Probabilistic Assessment Model Evaluation

Goodness of fit of the model

To further evaluate the model, two robust goodness-of-fit tests were performed since no single test can be considered to be comprehensive. The Hosmer and Lemeshow's *chi-square* test and the log-likelihood ratio test were performed to evaluate the overall significance of the model.

The value obtained from Hosmer and Lemeshow's test of the model for exterior joints with headed bars is 16.01 with a corresponding *p-value* obtained from chi-square distribution with 2 degrees of freedom as 0.00033. Since the *p-value* for the model is not significant, it is failed to reject the null hypothesis that there is no difference between observed and model-predicted values, implying that the model's estimates fit the data at an acceptable level.

Predictive efficiency of the model

To assess the predictive efficiency of the statistical model, the likelihood of unsatisfactory joint performance (*Event 1*), computed with β_i from Table 1, was plotted versus the observed event in Figure 3. Specimens from the data set exhibiting satisfactory joint performance (*Event 0*) are plotted as circles and unsatisfactory specimens (*Event 1*) are plotted as squares. If the model was perfect, all specimens exhibiting *Event 0* would have a computed probability of occurrence of *Event 1* of 0, while all specimens exhibiting *Event 1* would have a computed probability of occurrence of 1. The data in Figure 3 indicate that although the model is not perfect for exterior joints with headed bars, the model is able to predict satisfactory performance for 94% of the specimens and unsatisfactory performance for 89% of the specimens.

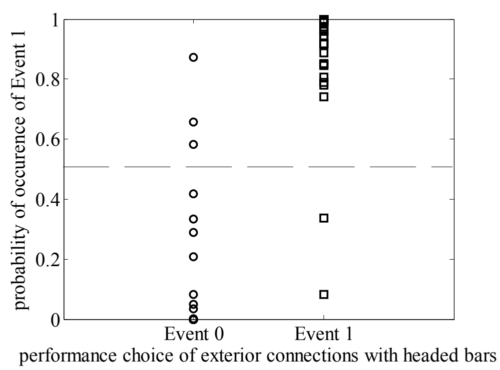

Figure 3. Assessment of predictive efficiency of the statistical binomial logit model used.

Conclusions

In this paper, the database spanning a wide range of design parameters for reinforced concrete interstory exterior beam-column joints with headed bars has been intuitively and empirically assessed to obtain joint performance trends and forensic evidence of the observed behavior. Subsequently, binomial logistic regression methodology has been developed to obtain a probabilistic estimate of satisfactory over unsatisfactory performance for those exterior joints with headed bars subject to seismic loading, given a set of design parameters. The probabilistic methodology also quantifies the effect of each of design parameter in determining the performance of the joint.

The deterministic study reveals that 1) ACI 352 development length for headed bars in beam-column joints is appropriate and thus can be included in §21.7.5 of ACI 318-08; 2) minimum net bearing area of $3A_b$ and minimum clear bar spacing of $2d_b$ could be suggested for both ACI 352R-02 and ACI 318-08, Ch. 21; and 3) ACI 318-08 requirements of the minimum side clear covers to the head and to the bar can be applied to headed bars in beam-column joints.

The probabilistic study reveals that 1) an increase in development length, head thickness and head size and a decrease in joint shear demand qualitatively result in better performance of the joint; 2) an increase in bar yield stress, joint transverse reinforcement, and column axial force results in probability of unsatisfactory performance. The joint shear demand and bar yield stress are two of the most influencing design parameters. The feasibility of the applications of very high strength headed bars in beam-column joints is highly questionable.

Acknowledgments

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